STA-6 2-D Hydraulic Modeling
Task 2.3.1 Final Draft Report

Science and Engineering Support Service (SESS)
Contract No. C-15988-WO04-05

South Florida Water Management District

October 27, 2004

Prepared by:
Sutron Corporation
Hydrologic Services Division (HSD)
6903 Vista Parkway N, Suite 5
West Palm Beach, FL 33411
Tel: (561)-697-8151

Prepared for:
South Florida Water Management District
Attn: Tracey Piccone, Project Manager
B-2 Building, 3rd Floor
3301 Gun Club Road
West Palm Beach, FL 32406

Table of Contents

1. Introduction **4**
2. Model Setup **6**
2.1 Site Description and Conceptual Model of STA-6 Section 1 6
2.2 Topography 8
2.3 Vegetation 10
2.4 STA-6 Structures 12
3. Model Calibration and Verification **15**
3.1 Calibration/Verification Dataset 15
3.2 Calibration strategy 22
3.3 Calibration Results 22
3.4 Verification (Validation) Results 38
3.5 Calibrated Model parameter sets 54
3.6 Sensitivity analysis 55
4. Simulation of Existing Conditions **64**
4.1 Design Flow Condition 65
4.2 Low Flow Condition 69
4.3 High Flow Condition 71
4.4 Sensitivity analysis on Existing Condition Results 74
5. Conclusions and Recommendations **77**
References **78**
Appendixes **80**
Appendix I. STA-6 Vegetation Map (December 2003) 80

List of Figures

Figure 1 Location Map of STA-6 5
Figure 2 Schematic of Existing STA-6 (not to scale) 7
Figure 3 Ground Surface Elevation of STA-6 (Ft NGVD) 9
Figure 4 Schematic of Vegetation Covers in STA-6 11
Figure 5 Illustration of Trapezoidal Weir Cross-section 12
Figure 6 Site Location of Water Depth Collection in STA-6 17
Figure 7 Water Surface Elevation at G392S_T and G393B_H (FT NGVD) 19
Figure 8 Water Surface Elevation at G352S_T and G354C_H (FT NGVD) 20
Figure 9 Stage Hydrograph at G600_T (Calibration) 25
Figure 10 Stage Hydrograph at G352S_H (Calibration) 26
Figure 11 Stage Hydrograph at G392S_H (Calibration) 27
Figure 12 Stage Hydrograph at G352S_T (Calibration) 28
Figure 13 Stage Hydrograph at G392S_T (Calibration) 29
Figure 14 Stage Hydrograph at G393B_H (Calibration) 30
Figure 15 Stage Hydrograph at G354C_H (Calibration) 31
Figure 16 Stage Hydrograph at G354C_T (Calibration) 32
Figure 17 Stage Hydrograph at G393B_T (Calibration) 33
Figure 18 Calibration Plot for Cell 3 (8/18/2003, interior sites) 34
Figure 19 Calibration Plot for Cell 3 (8/19/2003 12 pm, interior sites) 35
Figure 20 Calibration Plot for Cell 5 (8/12/2003, interior sites) 36
Figure 21 Calibration Plot for Cell 5 (8/13/2003, interior sites) 37
Figure 22 G600 Tailwater Level and Pumping Rate (2/22 to 3/3, 2004) 40
Figure 23 Stage Hydrograph at G600_T (Verification) 41
Figure 24 Stage Hydrograph at G352S_H (Verification) 42
Figure 25 Stage Hydrograph at G392S_H (Verification) 43
Figure 26 Stage Hydrograph at G352S_T (Verification) 44
Figure 27 Stage Hydrograph at G392S_T (Verification) 45
Figure 28 Stage Hydrograph at G354C_H (Verification) 46
Figure 29 Stage Hydrograph at G393B_H (Verification) 47
Figure 30 Verification Plot for Cell 3 (3/2/2004, interior sites) 48
Figure 31 Verification Plot for Cell 5 (2/23/2004, interior sites) 49
Figure 32 Stage Hydrograph for G600_T (8/20 to 8/25/2003 verification) 50
Figure 33 Stage Hydrograph for G352S_H (8/20 to 8/25/2003 verification) 51
Figure 34 Stage Hydrograph for G352S_T (8/20 to 8/25/2003 verification) 52
Figure 35 Stage Hydrograph for G392S_T (8/20 to 8/25/2003 verification) 53
Figure 36 Stage Hydrograph for G600_T and G352S_H (calibration by using G600_T as input) 56
Figure 37 Stage Hydrograph for G352S_T and G392S_H (calibration by using G600_T as input) 57
Figure 38 Stage Hydrograph for G392S_T and G393B_H (calibration by using G600_T as input) 58
Figure 39 Stage Hydrograph for G354C_T and G354C_H (calibration by using G600_T as input) 59
Figure 40 Hydrograph for G352S_H and G352S_T (2004 Verification by using G600_T as input) 60

Figure 41 Hydrograph for G392S_T (2004 Verification by using G600_T as input) 61
Figure 42 Effect of Manning's n value on Stage at G602 and G603 TW 62
Figure 43 Effect of Cw+30% on G603 Tailwater Level (G392S_T) 63
Figure 44 Impact of Time Step Size on Stage 64
Figure 45 Water Surface Elevation in STA-6 (ft NGVD) (Normal Flow, Existing Condition) 67
Figure 46 Velocity Plot for STA-6 (ft/s) (Normal Flow, Existing Condition) 68
Figure 47 Water Surface Elevation in STA-6 (Low Flow, Existing Condition) 70
Figure 48 Velocity Plot for STA-6 (Low Flow, Existing Condition)..... 71
Figure 49 Water surface Elevations in STA-6 (Weir controlled flow assumed) (High Flow, Existing Condition)..... 73
Figure 50 Water surface Elevations in STA-6 (Culvert controlled flow assumed) (High Flow, Existing Condition) 74
Figure 51 Changes in Velocity Magnitude (ft/s) with reduced Manning's n value (-30%)..... 76
Figure 52 Changes in Water Depth (ft) with reduced Manning's n value (-30%) ... 77

STA-6 2-D Hydraulic Modeling Final Report

STA Hydraulic Modeling Contract C-15988-WO04-05 (SESS Contract)

1. Introduction

Stormwater Treatment Area 6 (STA-6) is the first component of the Everglades Construction Project (ECP) to be constructed. The ECP was mandated by the 1994 Everglades Forever Act (section 373.4592, Florida Statutes).

STA-6 is situated on lands between L-3 Borrow Canal on the west and Rotenberger Wildlife Management Area on the east. Current STA-6 Section 1 is comprised of Treatment Cells 3 and 5. STA-6 Section 1 currently provides a total effective treatment area of 870 acres to treat stormwater runoff originating from an approximately 8,900 acres of agricultural production area. The location of STA-6 is shown in Figure 1.

Two-Dimensional (2-D) hydraulic models have not been previously developed for STA-6. The current project work is an attempt to develop a new 2-D hydraulic model for STA-6 Section-1 (Cell 3 and Cell 5) with new topographic data, project features, and to perform transient and steady flow simulations of the STA-6 hydraulics. Model Calibration and Verification work under transient conditions are also addressed.

The calibrated model under transient flow conditions is later used to simulate steady flow scenarios for STA-6 for Low, Design and High Flow conditions for existing STA-6 configuration as specified in the project scope of work. The majority of present tasks are spelled out under Task 2 of the contract scope of work, precisely under Subtask 2.1: STA-6 Existing Conditions Hydraulic Modeling.

This final report (Task 2.4) summarizes major results obtained in the modeling work for the Subtask 2.1 of this project for STA-6 and comments received from the District on the draft report (Subtask 2.4) have been incorporated.

.

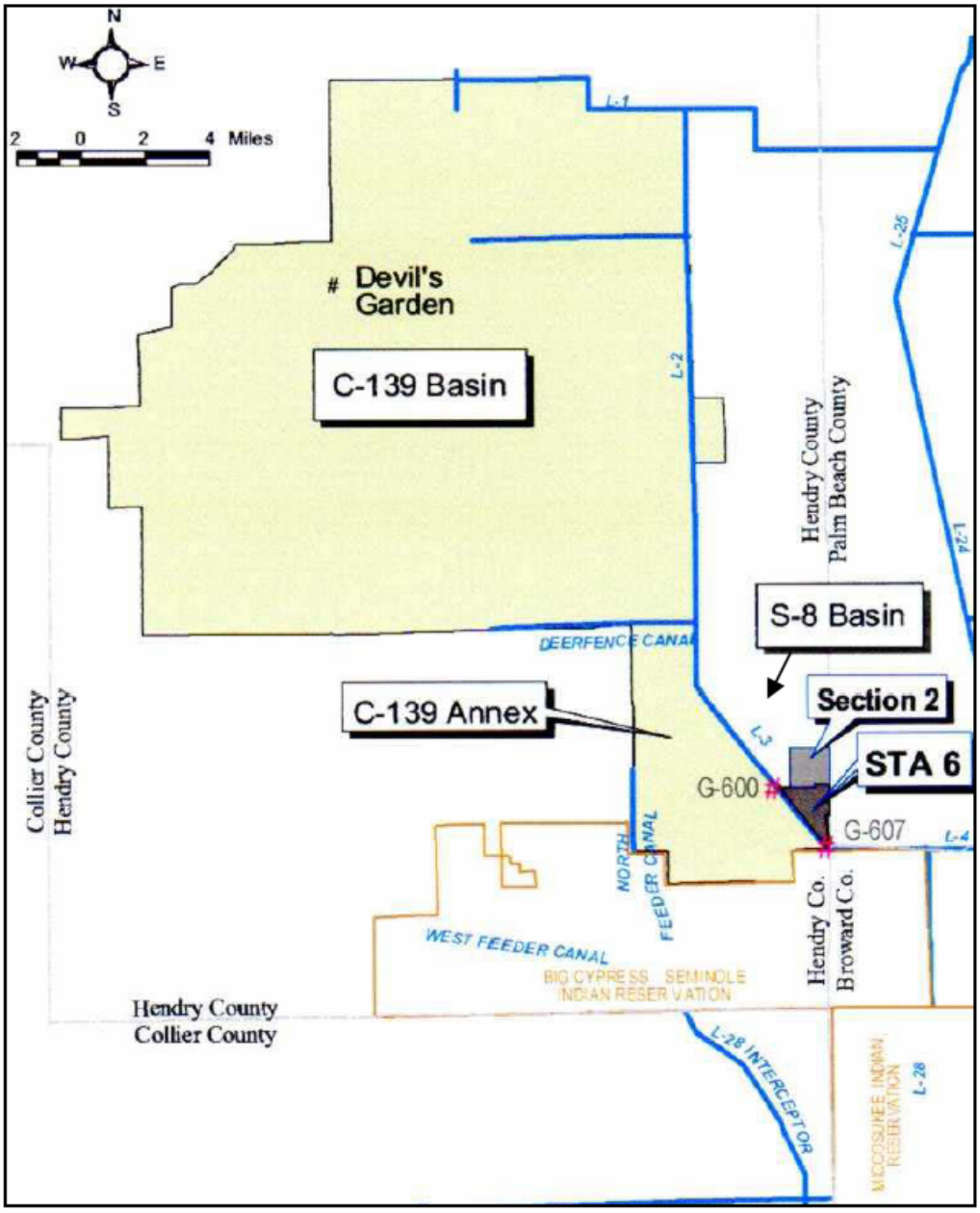


Figure 1 Location Map of STA-6

2. Model Setup

The FESWMS/FLO2DH computer program was selected by the District as the modeling tool for the current hydrodynamic modeling of STAs. The Flo2DH model engine is part of the Federal Highway Administration's Finite Element Surface-water Modeling System (FESWMS). It is a public domain model but the Graphic User Interface (GUI) through the Surface Water Modeling System (SMS) is not free. FLO2DH simulates two-dimensional depth averaged hydrodynamic flows of surface water bodies using the finite element method. Additional information about the theoretical background of the model code, its numerical method, its input and output data can be found in the User's Manual for FLO2DH 3.0 (reference [11]).

2.1 Site Description and Conceptual Model of STA-6 Section 1

STA-6 Section 1 is comprised of the Supply Canal, Inflow Weirs, Treatment Cells 3 and 5 and outflow drop-inlet structures (combined weir boxes and gated culverts) and the Discharge Canal. There is no manual control over the inflow distribution. G600 pumping station controls stage in the Supply Canal and inflow into the two treatment cells.

More specifically, the pumping station G600, Supply Canal, Inflow Weirs G601, G602 and G603, the Treatment Cells 3 and 5, outflow structures G354A-C and G393A-C and Discharge Canal comprise the current STA-6 Section 1. All these components are explicitly represented in the hydraulic model.

Control structure G604 has a permanent crest at 11.0 ft NGVD and contains stop logs to an elevation of 18 ft NGVD for normal operations. It acts as a dead end of the Supply Canal and allows all flows being directed through Treatment Cell 3 and 5.
Structures G605, G606 and G607 are not considered as part of the model domain and are located at the end of the Discharge Canal.

The perimeter levees are treated as no-flow boundary and in the hydraulic model, water cannot pass through them except through inflow and outflow structures. Seepage through the perimeter levees cannot be considered due to the limitation of FLO2DH. Semi-slip condition was applied to reflect the vertical friction on the levees.

Rainfall, evaporation and groundwater seepage are sources/sinks to the STA-6 hydraulic model. Seepage has been considered as an important factor in STA-6 water budget (References [2] and [3]). However, seepage rate depends on the head difference of both surface water and groundwater levels, and this cannot be adequately represented in FESWMS/FLO2DH. FLO2DH can only handle time-variable nodal point sources/sinks. By assuming short-term storm event flow conditions, the above sources/sinks are neglected with the exception that a rainfall

rate was applied as nodal point source term for the High Flow (Standard Project Storm) simulation.

All hydraulic structures are incorporated into the model domain.

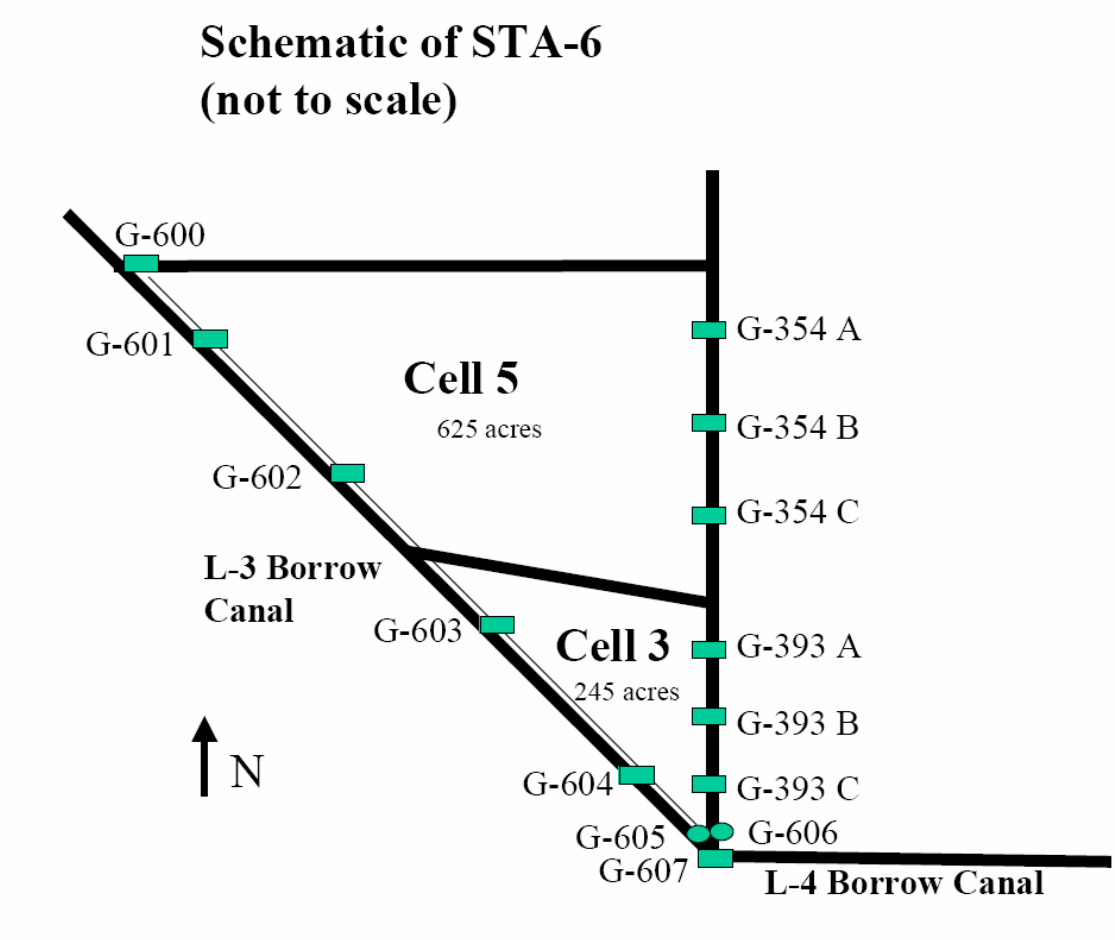


Figure 2 Schematic of Existing STA-6 (not to scale)

2.2 Topography

The Supply Canal was assigned a bottom elevation at 10.0 ft NGVD and a top width of 75 ft. The Discharge Canal was assigned a bottom elevation at 0.0 ft NGVD and a top width of 90 ft.

The ground elevation in the marsh area was interpolated from the topographic survey data provided by the District (Reference [5]).

In the marsh area, the surveyed ground elevation values in Cell 3 range from 11.51 ft NGVD to 13.15 ft NGVD. However, at a location close to its northern boundary (interior levee), there is a ridge near the northern boundary of Cell 3 with a top elevation at about 17.0 ft NGVD (Figure 3).

In Cell 5, in the marsh area, the ground elevation ranges from about 10.0 ft NGVD to 16.0 ft NGVD.

There are three transverse remnant farm ditches in Cell 5. The bottom elevation of these ditches is about 10.0 ft NGVD. The width of the ditches is about 20 ft. At the northeastern corner, there is a remnant ditch with a bottom elevation at about 4.0 ft NGVD.

The spreader and collection canals near the inflow and outflow structures have a bottom elevation of 10.0 ft NGVD.

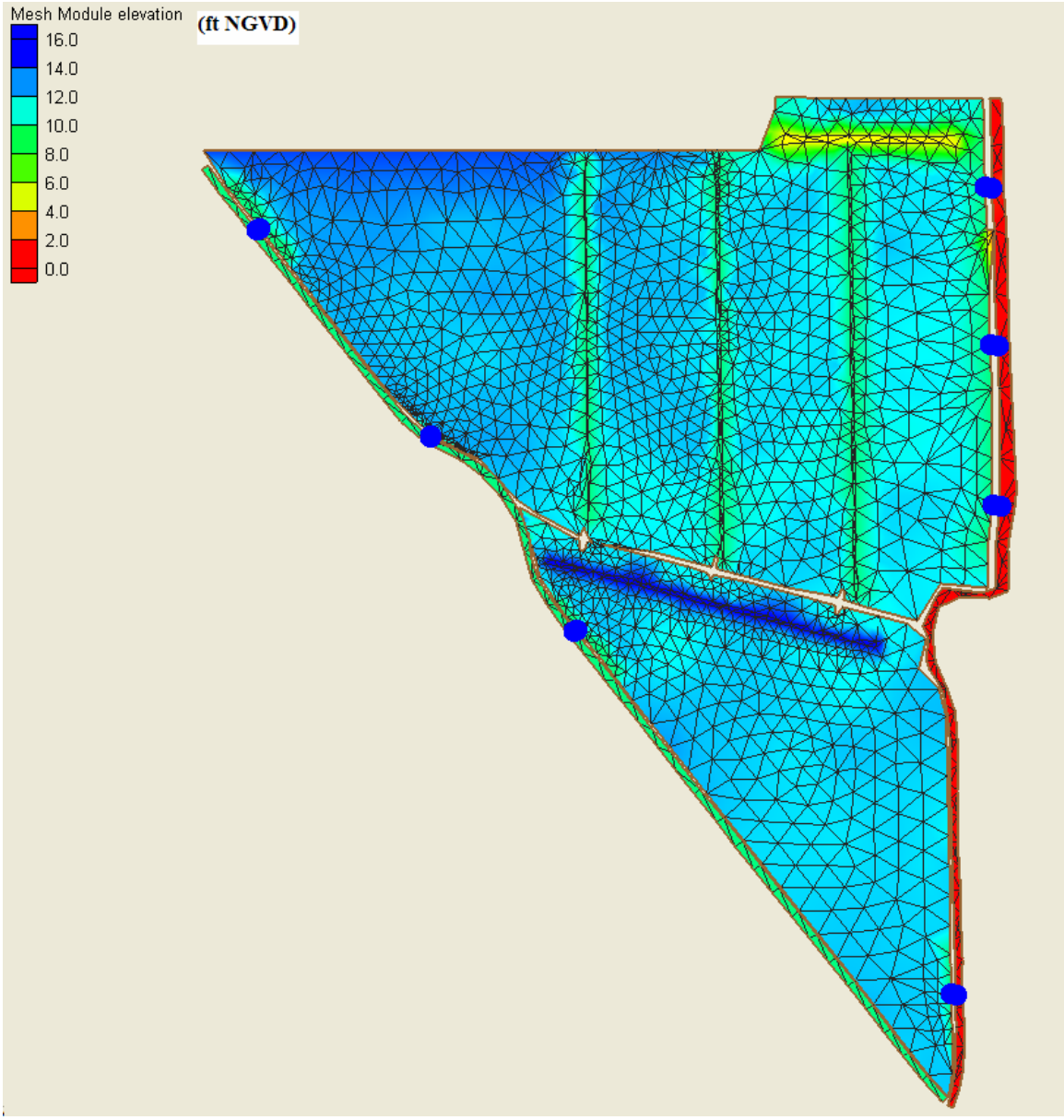


Figure 3 Ground Surface Elevation of STA-6 (Ft NGVD)

2.3 Vegetation

The December 2003 STA-6 vegetation map (Appendix I) was provided by the District and used to determine vegetative covers for hydraulic modeling.

The vegetation in Cell 3 is dominated by emergent sawgrass and some other wetland species. Cell 5 is dominated by emergent cattail marsh and open water with submerged aquatic vegetation (SAV).

For modeling purposes, the vegetative cover is classified as predominantly emergent sawgrass in Cell 3 and predominantly cattail and open water with or without SAV in Cell 5. All canals are assumed to be without vegetation (Figure 4).

Figure 4 Schematic of Vegetation Covers in STA-6

2.4 STA-6 Structures

The Inflow Weirs G601, G602 and G603 are three broad crested weirs providing inflow to STA-6 section 1, Treatment Cell 3s and 5.

Outflow structures G354A-C and G393A-C are drop-inlet structures. They are comprised of sharp-crested weirs and gated culverts.

The structure G604 is located at the southern end of the supply canal. It has a flap gate that prevents water from flowing out of the supply canal.

The Inflow Weirs are all trapezoidal broad-crested weirs. The District implemented the following empirical formula to compute the flow rate:

- **Case 1: Free flow over crest:**

$$Q_{free-crest} = C_d L H^{1.5} + 2.5 H^{2.5} \tan\theta \qquad (1)$$

Where H is the headwater depth above sill crest, L is the measured crest length and C_d is the discharge coefficient. And tan θ is defined as:

$$\tan\theta = \frac{TWD - L}{2D_{notch}} \qquad (2)$$

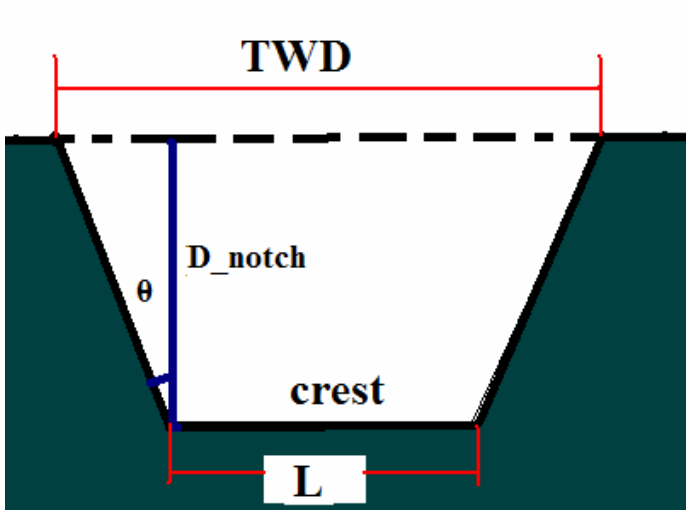


Figure 5 Illustration of Trapezoidal Weir Cross-section

- **Case 2: Submerged flow**

The submerged case is considered by a dimensionless coefficient:

$$C_s = \left[1 - \left(\frac{h}{H}\right)^{1.5}\right]^{0.385} \qquad (3)$$

$$Q_{submerged} = C_s Q_{free-crest}$$

This assumes the use of sharp-crested weir submergence formula for computing the submergence coefficient.

In FESWMS/FLO2DH, submerged flow over a weir segment, Q_w, is calculated as

$$Q_w = C_w C_s \sqrt{g} L H^{1.5} \qquad (4)$$

where C_s = weir submergence factor; C_w = dimensionless discharge coefficient for free weir flow; and L = length of the weir segment.

The submergence coefficients, C_s, are calculated as

$$C_s = (1 - \left(\tfrac{h}{H}\right)^a)^b \qquad (5)$$

where a, b = dimensionless submergence factor coefficients and their values depend on weir types. For a sharp-crested weir, a=1.5 and b=0.385; for a broad-crested weir, a=7.25 and b=0.5.

From the above descriptions, we have the following relationship for rectangular weir:

$$C_d = C_w \sqrt{g}$$
$$C_w = \frac{C_d}{\sqrt{g}} \qquad (6)$$

The calibration of weir equations for STA-6 inflow and outflow weirs with stream gauging data has only been recently completed (Reference [6, 7, 8 and 9]). The rating analyses results are not implemented into the District's FLOW Program. The historic flow data in DBHYDRO may have to be adjusted for the new calibrated weir coefficient value. We assume that the newly calibrated weir equations produce more accurate flows than those used in DBHYDRO, thus they are applied in the STA-6 hydraulic model. For modeling purpose, only the historic G600 pumping rate data can be used. Flow data at STA-6 inflow and outflow structures in DBHYDRO are computed before the calibration of the weir coefficients and are not used in model calibration and verification.

A summary of possible weir coefficient values relevant to the present modeling effort is given in Table 1.

Table 1: Weir Coefficients

Weir Name	G601		G602		G603		G354A	G354B	G354C	G393B
Section Parts	Rect.	V	Rect.	V	Rect.	V				
DBHYDRO	3.1	2.5	3.1	2.5	3.1	2.5	3.0	3.0	3.0	3.3
New calibration	2.077	1.807	2.077	1.807	1.911	1.208	2.288	2.288	2.288	2.7408
FLO2DH	0.366	N/A	0.366		0.337	N/A	0.403	0.403	0.403	0.483

Note: Rect. = rectangular section; V = V-notch (triangular section).

For Inflow Weirs G601, 602 and 603, the triangular section cannot be explicitly presented in FLO2DH. We have checked the FLO2DH model and it always uses the power factor n=1.5. This implies that only rectangular weir is correctly implemented.

