# Everglades Agricultural Area Regional Feasibility Study

## Deliverable 3.2 h – Final Optimum Allocation of Phosphorus and Hydraulic Loading to the Existing STAs, Compartment B & C, and the A-1 Reservoir, and Optimum Canal Improvements Associated with Optimum Allocation

(Work Order No. CN040912-WO04 Phase 2)

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# EVERGLADES AGRICULTURAL AREA REGIONAL FEASIBILITY STUDY

## DELIVERABLE 3.2 h– FINAL OPTIMUM ALLOCATION OF PHOSPHORUS AND HYDRAULIC LOADING TO THE EXISTING STAS, COMPARTMENTS B AND C, AND A-1 RESERVOIR, AND OPTIMUM CANAL IMPROVEMENTS ASSOCIATED WITH OPTIMUM ALLOCATION

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# 1.0 INTRODUCTION

This document and the analyses it summarizes were prepared by A.D.A. Engineering, Inc. (ADA) with assistance from Burns & McDonnell Engineering Co., Inc. under contract to South Florida Water Management District (SFWMD). The conduct of this work was authorized by SFWMD through its March 27, 2005 issuance of Work Order No. CN040912-WO04 to ADA.

## 1.1 Background

Under the Everglades Construction Project (ECP), the South Florida Water Management District (SFWMD) has constructed several stormwater treatment areas (STAs) and the U.S. Army Corps of Engineers has constructed STA-1E to help improve the quality of waters released to the Everglades Protection Area (EPA). In addition to the existing STAs, SFWMD is planning certain STA expansions and enhancements, Everglades Agricultural Area (EAA) canal improvements, construction of the EAA Storage Reservoir Project, and other EAA improvements (see **Figure 1**). With recognition of these planned improvements, the EAA Regional Feasibility Study (RFS) will evaluate alternatives for redistributing inflow volumes and phosphorus loads to the various STAs to optimize phosphorus removal performance. This study is not intended to define the final arrangement, location or character of these proposed projects but is a fact-finding exercise to develop the information necessary for the subsequent planning, design and construction of these future projects.

## 1.2 Scope of Work

This document was prepared in support of Task 3 “Optimum Allocation of Phosphorus and Hydraulic Loading to the Existing STAs and A-1 Reservoir, and Optimum Canal Improvements Associated with Optimum Allocation” and Task 4 “Detailed Alternative Analysis” of the SFWMD Work Order No. CN040912-WO04. The overall objective of the analyses reported herein is to evaluate the redistribution of hydraulic and total phosphorus loads to the STAs (both existing and the currently planned STA-6 Section 2, full conversion of Compartments B and C of the Talisman Land Exchange to use in STAs) to optimize phosphorus reduction, given the presence of the Everglades Agricultural Area Storage Reservoir (EAASR) Compartment A-1. This analysis is specific to the period 2010-2014 (following completion of the above identified projects, but prior to the completion of the planned EAASR Compartment A-2), and addresses Alternatives 1 through 5.

Estimates of the overall flow rates during peak flow conditions, inflow volumes and TP loads to be accommodated in the various STAs were developed under Task 1 of Contract CN040912-WO04. Basins considered include the following:

- C-51 West Canal
- S-5A (West Palm Beach Canal)
- Chapter 298 Districts:
  - East Beach Water Control District
  - East Shore Water Control District
  - 715 Farms (State Lease No. 3420)
  - South Shore Drainage District
  - South Florida Conservancy District, Unit 5 (S-236 Basin)
- S2/S-6/S-7 (Hillsboro and North New River Canals)
- S-3/S-8 (Miami Canal)
- C-139 and C-139 Annex

- L-8 Canal
- Lake Okeechobee deliveries south to the STAs and Everglades

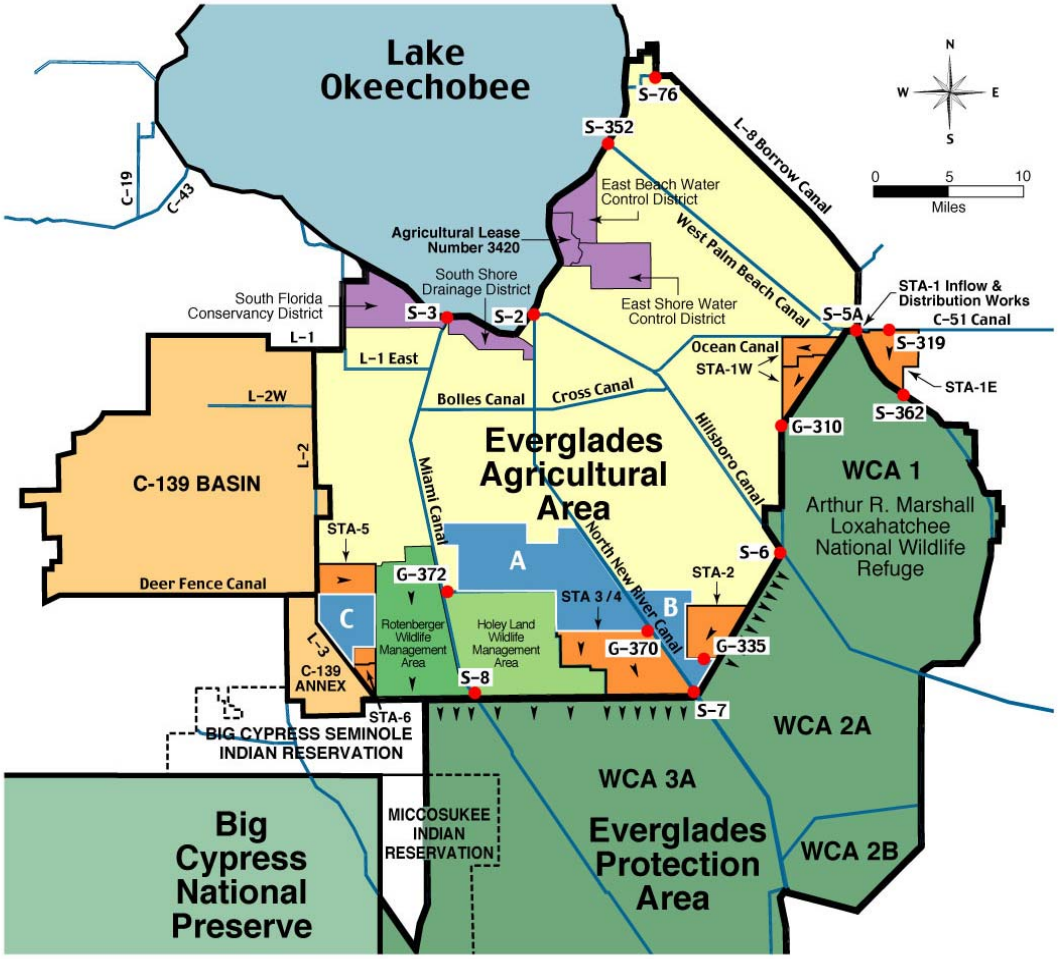



**Figure 1-1 – EAA Project Area**

## 1.3 Analytical Methods for Determining Optimum Canal Improvements

The determination of optimum canal improvements was based on hydrologic and hydraulic modeling performed as part of Phase 1 Task 3 of this project (authorized by SFWMD Work Order CN040912-WO03). Phase 1 Task 3 defined the operating strategy for optimizing STA performance with existing EAA canals. Phase 1 Task 3 utilized a MIKE 11 hydrologic/hydraulic model of the EAA canals system as of the end of 2006. The model includes all the major EAA canals, the L-canals, the existing STAs, STA 2, 5, and 6 expansions as of December 2006, and selected minor private farm canals. The Phase 1 Task 3 work was conducted assuming a uniform rate of runoff equal to 3/8 inch runoff from all EAA farms plus analysis of 3/4 inch runoff from selected basins. This analysis of 2006 conditions concluded that there were limited possibilities for re-directing runoff from the S-5A basin to the S-7 basin. After completion of the

Phase 1 Task 3 report, ADA conducted another simulation for 2006 conditions and used 3/4" runoff in all basins of the EAA except those basins discussed in Section 1.5.1 below.

The analysis for this task is specific to the period 2010 to 2014. It assumes that Compartment B, Compartment C, and the A-1 Reservoir will be in place. This hydraulic assessment will define the hydraulic response of alternative hydraulic configurations of EAA canals, Compartment B, Compartment C, and the A-1 Reservoir. Effects of alternative internal configurations of Compartment B and Compartment C will be defined by this task. This task will also define the hydraulic response of alternative canal enlargements and potential new gates to redistribute basin runoff.

## 1.5 Reference Information

This section summarizes previous studies, reports, and data employed in the conduct of the analyses presented herein.

### 1.5.1 Inflow Rates

The analysis will be conducted assuming 3/4 inch runoff from EAA farms. Runoff from approximately 200 farms has been aggregated into runoff inputs from approximately 120 locations (see **Figure 1-2** and **Table 1-1**). Each of these locations represent runoff from one or more permitted EAA farms and/or water control districts. Runoff from the C-139 Basin, the C-139 Annex, C-51W Basin, and the L-8 was determined using alternative means, as described below. Measured runoff as described in Phase 2 Deliverable 1.3 (Burns & McDonnell, 2005a) and permitted discharges from those basins were reviewed. The runoff rates presented in **Table 1-2** were selected based on this review. Certain Chapter 298 Districts have historically discharged to Lake Okeechobee and have been partially re-directed to discharge to EAA canals. The runoff rates used for these 298 Districts have been based on permitted pump station capacities since runoff rates are higher than the 3/4 inch rate assumed for most of the EAA. Table 1-3 presents the pump station capacities for the 298 Districts.

**Table 1-1 – EAA Runoff INSERT**

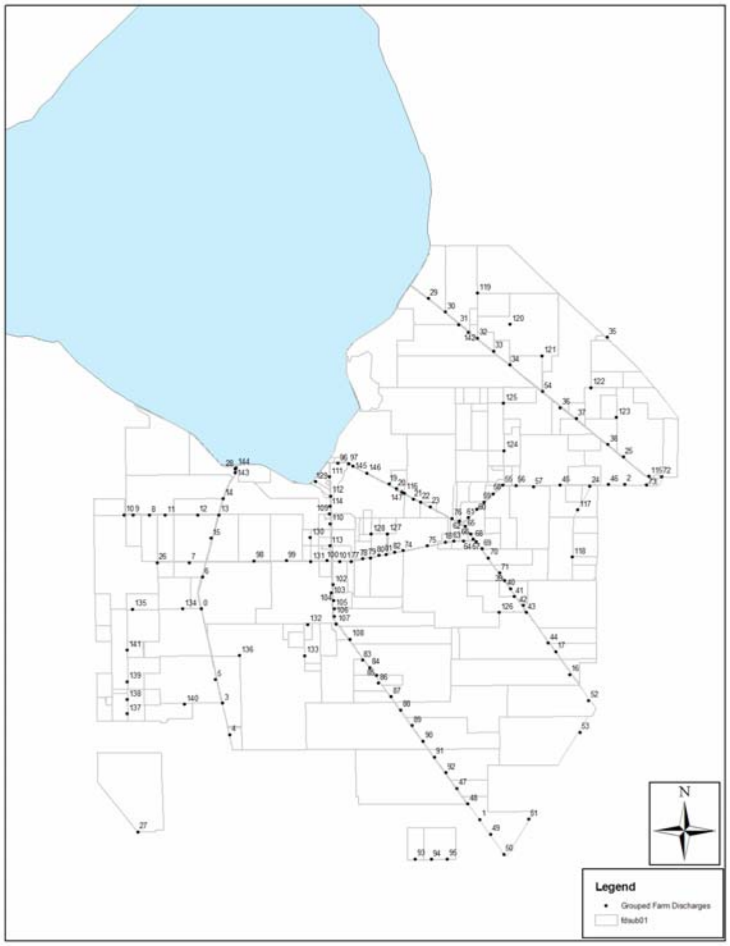



**Figure 1-2 – Locations of Runoff Inputs**

**Table 1-2 – Runoff Rates Assumed for C-139, C-139 Annex, C-51W, and L-8 Basins**

| Basin | Runoff Rate (cfs) | Source |
|---|---|---|
| C-139 | 2,000 | Burns & McDonnell, 2005a; URS, 2005 |
| C-139 Annex | 452 | Burns & McDonnell, 2005a; URS, 2005 |
| C-51W | 2,000 | Burns & McDonnell, 2005a |
| L-8 | 1,500 | Burns & McDonnell, 2005a |

**Table 1-3 – Runoff Rates for Chapter 298 Districts and Receiving Water Body**

| 298 District | Flow at 3/4" Runoff (cfs) | Permitted Pump Capacity (cfs) |
|---|---|---|
| SSDD - Miami Canal | 77 | 178 |
| SFCD - Miami Canal | 75 | 504 |
| EBWCD #3 - WPB | 206 | 338 |
| ESWCD PS2 - Hillsboro | 260 | 439 |

### 1.5.2 Inflow Volumes, TP Concentrations and TP Loads

Inflow volumes, Total Phosphorus (TP) concentrations and TP loads employed in this analysis are based on information presented in the following reports, all prepared for the South Florida Water Management District by Burns & McDonnell Engineering Co., Inc. under subcontract to ADA Engineering, Inc. as elements of Task 1 of the scope of work under District Contract CN040912-WO04:

- Deliverable 1.1.2: *Evaluation of 2006 Hydrologic Simulation Results*, Final Report dated June 27, 2005;
- Deliverable 1.2A: *Inflow Data Sets for the Period 2010-2014,* Draft Report dated August 23, 2005;
- Deliverable 1.3.2: *Historic Inflow Volumes and Total Phosphorus Concentrations by Source,* Final Report dated June 27, 2005;
- Deliverable 1.4.2: *Methodology for Development of Daily Total Phosphorus Concentrations,* Final Report dated June 30, 2005;
- Deliverable 1.5.2: *Inflow Data Sets for the Period 2006-2009,* Final Report dated August 9, 2005;
- Deliverable 2.2: *Optimum Allocation of Loads to the STAs for the Period 2006-2009,* Final Report dated September 7, 2005.

### 1.5.3 Basic Designs of Proposed STA Expansions

Information on the presently planned configuration and basic layout and design of STA-6 Section 2; Cell 4 of STA-2; and the third flow-way of STA-5 was taken from the following documents:

- *Basis of Design Report (BODR) Stormwater Treatment Area 6 – Section 2 and Modifications to Section 1*; prepared for the South Florida Water Management District by URS Corporation under Contract CN040936-WO02; June 1, 2005;
- *Basis of Design Report (BODR) STA-2/Cell 4 Expansion Project*; prepared for the South Florida Water Management District by Brown & Caldwell under Contract CN040935-WO04; May 12, 2005;
- Draft Basis of Design Report (BODR) Stormwater Treatment Area 5 Flow-way 3; prepared for the South Florida Water Management District by URS Corporation under Contract CN040936-WO05; April 20, 2005.

No information is presently available for the planned configuration and basic layout and design of the full conversion of Compartments B and C of the Talisman Land Exchange to use as stormwater treatment areas. The layout and configuration of those expanded stormwater treatment areas assumed for use in this analysis is described in Part 2 of this document.

The layout, configuration and operation of the EAASR Compartment A-1 assumed for use in this analysis is based on discussions with Black & Veatch and Acceler8 staff and is described in Part 2 of this document.

### 1.5.4 Previous Studies and Reports

Certain background data and information discussed in this document was taken from the following previous studies and reports:

- (Draft) *Supplemental Analysis, Everglades Protection Area Tributary Basins*, prepared for the Everglades Agricultural Area Environmental Protection District by Burns & McDonnell; March 2, 2005 (hereinafter referred to as the Supplemental Analysis);
- Final Report, *Everglades Protection Area Tributary Basins, Long-Term Plan for Achieving Water Quality Goals*; prepared for the South Florida Water Management District by Burns & McDonnell; October, 2003 (hereinafter referred to as the Long-Term Plan), together with such modifications to the Long-Term Plan that are embodied in a revised Part 2 (dated November, 2004) submitted to the Florida Department of Environmental Protection (FDEP), and approved by FDEP in December, 2004;
- *Basin-Specific Feasibility Studies, Everglades Protection Area Tributary Basins; Evaluation of Alternatives for the ECP Basins*; prepared for the South Florida Water Management District by Burns & McDonnell; October 23, 2002 (hereinafter referred to as the BSFS Evaluation of Alternatives).
- *Addendum to Design Documentation Report, Stormwater Treatment Area 1 East*; prepared for the Jacksonville District, U.S. Army Corps of Engineers by Burns & McDonnell; November 2000;
- *(Draft) Stormwater Treatment Area 1-East (STA-1E) Water Control Plan*, Jacksonville District, U.S. Army Corps of Engineers; August, 2005;
- *(Draft) Design Analysis Report for the STA*-1E Cells 1-2 PSTA/SAV Field-Scale Demonstration Project, Palm Beach County, Florida; prepared for the Jacksonville District, U.S. Army Corps of Engineers by SAIC Engineering, Inc.; June 28, 2005.
- *STA 1W, 2, 3/4, 5, and 6 Operating Plans*; prepared by SFWMD, 2000 (STA 5), 2001 (STA 2), 2004 (STA 1W, STA 3/4, STA 6)
- *STA 1E Operating Plan* – draft; prepared by SFWMD, In Review.

# 2.0 OVERVIEW OF ALTERNATIVES

The long-term Everglades water quality goal is for all discharges to the Everglades Protection Area (EPA) to achieve and maintain compliance with water quality standards, including phosphorus, as established in Rule 62-302.540, F.A.C., in the EPA. Substantial progress towards reducing phosphorus levels discharged into the EPA has been made by the State of Florida and other stakeholders. The combined performance of the source controls in the EAA and the STAs of the Everglades Construction Project (ECP) has exceeded expectations. In addition, some source control measures have been implemented in urban and other tributary basins included in the Everglades Stormwater Program (ESP). Nonetheless, additional measures are necessary to achieve the Everglades water quality goal.

The projects in the October 27, 2003 *Everglades Protection Area Tributary Basins Long-Term Plan for Achieving Water Quality Goals* (Long-Term Plan) were designed to achieve compliance with the water quality standards for the EPA by December 31, 2006. One of the key assumptions during the development of the Long-Term Plan was that Compartments B and C (see Figure 1) would be under consideration for use as part of the EAA Storage Reservoir Project through Fiscal Year (FY) 2010 and for this reason should not be considered for other Everglades restoration uses until FY 2011. Subsequent to completion of the Long-Term Plan, it was determined that all of the EAA Storage Reservoir Project's water storage goals could be achieved on Compartment A, and that Compartments B and C would not be needed to meet the storage objectives of the EAA Storage Reservoir Project (Phase 1 and 2). In light of the recent availability of land in Compartments B and C, construction of additional stormwater treatment areas is proposed in association with STA-2, STA-5 and STA-6 to assist the STAs in improving water quality entering the EPA. It is also the SFWMD's intent to construct additional stormwater treatment areas on the remaining acreage of Compartments B and C.

The intent of this Feasibility Study is to determine the optimal configuration of stormwater treatment areas on Compartments B and C with the objective of assisting the STAs in improving water quality in the EPA. The Study will take into account the anticipated flows and phosphorus loads to the existing STAs, the currently planned STA expansions and enhancements, the EAA Canal Improvements, the Everglades Agricultural Area Storage Reservoir Project and other currently planned improvements in the EAA region.

The alternatives described below are intended to address 2010 conditions. Alternatives that appear to be feasible from a water quality and hydraulics perspective will be analyzed using the evaluation methodology and evaluation criteria developed as part of Phase 1, Task 2. The alternatives evaluation will also include assessment of project costs, land acquisition, and implementation schedules. This report presents results for hydraulic modeling of the proposed alternative modifications.

**Alternative 1**

The main purpose of this alternative is to re-direct runoff from the S-5A basin to the S-2-S-7 basin. Components are summarized below and are shown in **Figure 2-1**:

1. S-5AW will be closed and S-5AE will be doubled.
2. Addition of new gate in West Palm Beach (WPB) Canal to separate the north S-5A drainage basin.