From comparison of the District's trapezoidal weir equation and FLO2DH's rectangular weir equation, we may try to approximate the weir coefficient in FLO2DH as:

$$Q_{FLO2DH} = C_W C_S \sqrt{g} L H^{1.5} = Q_{District} = C_S (C_{d(R)} L H^{1.5} + C_{d(V)} H^{2.5} \tan\theta)$$

$$= C_S L H^{1.5} \sqrt{g} [C_{d(R)} + \frac{C_{d(V)}}{L} H \tan\theta] \frac{1}{\sqrt{g}}$$

$$C_w = [C_{d(R)} + C_{d(V)} (\tfrac{H}{L}) \tan\theta] * \frac{1}{\sqrt{g}} \qquad (7)$$

Where $C_{d(R)}$ and $C_{d(V)}$ are the discharge coefficients for rectangular weir and triangular section weir, respectively. It is obvious that the equivalent C_w value is not constant. It is a linear function of H.

For the new calibrated G602 and G603 coefficient values, with tan θ = 6 and 10, L=11 ft and 14 ft for G602 and G603, respectively and g=32.2 ft/s^2:

For G602, C_w= (2.077+0.9856*H)/5.6745;
For G603, C_w = (1.911+0.8629*H)/5.6745.

A constant equivalent weir coefficient Cw is used to represent the whole G601, G602 and G603 trapezoidal weirs in FLO2DH for different model runs. This is an approximation, because the weir coefficients can only be input as a constant in the model code FLO2DH.

The constant weir coefficients for G601, G602 and G603 were obtained by trying to minimize the difference between measurements (stream gauging data) and computed discharges. They were 0.622, 0.70 and 0.544, respectively for G601, G602 and G603.

For model calibration and verification, we applied the submergence coefficient for broad-crested weirs for G601, G602 and G603.

The flow rate at inflow and outflow weirs during time periods of calibration/verification will be adjusted by the newly calibrated weir coefficients. Thus, no attempts have been made to match the historic flow data stored in DBHYDRO.

There may be a water budget issue with the new weir coefficient values. Since the flow rate at G600 may have not been recalibrated, a comparison of the computed flow rate at Inflow Weirs G601, 602 and 603 with G600 may reveal problem. For example, the weir coefficient values have been reduced by 30% to 50% in the new calibration. If these coefficient values are to be applied to recalculate the historic flow data, then STA-6 water budget may be quite different from the old flow data. A detailed analysis is out of current scope of work. In the hydrodynamic model, the weir flow rate is dynamically computed and responses to G600 inflow rate, so this issue is less important, i.e., what comes in will flow out.

The trapezoidal weir equations may be calibrated to a form of rectangular weir equation with stream gauging data. As described in reference [10], the triangular section weir flow can be lumped to the rectangular weir coefficient, thus it can be compatible with the FLO2DH weir equation. But it should be noted that the lumped weir coefficient is a function of headwater level.

3. Model Calibration and Verification

3.1 Calibration/Verification Dataset

Quality historic hydraulic data for model calibration and verification has been a major concern from the beginning of this modeling project. Our previous work on the STA-2 2-D hydraulic models was limited by the scarcity of measured stage or velocity data (Reference [15]).

The linked hydrodynamic model built for STA-6 under this current effort, uses several stage monitoring sites as interior locations. Both continuous stage stations and water quality collection sites were used for model calibration and verification target sites.

1. STA-6 continuous stage historic data

Since both the supply canal and discharge canal are incorporated into the model domain, existing stage recording locations are within the model domain. Historic stage data at these sites can be used for model calibration and verification.

The following locations as listed in Table 2 are used for model calibration and verification.

Table 2: Stage Recording Locations at STA-6

Stations	Description
G600_T	Tailwater elevation for the G600 pump station
G352S_H	Headwater elevation for the Inflow Weirs G601 and G602
G352S_T	Tailwater elevation for the Inflow Weirs G601 and G602
G392S_H	Headwater elevation for the Inflow Weir G603
G392S_T	Tailwater elevation for the Inflow Weirs G603
G354C_H	Headwater elevation for the outflow structures G354A-C
G354C_T	Tailwater elevation for the outflow structures G354A-C
G393B_H	Headwater elevation for the outflow structure G393A-C
G393B_T	Tailwater elevation for the outflow structure G393A-C

2. STA-6 measured water depth data (Water Quality Data sites)

Water depth data collected for water quality monitoring purpose were available for a few limited dates at interior locations of Cell 3 and 5. The spatial distribution of the water quality sites is quite uniform over the treatment cells (Fig. 6). This would have been a very good data set for calibration/verification if the exact bottom elevation at these sites had been surveyed.

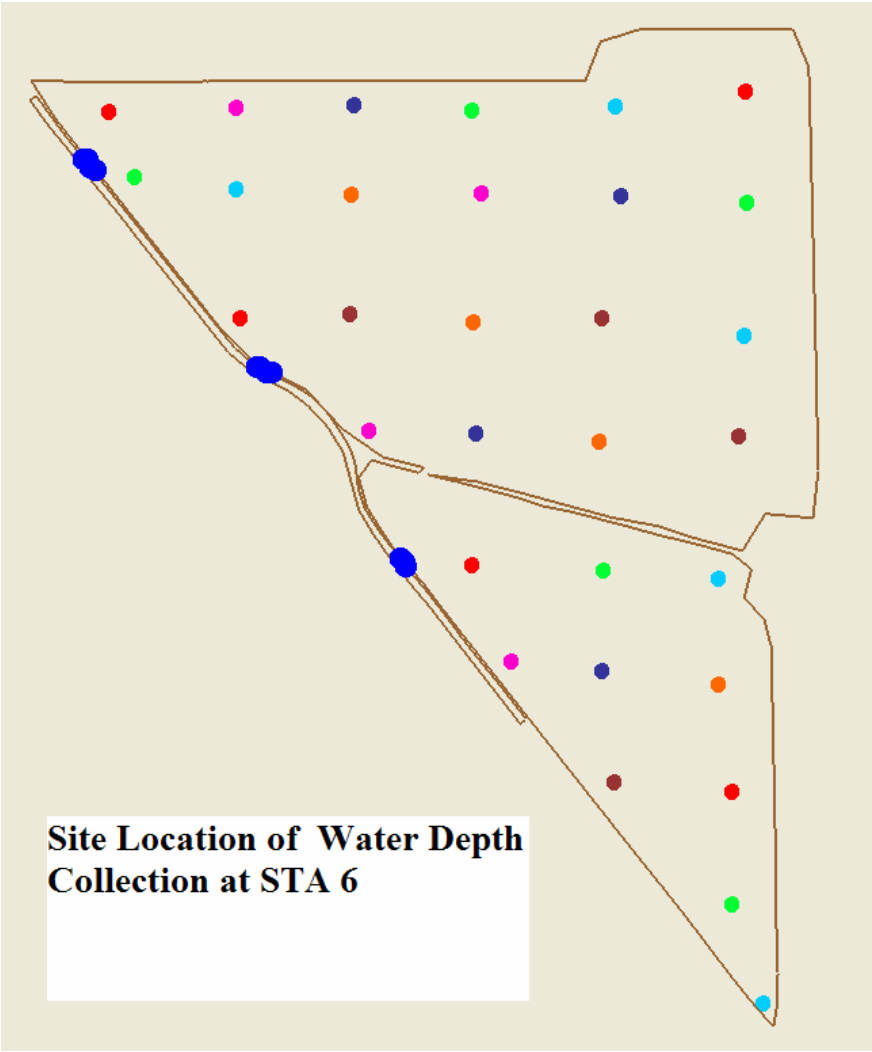


Figure 6 Site Location of Water Depth Collection in STA-6

Although the water depth data are measured, the bottom elevation values at these sites are interpolated from the topographic survey. These data are then combined to provide the (observed) water surface elevation data needed for calibration/verification. Many of the water depth data that were collected are not within reasonable ranges when comparing the corresponding water surface elevations with upstream and downstream level in the same time period (Figures 7 and 8).

Table 3 lists the water depth data collected in Cell 5 in August, 2003.

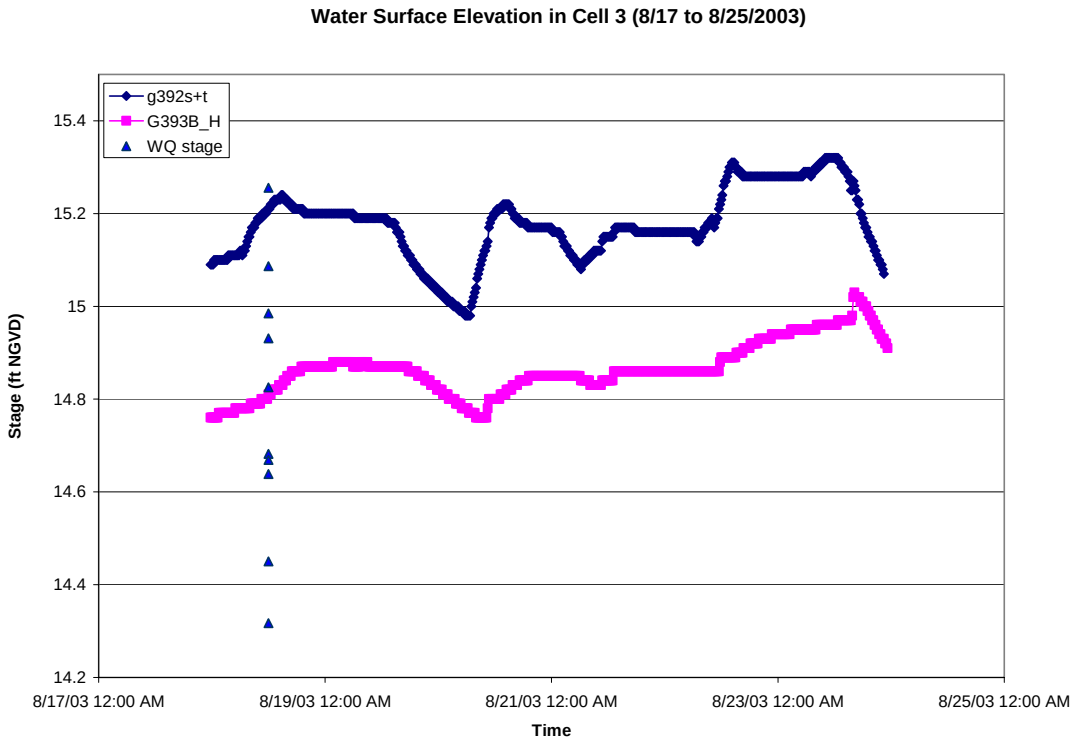
Table 3: Water Depth Data Collected in Cell 5 (2003 Data)

Site	Date	Time	Lat	Long	Water Depth (cm)	Water Depth (ft)
C5NA	12-Aug	1200	26.35892	80.90266	46	1.51
C5NB	12-Aug	1230	26.359	80.89876	52	1.71

C5NC	12-Aug	1250	26.35911	80.89515	46	1.51
C5ND	12-Aug	1315	26.35894	80.89155	42	1.38
C5SMIDC	12-Aug	1155	26.35334	80.89531	24	0.79
C5SMIDD	12-Aug	1215	26.35313	80.89149	51	1.67
C5SMIDE	12-Aug	1301	26.35319	80.88757	56	1.84
C5NMIDB	13-Aug	1330	26.35679	80.8988	45	1.48
C5NMIDC	13-Aug	1300	26.35666	80.89527	49	1.61
C5NMIDD	13-Aug	1245	26.35664	80.89131	55	1.80
C5NE	13-Aug	1110	26.35907	80.88715	38	1.25
C5NF	13-Aug	1140	26.35943	80.88316	35	1.15
C5NMIDE	13-Aug	1230	26.35658	80.887	57	1.87
C5NMIDF	13-Aug	1200	26.35641	80.88316	72	2.36
C5SMIDB	13-Aug	1315	26.35321	80.89867	80	2.62
C5SMIDF	13-Aug	1110	26.35275	80.88323	58	1.90
C5SC	13-Aug	1245	26.35012	80.89474	58	1.90
C5SD	13-Aug	1220	26.35006	80.89145	89	2.92
C5SE	13-Aug	1150	26.34983	80.8877	60	1.97
C5SF	13-Aug	1130	26.34998	80.8834	60	1.97
C5NMIDA	18-Aug	1130	26.35711	80.90187	45	1.48

There are 21 water quality sampling sites having water depth data in Cell 5. The water depth data provided were measured on 8/12/2003 - 8/13/2003 and 2/23/2004 - 2/24/2004.

For Cell 3, there are only 10 water depth collection sites. The water depth data provided were measured on 8/18/2003, 8/19/2003 and 3/2/2004. There were no time stamps for the 2/24/2004 and 3/2/2004 data. The collection time for the data collected on these dates was assumed to be 12 pm.



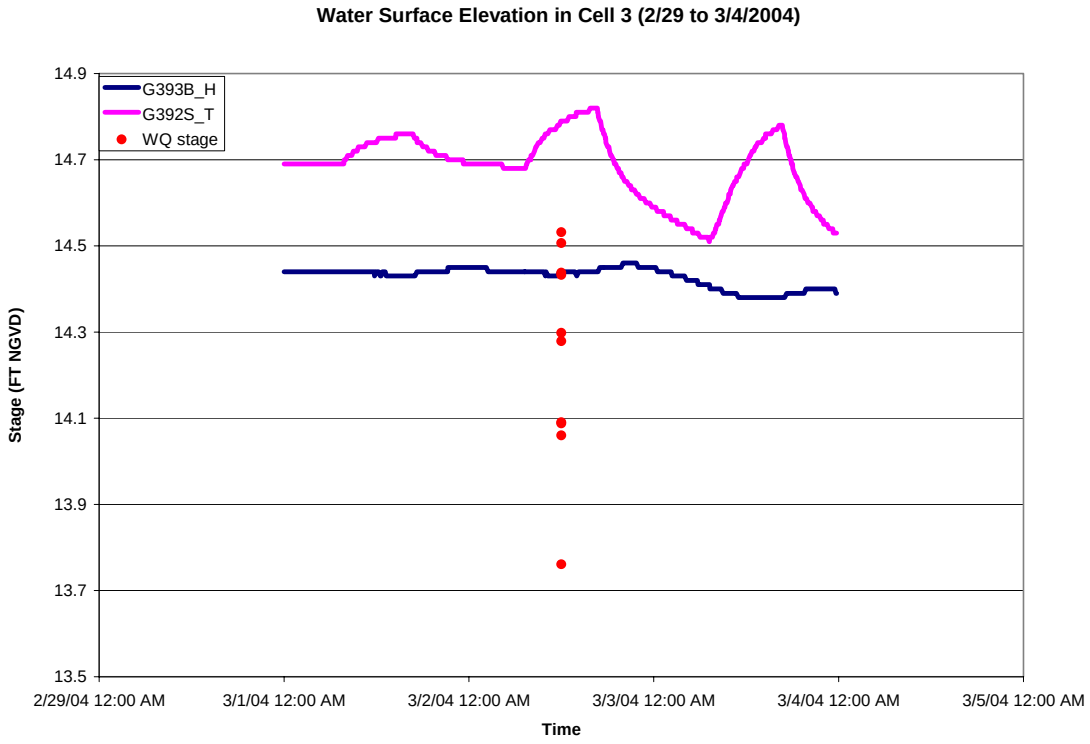
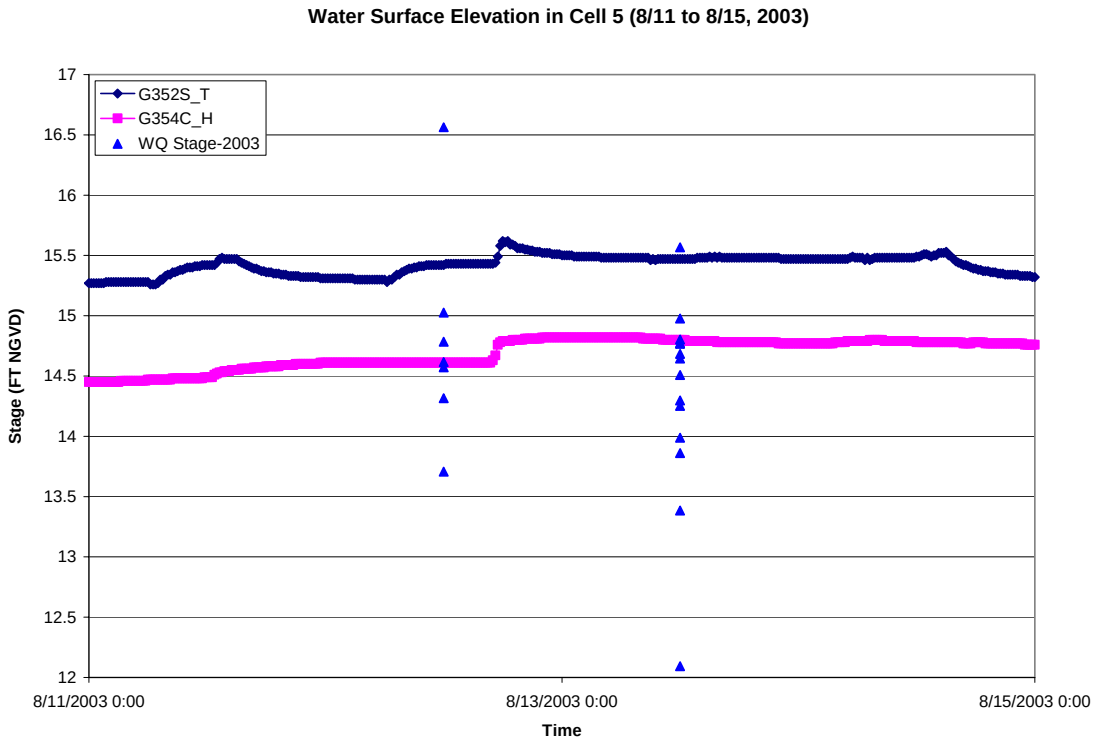


Figure 7 Water Surface Elevation at G392S_T and G393B_H (FT NGVD)



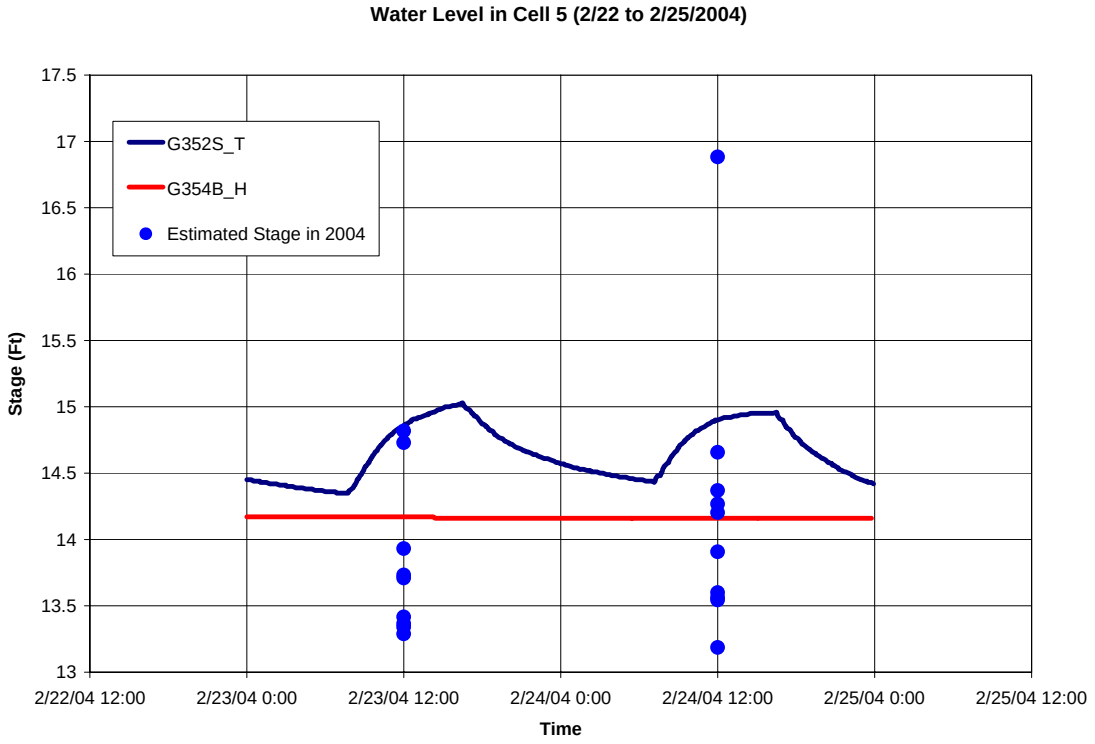


Figure 8 Water Surface Elevation at G352S_T and G354C_H (FT NGVD)

The discrepancies in the observed water quality depth data can be explained by the fact that exact ground elevations at these sites are not available. This also means that these observed data are not always reliable for model calibration/verification, since the error in the bottom elevation can be possibly higher than 0.25 ft.

The possible errors in the derived water surface elevation data are a combination of the error in the water depth measurement and the bottom elevation interpolation:

$$H = h + Z0$$
$$\text{Error}(H) = \text{Error}_{h_meas} + \text{Error}_{Z0_interp}$$

In the above equation, H is the water surface elevation, h is water depth and Z0 is ground elevation.

Since the land surface elevations in the treatment cells are not uniform and the water quality data collection sites are not always close to topographic survey points, interpolation error can be significant. It is thought that the existing water depth data collected for water quality monitoring purposes are not always reliable for model calibration and verification, and therefore, the matching between these derived water surface elevations and their corresponding computed values are not always an accurate indicator of model performance.

The following description of the water depth measurement method for STA monitoring (provided by the District's STA water quality monitoring staff) also confirms the uncertainty in local ground elevations:

- Locate sample site by use of GPS navigation system.
- Secure boat with anchor at specified sample location.
- Take calibrated PVC pole (1 meter +) and get clear of the boat trail influence.
- Be sure to carefully move aside any vegetation that would influence depth measurement.
- Push calibrated pole into water column and down to sediment until resistance is established.
- Record measurement and repeat 3 times in a square meter area.
- A total of 4 depths should be recorded per site for average depth.
- **Keep in mind the bottom of the STAs are very uneven. Depths may very by 30-80 cm per site. There are many old canals that have not been filled in properly as well as terrain elevation inconsistencies.**

It is recommended that better quality data such as measured interior water surface elevations and velocity be collected to refine future hydraulic models.

3.2 Calibration strategy

Because the Supply Canal and Discharge Canal are within the model domain, several continuous recording stations become interior sites. These sites provide good historic data for model calibration and verification.

The observed interior water depth data provide the spatial pattern at a specific date and time; however, there are some discrepancies in the data. When the water depth value is added to the bottom elevation, the estimated water level should be within a reasonable range. If the derived water level at a site is out of range as shown in the previous description, this data was considered abnormal and therefore was considered to be inappropriate for calibration/verification. As a result, only a small portion of the interior water depth data was found to be suitable for use in model calibration.

The calibration process is a multi-step process of adjustments and comparison. The model discretization, the model parameters and hydraulic data for inflow and outflow conditions were examined and the historic matching results were judged by a general target of ±0.25 ft for stage values and relevant statistics. Measured velocity data were not available for current calibration.

3.3 Calibration Results

The time period selected for model calibration was between 8/10/2003 and 8/20/2003. The selection of this time period was based on the fact that we have some observed interior water depth data during this time period. The target sites with continuous observed stage data are those headwater/tailwater locations at G602 and G603, G354_C and G393B.

The 10-day time period used for calibration includes a 2-day warm-up (8/10, 8/11) to dampen the effect of initial condition. The interior observed water depth data were collected on 8/12, 8/13, 8/18 and 8/19 between 11:00 am and 3:00 pm.

The following continuous stage sites are used as calibration target:

- Water level in the Supply Canal is represented by G600_T, G352S_H and G392S_H.
- Water level in the Treatment Cell 3 is represented by G392S_T and G393B_H.
- Water level in the Treatment Cell 5 is represented by G352S_T and G354C_H.
- Water level in the Discharge Canal is represented by G354C_T and G398B_T.

The calibration process is a trial and error process. With G600_P as inflow boundary condition and the stage at G393B_T at the Discharge Canal as outflow boundary, the

STA-6 linked hydraulic model was run with a time step of one hour for 240 hours. The convergence criteria for both water depth and unit discharge are 0.01.

The boundary condition values were from the breakpoint data in the District's corporate database DBHYDRO (15 minute intervals).

The Manning's n values and the weir coefficients are two of the most sensitive model parameters in model calibration.

We carefully estimated the approximate weir coefficients to be used for G601, G602 and G603. Weir coefficients were also used in the outflow structures. The weir controlled flow condition was satisfied in the outflow structure computation during the selected period of record. The newly calibrated weir coefficients were used.

The water levels in the treatment cells are also sensitive to the Manning's n values. It was found that the water level at the Inflow Weir G602, which represents the water level in the western part of Cell 5, is hard to maintain in the model. As a result, the Manning' n value for the emergent cattail dominant vegetation was increased to n=1.5 ~ 2.0 to match the historic stage data. For sawgrass dominant vegetation in Cell 3, a Manning's n value of 1.2 ~1.5 was applied to match historic stage data. With other model parameters being unchanged, smaller Manning's n values will result in poorer matches to the historic stage data.

The statistics for model calibration results were summarized in Table 4. The R^2 Value is obtained by linear regression analysis between computed and measured stages. An R^2 value of 1.0 indicates perfect match between two time series data.

Table 4. R^2 Value and Mean Absolute Difference between Measured and Computed Stage (Calibration: 8/12/2003 to 8/20/2003)

Station	R^2	Mean Absolute Difference (ft)
G600_T	0.93	0.08
G352S_H	0.95	0.08
G352S_T	0.95	0.09
G392S_H	0.95	0.06
G392S_T	0.93	0.11
G393B_H	0.90	0.04
G393B_T	As boundary condition	
G354C_H	0.84	0.05
G354C_T	0.97	0.06

Most of the time, the differences of measured and computed stage values are within ±0.1 ft. Thus, we can conclude that the calibration results are satisfactory, considering the fact that there are so many uncertainties in inflow, weir coefficients and the Manning's n values.

The stage hydrograph plots (Fig. 9 to 17) show the matching between historic time series and computed stage. The matching at some interior sites was also plotted in Figures 18, 19, 20 and 21. The calibration target is also ±0.25 ft in these plots. They can only be considered a qualitative indicator of model performance due to previously mentioned quality issues.

Because the final parameter set was successfully applied in model verification with satisfactory results, it is proposed that no further adjustment of the model parameters is needed.

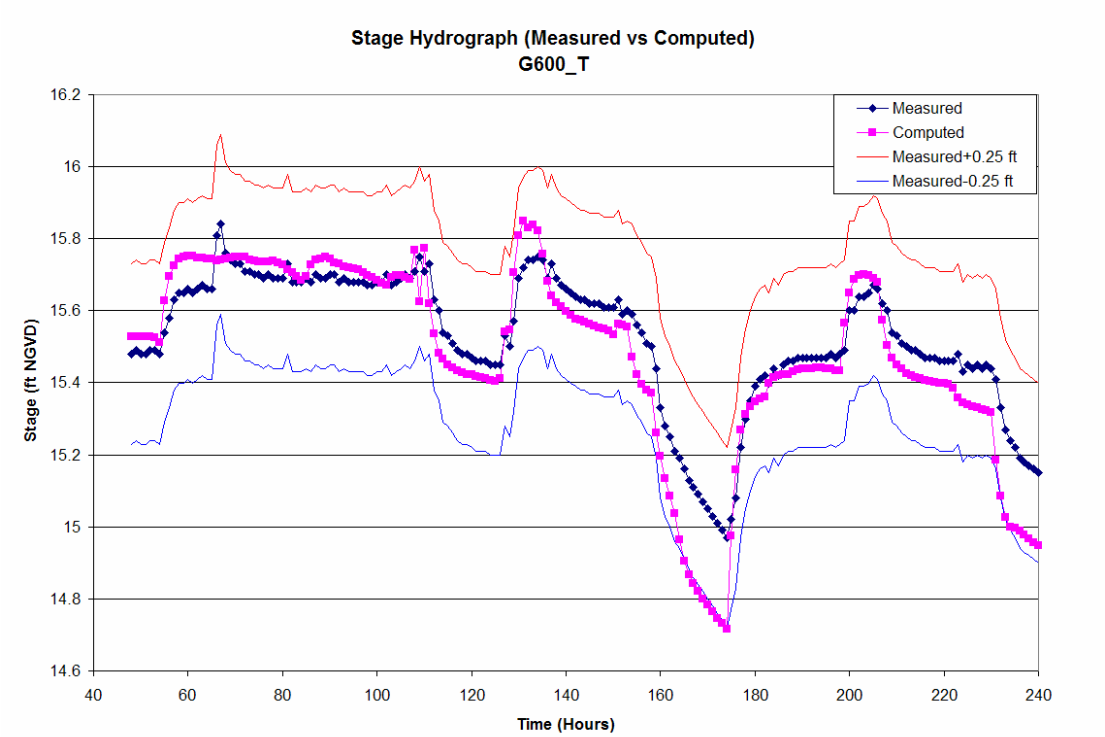


Figure 9 Stage Hydrograph at G600_T (Calibration)

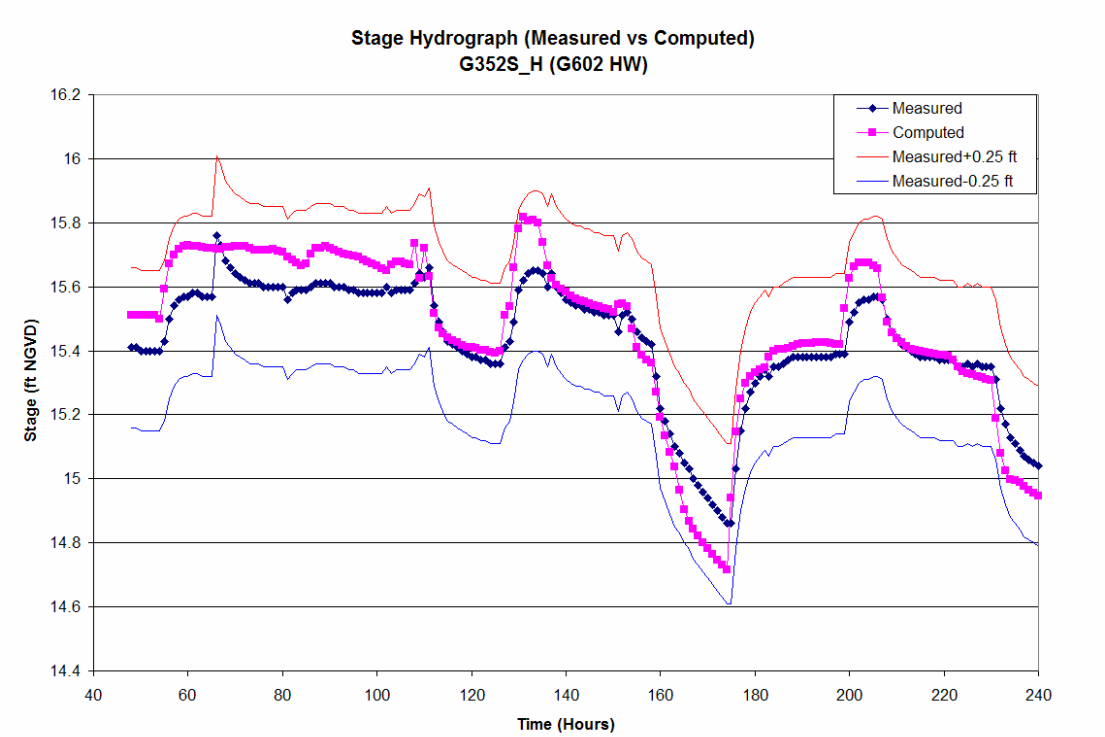


Figure 10 Stage Hydrograph at G352S_H (Calibration)

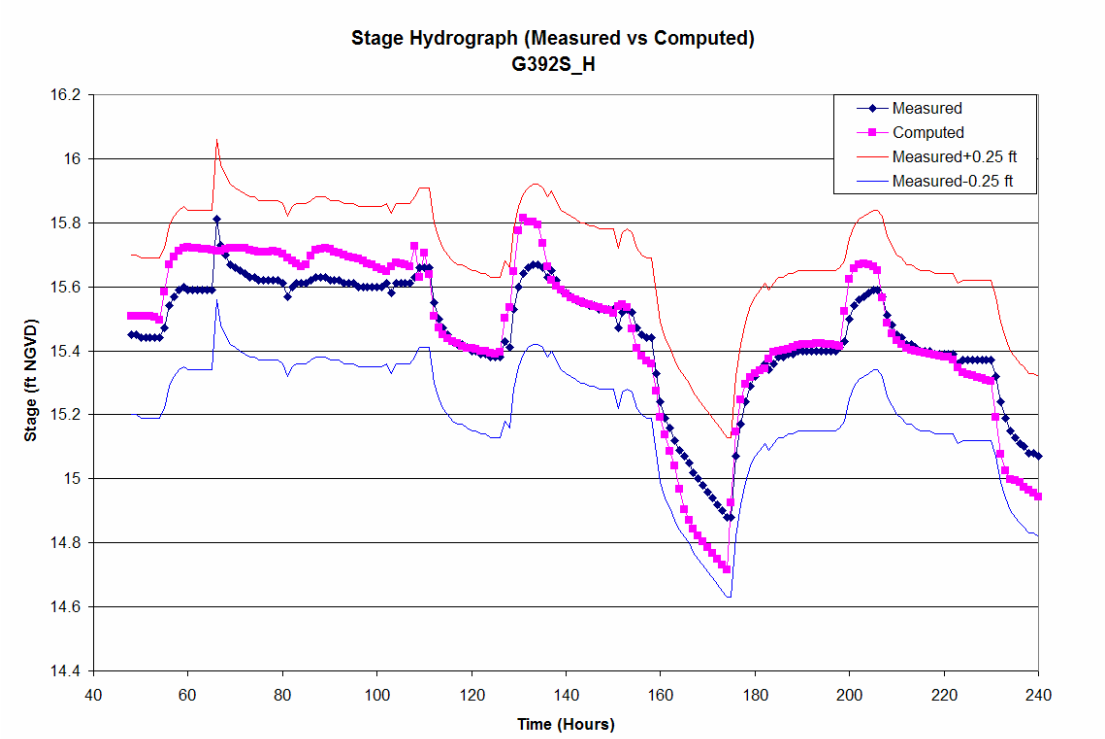


Figure 11 Stage Hydrograph at G392S_H (Calibration)

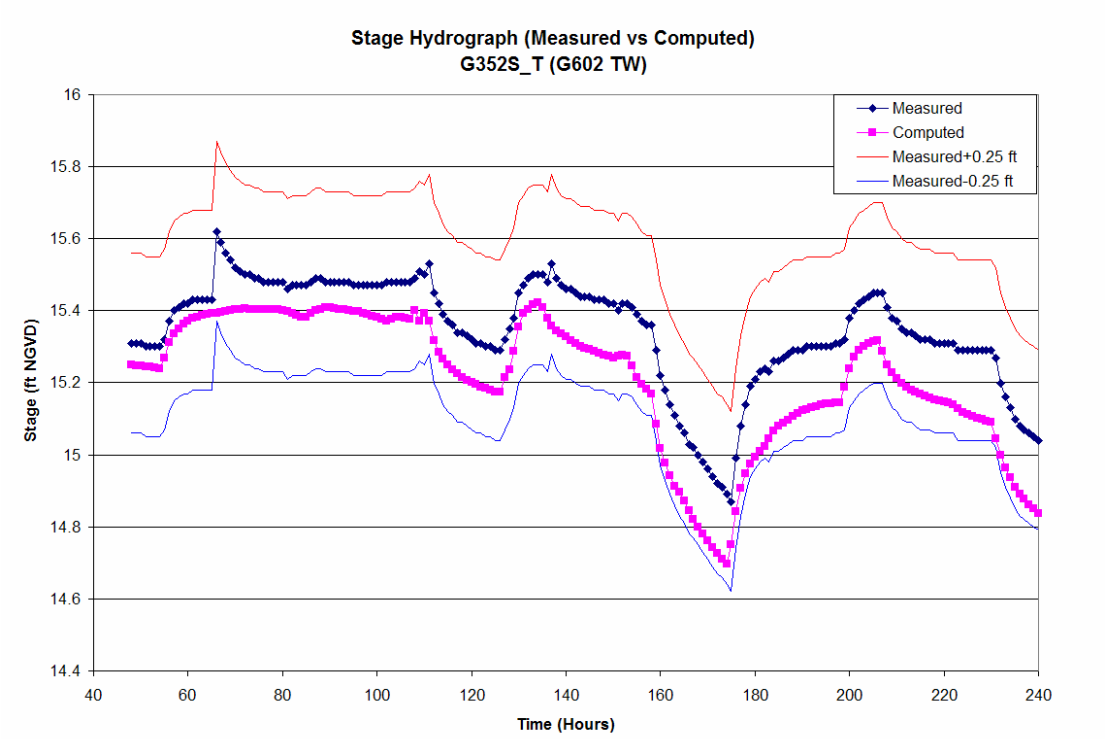


Figure 12 Stage Hydrograph at G352S_T (Calibration)

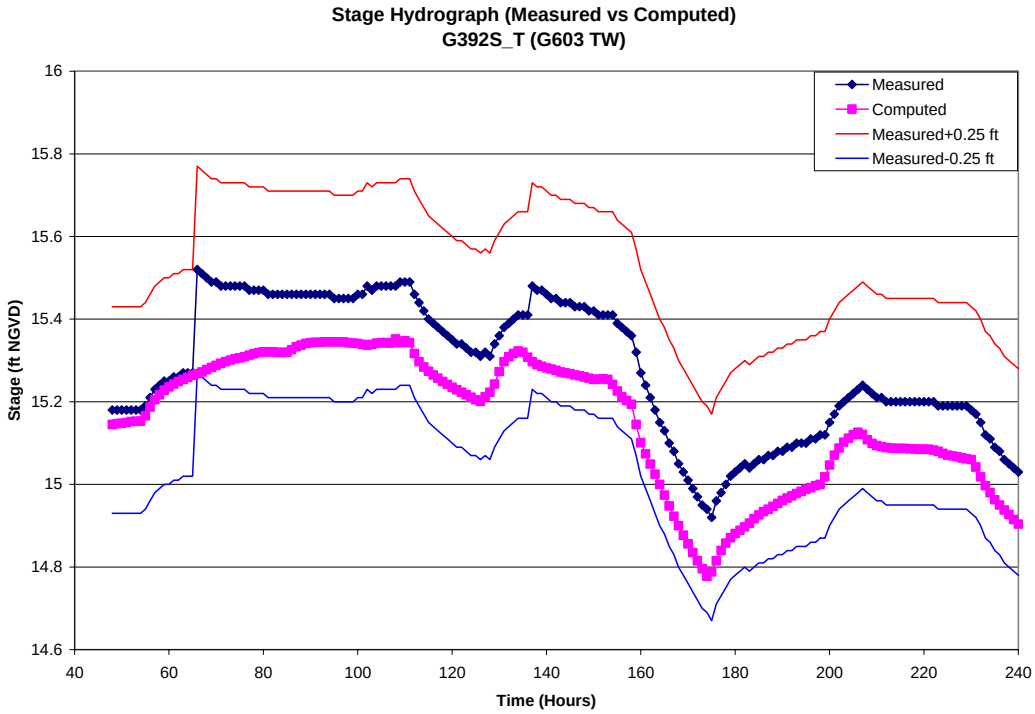


Figure 13 Stage Hydrograph at G392S_T (Calibration)

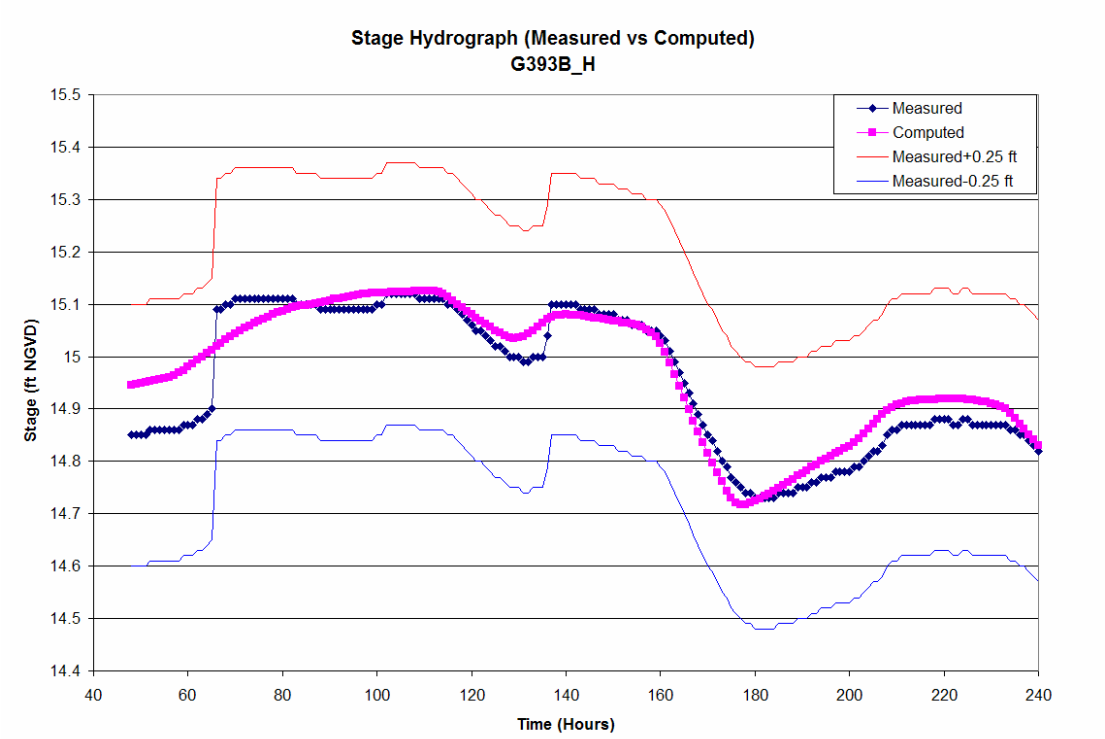


Figure 14 Stage Hydrograph at G393B_H (Calibration)

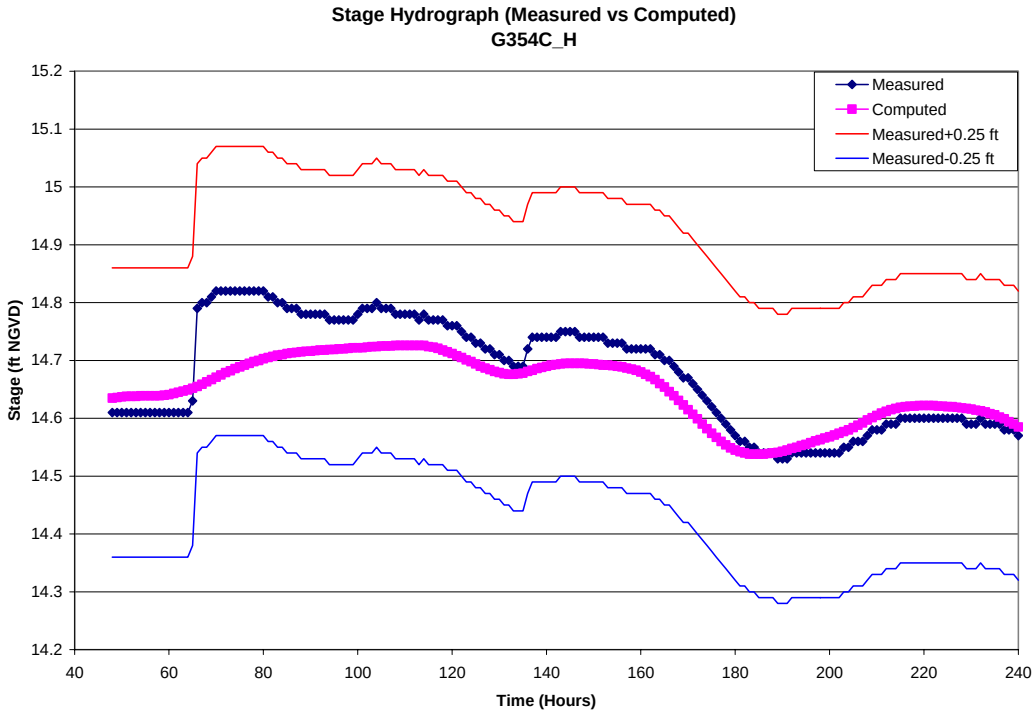


Figure 15 Stage Hydrograph at G354C_H (Calibration)

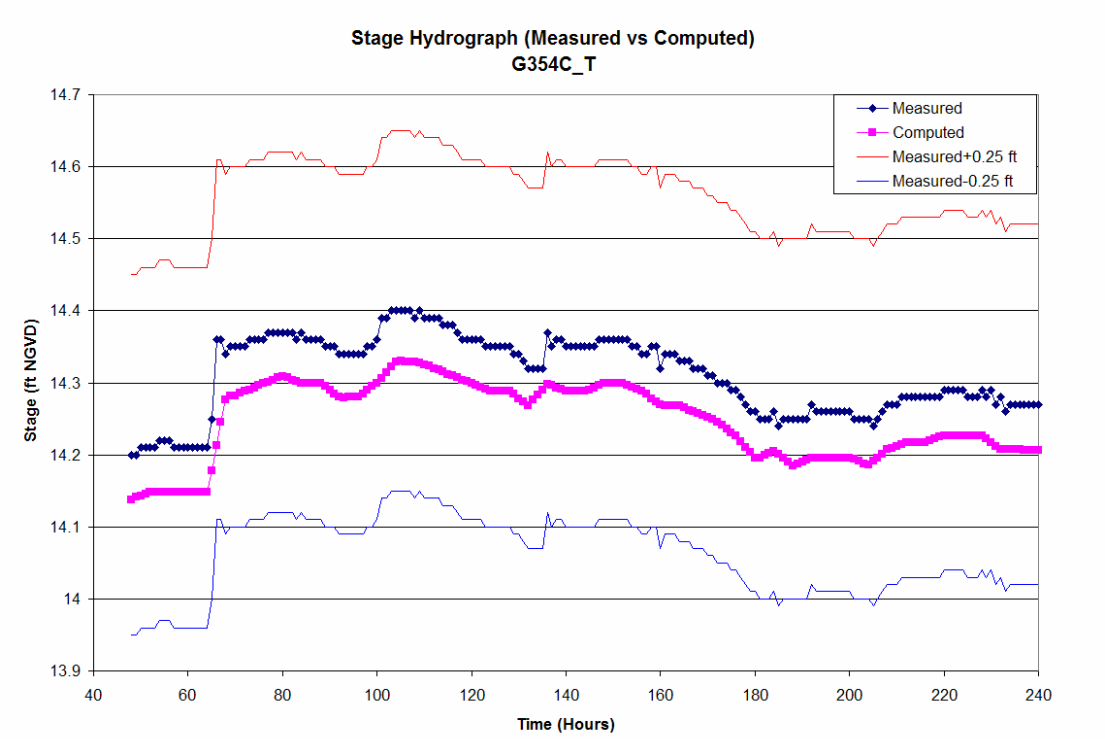


Figure 16 Stage Hydrograph at G354C_T (Calibration)

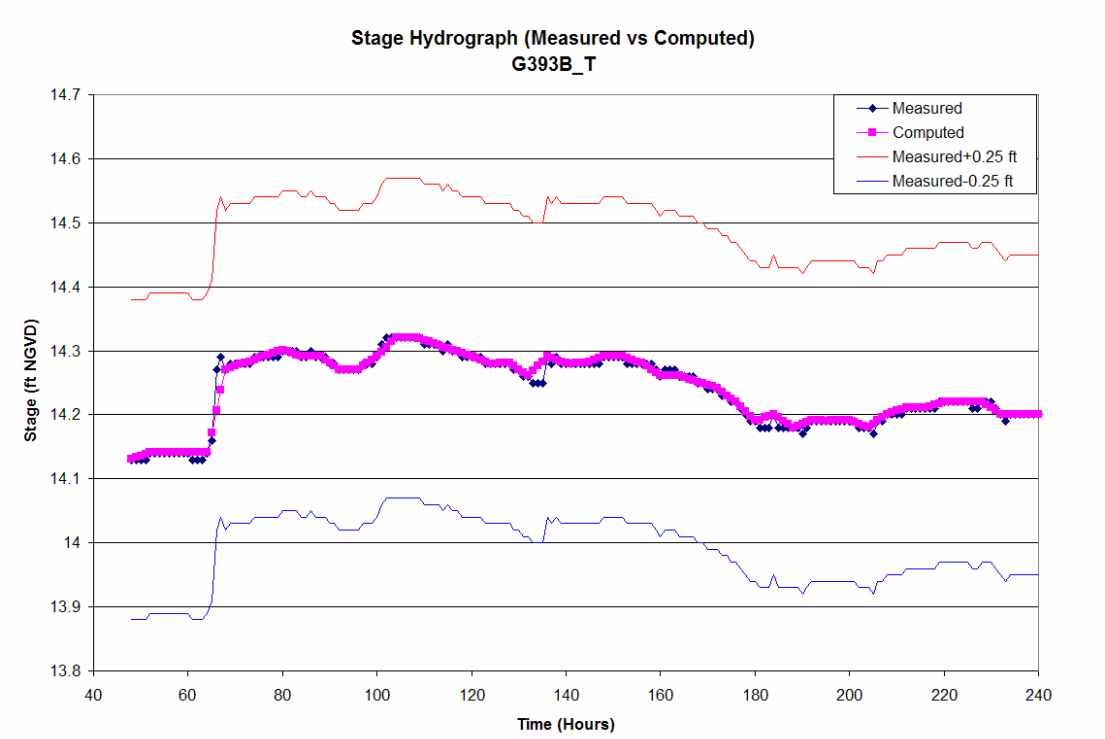


Figure 17 Stage Hydrograph at G393B_T (Calibration)

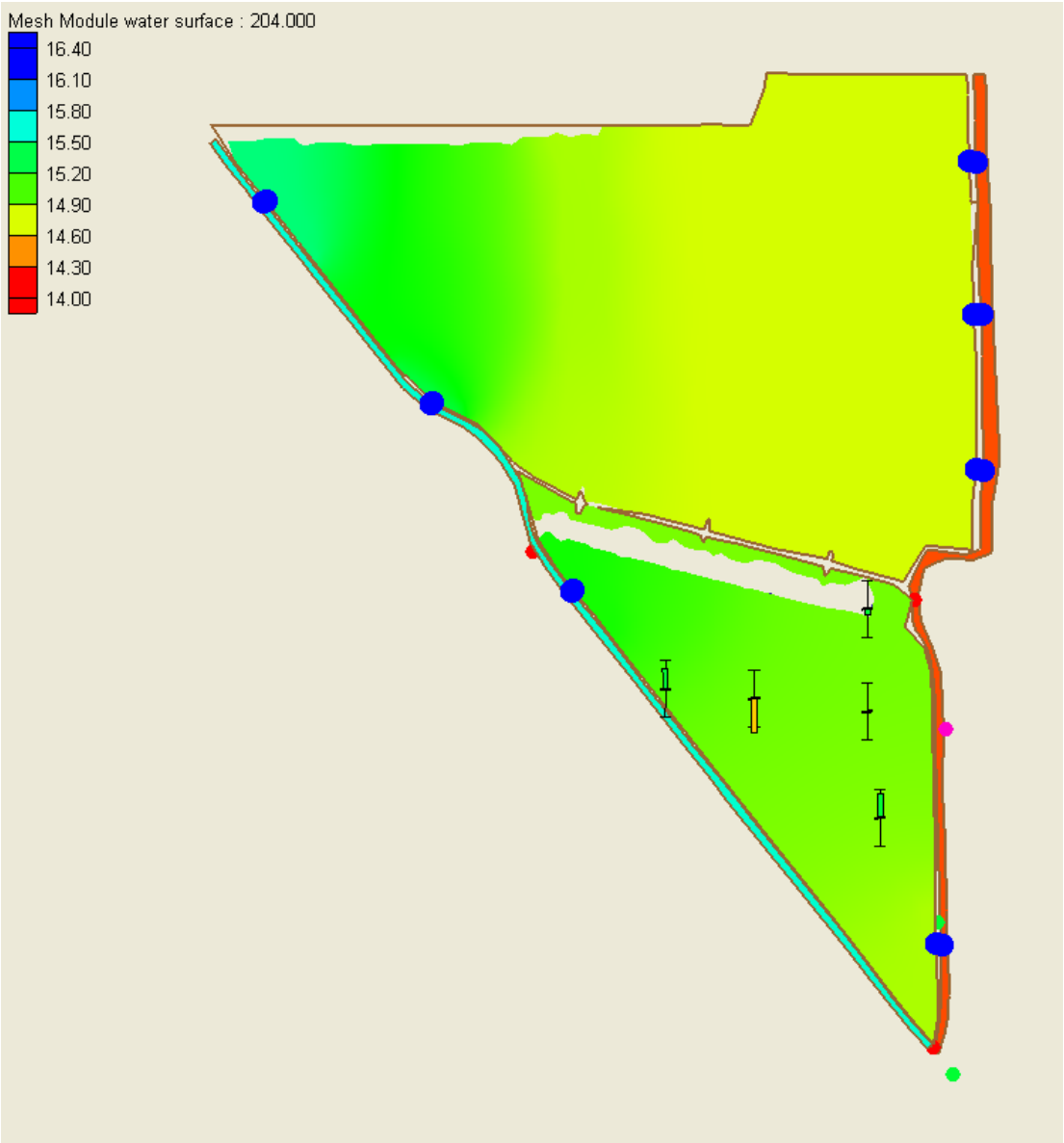


Figure 18 Calibration Plot for Cell 3 (8/18/2003, interior sites)