3. Addition of new Canal from WPB Canal to the Sam Senter Canal.
4. Expanded Sam Senter Canal.
5. Addition of new gate in the Hillsboro Canal south of the Cross Canal. This gate will divert a portion of S-2/S-6 flows to the Cross Canal and then to the North New River Canal.
6. Expand capacity of the Ocean Canal from the Sam Senter Canal to the Hillsboro Canal.
7. Expand capacity of the Hillsboro Canal from the Ocean Canal to the Cross Canal.
8. Expand capacity of the Cross Canal and enlarge farm bridges along the Cross Canal.
9. Expand capacity of North New River Canal (NNRC).
10. Addition of A-1 Reservoir and Compartment B with inflow pumps on the NNRC (3000 cfs for A-1, 1600 cfs for Compartment B).
11. The Compartment C STA receives runoff from only C-139 and C-139 Annex.
12. The new STA-2 Cell 4 is connected to Compartment B and is not connected to cells 1, 2, and 3 of STA 2.

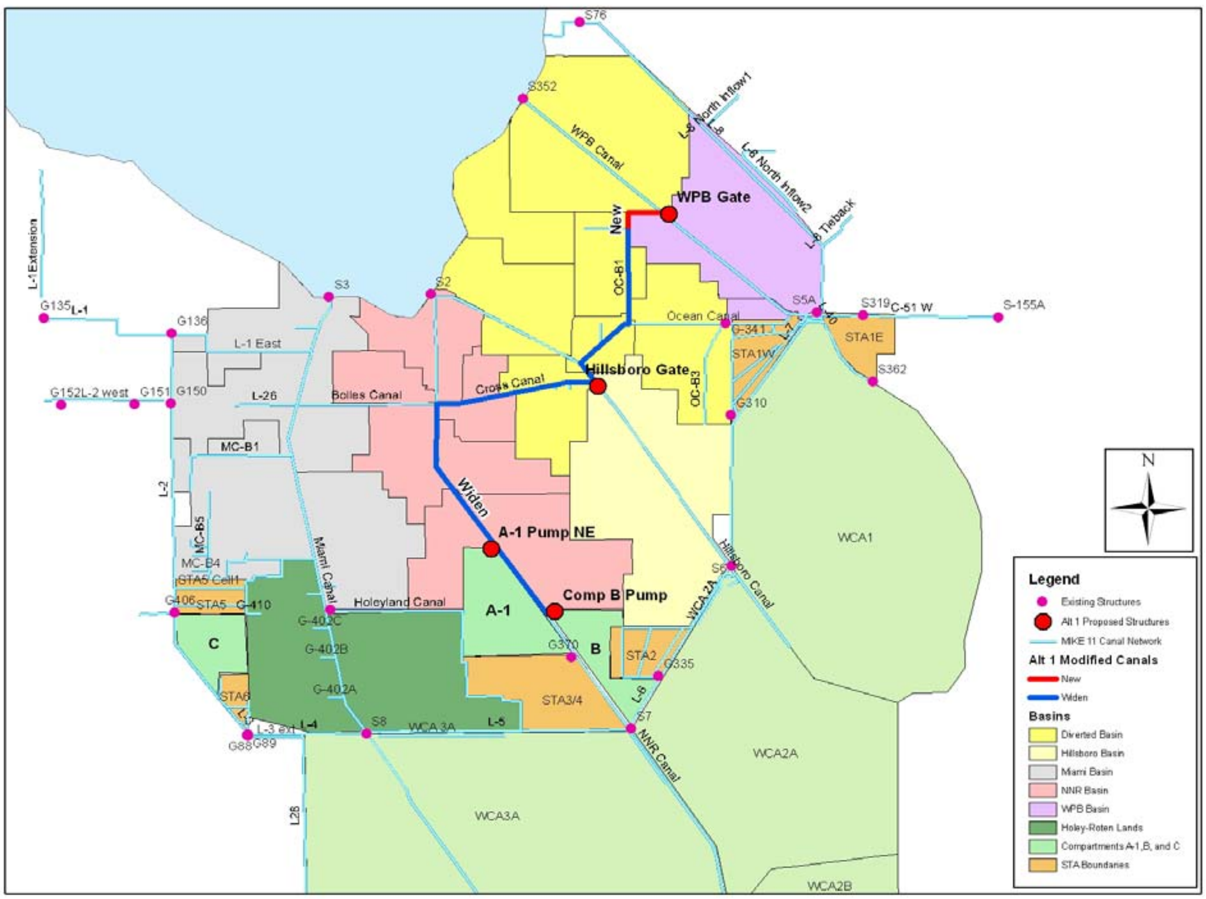



**Figure 2-1 – Alternative 1**

### Alternative 2

The purpose of this alternative is to minimize inter-basin transfers while achieving low TP concentrations in STA discharges. A summary of the components of this alternative is presented in **Figure 2-2** and is described below:

1. Enlargement of S-5AE to twice the existing capacity and close Structure S-5AW.
2. Enlargement of the L-7 Borrow Canal and separation of the Borrow Canal from the Loxahatchee National Wildlife Refuge (see figure below).
3. Installation of a gate on the east bank of L-7 near G-251 to allow STA 1W discharges to enter the north portion of the Refuge during periods when discharges are within acceptable concentration limits.
4. Replacement of G-338 (near S-6) with a new gated control structure that would allow L-7 flows to be delivered to the STA-2 Inflow Canal. (see **Figure 5.7** for a detail of the area around G-338 and S-6).
5. Construction of a new canal from the STA-2 Inflow Canal to Compartment B.
6. Removal of G-336G on L-6.
7. Enlargement of the Cross Canal and the North New River Canal.
8. A new inflow pumping station on the NNRC to the A-1 Reservoir. This is the pump station anticipated by Black & Veatch, however the pump capacity could be different than what is assumed by Black & Veatch.
9. STA-1 complex treats average annual amount of 35,000 ac-ft. L-8 runoff (approximate amount that historically discharged to the Refuge).
10. Hydraulic connection of the new STA-2, Cell 4 to the new Compartment B STA, i.e., no longer hydraulically connected to STA-2 Cells 1, 2, and 3.
11. Enlargement of the Ocean Canal and Hillsboro Canal.
12. Addition of a new gated structure on the Hillsboro Canal to limit flow into STA-2. This gate will close when the S-6 flow exceeds approximately 1300 cfs, such that the combined STA 2 inflow is no greater than 4720 cfs.
13. Modification of the operations of S-5A, G-300, G-370, G-335, G-302, and S-155A.
14. Modification length and cross sections of the STA 2 Cell 4 Discharge Canal.

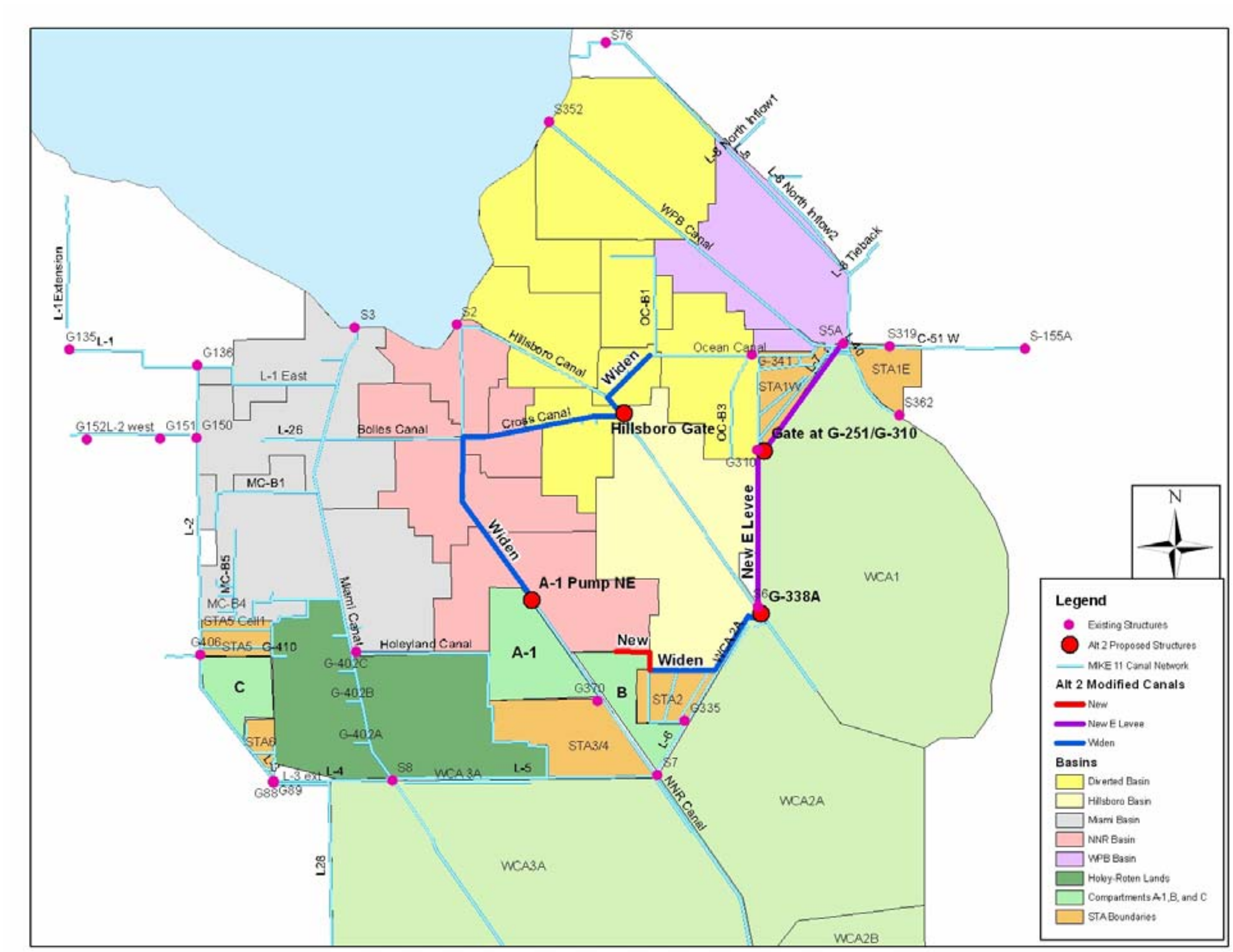



**Figure 2-2 – Alternative 2 Features**

### Alternative 3

This is a modification of Alternative 1 above. The purpose of this alternative is to convey the optimal amount of water (both agricultural runoff and lake releases) to Compartment C to balance the flows and loads across the six STAs. This Alternative includes all features of Alternative 1 (**see Figure 2-1 above**), but also includes a pump station in the Manley Ditch (called MC-B5 in the Mike 11 network) to convey additional water to L-2 just north of STA 5. **Figure 2-3** shows the STA 5/6 complex.