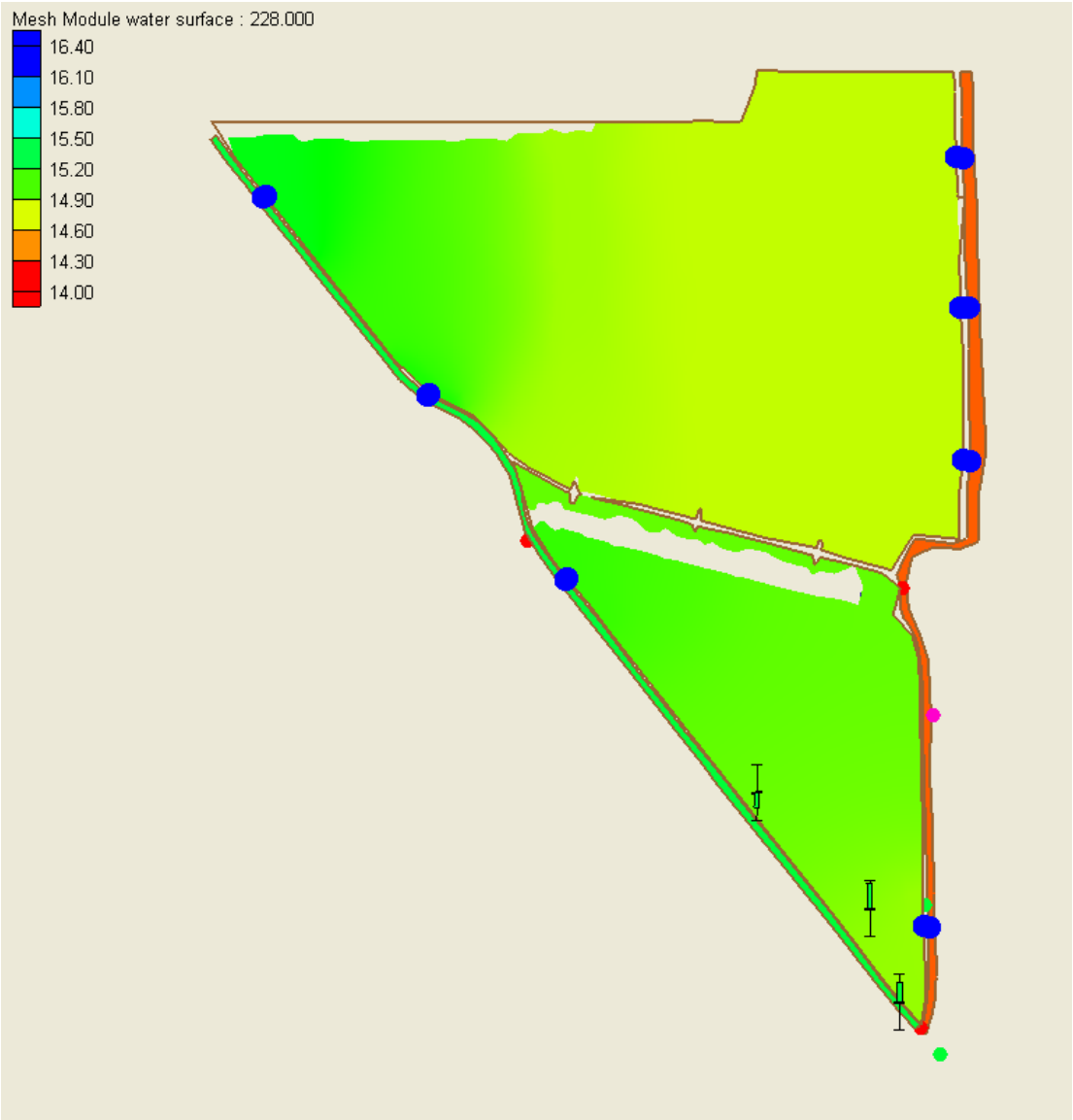


Figure 19 Calibration Plot for Cell 3 (8/19/2003 12 pm, interior sites)

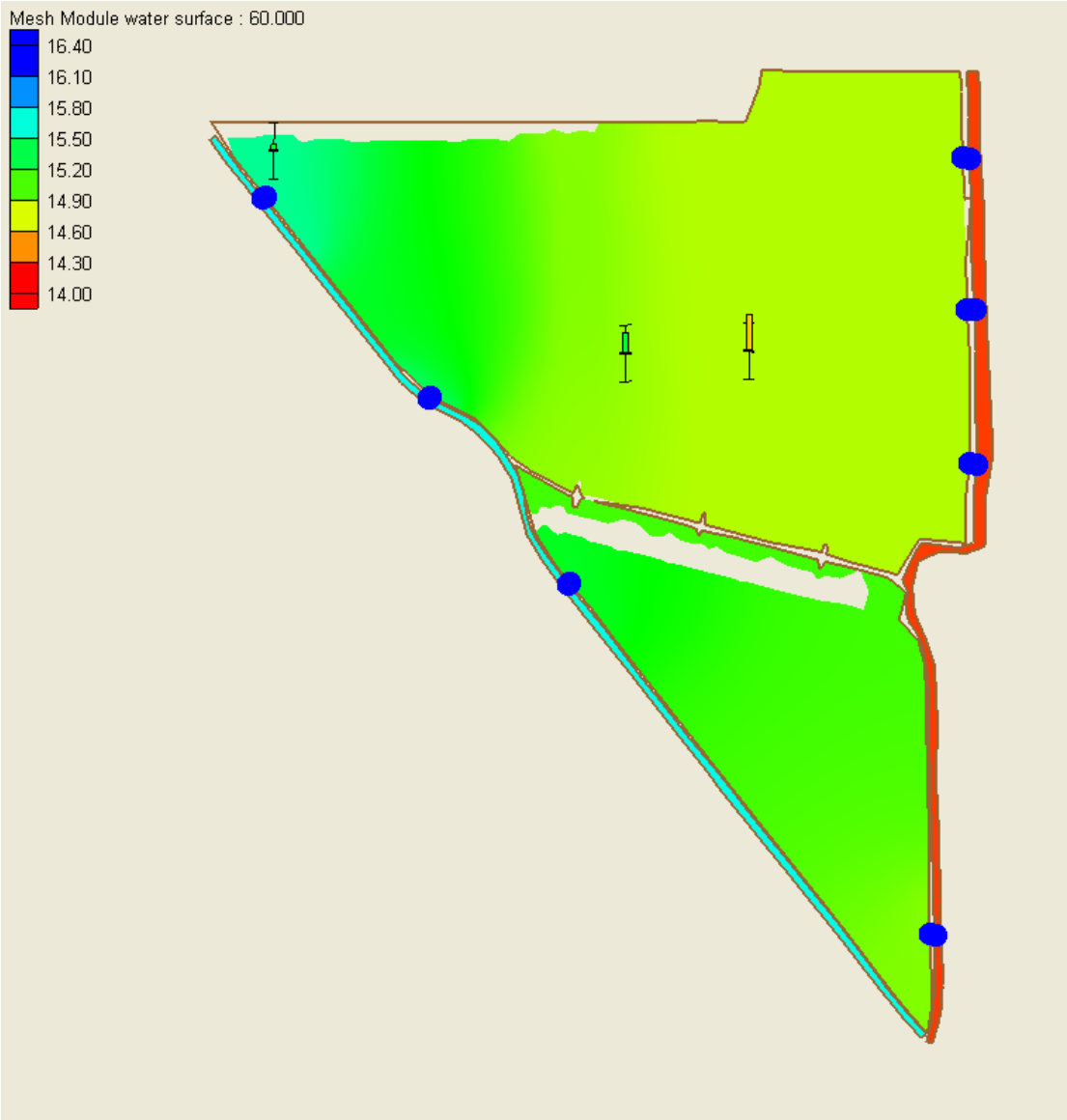


Figure 20 Calibration Plot for Cell 5 (8/12/2003, interior sites)

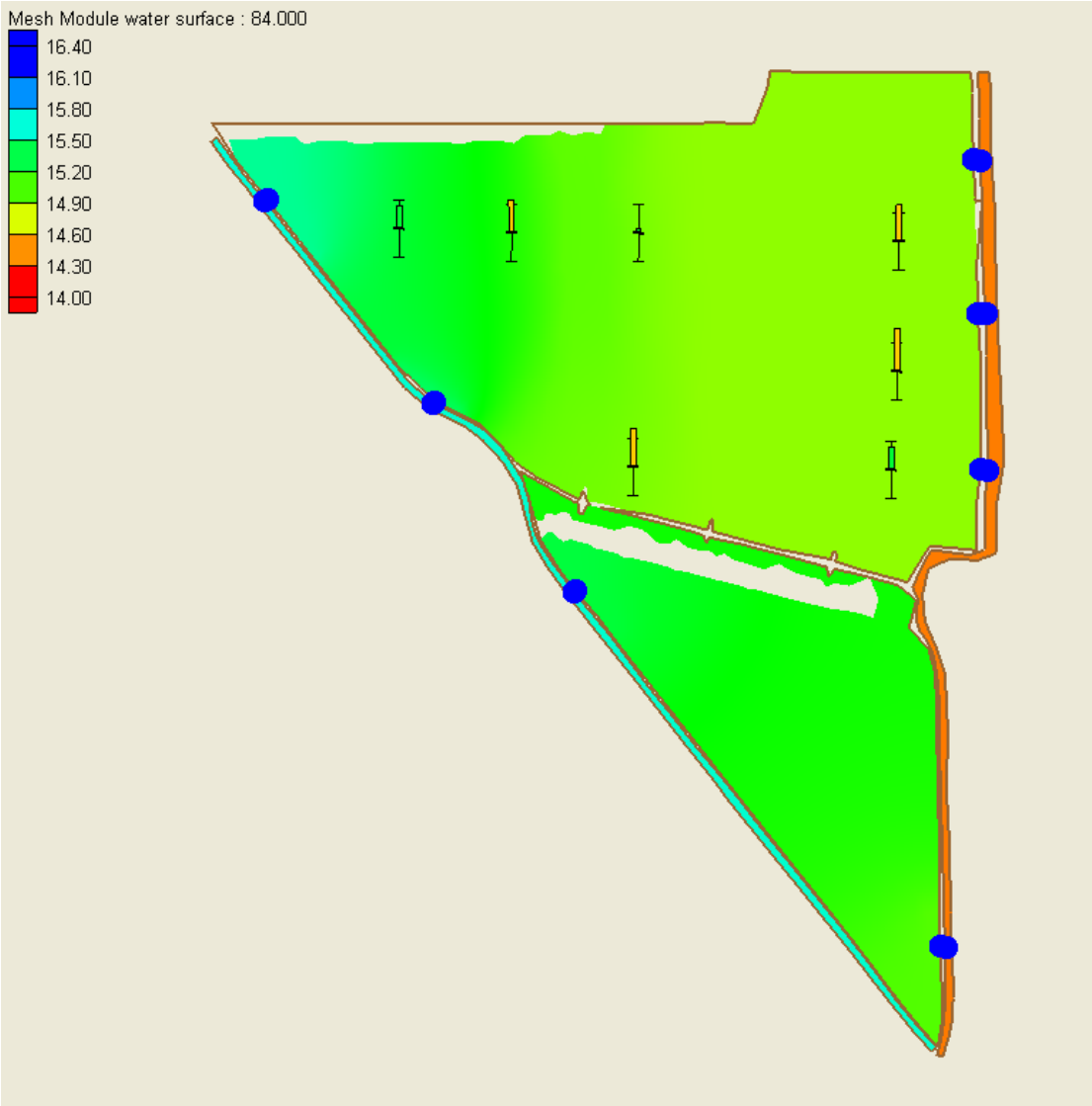


Figure 21 Calibration Plot for Cell 5 (8/13/2003, interior sites)

3.4 Verification (Validation) Results

Independent observed data are used to verify the capability of the model to predict hydraulic behavior without adjusting the calibrated model parameters.

The time period 2/22/2004 0:00 - 3/3/2004 0:00 was chosen for model verification. All parameters applied in the calibration were kept unchanged during verification. During this time period in the dry season, the tailwater level of all outflow structures was below weir crest elevation, thus free weir flow occurred all the time. After some sensitivity analyses, it was decided to apply specified stage boundary conditions at the location of outflow structures. Thus, the Discharge Canal was not included. The first 36 hours of the verification run was warm-up period to dampen the initial condition effect on the solution and a one-hour time step was applied through out the whole 240-hour simulation.

The uncertainty in inflow during the dry season is caused by the fact that unknown quantity of water might have been diverted from the supply canal for sugarcane irrigation. In order to verify the model prediction accuracy in wet season, the time period 8/20/2003 0:00- 8/25/2004 0:00 was also chosen for model verification.

- **2004 Verification run results**

The statistics shown in Table 5 demonstrates that the model verification is satisfactory. During most of the time period, the difference between measured and computed stage was within ± 0.1 ft.

Table 5: R^2 Value and Mean Absolute Difference between Measured and Computed Stage (Verification: 2/22/2004 to 3/3/2004)

Station	R^2	Mean Absolute Difference (ft)
G600_T	0.92	0.09
G352S_H	0.92	0.08
G352S_T	0.94	0.11
G392S_H	0.93	0.07
G392S_T	0.95	0.04
G393B_H	as boundary condition	
G354C_H	as boundary condition	

- **Supply Canal:**

The historic flow and stage data at G600_T is plotted in Figure 22. G600_T is at the inflow boundary location and the water level closely follows the trend in G600_P pumping rate. The pumping rate was computed by the District with a rating equation and stored in DBHYDRO.

The computed stage values and the historic measured stage data matched quite well when there were pumping activities. When G600 stopped pumping, the computed stage level quickly declined (Figure 23). Similar trends were also found in G352S_H and G392S_H (Figures 24 and 25). On the other hand, there are times of high pumping rate where the computed stage level is consistently higher than historic stage values. Whether this can be attributed to possible irrigation water withdrawal from the Supply Canal or due to G600_P pump rating error needs further investigation.

- **Treatment Cells 3 and 5:**

Computed G603 (G392S_T) and G602 (G352S_T) tailwater levels follow similar trends as in the Supply canal (Figures 26 and 27). They matched the measured stage value with similar mean absolute difference as in model calibration. In fact, there is a better match at G392S_T than model calibration results.

- **Interior sites**

The verification model run results were also compared to the selected interior water quality sites. The results for Cell 3 (3/2/2004 12:00 pm) were satisfactory (Figure 30). As for Cell 5, very few water quality sites can be used (2/23/2004 12:00 pm) (Figure 31) and there are two sites with stage difference exceeding 0.25 ft. Since the exact land surface elevations are interpolated values, they provide less reliable verification.

- **2003 Verification run results**

The wet season (8/20 to 8/25/2003) verification simulation was selected to check whether the calibrated model can also perform well under wet season flow condition. The model run results are partially plotted in Figure 32 to 35. It can be seen that similar matching between measured and computed stage were obtained. Thus the calibrated parameters were further verified.

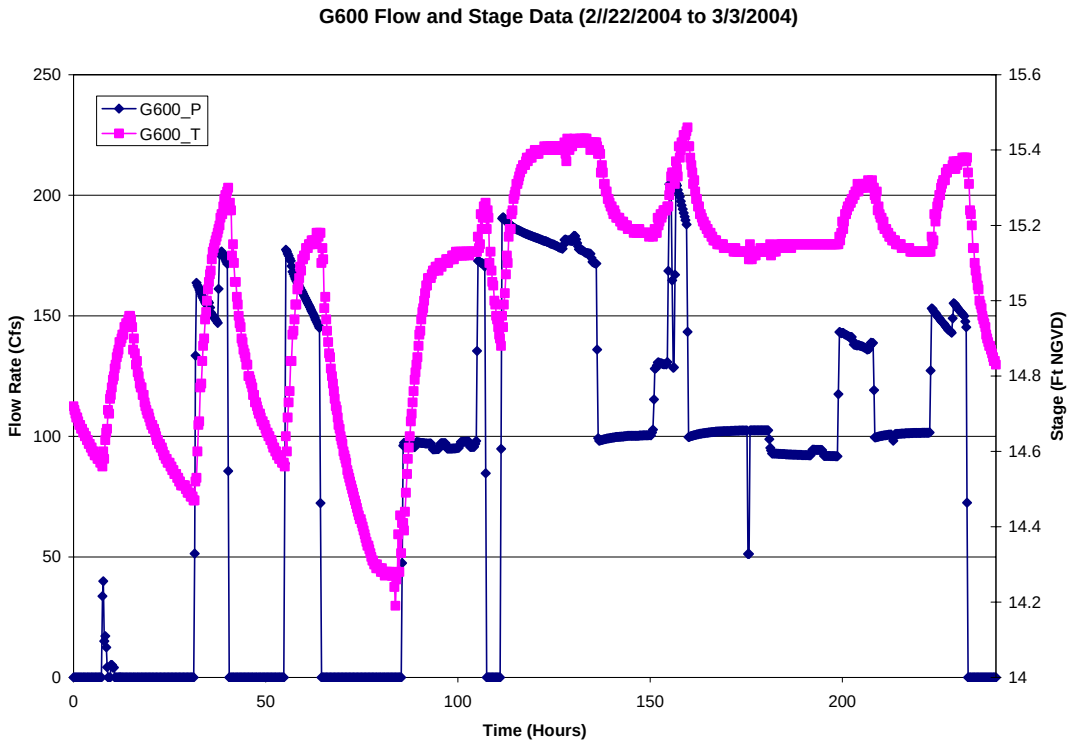


Figure 22 G600 Tailwater Level and Pumping Rate (2/22 to 3/3, 2004)

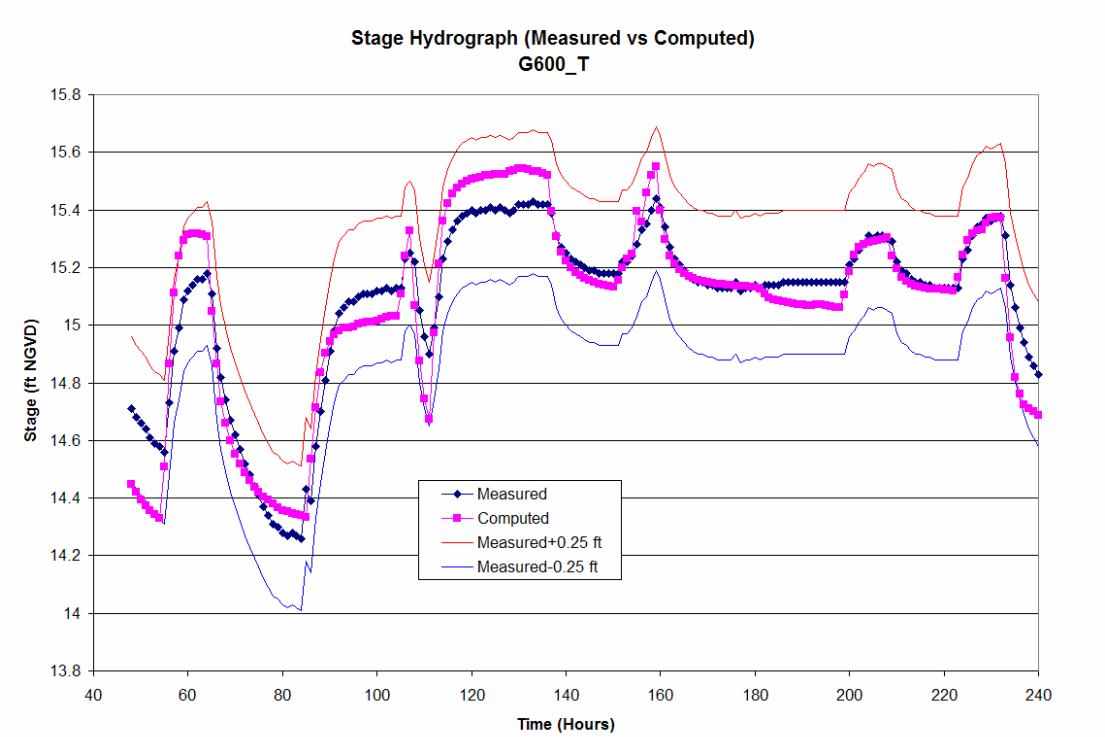


Figure 23 Stage Hydrograph at G600_T (Verification)

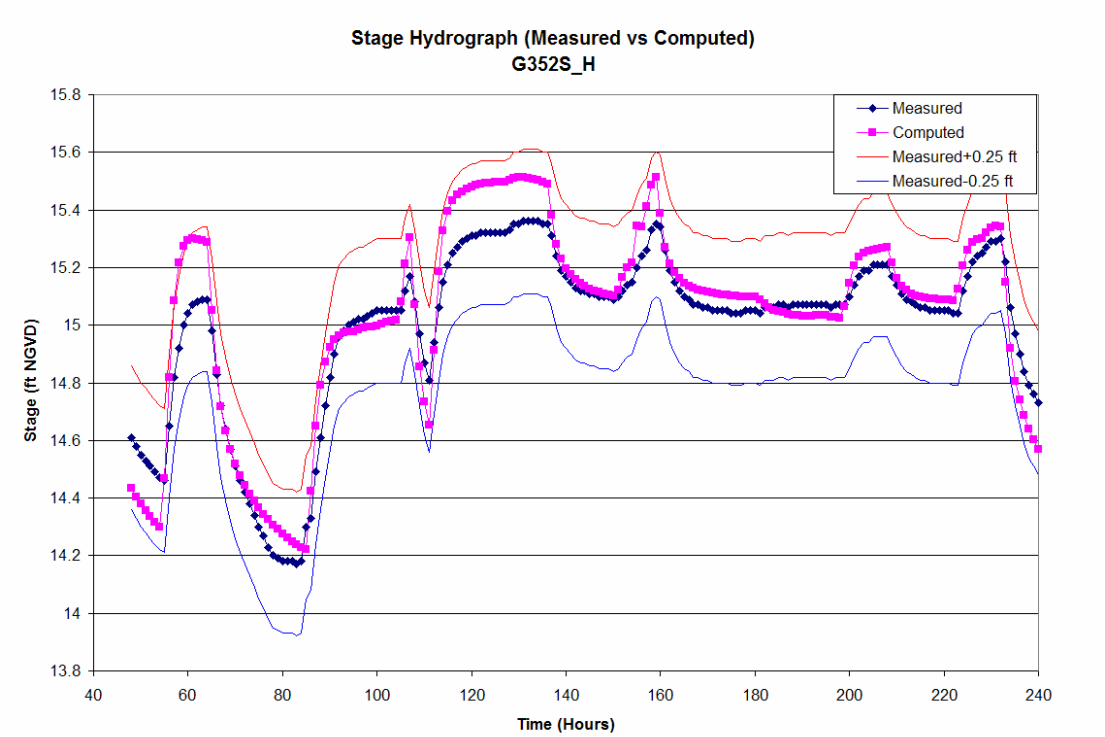


Figure 24 Stage Hydrograph at G352S_H (Verification)

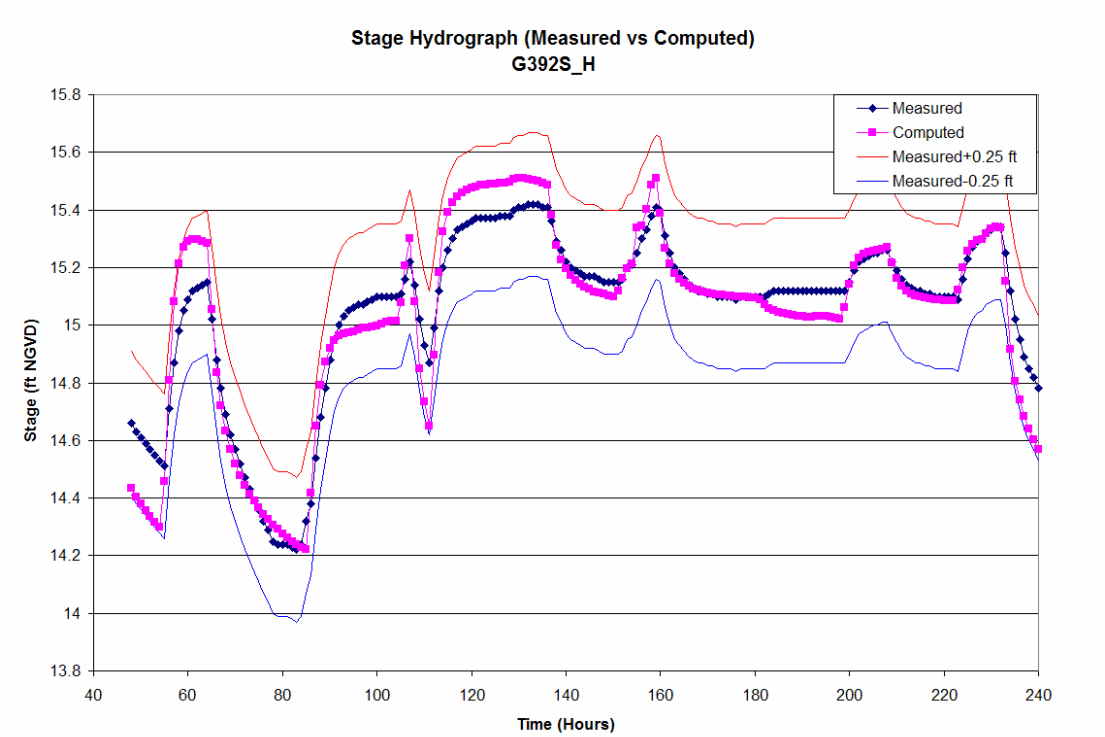


Figure 25 Stage Hydrograph at G392S_H (Verification)

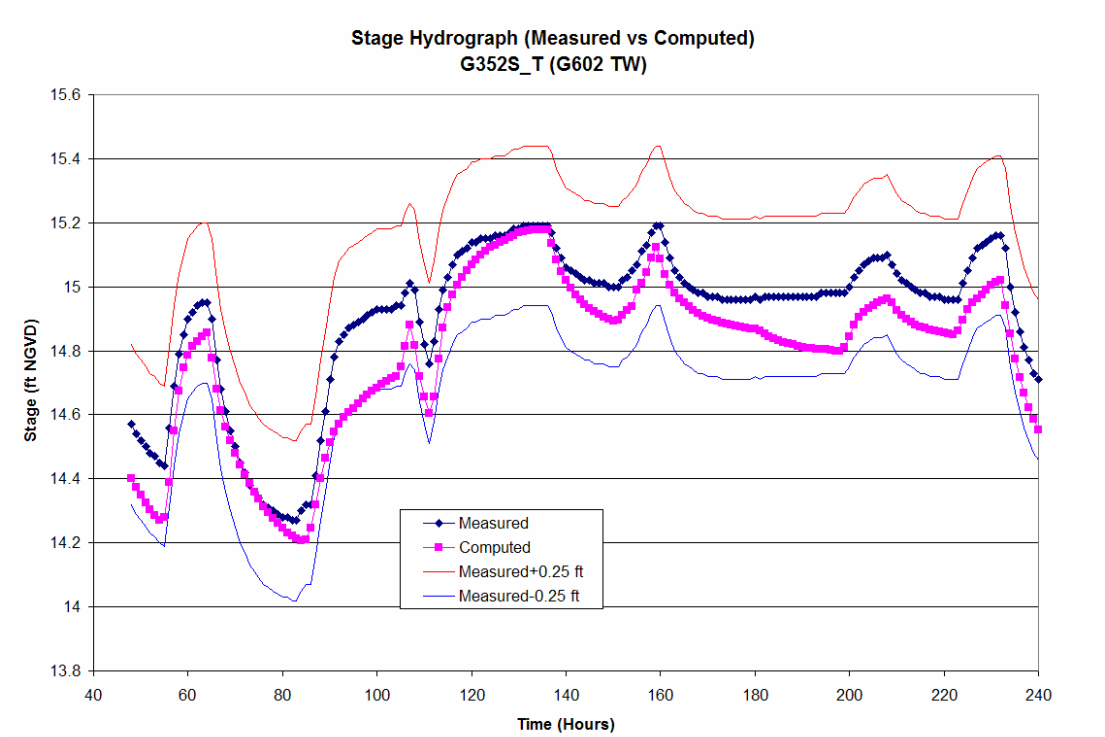


Figure 26 Stage Hydrograph at G352S_T (Verification)