Alternative 3 was modeled with and without an outflow pump in the STA 6 Discharge Canal. Alternative 3A includes the pump while Alternative 3B does not. The purpose of this pump will be to enhance flow-through capacity of the STA 5/Compartment C/STA 6 complex.

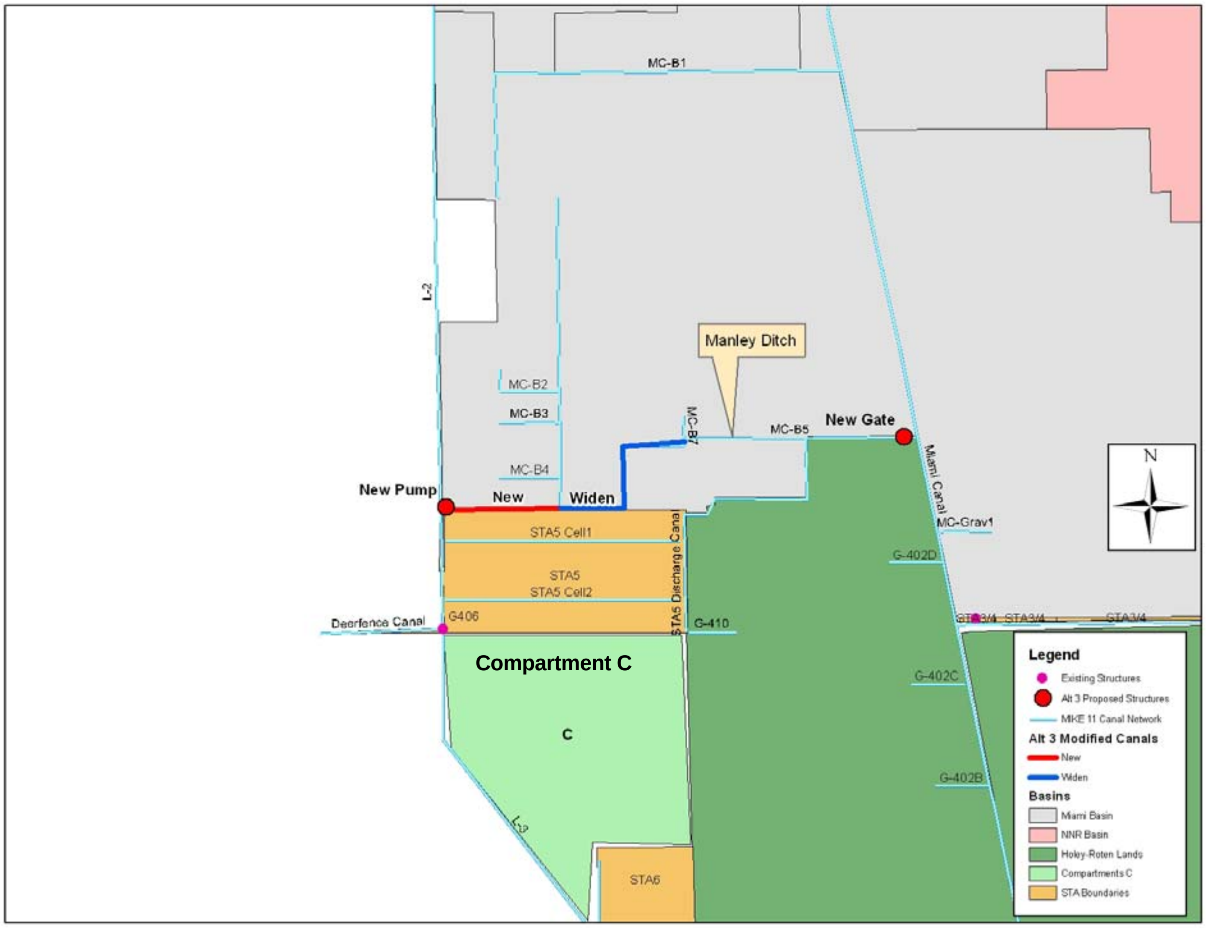



**Figure 2-3. Alternative 3 Manley Ditch Modifications**

**Alternative 4**

This alternative will be a mix of components for Alternatives 1, 2, and 3. The objective of this alternative is to take the best features of previous alternatives to reduce the overall cost while maintaining desired nutrient removal performance. Details are presented below:

1. STA 1E will be enlarged to incorporate Section 24 of the Acme Improvement District.
2. Enlargement of the L-7 Borrow Canal and separation of the Borrow Canal from the WCA 1, the Loxahatchee National Wildlife Refuge (see **Figure 2-2**).
3. Addition of new gate in the Hillsboro Canal south of the Cross Canal. This gate will divert a portion of S-2/S-6 flows to the Cross Canal and then to the North New River Canal.
4. Connect the Manley Ditch to the STA 5 Seepage Canal and construct a 600-cfs pump station to increase runoff to the STA 5/Compartment C/STA 6 complex.
5. Enlarge Cross and Bolles Canals.
6. Limited widening of the North New River Canal.
7. The A-1 Reservoir will receive water from existing G-370 and a new 3000 cfs pump station on the NNRC.
8. Construct a siphon from the A-1 Reservoir to Compartment C.

**Alternative 5**

Alternative 1 with a modification of Compartment B internal flow patterns to keep STA-2 Cell 4 hydraulically linked to STA-2 Cells 1, 2, and 3, i.e., operating separately from the remainder of Compartment B.

# 3.0 DETAILED DESCRIPTION OF ALTERNATIVES AND FINDINGS

This section provides a more detailed description of how the alternatives were modeled with MIKE 11 and presents results of the hydraulic and nutrient removal modeling.

## 3.1 Alternative 1

### 3.1.1 Detailed Description of Alternative 1

The implementation of this alternative in MIKE 11 is described below:

1. S-5AW gates are assumed to be closed, and S-5AE gates are assumed to be doubled from two to four gates. Another gate was added to S-155A because target flows could not be achieved with the existing gates.
2. The WPB Gate was called Alt1_WPBC and will be located at chainage 54,740 ft. There will be two 20-foot wide gates with an invert elevation at -2 ft NGVD. The gates were assumed to open to elevation 8.5 ft NGVD (total height = 10.5 ft). The gates were initially programmed to open when the upstream water level exceeded 11.5 ft NGVD and to open no more than once per hour.
3. A new branch called Alternative1 was added between the WPB Canal at WPB chainage 52,861.3 ft and the Sam Senter Canal (called OC-B1 in MIKE 11) at OC-B1 chainage 13,138.1 ft. Dimensions for this new canal are presented in **Table 3-1**.
4. The Sam Senter Canal cross section area was changed as shown in **Table 3-1**. The existing Sam Senter Canal dimensions in the existing conditions model **were assumed** to have a bottom width of 12 ft, invert elevation at 2 ft NGVD, 1:1 side slopes, and a top-of-bank elevation of 12 ft. Note that surveyed cross sections of the existing Sam Senter Canal were not available.
5. The Ocean Canal cross section area from Sam Senter Canal to the Hillsboro Canal (Chainage 0 to 18,691) was increased. The cross section dimensions for various runs are presented in **Table 3-1**.
6. The Hillsboro Canal capacity was modified between the Ocean and Cross Canals. Dimensions are also presented in **Table 3-1**.
7. A new gate was installed on the Hillsboro Canal just south of the intersection with the Cross Canal. There are two (later changed to three) gates at chainage 59600 ft that are 20 ft wide, invert elevation at -2 ft NGVD, and max opening at 8.5 ft NGVD. This gate opens when upstream water levels exceed 11.5 ft NGVD.
8. Cross Canal and North New River cross sections were modified a number of times in an attempt to minimize expansion while minimizing canal water levels during simulations. The various dimensions evaluated are presented below in **Table 3-1**. **Figure 3-1** illustrates the difference between the existing and proposed cross sections at one location.
9. Bolles Canal cross sections were not modified until Run 16 because the first 15 runs focused on canal dimensions for the flow path from WPB to NNR. Dimensions are not shown as improvements to the Bolles Canal are not recommended at this time.
10. Existing bridges and culvert on the Cross Canal were removed and new bridges were not included in the model as it was assumed that the low chord of the bridges would be at least 13 ft NGVD and that the bridge concrete foundations would not constrict flow.
11. Compartment B was added in the following manner: There will be a 1,600 cfs inflow pump station from the NNR to an inflow distribution canal called CompB North EW1. The north

cell was called CompB North NS, which flows to a collection canal called CompB-EW2. This collection canal then flows south through Cell 4 (disconnected from STA 2) to the south cell inflow distribution canal which was called CompB-South-EW1. Interior water levels will be controlled by inflow and outflow gates in Cell 4. The number of gates is larger than what designed by Brown and Caldwell for Cell 4. This assessment assumes six (rather than 4 assumed by Brown and Caldwell) 6 x 6 ft inflow gates with a sill level of 2.25 ft NGVD. This assessment assumed three (rather than two assumed by Brown and Caldwell) outflow gates with a sill elevation of -2.5 ft. They are 8 ft wide and 7 ft high. The south cell was called CompB-South-CL. This south cell discharges to a collection canal called CompB-South-Collection. The collection canal connects to existing STA 2 discharge pump G-335 and a new 1,600 cfs outflow pump station called CompB-Outflow. CompB-Outflow will be activated only if G-335 is operating at full capacity. Dimensions of Compartment B are presented in **Figure 3-2**.