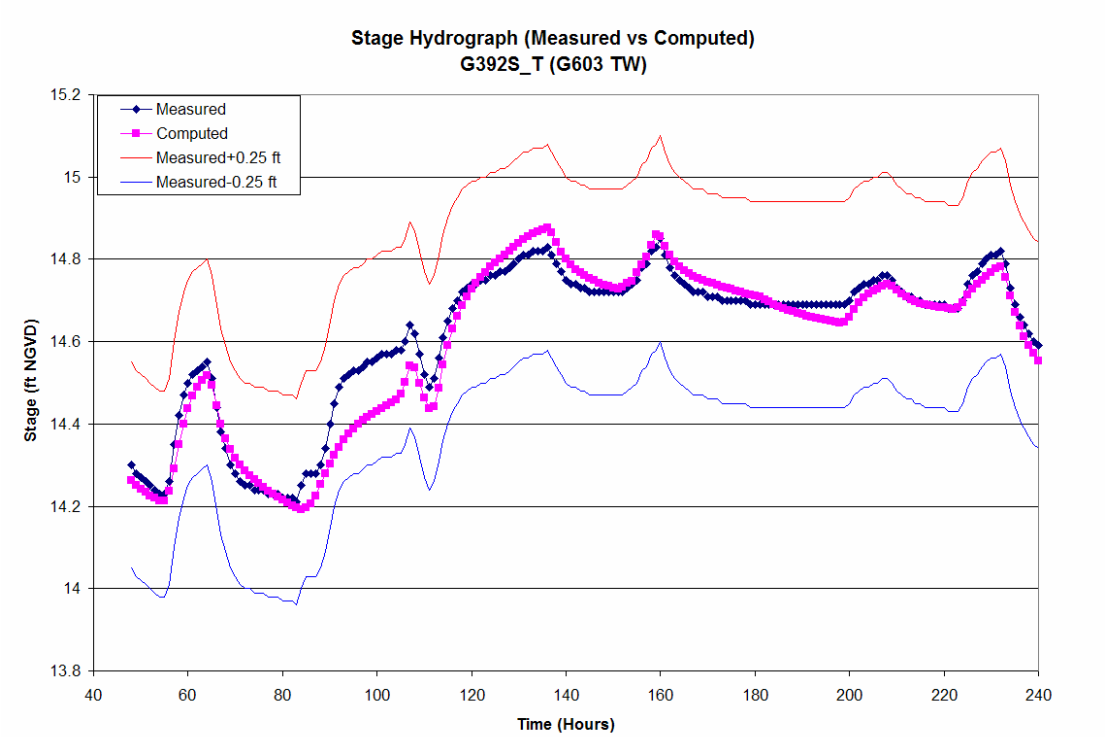


Figure 27 Stage Hydrograph at G392S_T (Verification)

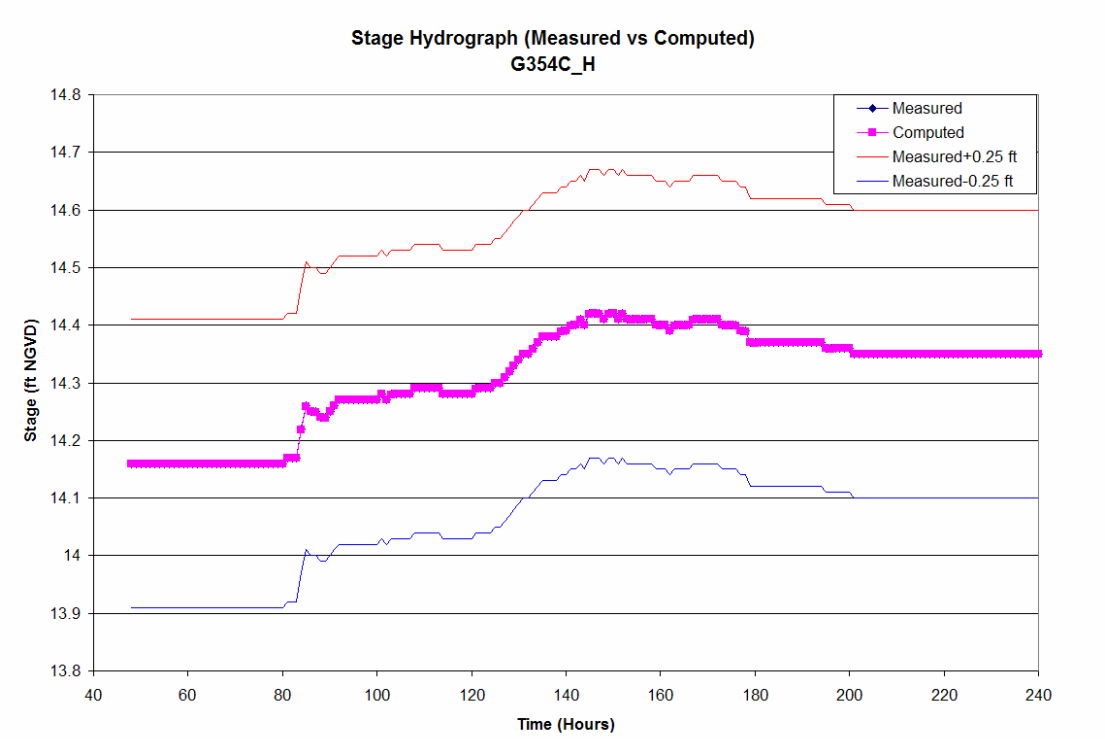


Figure 28 Stage Hydrograph at G354C_H (Verification)

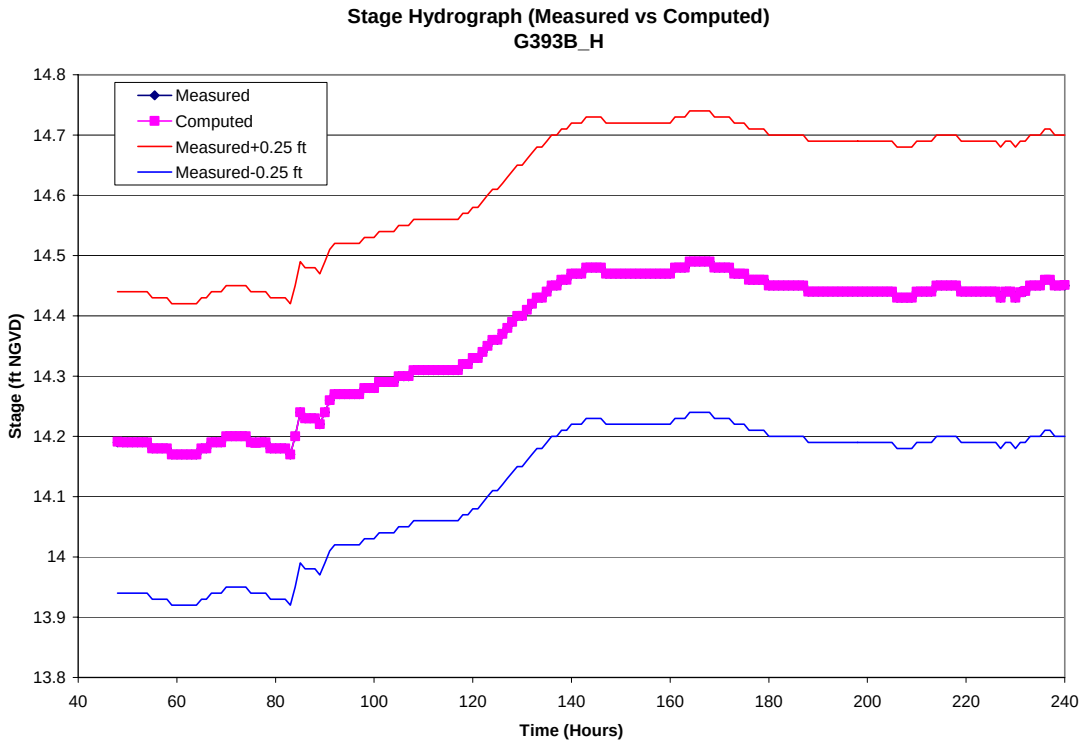


Figure 29 Stage Hydrograph at G393B_H (Verification)

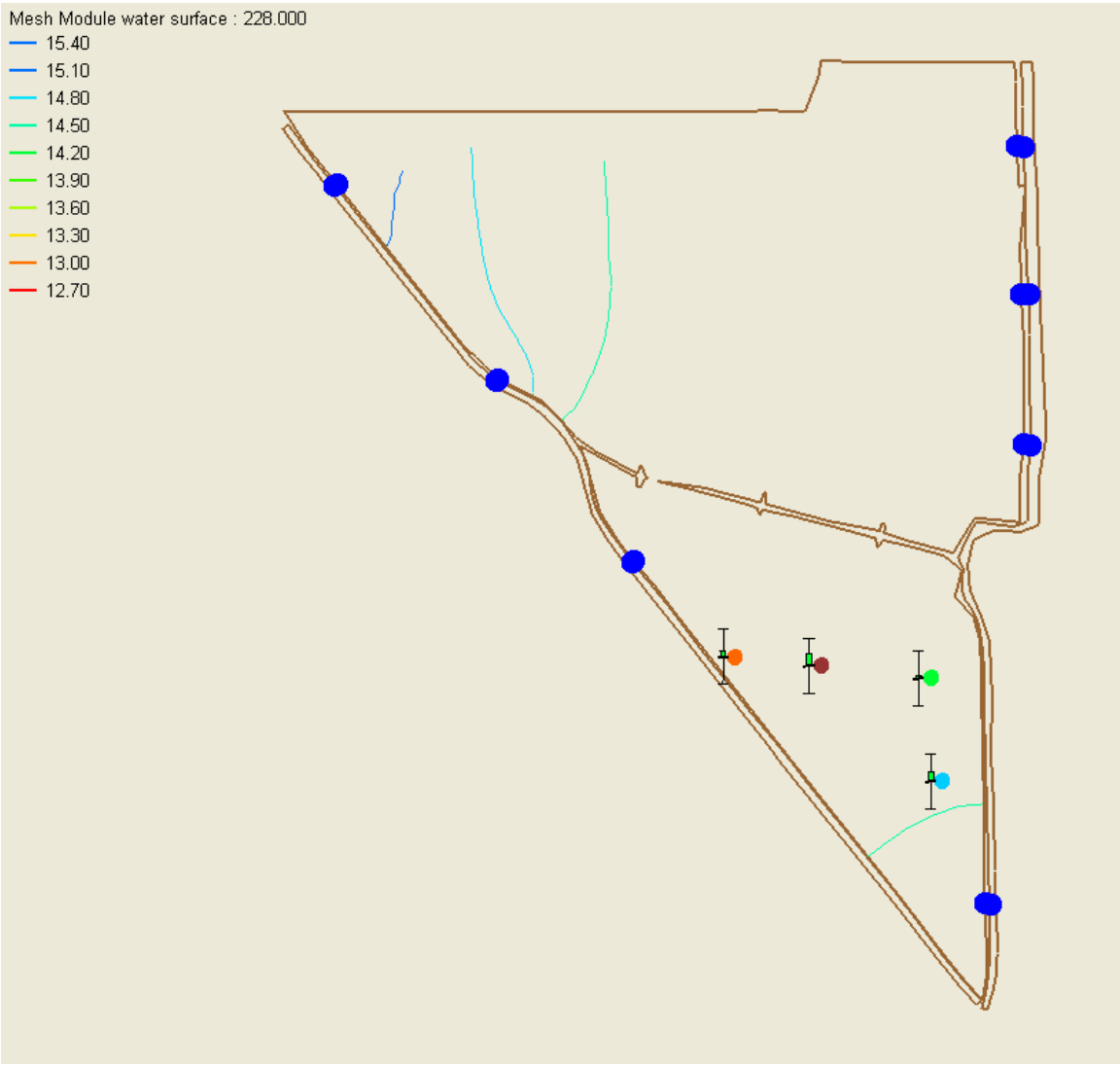


Figure 30 Verification Plot for Cell 3 (3/2/2004, interior sites)

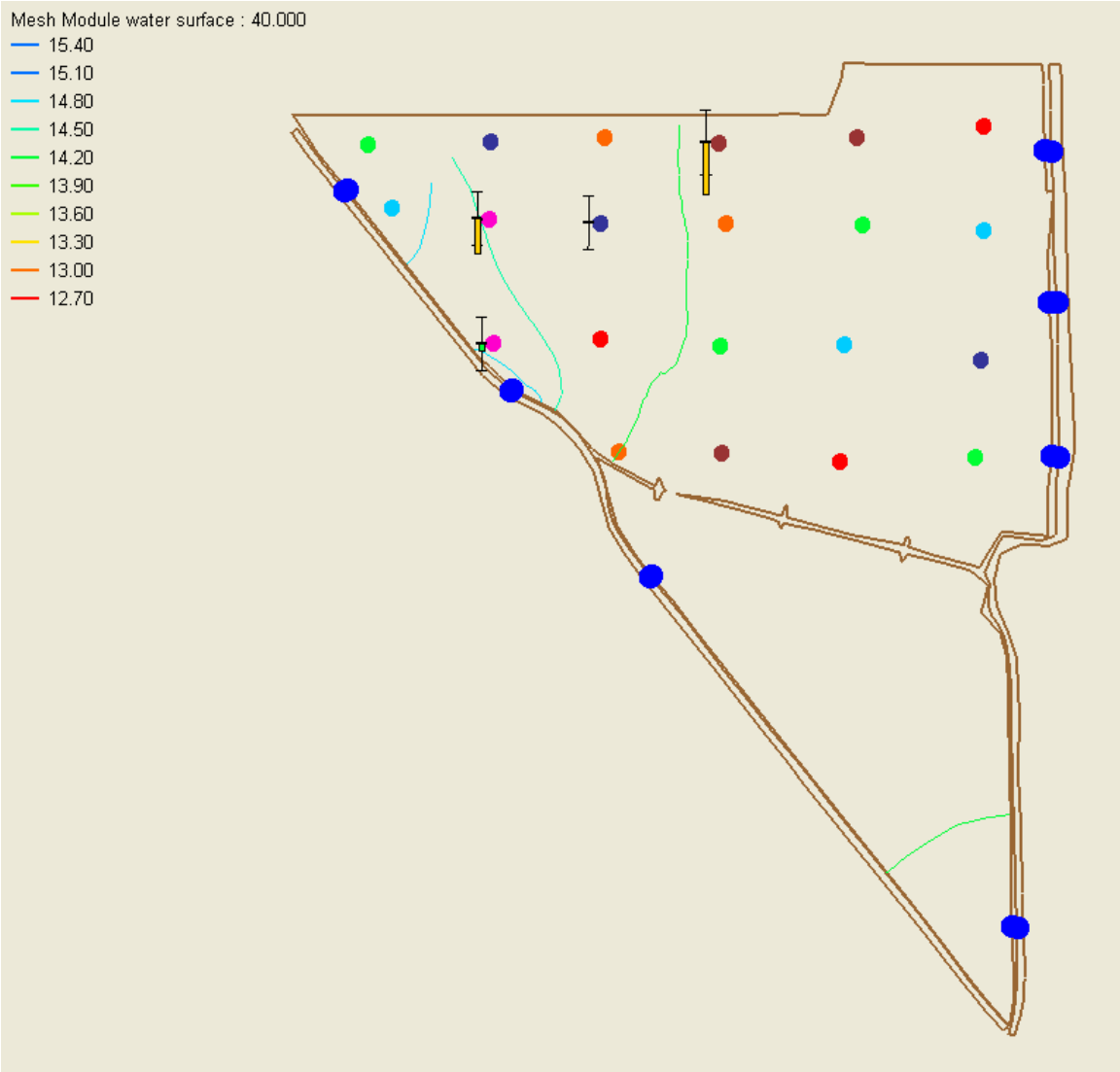


Figure 31 Verification Plot for Cell 5 (2/23/2004, interior sites)

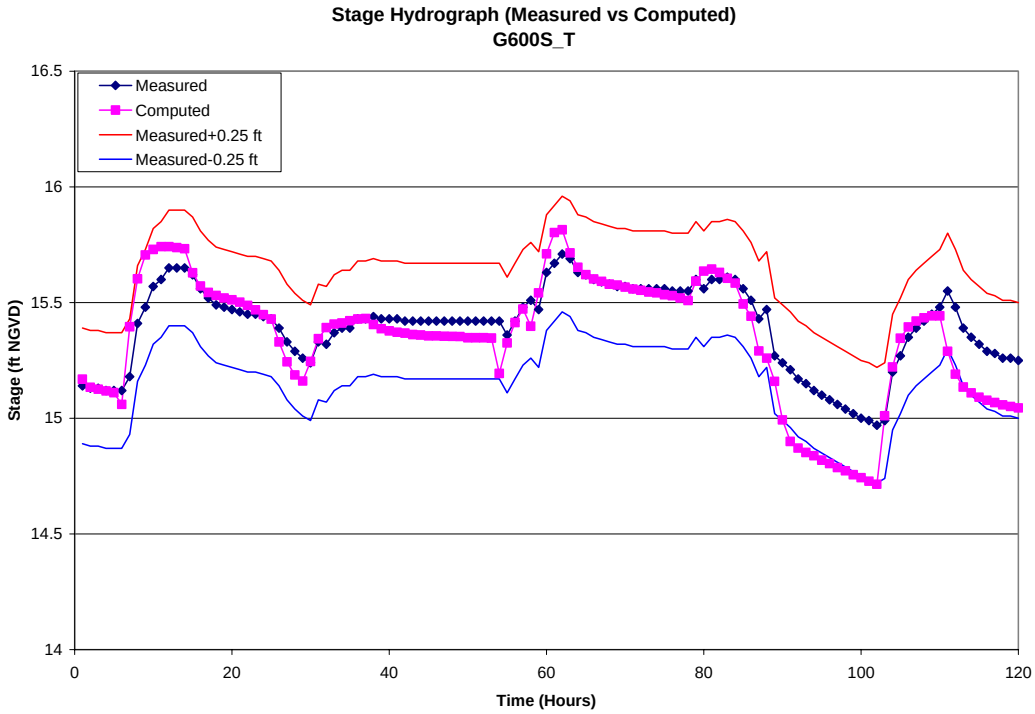


Figure 32 Stage Hydrograph for G600_T (8/20 to 8/25/2003 verification)

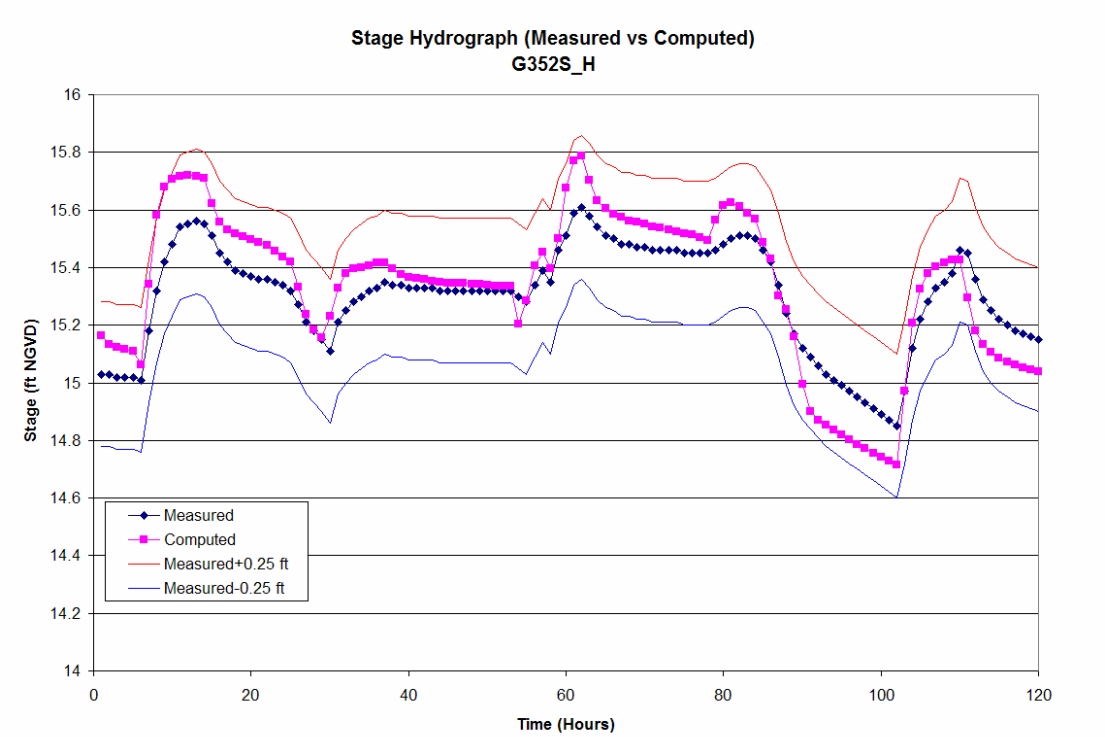


Figure 33 Stage Hydrograph for G352S_H (8/20 to 8/25/2003 verification)

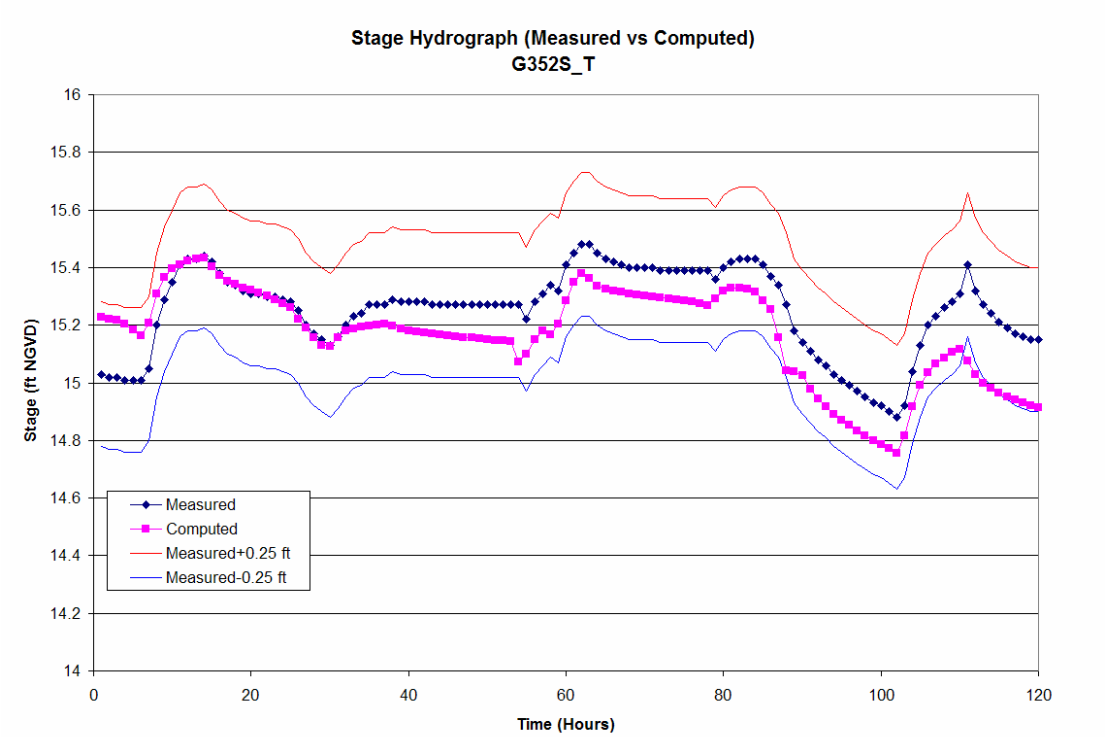


Figure 34 Stage Hydrograph for G352S_T (8/20 to 8/25/2003 verification)

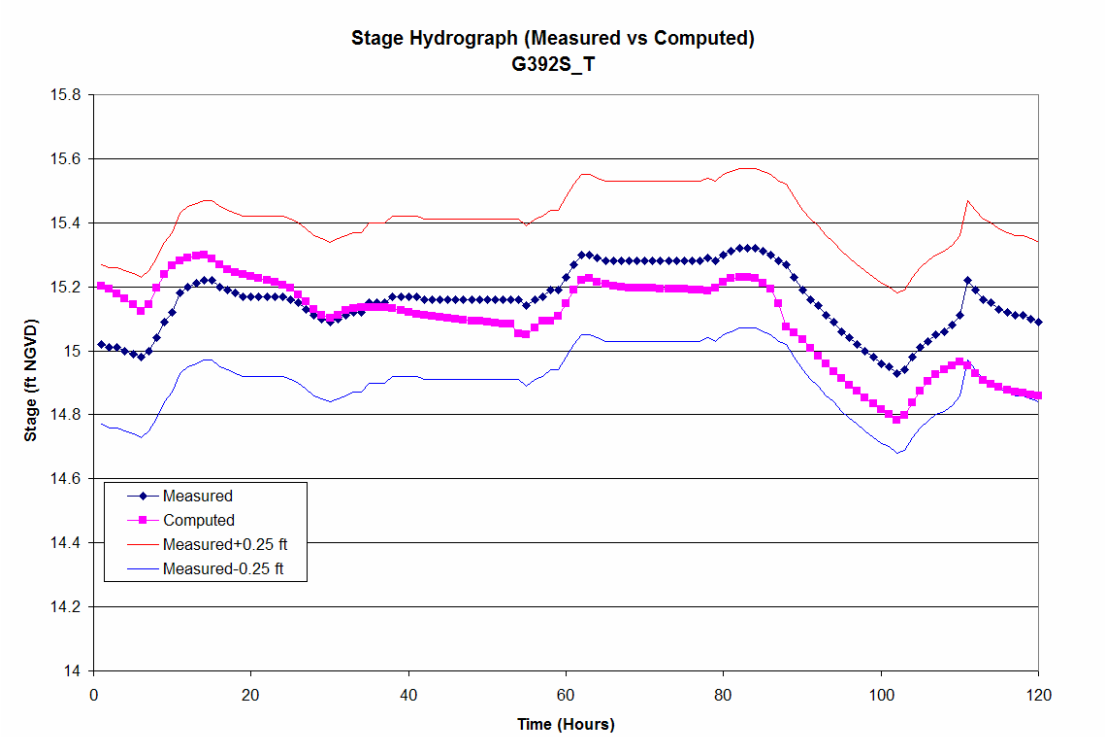


Figure 35 Stage Hydrograph for G392S_T (8/20 to 8/25/2003 verification)

3.5 Calibrated Model parameter sets

The value of model parameters was selected based on model calibration/verification results.

Table 6: Manning's n Values used for STA-6

Depth (ft)	Cattail (Cell 5)	Sawgrass (Cell 3)	Open water with SAV	Canals
3.0	1.5	1.2	0.1	0.03
1.5	Linearly varies	1.2	0.1	
1.0	2.0	Linearly varies	0.1	
0.5	2.0	1.5	0.3	

It is thought that the calibrated Manning's n values for the STA-6 wetland vegetation are at the higher end of the physically reasonable range. The linear relationship of Manning's n value with water depth is implemented in FLO2DH as follows.

$$n = \begin{cases} n_1 & if \quad d \le d_1 \\ n_1 \dfrac{d - d_1}{d_2 - d_1} + n_1 \dfrac{d_2 - d}{d_2 - d_1} & if \quad d_1 < d < d_2 \\ n_2 & if \quad d \ge d_2 \end{cases}$$

where d_1 and d_2 are water depth threshold under and above which n value is constant, and a linear interpolation is applied for n value when water depth is between d_1 and d_2.