12. Structure G-336G in canal L-6 was removed to allow a balancing of outflows from both Compartment B and STA 2.
13. Compartment C dimensions were taken from URS (2005). **Figure 3-3** presented below is Figure 4 from the URS Hydraulic Modeling Report. The floor elevations at the upstream end of the cells were assumed to be 14 feet. Existing ground elevations on the west side of the cells are in the range of 17 feet. This change was made to allow for gravity inflows from L-3 to Compartment C during the dry season and to extend the hydroperiod of the western portion of the cells.

**Table 3-1 – Alternative 1 Cross Section Dimensions (dimensions in feet)**

| Canal | Start Ch | End Ch | Bottom Width | Invert | Side Slopes | Top-of-bank |
|---|---|---|---|---|---|---|
| Alternative 1 (Connecting WPB Canal to Sam Senter Canal) | 0 | 14,350 | 95 | -10.5 | 2.5 | 25 |
| Sam Senter (OC-B1) | 13,138 | 41,883 | 65 | -10.0 | 2.5 | 25 |
| Ocean Canal | 0 | 18,691 | 85 | -10.0 | 2.5 | 25 |
| Hillsboro Canal | 48,313 | 54,313 | 100 | -10.0 | 2.5 | 25 |
| Cross Canal | 0 | 46,759 | 77 | -10.0 | 2.5 | 25 |
| North New R. Runs 36 | 33,133 | 125,889 | 110 | -9.5 | 2.5 | 25 |
| North New R. Run 37 | 33,658 | 125,889 | 27.5 | -15.0 | 2.5 | 25 |

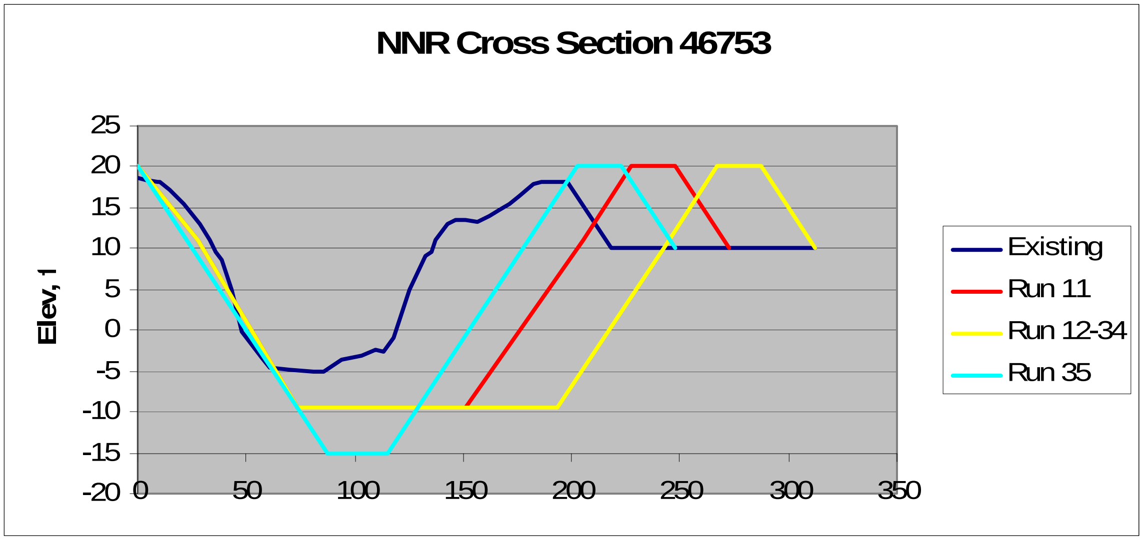
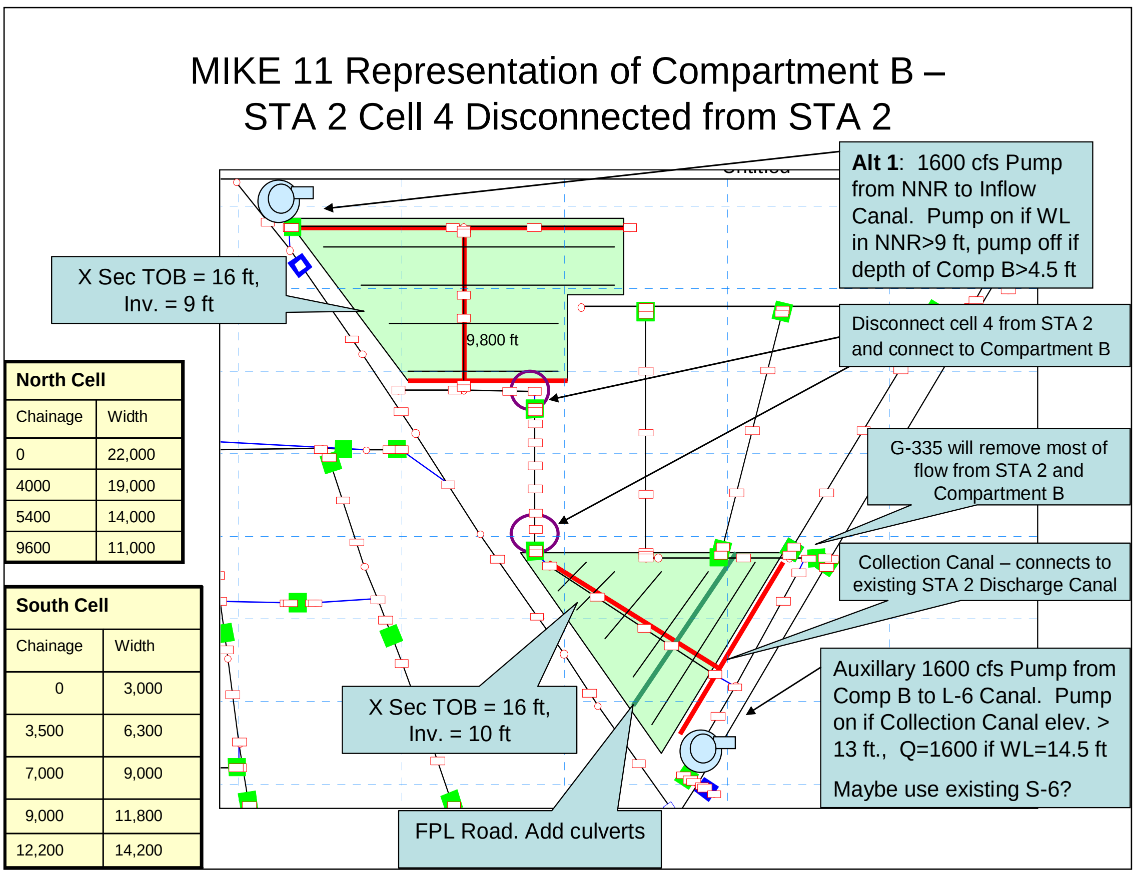



**Figure 3-1 – Existing and Alternative 1 Cross Sections, North New River Canal at Chainage 46753**




| North Cell | |
|---|---|
| Chainage | Width |
| 0 | 22,000 |
| 4000 | 19,000 |
| 5400 | 14,000 |
| 9600 | 11,000 |

| South Cell | |
|---|---|
| Chainage | Width |
| 0 | 3,000 |
| 3,500 | 6,300 |
| 7,000 | 9,000 |
| 9,000 | 11,800 |
| 12,200 | 14,200 |