The uncertainty in inflow boundary condition, error in weir equations and possible neglected source/sink terms must be considered when these values are judged for use outside of STA-6 model application. The scarcity of quality measured interior water level data also need to be addressed by future data collection.

Manning's roughness coefficient (n value) is related to vegetation type and density, surface irregularity, soil type and other hydraulic characteristics. Ideally, field measured data can be used and calibration of this parameter can be minimal. There is very limited information on the friction roughness values used for natural or constructed wetlands. The material types in STA-6 include sawgrass and cattail dominant marshes and open water with SAV. USGS conducted some field and laboratory experiments on friction roughness of wetland vegetations in the Everglades. The Manning's n value was in the range of 0.4 to 0.6 for sawgrass in the natural southern Everglades (Reference [4]). However, bigger Manning's n value has been used in previous STA hydraulic analyses and modeling studies.

3.6 Sensitivity analysis

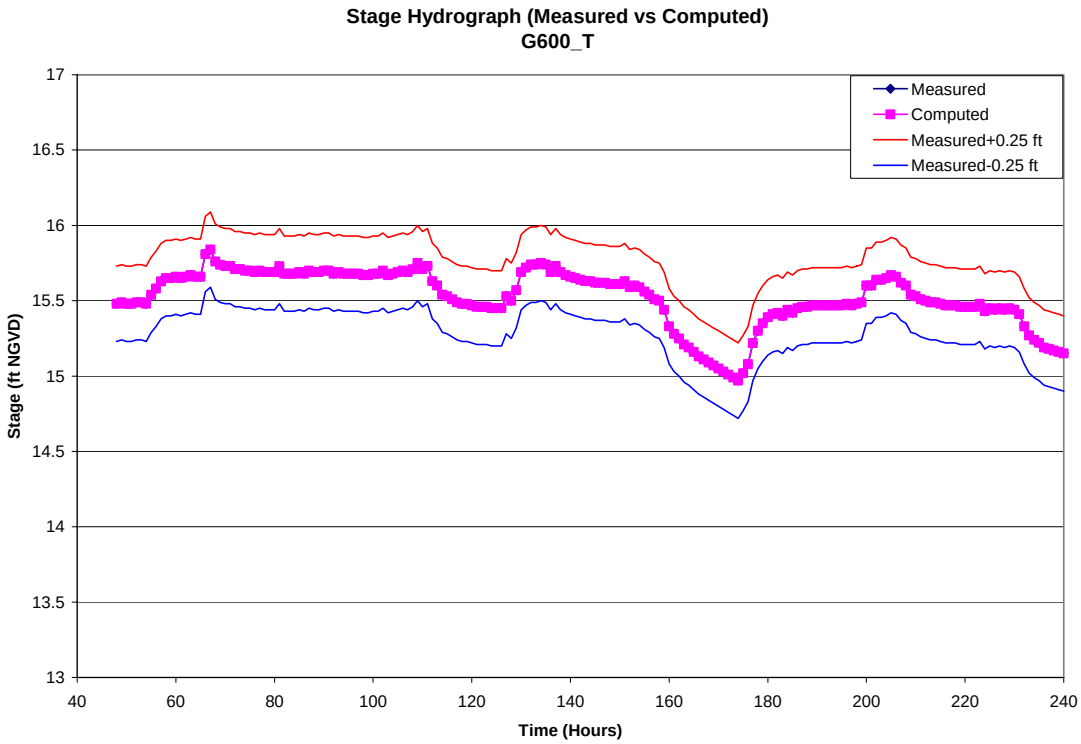
Considering the uncertainty in boundary conditions and observed hydraulic data, some sensitivity analyses were also performed. The 2004 verification run was used for this purpose.

By systematically studying the sensitivity of the simulation results to the changes in model parameters, we can get more insight about the model behavior and model performance.

- **Inflow uncertainty**

  We applied a specified stage hydrograph at G600_T instead of an inflow discharge hydrograph for both calibration/verification runs. This was done to investigate whether or not the uncertainty in inflow quantity and distribution (for example, the possible diversion for irrigation) will significantly affect the verification results.

  The model results show that the calibration/verification results are very similar to those previous results obtained by using a specified discharge hydrograph. This can be observed from the following stage hydrograph plots (Figures 36 to 41). This can be considered as an indicator that the weir equations are well behaved to produce similar results. Since a specified stage hydrograph was applied at upstream, the weir formula determined how much water would be delivered into the Treatment Areas.



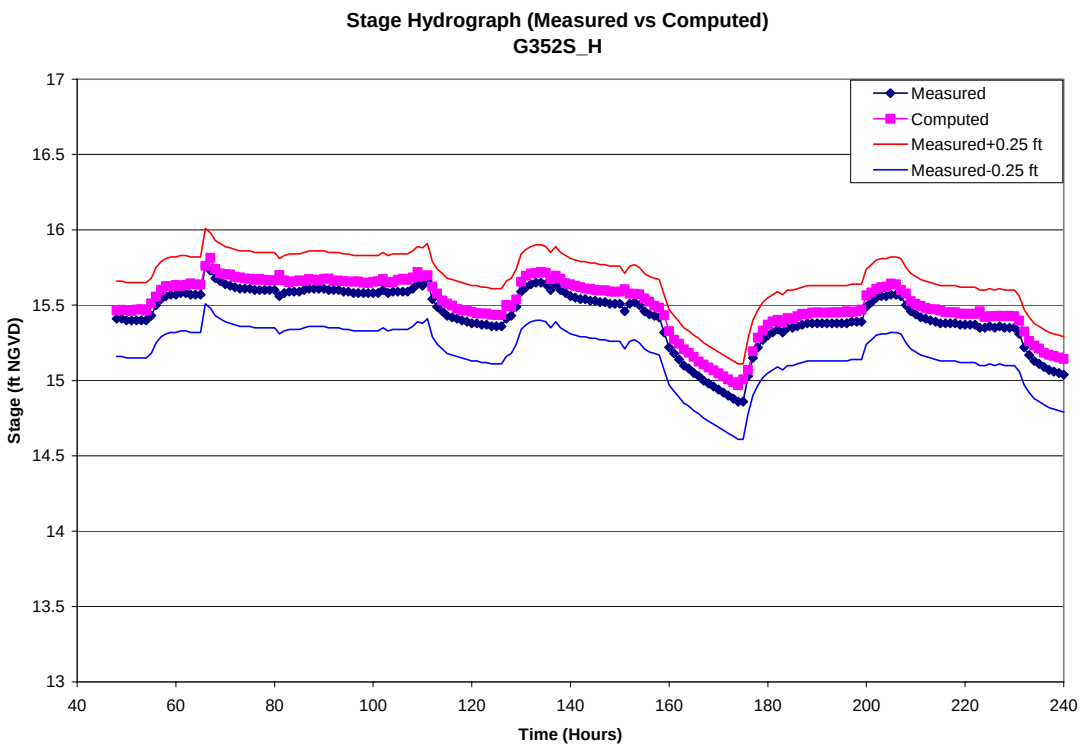
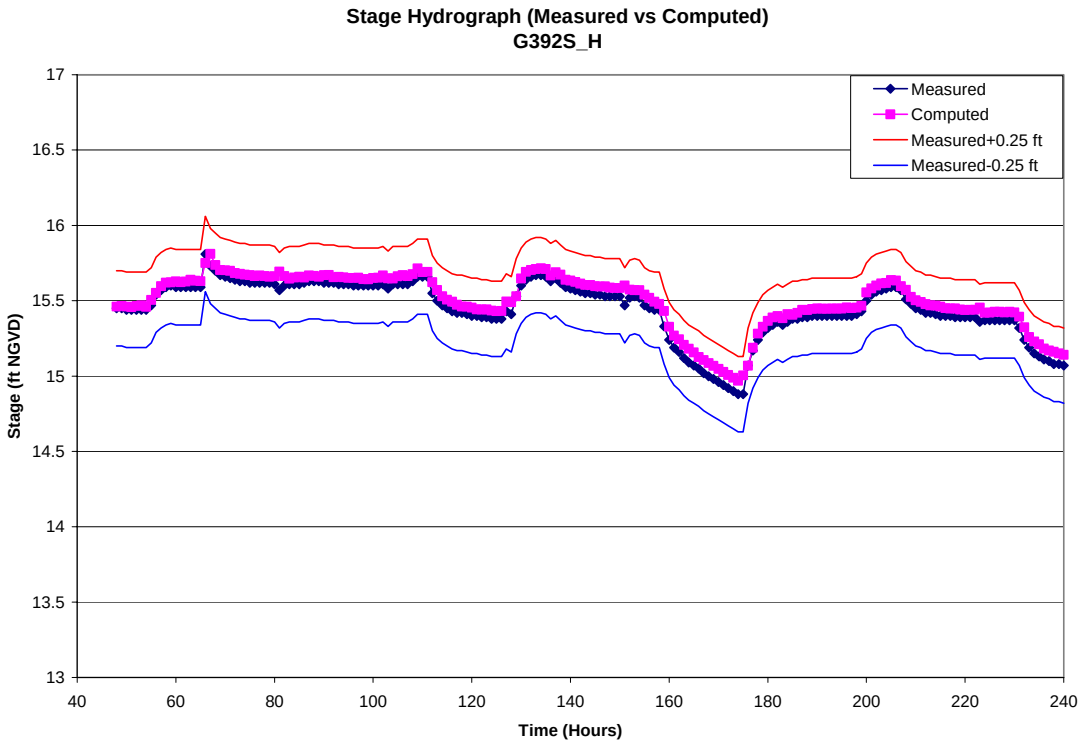


Figure 36 Stage Hydrograph for G600_T and G352S_H (calibration by using G600_T as input)



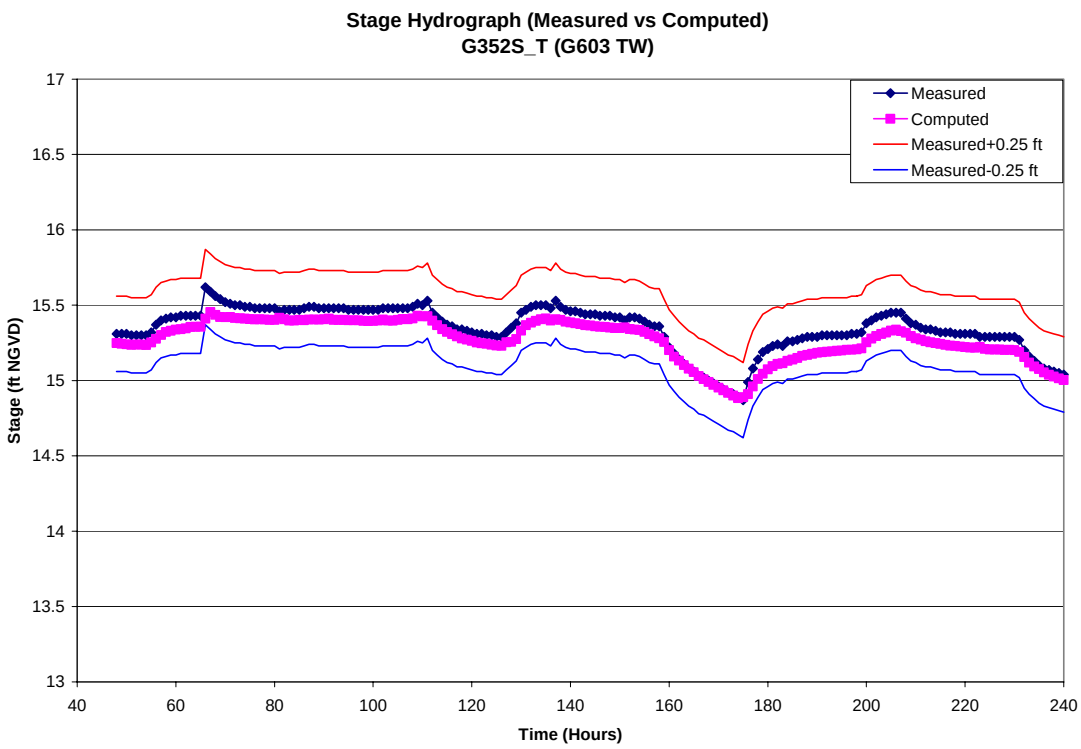
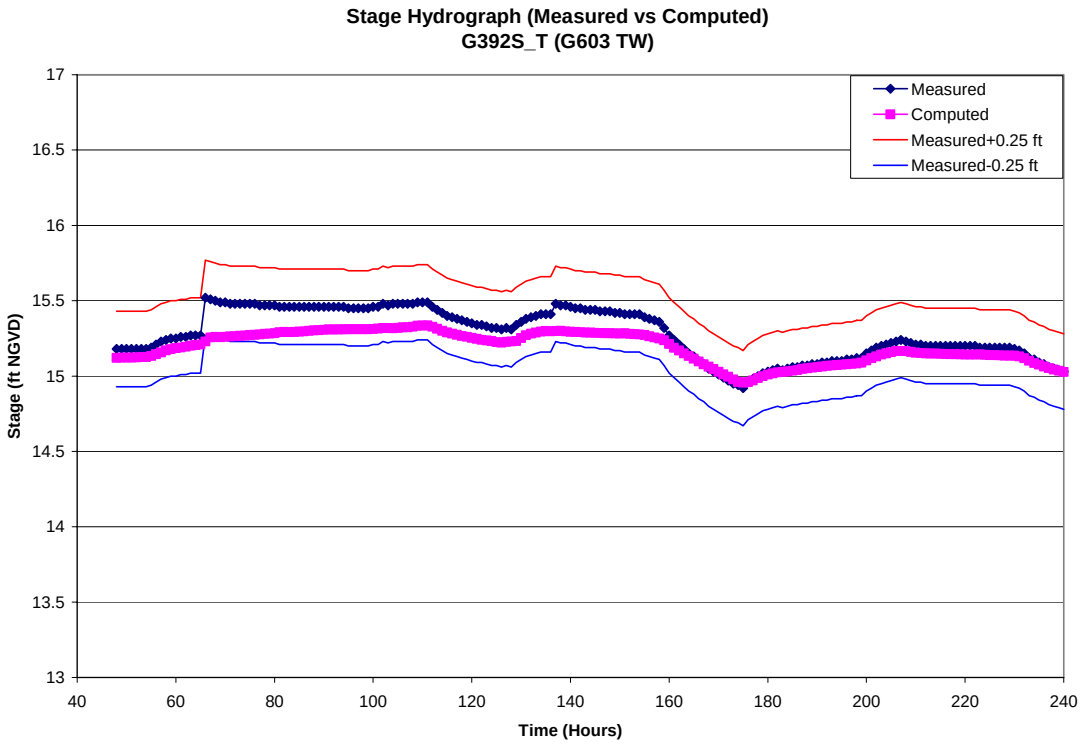


Figure 37 Stage Hydrograph for G352S_T and G392S_H (calibration by using G600_T as input)



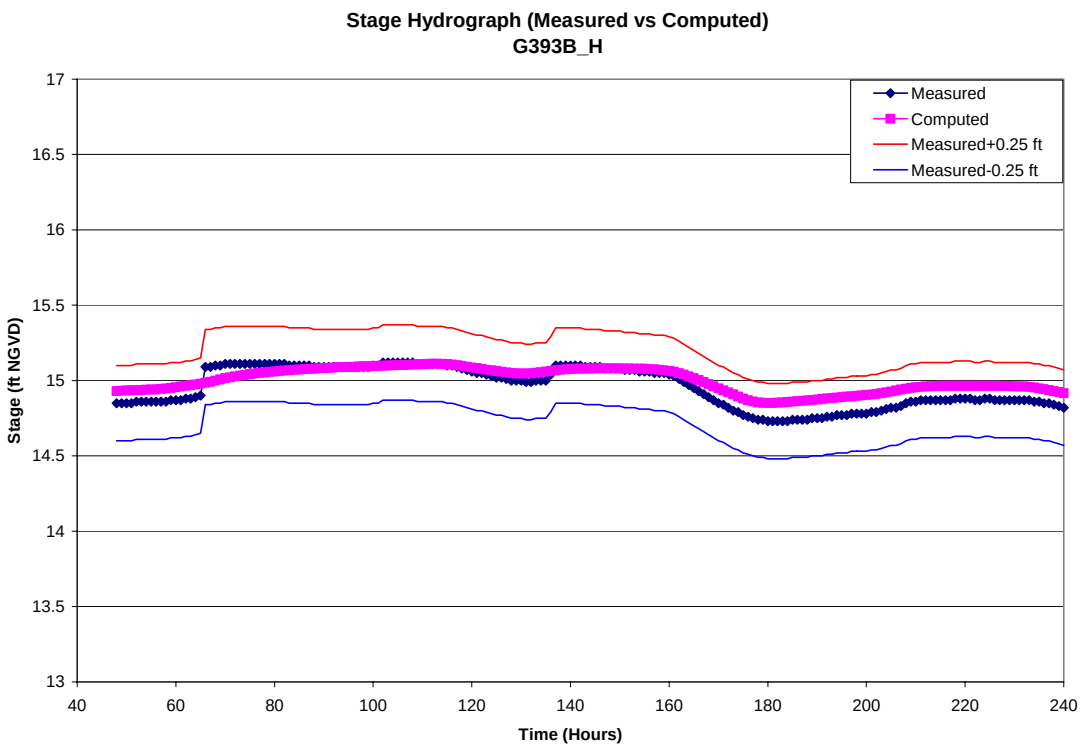
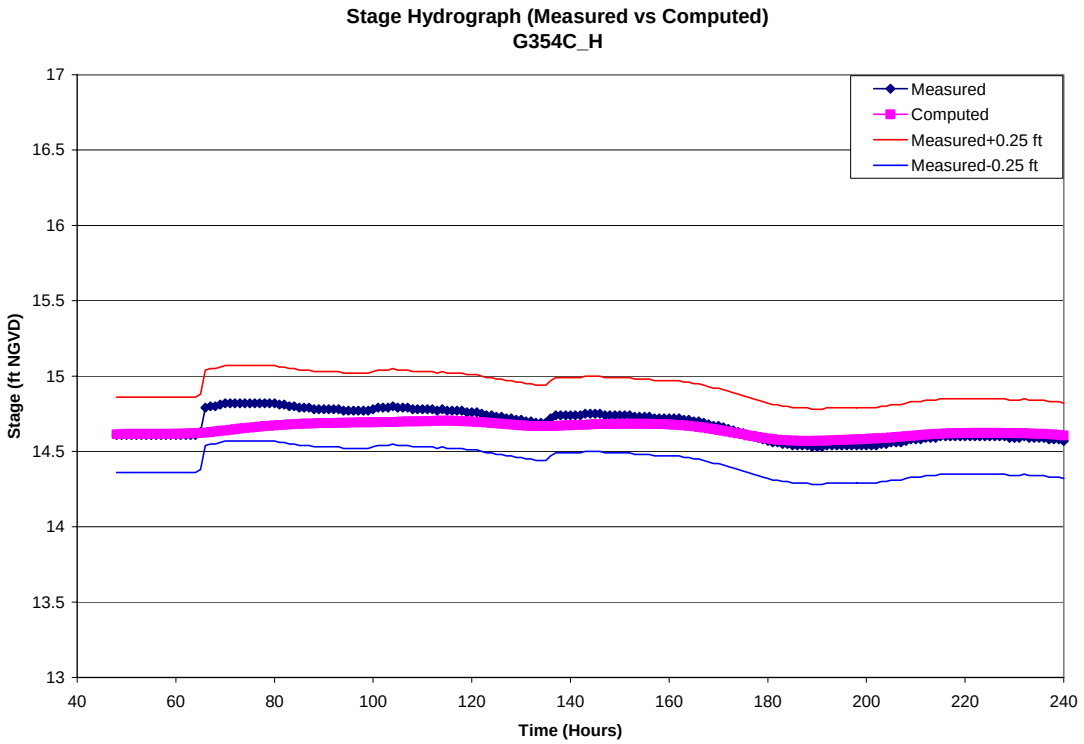


Figure 38 Stage Hydrograph for G392S_T and G393B_H (calibration by using G600_T as input)



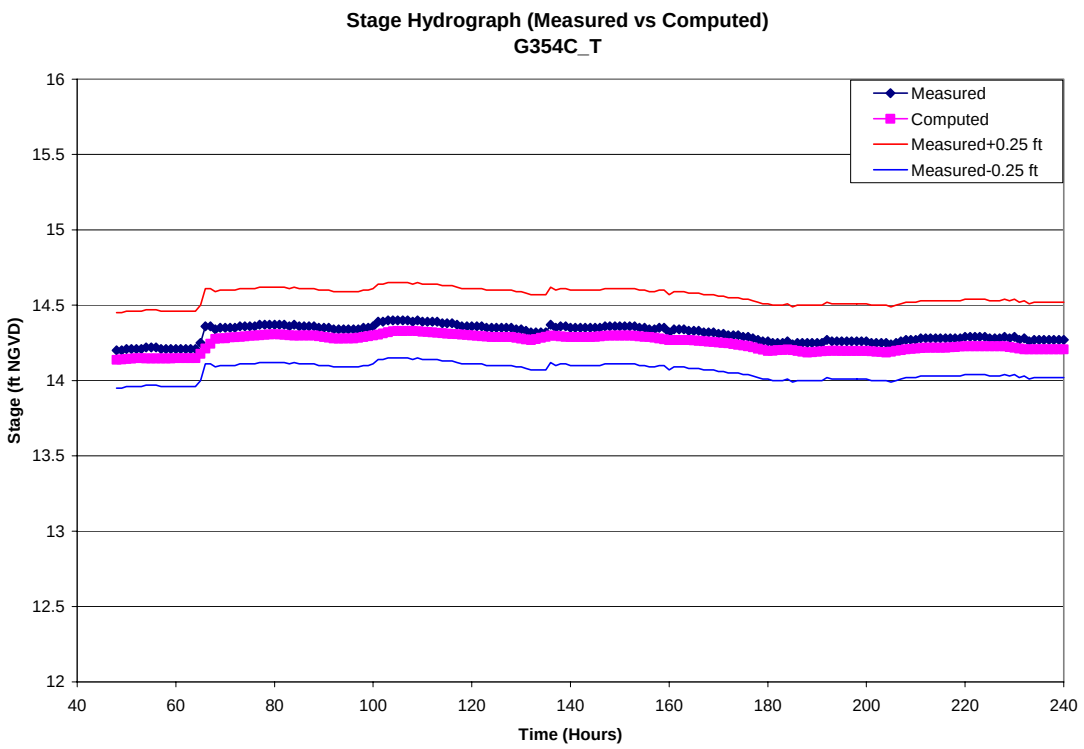
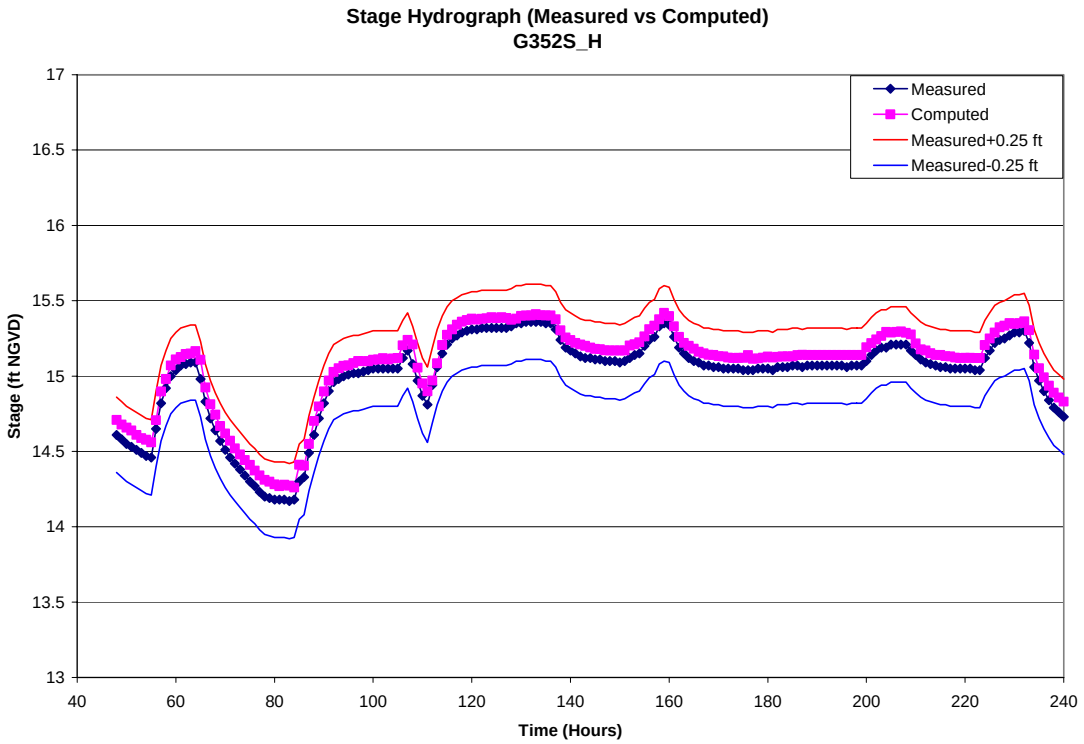


Figure 39 Stage Hydrograph for G354C_T and G354C_H (calibration by using G600_T as input)



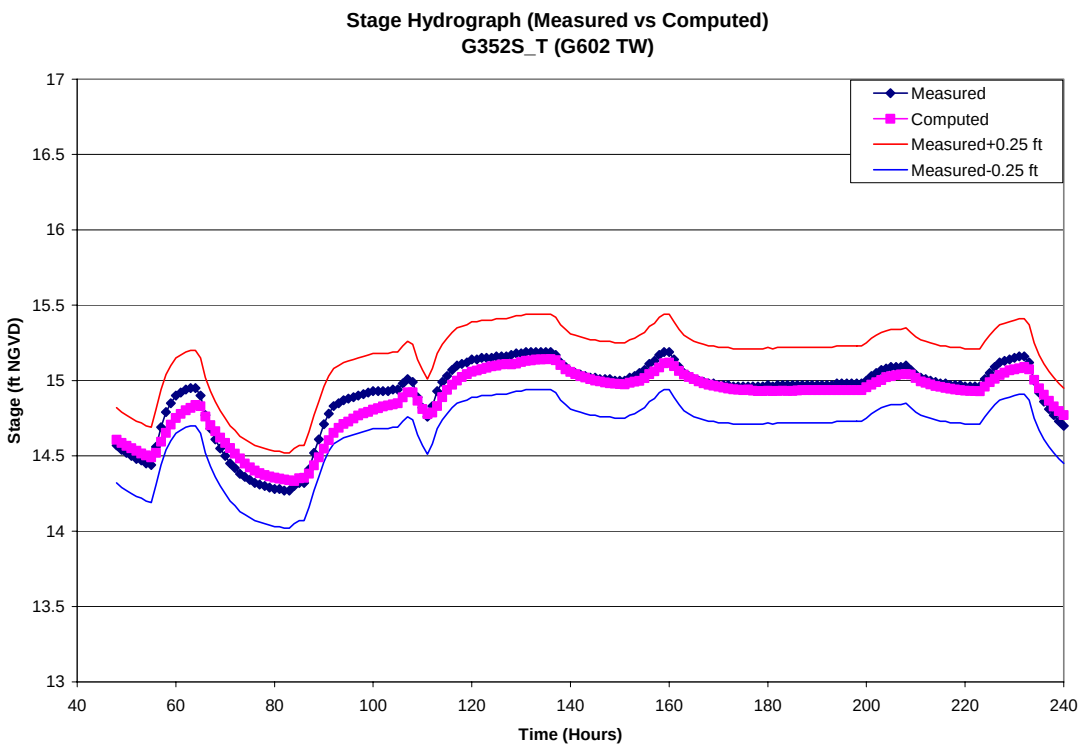


Figure 40 Hydrograph for G352S_H and G352S_T (2004 Verification by using G600_T as input)

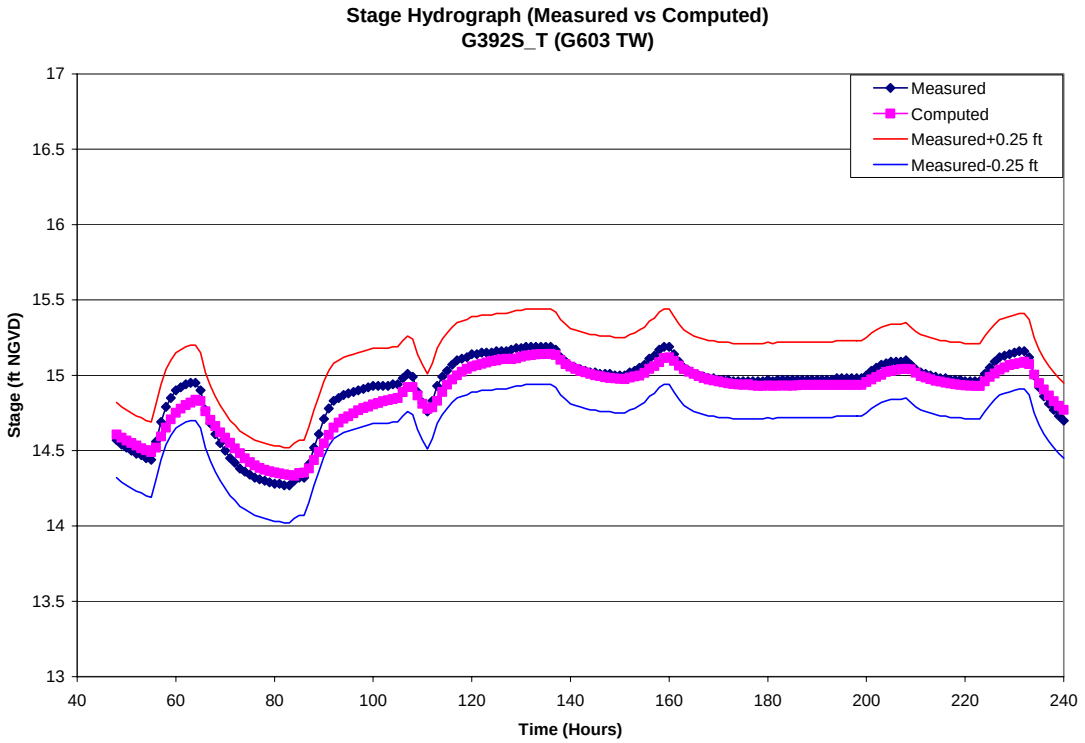


Figure 41 Hydrograph for G392S_T (2004 Verification by using G600_T as input)

- **Manning's n value**

By decreasing the Manning's n value by 30% from the calibrated value, the stage level at G602 TW and G603 TW dropped by about 0.2 ft (Figure 42). This is consistent with what was observed during model calibration. Use of smaller Manning' n values resulted in not matching the historic stage values as well as the calibrated values.

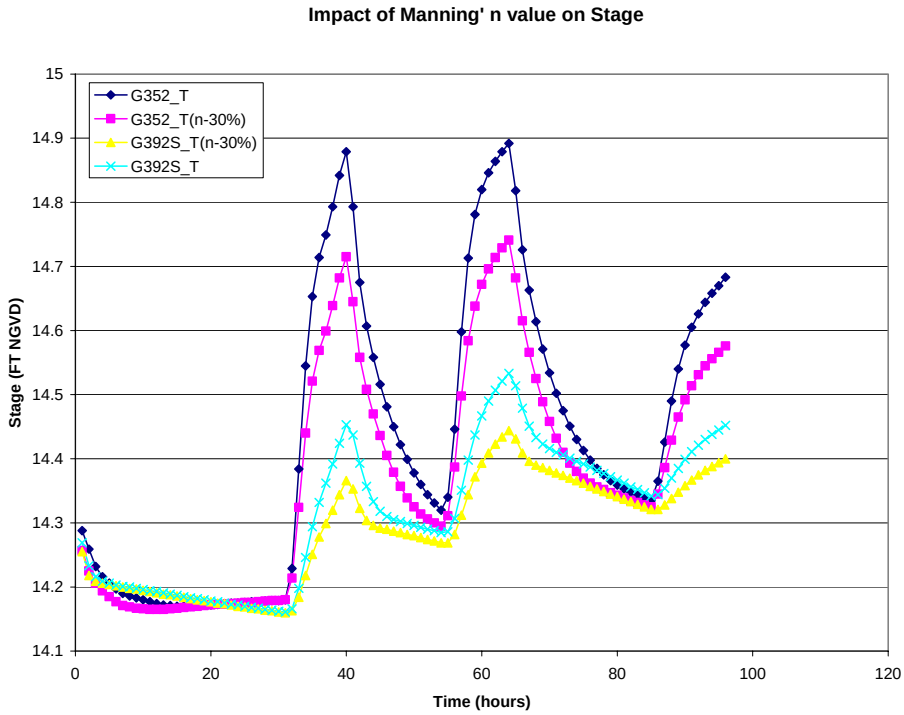


Figure 42 Effect of Manning's n value on Stage at G602 and G603 TW

- **Weir coefficients**

The weir coefficient values used for Inflow Weirs G601, G602 and G603 were simultaneously increased by 30%, while the weir coefficient value for outflow structures G354 A, B and C and G393B were kept unchanged.

The change in water level is not significant (Figure 43). This may be attributed to the fact that the total inflow volume is fixed at G600. The total flow passing through the three Inflow Weirs is thus determined by the available quantity from the Supply Canal.

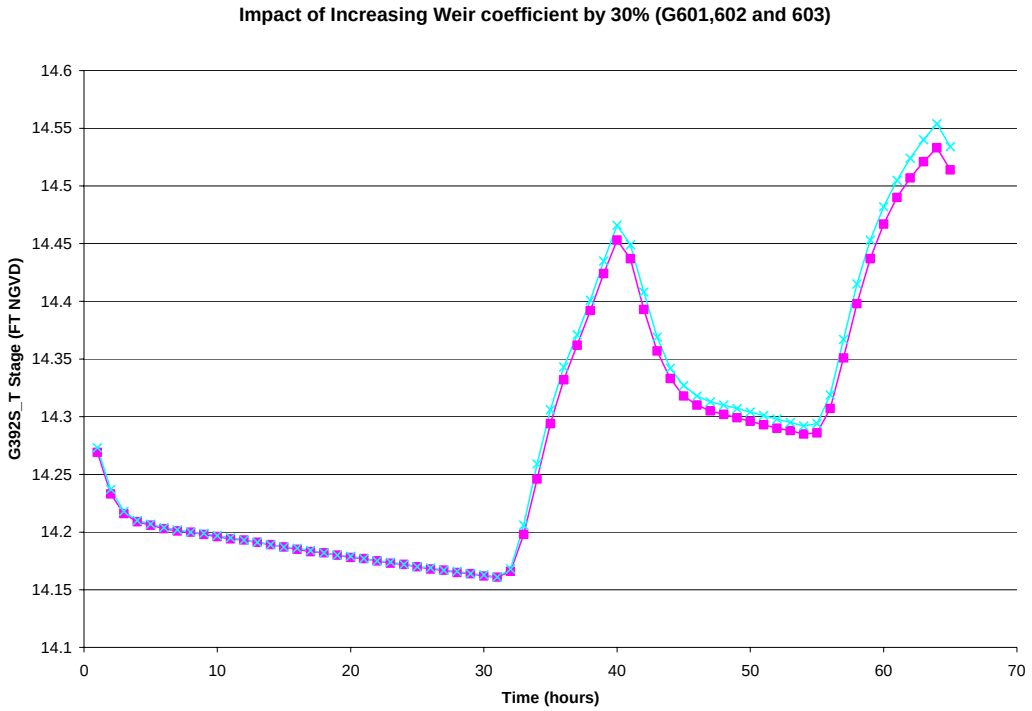


Figure 43 Effect of Cw+30% on G603 Tailwater Level (G392S_T)

- **Numerical parameters**

The impact of finite element mesh size and time step size on the simulation results was also investigated.

The finite element mesh size used was determined by consideration of reducing model run time without affecting model accuracy, because numerous model runs have to be completed within the time frame of the project work.

The time step sizes of one hour and 0.25 hour (15 minutes) were used for the same verification run (2/22/2004 to 3/3/2004), and the water level and velocity magnitude were compared. A plot of stage at G602 TW (G352S_T) (Figure 44) demonstrated that the difference due to time step size is very small and similar results were observed for velocity. Thus it is proposed that the use of a one-hour time step in model calibration and verification is adequate.

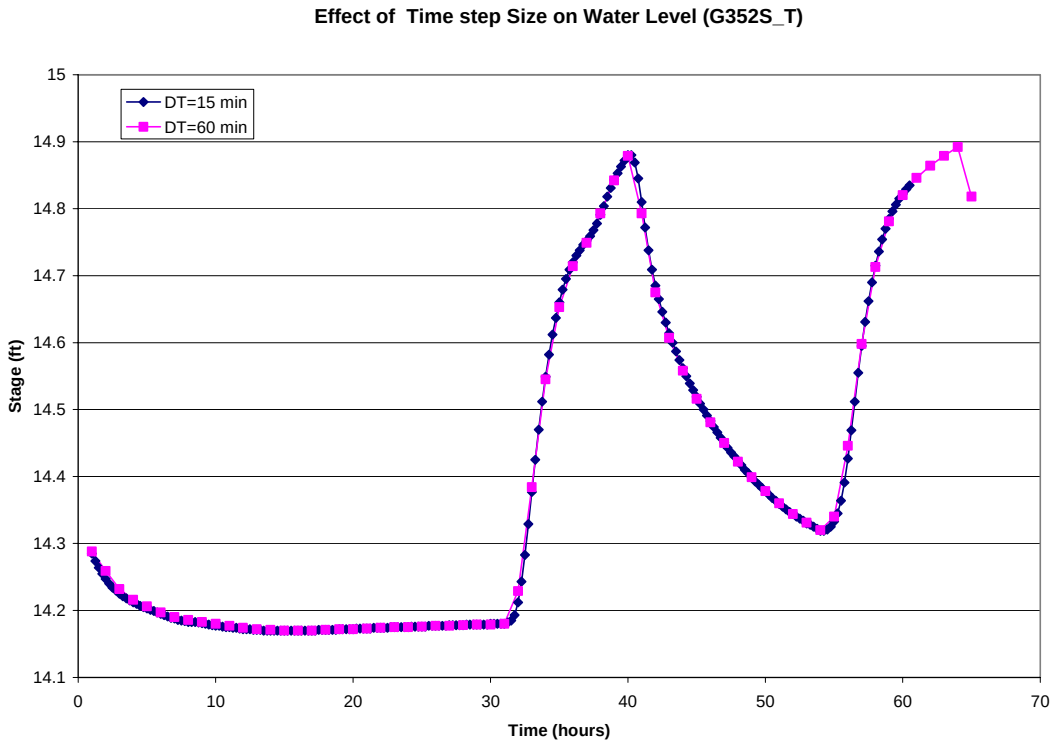


Figure 44 Impact of Time Step Size on Stage

4. Simulation of Existing Conditions

2-D hydrodynamic simulations for the STA-6 existing configuration, under Normal Design Flow, Low Flow and High Flow Conditions were made with the newly developed 2-D hydraulic model.

All Existing Condition model runs are transient simulations. Since hydraulic structures are incorporated into the model, numerical instabilities were encountered in direct steady state flow model runs. All scenario transient runs were conducted with constant boundary conditions for a certain time period until the changes in mass balance and water levels reached a desired accuracy level and steady state could be assumed.

The Design, Low and High Flow value and the desired outflow structure maximum headwater levels are from the STA-6 Operation Plan (Reference [1]). The Design Flow condition is the Normal Flow condition in the STA-6 Operation Plan. The High Flow condition is the Standard Project Storm Flow in the same reference. The Low Flow condition is assumed as a total flow of 100 cfs condition.

The design rainfall for the High Flow condition is the Standard Project Storm (21.6 inches in 24 hours) described in the STA-6 Operation Plan. Table 7 lists the flow

values used for Low, Design and High Flow conditions. Table 8 summarizes the specified water levels at the Discharge Canal for these model runs. For the Low Flow, the target stage in the Discharge Canal is set to a low value so that free weir flow is maintained at the outflow structures.

Table 7: Flow Rate for Different Steady Flow Conditions (cfs)

Item	Total	Cell 3	Cell 5
Low	100		
Design	500		
High (inflow + rainfall)	1,289.5		
Structure inflow for High Flow condition	500		
Rainfall rate	789.5	222.3	567.2

Table 8: Water Level at the Discharge Canal (ft)

Item	Stage (ft)
Low	13.0
Design	15.15
High	16.4

4.1 Design Flow Condition

A constant pumping rate of 500 cfs at G600 location and a constant specified stage of 15.15 at G393B_T were applied for the Design Flow Condition model run.

Water mass balance and water surface elevation were verified for steady state after a 720-hour transient simulation. A summary of STA-6 water budget under Design Flow condition is shown in Table 9. It can be seen that good mass balance was achieved.

Table 9: Water Budget computed by the Design Flow simulation

Supply Canal:		Discharge (Cfs)
Inflow		**500**
Outflow	G601	154.153
	G602	179.474
	G603	166.395
	Total outflow	**500.022**
Cell 3:		
Inflow	G603	166.395

Outflow	G393B	166.387
Cell 5:		
Inflow	G601	154.153
	G602	179.474
Outflow	G354A	108.788
	G354B	108.839
	G354C	115.985
	Total outflow	333.612
STA-6 Total Outflow		**499.999**

The water surface elevation and the velocity magnitude/vector are plotted in the following figures 45 and 46.

The water surface elevation in Cell 3 ranges from 16.0 to 16.5 ft NGVD and in Cell 5, it ranges from 15.8 to 16.5 ft NGVD. The higher values are only distributed in the local areas closest to G601 and G602. Water level is below 16.0 ft for most of the area of Cell 5.

The maximum velocity magnitude is 1.44 ft/s in the Supply Canal. In the marsh area, the velocity is smaller than 0.1 ft/s. There are no obvious short-circuiting routes in STA-6.

Figure 45 Water Surface Elevation in STA-6 (ft NGVD) (Normal Flow, Existing Condition)

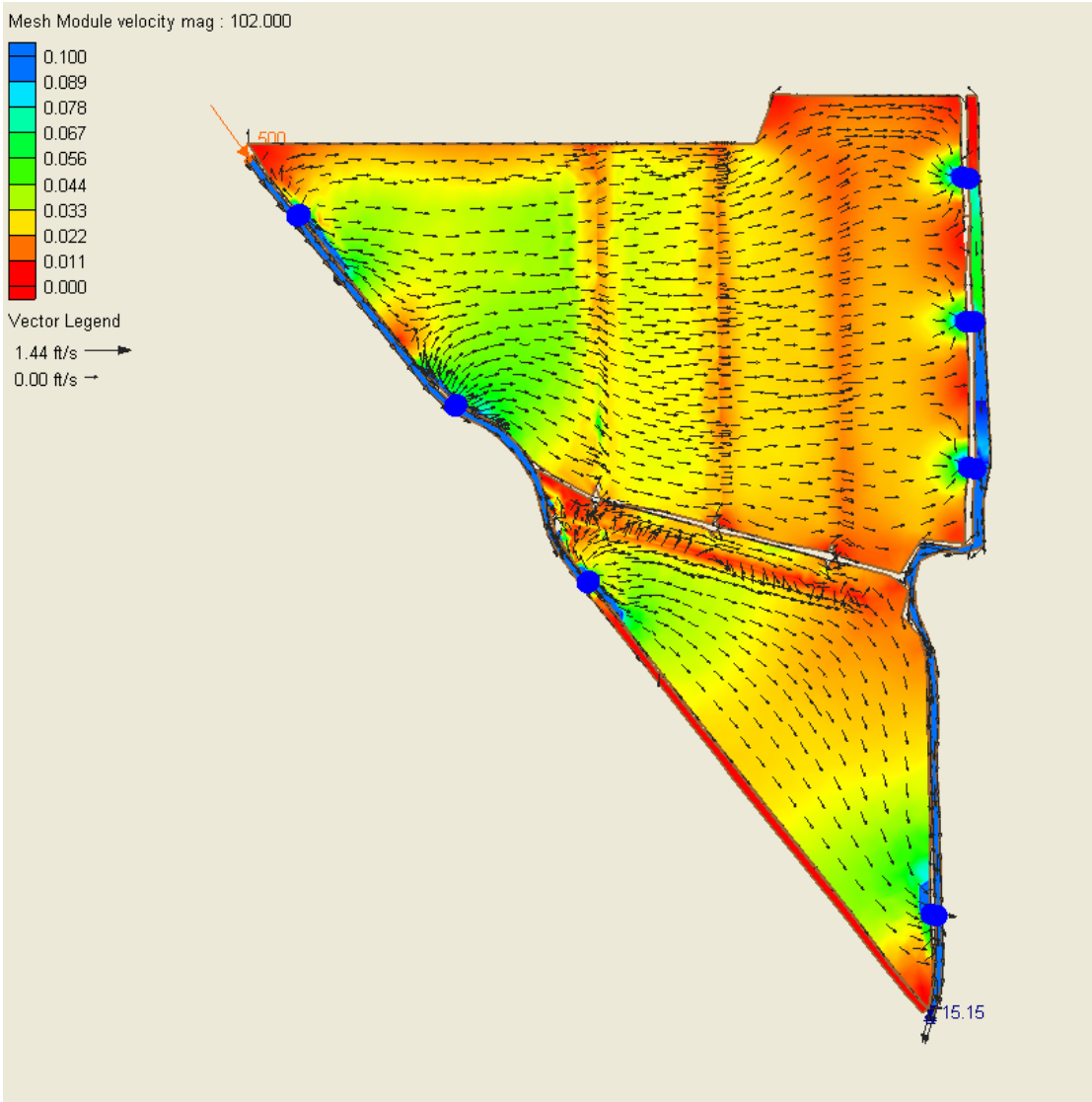


Figure 46 Velocity Plot for STA-6 (ft/s) (Normal Flow, Existing Condition)

4.2 Low Flow Condition

A constant pumping rate of 100 cfs at the G600 location and a constant specified stage of 13.0 at G393B_T were applied for the Design Flow Condition model run.

Water mass balance and water surface elevation were verified for steady state after a 720-hour transient simulation (Table 10).

Table 10: Water Budget computed by the Low Flow simulation

Supply Canal:		Discharge (Cfs)
Inflow		**100**
Outflow	G601	27.675
	G602	37.609
	G603	34.716
	Total outflow	**100.0**
Cell 3:		
Inflow	G603	34.716
Outflow	G393B	34.716
Cell 5:		
Inflow	G601	27.675
	G602	37.609
Outflow	G354A	26.167
	G354B	19.559
	G354C	19.556
	Total outflow	65.282
STA-6 Total Outflow		**100.0**

The water surface elevation and the velocity magnitude/vector are plotted in the following figures 47 and 48.

The water surface elevation in Cell 3 ranges from 14.60 to 14.7 ft NGVD and in Cell 5, it ranges from 14.5 to 14.9 ft NGVD. The higher values are only distributed in the local areas closest to G601 and G602. Water level is below 14.60 ft for most of the area of Cell 5.

The Low Flow Condition simulation shows that with a 100 cfs inflow, about 0.5 ft of mean water depth can still be maintained in the marsh area. This is due to the raised crest elevations at the outflow structures (above 14.0 ft).

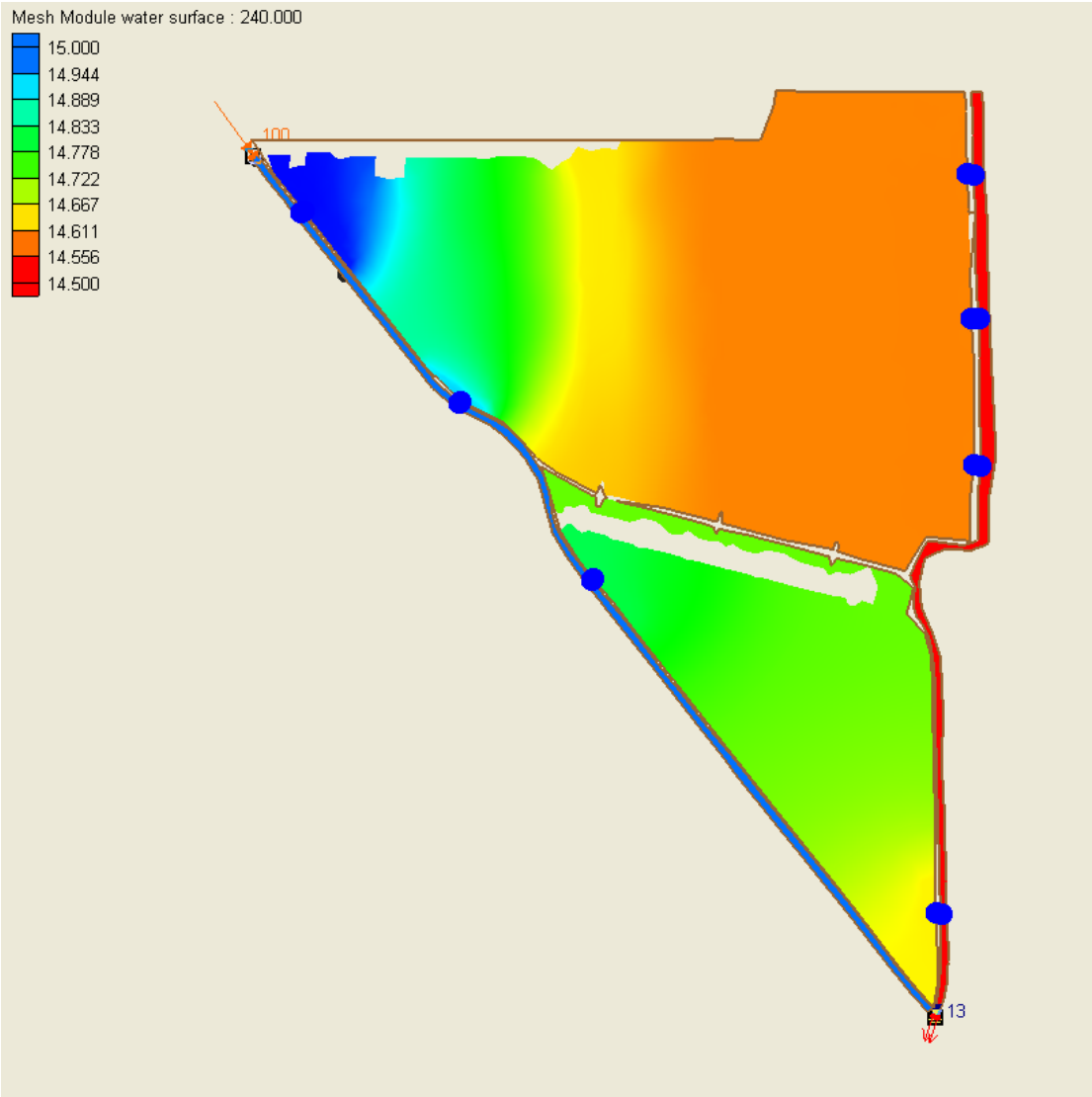


Figure 47 Water Surface Elevation in STA-6 (Low Flow, Existing Condition)

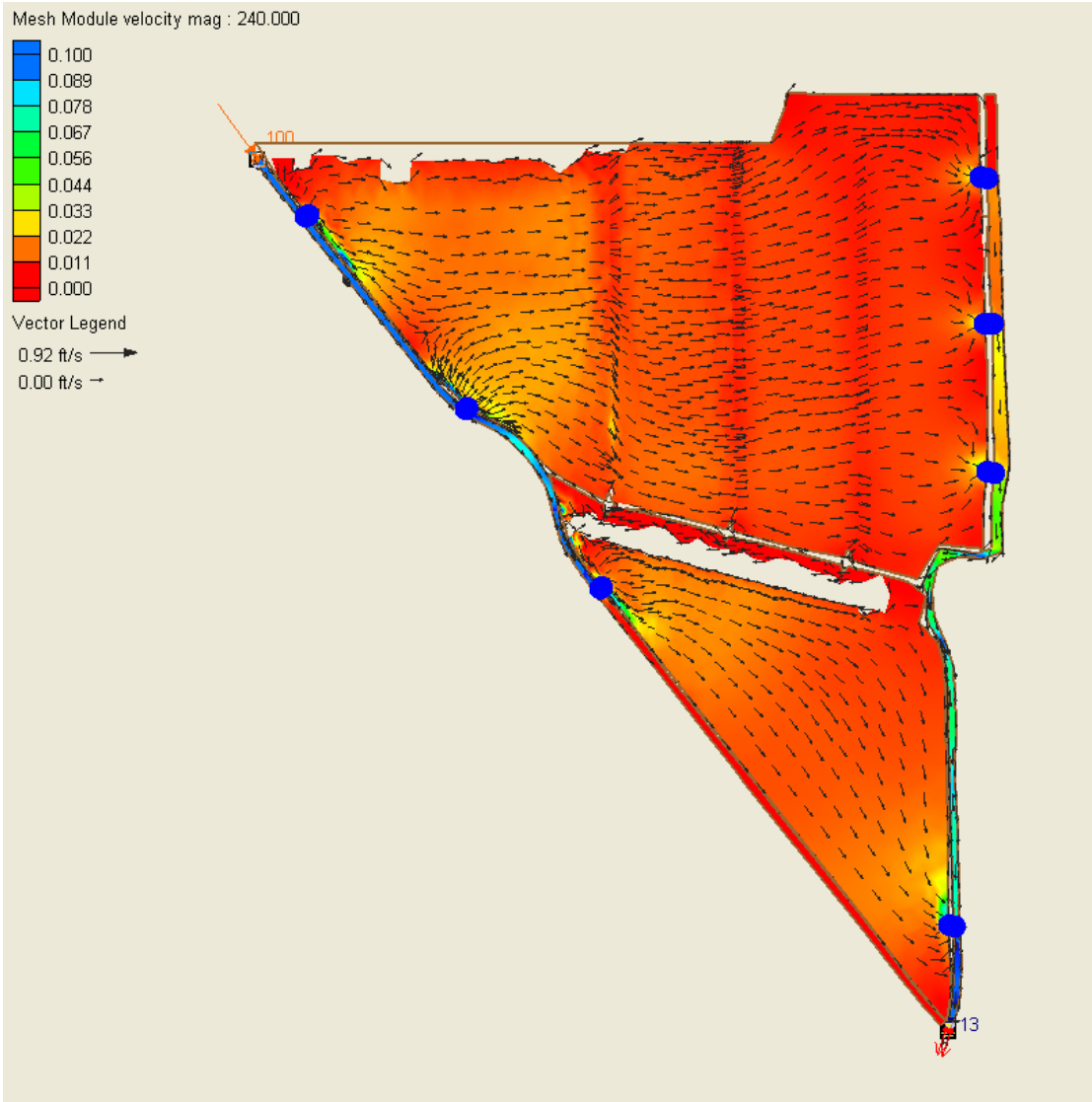


Figure 48 Velocity Plot for STA-6 (Low Flow, Existing Condition)

4.3 High Flow Condition

For the High Flow condition, the major concern is whether or not the storage capacity can accommodate the extreme storm rainfall volume and if water elevations will exceed desired values and cause over-topping of the levees.