**Figure 3-2 – MIKE 11 Representation of Compartment B**

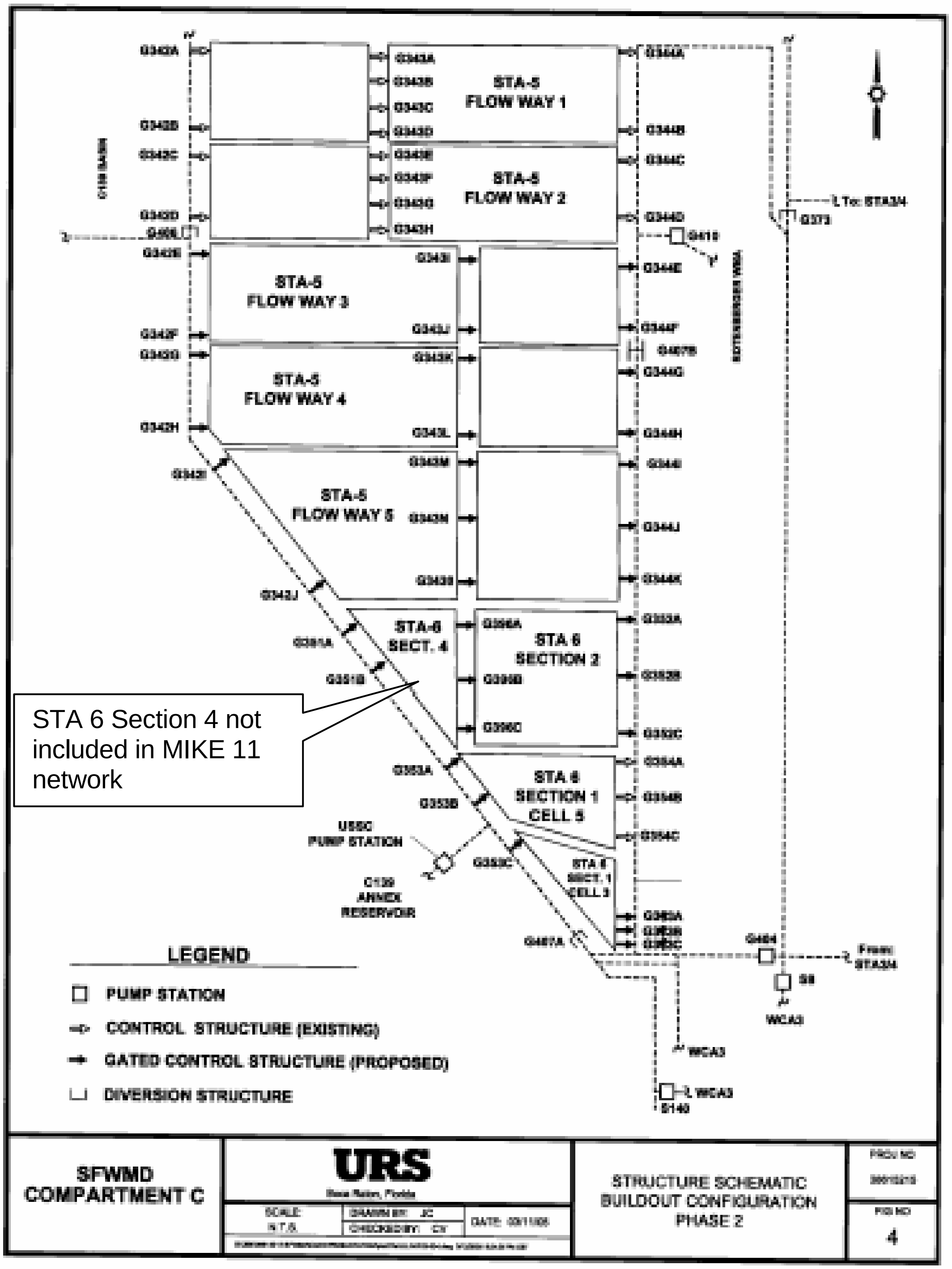



**Figure 3-3 – Figure 4 from URS (2005) Basis of Design Report for STA 5 Flow-way 3**

### 3.1.2 Hydraulic Analysis

The dimensions of canal enlargements and the gate operations for the WPB and Hillsboro gates were selected to accomplish diversion of flows to achieve a balanced distribution of flows to the STAs. The diversions were based on an analysis of projected water quality conditions using DMSTA (see **Attachment 1**). **Table 3-2** presents flows and canal stages for selected stations in the EAA for two options of Alternative 1. Run 36 assumes wider canals than Run 37 (see Figure 3-3 above for details). The target flows to be diverted and the Alternative 1 diversion flows are also presented below in **Table 3-2**. **Table 3-2** illustrates that Alternative 1 Run 37 comes closest to meeting the diversion requirements to balance flows and loads to the STAs, however Run 37 results in higher stages in the Ocean Canal near G-341 that results in eastward flows through G-341 toward pump station S-5A.

**Table 3-2 - Flows and Stages for Existing Conditions and Alternative 1**

| **PEAK FLOWS, cfs** | | Large Canals | Limited Widening | Target Flows |
|---|---|---|---|---|
| **Station** | **2006 EX** | **Run36** | **Run37** | |
| Cross Ch 100, West End | -525 | -2,760 | -2,380 | |
| Cross Ch 20000, Middle | -216 | -2,420 | -2,036 | -2,000 |
| Cross Ch 45271, East End | 161 | -2,047 | -1,690 | |
| Ocean Ch 3400, Near Hillsboro | -561 | -1,966 | -1,526 | |
| G-341 East Flow | 770 | 0 | 71 | |
| WPB 51430, U/S of New Gate | 1493 | 1,484 | 1,500 | |
| WPB 54740, D/S of New Gate | 1493 | 880 | 1,237 | |
| Sam Senter Ch 35368, near Ocean Canal | 412 | 1,028 | 667 | 1,200 |
| Bolles Ch 211, West End | -271 | -102 | -75 | |
| Hillsboro Canal, Ch 54697, U/S of New Gate | 675 | 674 | 670 | |
| Hillsboro Canal, Just U/S of S-6 | 2728 | 2,723 | 2,722 | |
| STA 1W Inflow Ch 100 | 3,208 | 2,783 | 2,952 | |
| G-311, to STA 1E | 0 | 0 | 0 | |
| STA 1E S-319 Inflow Pump Station | 1,800 | 1,900 | 1,900 | |
| STA 2 Inflow Canal Ch 1100 | 3,300 | 3,201 | 3,203 | |
| Comp B NNR Pump Station | 0 | 1,146 | 700 | 1,600 |
| A-1 Pump NE Pump Station | 0 | 2,167 | 2,200 | |
| STA 3/4 G-370 East Pump Station | 2,742 | 2,578 | 2,566 | |
| STA 3/4 G-372 West Pump Station | 3662 | 3,645 | 3,646 | |
| STA 5 (Cells 1-2, Compartment C cells) | 1,724 | 2,803 | 2,803 | |
| STA 6, Cells 3 and 5 (EX incl. Section 2) | 1,285 | 184 | 184 | |
| Sum of STA Inflows | 20,449 | 17,624 | 17,202 | |
| C-51W at S-155A (bypass flow) | 830 | 1,367 | 1,401 | |
| **STAGES, ft NGVD** | | | | |
| **Station** | **2006 EX** | **Run 36** | **Run 37** | |
| NNR at A-1 Pump NE Pump Station | 12.37 | 9.74 | 9.74 | |
| NNR at Compartment B Pump Station | 11.40 | 9.36 | 9.22 | |
| NNR at G-370 | 9.78 | 9.21 | 9.02 | |
| Cross Ch 200, West End | 13.09 | 11.20 | 11.76 | |
| Cross Ch 23622, Middle | 14.32 | 11.33 | 11.89 | |
| Cross Ch 43983, East End | 15.14 | 11.48 | 12.00 | |
| Ocean Ch 6800, halfway betw Hills & bend | 14.62 | 11.58 | 12.05 | |
| Ocean Ch 46400, at Gladeview Canal | 14.68 | 12.15 | 12.50 | |
| Sam Senter Ch 13139 (north end) | 14.90 | 11.66 | 12.10 | |
| WPB Ch 50000, U/S of new Gate | 12.31 | 11.71 | 12.14 | |
| Bolles Canal Ch 422, West End | 12.91 | 12.15 | 12.50 | |
| Target | 12.50 | 12.50 | 12.50 | |

*Note: U/S and D/S: Upstream and Downstream, Ch: chainage (canal location, feet from Lake Okeechobee). Highlighted cells exceed target flows and/or canal stages.*

Because the diversion of flows is accomplished using new gates in the WPB and Hillsboro Canals, Alternative 1 water levels are compared to 2006 existing conditions water levels, as shown in **Table 3-2**. Run 37 results in opening of G-341, elevating flows to STA 1W. **Figure 3-4** illustrates the stages and flows at G-341.

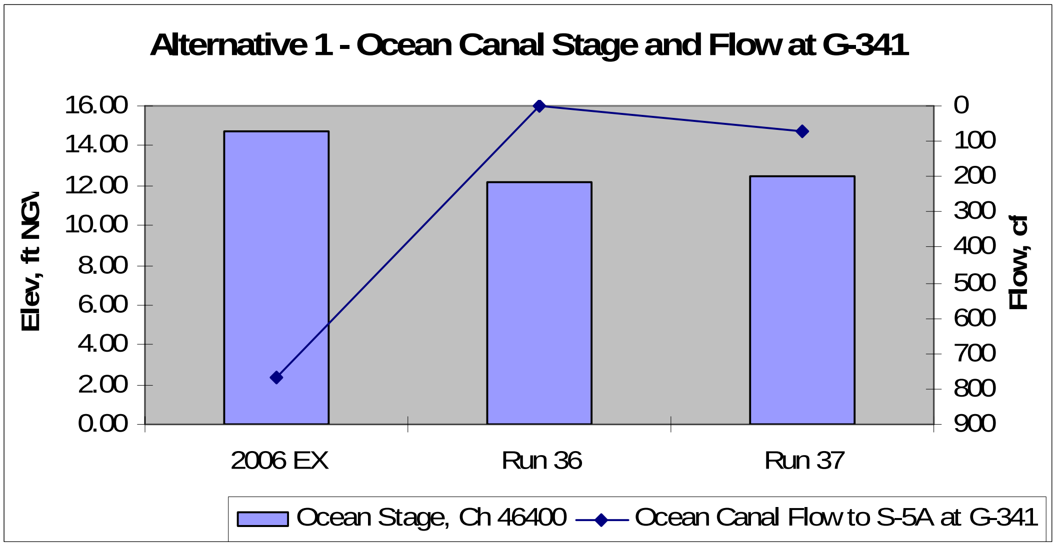



**Figure 3-4 – Flows and Stages at G-341 in the Ocean Canal**

**Figures 3-5** and **3-6** present water levels in STA 2 and Compartment B. Water levels in STA 2 are within desired depths for 2006 conditions, yet are higher than desirable depths for the north cell of Compartment B. The desired flow in Compartment B is 1,600 cfs, yet the peak flow is only 1,084 cfs. Two-dimensional modeling conducted by Brown and Caldwell also indicated that the peak depth may be exceeded above 1,000 cfs (personal communication, Emily Mott, Brown and Caldwell, September 18, 2005). Further modeling may be appropriate to resolve this situation. The cross section file for Compartment B used in the MIKE 11 model were assumed as no detailed survey information was identified, and the assumptions made for this assessment should be checked.