For the High Flow condition, the Normal 500 cfs structure inflows were combined with Standard Project Storm rainfall to predict whether or not the treatment cells can accommodate the high volume of water within the treatment cells. Due to the lack of explicit representation of rainfall in the FESWMS/FLO2DH code, the rainfall rate was converted into a volumetric rate for each cell and distributed relatively uniformly over the surface area of each cell at certain nodes.

Under the High Flow condition, all outflow structures will be fully opened. G393A, B and C will be opened to drain the high flow rate through Cell 3.

Due to the high flow rate passing through the drop-inlet structures, the flow is more likely culvert controlled flow. For this reason, both the weir controlled flow and the culvert controlled flow cases were simulated and compared.

The simulation results are as follows (Figures 49 and 50):

- Weir controlled flow model run:

The supply canal water level is 18.20 ft NGVD at G600_T and 18.14 ft NGVD at G603 headwater (G392S_H).

Cell 3 water level ranges from 17.70 ft NGVD at the upstream (inflow) end to 16.70 ft NGVD at the downstream (outflow) end.

Cell 5 water surface elevation ranges from 18.0 ft NGVD at the upstream (inflow) end to 17.60 ft NGVD at the downstream (outflow) end.

- Culvert controlled flow model run:

The water surface elevations are very close to those values obtained in model runs where weir controlled flow condition was assumed at outflow structures.

The supply canal water level is 18.18 ft NGVD at G600_T and 18.15 ft NGVD at G603 headwater (G392S_H).

Cell 3 water level ranges from 17.70 ft NGVD at the upstream (inflow) end to 16.75 ft NGVD at the downstream (outflow) end.

Cell 5 water surface elevation ranges from 18.0 ft NGVD at the upstream (inflow) end to 17.65 ft NGVD at the downstream (outflow) end.

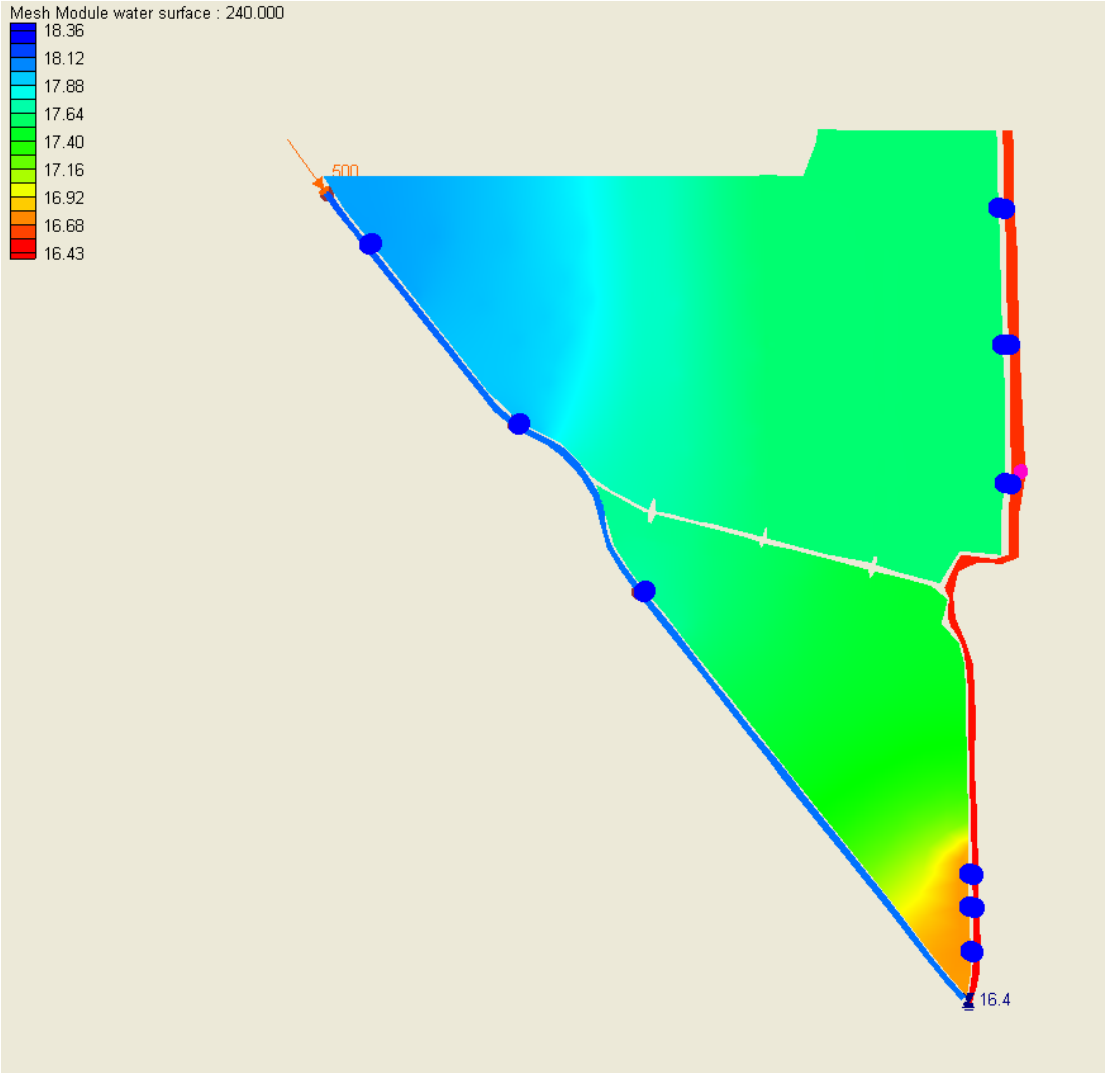


Figure 49 Water surface Elevations in STA-6 (Weir controlled flow assumed) (High Flow, Existing Condition)

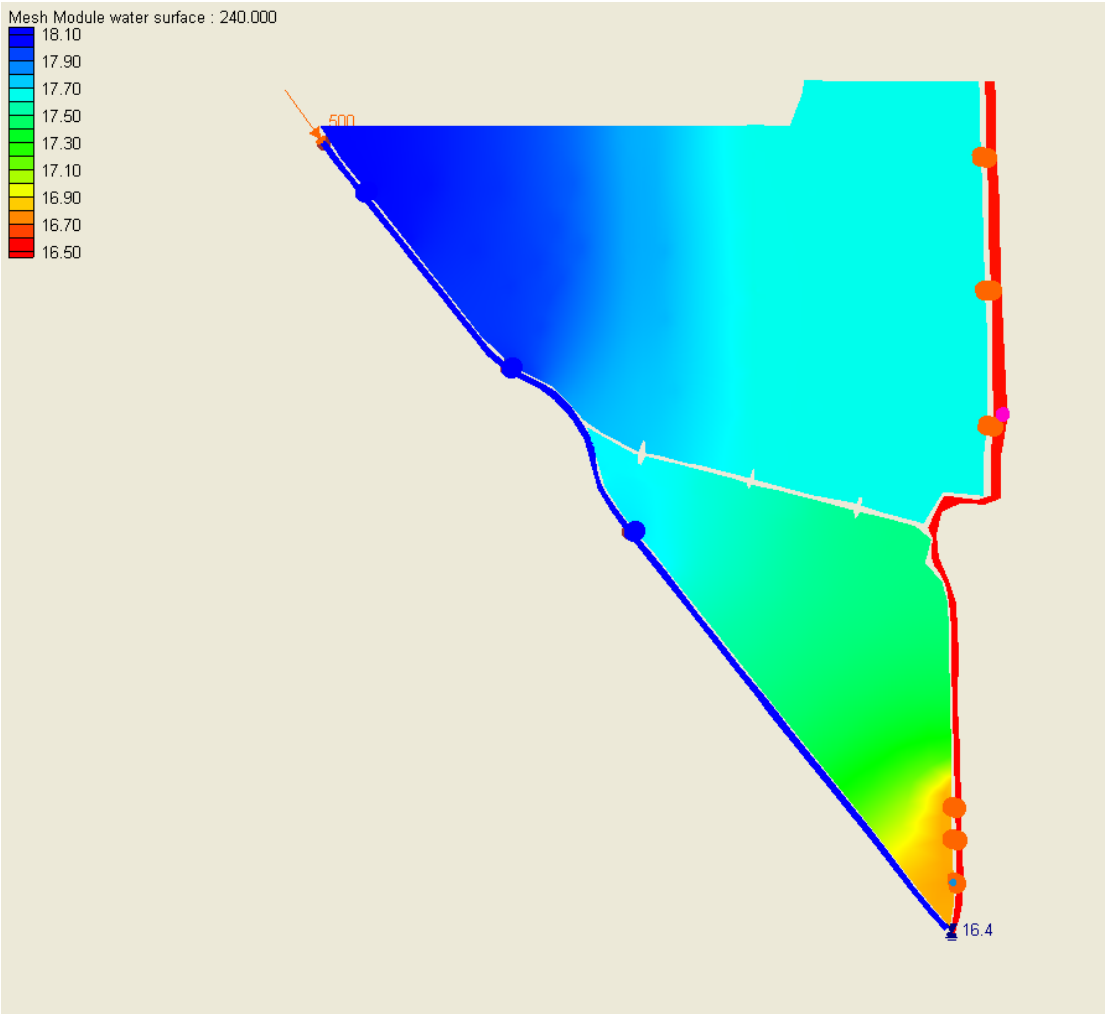


Figure 50 Water surface Elevations in STA-6 (Culvert controlled flow assumed) (High Flow, Existing Condition)

4.4 Sensitivity analysis on Existing Condition Results

Previous post-calibration sensitivity analyses have demonstrated the effects of uncertainty on model results. The Manning' n values are the most sensitive parameters affecting computed stage and velocity in the marsh areas.

The calibrated model is non-unique since the calibrated model parameters were selected through a trial and error process by using selected hydraulic data. They can be considered as a result of the integration of all factors affecting calibration and verification results (inflow, topography, vegetation, weir equations, etc.).

The impact of non-uniqueness on the Existing simulation results was investigated for the Normal Flow Condition.

The concern is that whether the non-uniqueness of the calibrated model will lead to different results for the intended applications. From previous STA studies, the following qualitative hydraulic performance measures are desirable:

- The velocity magnitude in the marsh area should be smaller than 0.1 ft/s.
- The average water depth in the marsh is between 0.5 ft and 4.5 ft.
- No obvious short-circuiting water flow routes should be presented.

If two scenario model runs have similar results on these indicators, we can assume that we obtain the same results for intended objectives.

In a sensitivity model run for the existing Design Flow condition, the Manning's n values were reduced by 30% from the calibrated n values for Cell 3 and 5 vegetations. A comparison of velocity magnitude and water depth shows that there are very small changes in velocity magnitudes (about 0.001 ft/s) (Figure 51). So the same conclusion can be made about velocity distribution in the marsh area. As for the water depth, the change ranges from about -0.3 ft to 0.001 ft (Figure 52). This also will not change the conclusion on water depth distribution in the marsh area.

In conclusion, even the Manning' n values can be varied from the calibrated values by a certain range, the model runs for existing conditions still lead to similar results. The problem of equifinality (many different model structures and parameter sets may give similar calibration results based on available observation data) can be further investigated by uncertainty analyses.

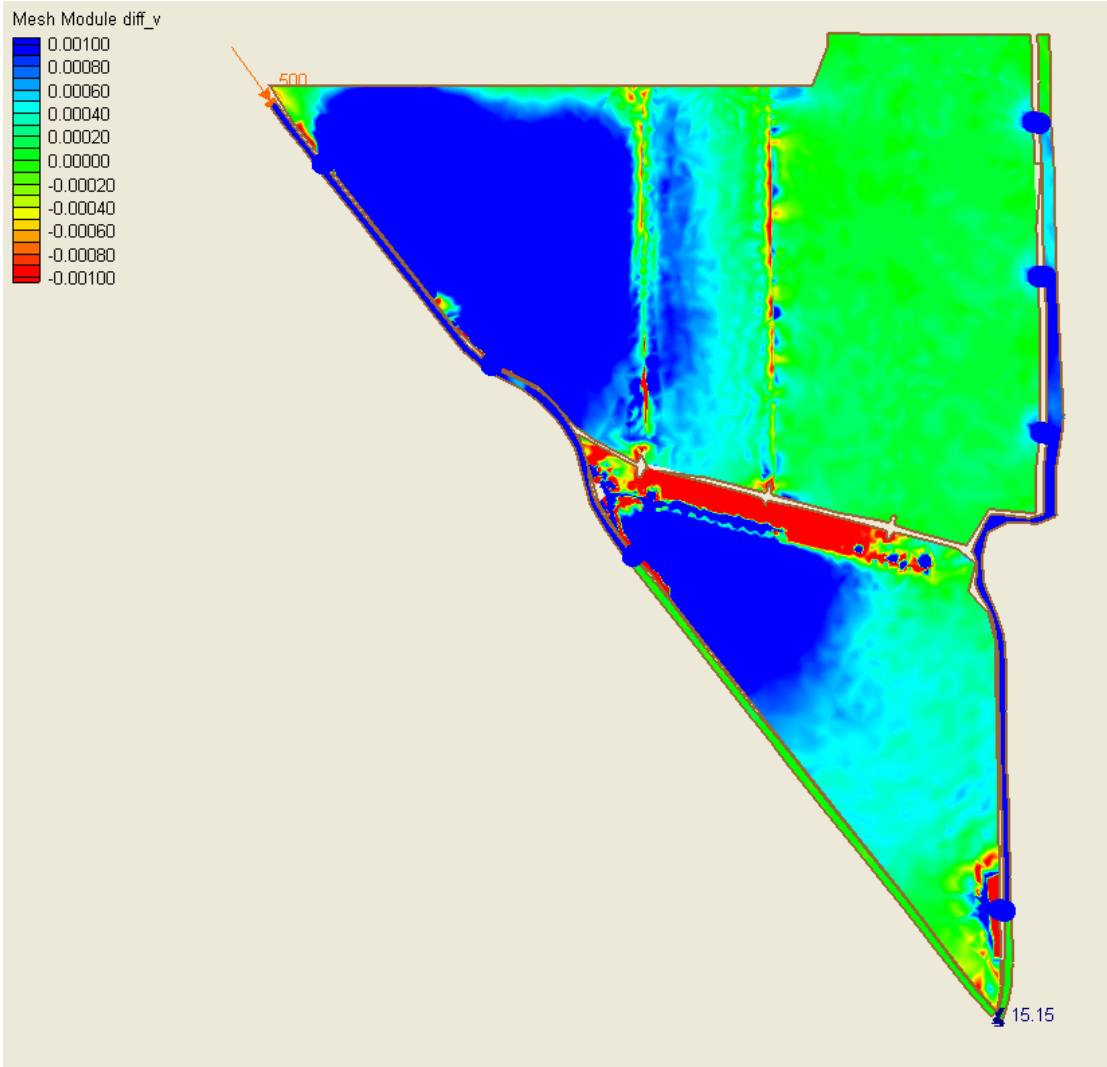


Figure 51 Changes in Velocity Magnitude (ft/s) with reduced Manning's n value (-30%)

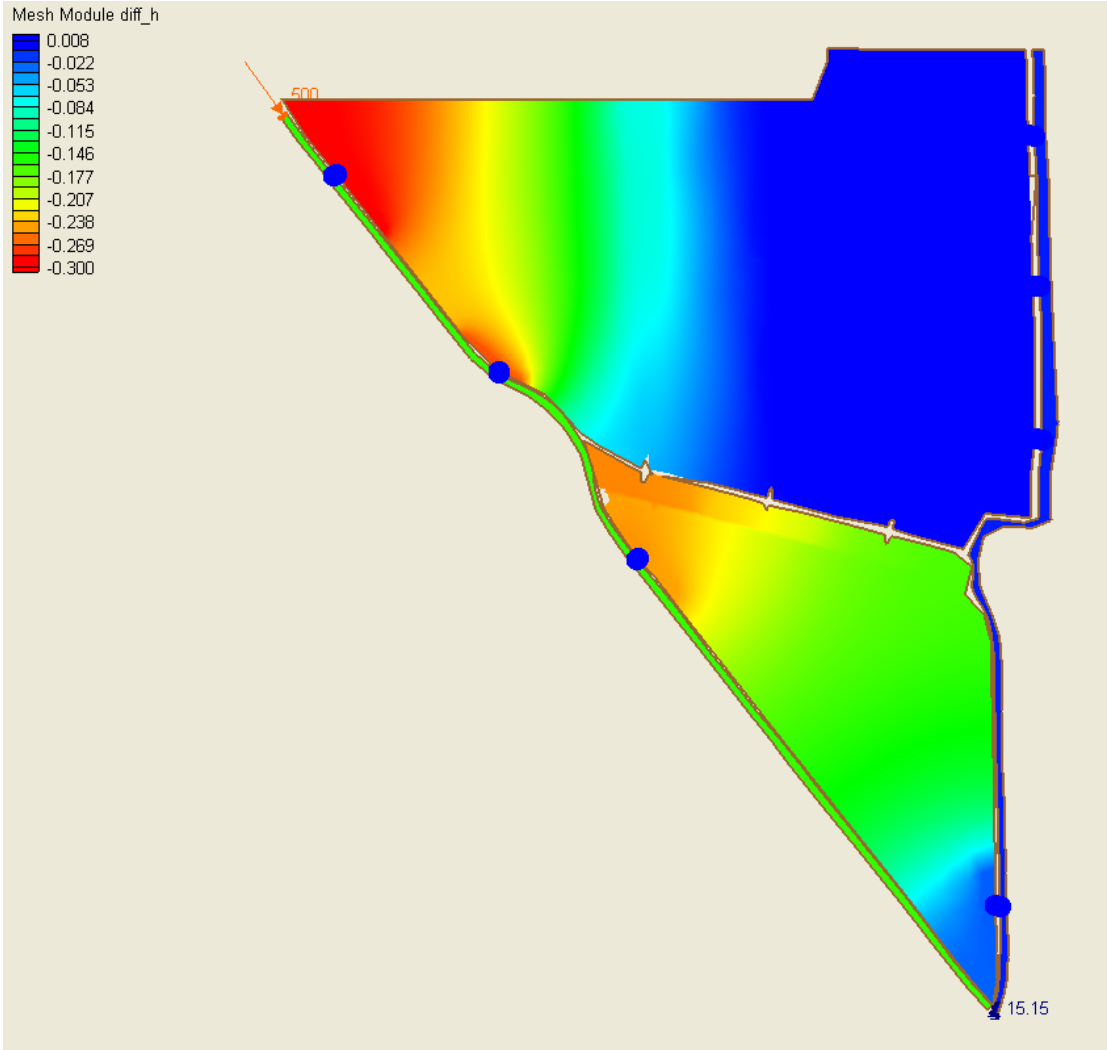


Figure 52 Changes in Water Depth (ft) with reduced Manning's n value (-30%)

5. Conclusions and Recommendations

A two-dimensional hydrodynamic model of STA-6 has been developed. The latest information on vegetation and topography were used in building the model. Under transient flow condition, the hydraulics of STA-6 as a whole was simulated simultaneously as a linked model. Historic time series stage data were used in model calibration and verification, and sensitivity analyses were conducted for some important model parameters.

Future efforts to improve the STA-6 2-D hydraulic model can be directed to the following issues:

- Interior water level and velocity data collection for calibration/verification.

- Experiment or field study on vegetative friction coefficients.
- Integrated model to address seepage issues.
- Continue to improve the accuracy of structure flow equations (pumping station, weirs and gated culverts).

References

[1] South Florida Water Management District (SFWMD), Stormwater Treatment Area 6 Operation Plan, April, 2004.

[2] Scott Huebner, Water Budget analysis for stormwater Treatment area 6, section 1, Technical Publication EMA #391. South Florida Water Management District, West Palm Beach, FL. February 2001.

[3] Scott Huebner, Water Budget analysis for stormwater Treatment area 6, section 1, Technical Publication EMA #408. South Florida Water Management District, West Palm Beach, FL. January 2003.

[4] Eric D. Swain, Melinda A. Wolfert, Jerad D. Bales, and Carl R. Goodwin, Two-Dimensional Hydrodynamic Simulation of Surface-Water Flow and Transport to Florida Bay through the Southern Inland and Coastal Systems (SICS), U.S. Geological SurveyWater-Resources Investigations Report 03-4287, 2004.

[5] Betsy Lingsay,Inc. prepared for South Florida Water Management District, Topographic Survey of Stormwater Treatment Area 6. April 2004.

[6] Hydrologic Associates USA,inc., prepared for South Florida Water Management District, Calibration of Structure G-602 of Stormwater Treatment Area 6, June 2004.

[7] Hydrologic Associates USA,inc., prepared for South Florida Water Management District, Calibration of Structure G-603 of Stormwater Treatment Area 6, July 2003.

[8] Hydrologic Associates USA,inc., prepared for South Florida Water Management District, Calibration of Structure G-354 of Stormwater Treatment Area 6, September 2003.

[9] Hydrologic Associates USA,inc., prepared for South Florida Water Management District, Calibration of Structure G-393B of Stormwater Treatment Area 6, September 2003.

[10] U.S. Department of the Interior, Bureau of Reclamation. Water Measurement Manual, USBR, 1997.

[11] User's Manual for FESWMS Flo2DH, Release 3, Publication No. FHWA-RD-03-053, Federal Highway Administration, September 2002.

[12] South Florida Water Management District (SFWMD), Two dimensional Hydraulic Analysis for Cell 3 of STA-2, an SAV-dominated Treatment Cell, March 21, 2001.

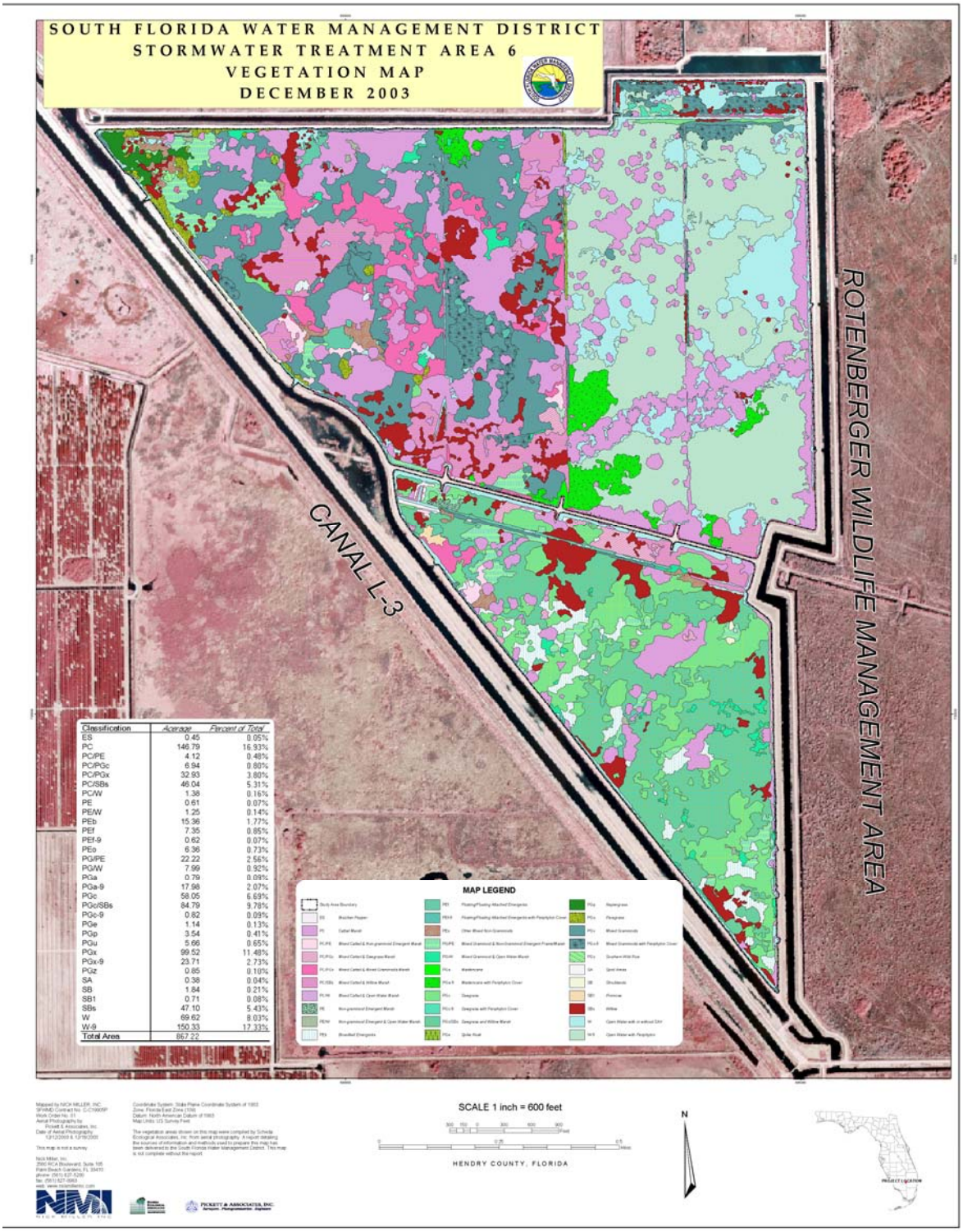
[13] Burns & McDonnell, 2D Hydrodynamic Modeling Stormwater Treatment Area 3/4 and East WCA-3A Hydropattern Restoration, January 2000.

[14] Burns & McDonnell, Long Term Plan for Achieving Water Quality Goals, October 27, 2003.

[15] Sutron Corporation, prepared for South Florida Water Management District, STA-2 Hydraulic Modeling Task 1.4 Final Report, May 28, 2004

Appendixes

Appendix I. STA-6 Vegetation Map (December 2003)



Classification	Acreage	Percent of Total
ES	0.45	0.05%
PC	146.79	16.93%
PC/PE	4.12	0.48%
PC/PGc	6.94	0.80%
PC/PGx	32.93	3.80%
PC/SBs	46.04	5.31%
PC/W	1.38	0.16%
PE	0.61	0.07%
PE/W	1.25	0.14%
PEb	15.36	1.77%
PEf	7.35	0.85%
PEf-9	0.62	0.07%
PEo	6.36	0.73%
PG/PE	22.22	2.56%
PG/W	7.99	0.92%
PGa	0.79	0.09%
PGa-9	17.98	2.07%
PGc	58.05	6.69%
PGc/SBs	84.79	9.78%
PGc-9	0.82	0.09%
PGe	1.14	0.13%
PGp	3.54	0.41%
PGu	5.66	0.65%
PGx	99.52	11.48%
PGx-9	23.71	2.73%
PGz	0.85	0.10%
SA	0.38	0.04%
SB	1.84	0.21%
SB1	0.71	0.08%
SBs	47.10	5.43%
W	69.62	8.03%
W-9	150.33	17.33%
Total Area	867.22	