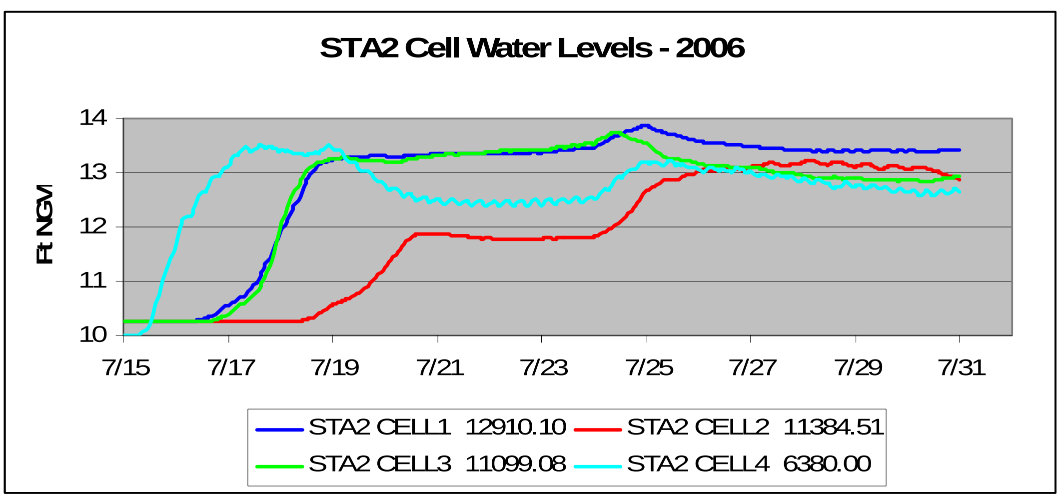



**Figure 3-5 – Water Levels in STA 2 for 2006 Existing Conditions**

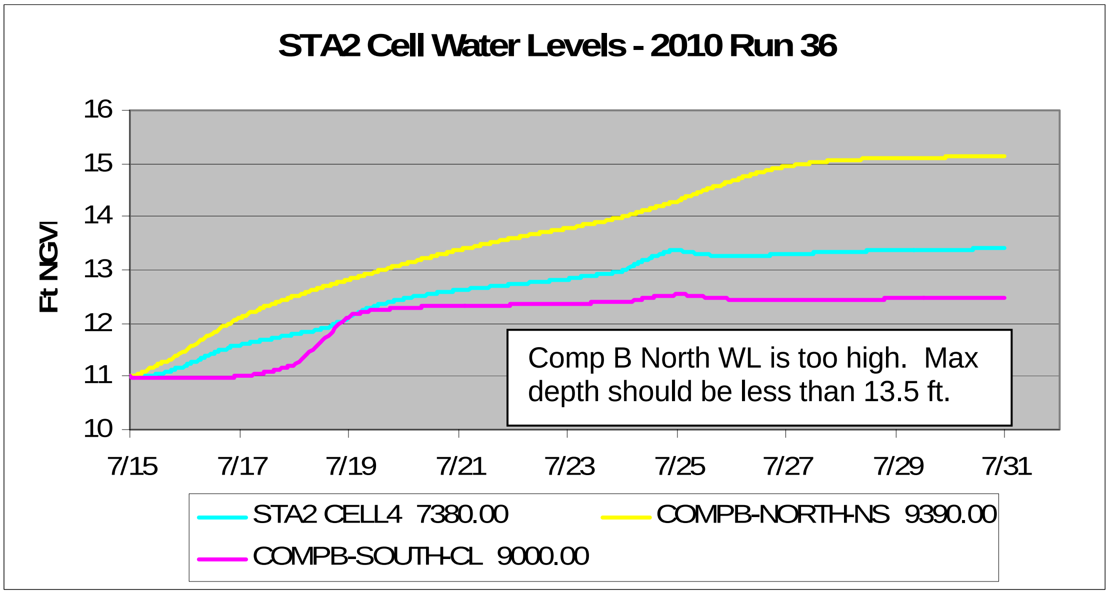



**Figure 3-6 – Water Levels in Compartment B for 2010 Alternative 1 Run 36**

## 3.2 Alternative 2

### 3.2.1 Detailed Description of Alternative 2

The implementation of this alternative in MIKE 11 is described below:

1. S-5AW gates are assumed to be closed, and S-5AE gates are assumed to be doubled from two to four gates. An additional gate was added to S-155A because target flows could not be achieved with the existing gates. This may not be necessary and should be evaluated with more detailed hydraulic assessments that better represent target C-51W canal elevations and S-155A tailwater submergence.
2. The bottom width of the L-7 Borrow Canal, chainage 6,416 to chainage 86,438 was increased by approximately 60 ft. Additionally, a separation levee was created between the L-7 and the Loxahatchee National Wildlife Refuge (LNWR). The east top of levee was adjusted to an elevation of 20 ft-NGVD; the side slope of the levee is 3H:1V. Refer to **Table 3-4** and **Figure 3-7**.
3. The gate in the east bank of L-7 near G-251 to allow STA 1W discharges to enter the north part of the Refuge was not modeled in MIKE 11.
4. The L-7 canal was diverted from its original connection to the Hillsboro Canal, downstream of G-338, to a new connection point upstream of G-338 in the vicinity of the S-6 Diversion Canal. New structure G-338A was added to the L-7 Canal at chainage 86,538. This structure consists of four 20-ft wide gates with a sill level at 7 ft-NGVD. The gate was programmed to begin opening when the head upstream reaches a level of 15 ft-NGVD. The gate will gradually open with the increase in head upstream until fully open at a level of 19 ft-NGVD. See **Figure 3-8** for the modifications to the L-7 and location of G-338A.
5. The existing STA 2 Inflow Canal was enlarged and was connected to the north spreader canal for Compartment B. To accomplish this, Branch CompB-North-EW1 was extended south to the STA-2 Inflow Canal (approximately 5,110 feet). A portion of the flows were routed through the entire flow-path of Compartment B, while a portion of the flows were routed through a new gate on the existing STA 2 Inflow Canal around the North cell and

cell 4. These diverted flows are treated in CompB-South-CL. See **Figure 3-9** representations of Compartment B.

6. Pump station G-335 operation was altered to begin pump operation when the head upstream exceeded 12 ft-NGVD, and reach peak flow (3,000 cfs) when the head upstream is 13 ft-NGVD. The CompB-South-CL pump station (located on the CompB-South-Collection Canal) turns on when the upstream head reaches 13 ft-NGVD and reaches a peak flow of 3,000 cfs at elevation 14 ft-NGVD. The operation of these two pumps has been established so that G-335 handles the majority of flows and the second pump station provides peak service. During design, a detailed investigation should be conducted to determine if S-6 can provide the peak service, thereby eliminating the need for the second pump station.
7. Structure G-336G was removed from the Mike11 network.
8. The cross sections of the S-6 Diversion Canal (chainage 0 to 3,037), STA-2 Supply Canal (chainage 0 to 17,953), and STA-2 Inflow Canal (chainage 0 to 39,900) were modified to widen the canal. As shown in **Table 3-5**, the bottom widths for the STA-2 Supply and S-6 Diversion Canals were increased by 180 ft. The side slope remained unchanged thus, offsetting the top of bank by 180 ft. The bottom width for the STA-2 Inflow Canal was increased by 100 ft. The side slope for this canal also remained unchanged, thus offsetting the top of bank by 100 ft.
9. The North New River Canal (NNR), Ocean Canal, Hillsboro Canal, and Cross Canal cross sections were modified to increase the bottom width of the canal. See **Table 3-5** for a summary of these changes.
10. The Alternative 1 Hillsboro River gate was included to divert runoff from the Ocean Canal to the NNR. Farm 50-011-03 has the ability to discharge to both the Hillsboro and Cross Canals. In this alternative, runoff from this farm is discharged to the Cross Canal.
11. The inflow pump station located on the NNR Canal originally proposed for Compartment B in Alternative 1 was turned off.
12. Pump station G-370 was modified so that the pump station continues to operate after the A-1 Reservoir elevation reaches 18 ft-NGVD. Pump station will deliver water to STA 3/4 when A-1 Reservoir water levels are equal to or greater than18 ft-NGVD.

**Table 3-3 – Alternative 2 L-7 Cross Section Dimensions**

| Cross-Section Chainage (FT) | Existing Bottom Width (FT) | Proposed Bottom Width (FT) | Proposed East Top of Bank (FT-NGVD) |
|---|---|---|---|
| 6416 | 100 | 160 | 20 |
| 6595 | 140 | 200 | 20 |
| 11594 | 70 | 130 | 20 |
| 16998 | 110 | 170 | 20 |
| 21597, 26496, 31596 | 120 | 180 | 20 |
| 36755, 43114 | 100 | 160 | 20 |
| 48519 | 70 | 130 | 20 |
| 53746, 58848, 64223 | 80 | 140 | 20 |
| 69540 | 90 | 150 | 20 |
| 74829 | 130 | 190 | 20 |
| 80016, 86438 | 140 | 200 | 20 |

**Table 3-4 – Alternative 2 Miscellaneous Cross Section Dimensions**

| Canal | Start Ch (FT) | End Ch (FT) | Bottom Width (FT) | Invert (FT-NGVD) | Side Slopes H:1 | Top-of-bank (FT-NGVD) |
|---|---|---|---|---|---|---|
| Ocean Canal | 0 | 18,691 | 85 | -10.0 | 2.5 | 18 |
| Hillsboro Canal | 48,313 | 54,313 | 100 | -10.0 | 2.5 | Existing |
| Cross Canal | 0 | 46,759 | 140 | -6.25 | 2.5 | 15 |
| North New River Canal | 33,133 | 125,889 | 110 | -9.5 | 2.5 | 20 |
| S-6 Diversion Canal | 0 | 3,037 | 140 | -4.0 | Existing | Existing |
| STA 2 Supply Canal | 0 | 17,953 | 140 | -4.0 | Existing | Existing |
| STA 2 Inflow Canal | 0 | 30,000 | 100 | -4.0 | Existing | Existing |
| STA 2 Cell 4 Discharge | 100 | 1,600 | 100 | -4.0 | Existing | Existing |

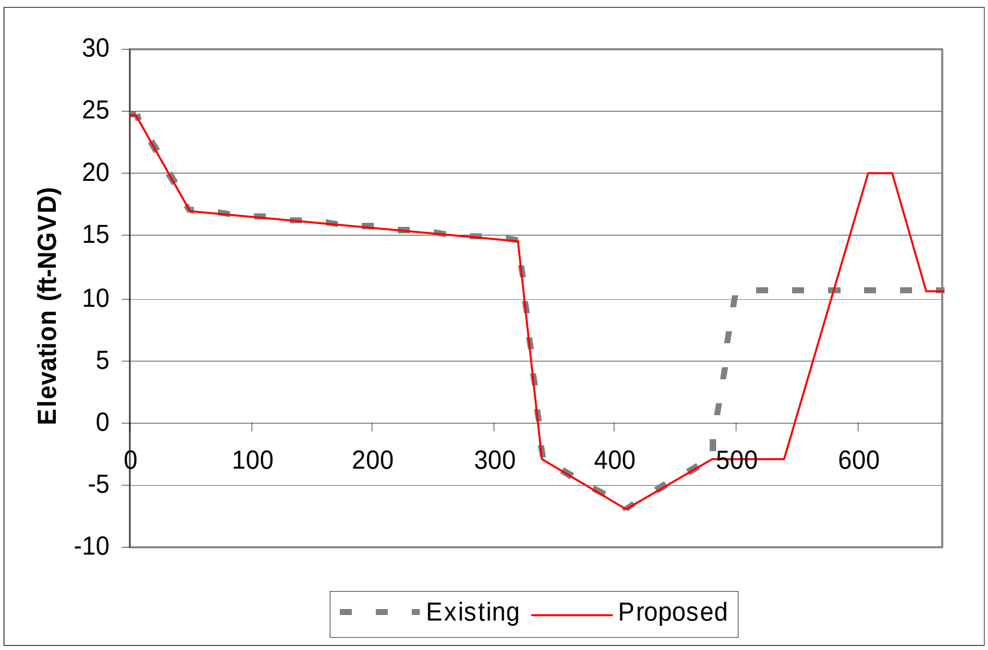



**Figure 3-7 – Details of L-7 East Levee Modifications**

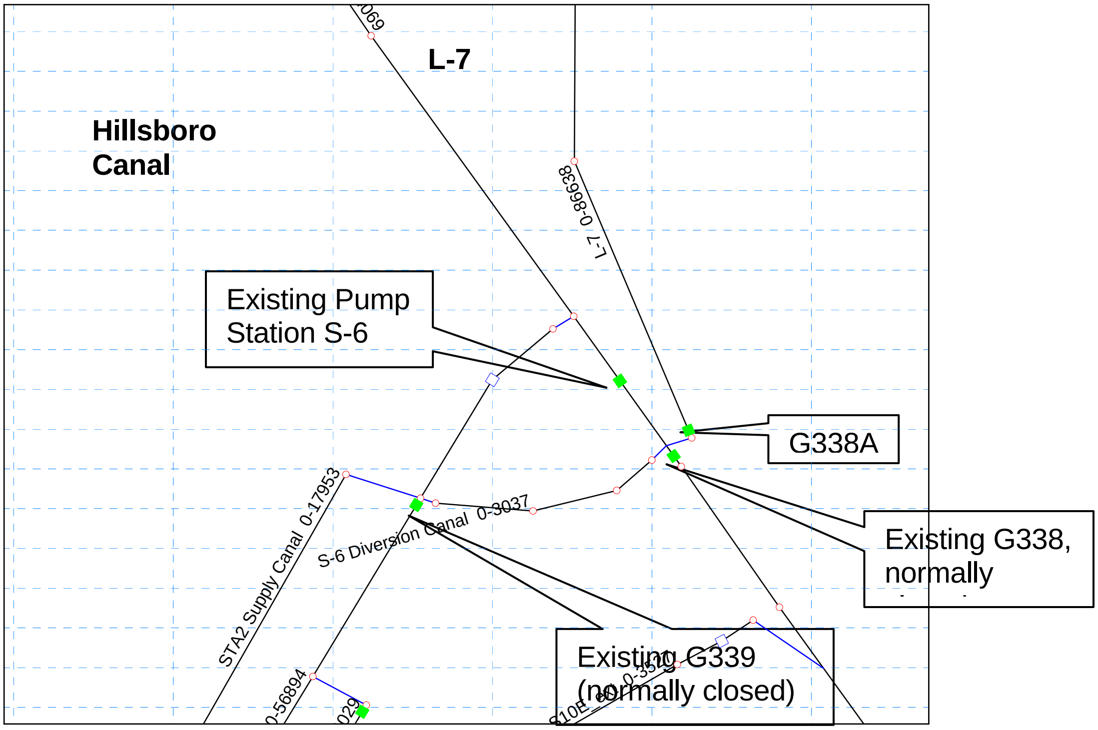



**Figure 3-8 – Alternative 2 - L-7 and Hillsboro Canal at Pump Station S-6 and Gate G-338**

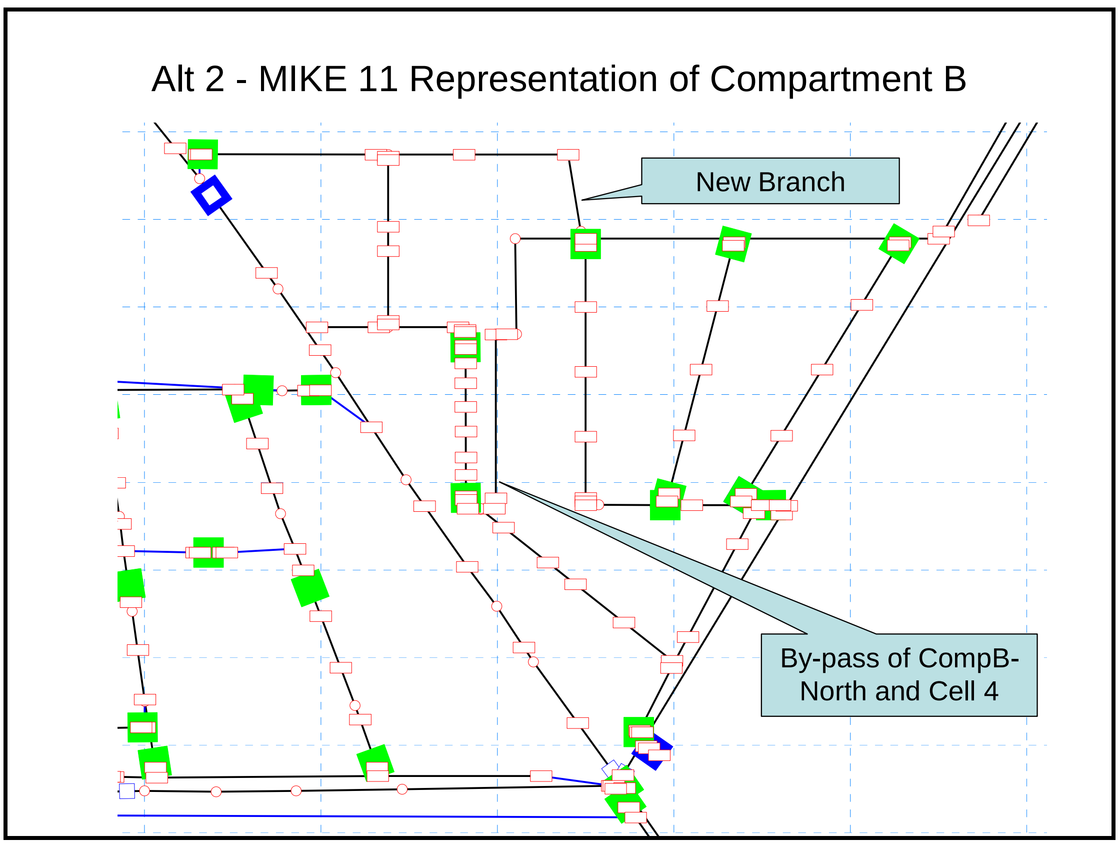



**Figure 3-9 – Alternative 2 Layout**

### 3.2.2 Hydraulic Analysis

Development of the Alternative 2 hydraulic model preceded receipt of DMSTA analysis. This concurrent analysis was conducted because prior investigations conducted by others suggested that this alternative would be effective in achieving a balanced discharge from the EAA STAs without extensive inter-basin transfers. Initial hydraulic modeling results conducted as part of this assessment indicated that STA 2/Compartment B could handle the hydraulic inputs from STA 1W and the S-2/S-6 basin. As results of DMSTA modeling were received, it became apparent that modifications to this alternative would be required to deliver the appropriate flows to each STA. Numerous runs were conducted in an attempt to route the desired flows to the STAs and the A-1 Reservoir. Changes made to improve the performance of this alternative are summarized below:

A. The initial concept was to use STA 2 and Compartment B to treat the S-6 Hillsboro Basin and to further treat STA 1W discharges. A portion of this combined inflow to the STA 2/Compartment B inflow canal was routed around STA 2 and Compartment B North to avoid flow constrictions in Cell 4, the middle cell of Compartment B.
B. Cross sectional area of the North New River, Cross Canal, Hillsboro Canal, and the Ocean Canal were increased in increments to increase conveyance from the Hillsboro Canal to the North New River via the Cross Canal
C. A gate was added to the Hillsboro Canal just south of the junction with the Cross Canal, and this gate was programmed to stay closed during the entire period of simulation.
D. Runoff from farm 50-011-03 that has the ability to discharge to both the Hillsboro and Cross Canal was routed to the Cross Canal, which reduced runoff to the Hillsboro Canal by 230 cfs.
E. Existing pump station S-6 operation was modified to reduce the inflow rate to STA 2 (this test was not successful due to high stages north of S-6).

After evaluation of model results for these tests, Alternative 2 incorporated items A-D listed above. This version is referred to as Run 51. Run 51 came close to meeting flow targets for the STAs but resulted in high stages in the Ocean Canal, Sam Senter Canal, Cross Canal, and portions of the Hillsboro Canal.

Due to the problems in Run 51, a number of additional tests were conducted modifying a variety of model parameters in an attempt to meet both the flow and the flood control targets. Changes included modifying flow distribution to STA 1E and 1W, using a number of different combinations of canal enlargements, and modifying the pump capacity for the A-1 Reservoir NE pump station. Run 55 assumed that the A-1 NE pump station capacity would be 5,000 cfs rather than 3,000 cfs used for Alternative 1, and reduced the pump discharge capacity in STA 1W (G-251 was turned off). This run was able to achieve both the flow targets to the STAs and the flood control targets (stages less than 12.5 ft-NGVD in the Ocean, Sam Senter, Hillsboro, and Cross Canals). Stages and flows for Alternative 2 Run 51 and Run 55 are compared to existing conditions and target stages and flows in **Table 3-6**.

**Table 3-5 - Flows and Stages for Existing Conditions and Alternative 2**

| PEAK FLOWS, cfs<br>Station | 2006 EX | Run51 | Run 55 | Target Flows |
|---|---|---|---|---|
| Cross Ch 100, West End | -525 | -2,593 | -3,093 | |
| Cross Ch 20000, Middle | -216 | -2,286 | -2,731 | |
| Cross Ch 45271, East End | 161 | -1,631 | -2,110 | -1,460 |
| Ocean Ch 3400, Near Hillsboro | -561 | -856 | -1,340 | |
| G-341 East Flow | 770 | 517 | 0 | |
| Hillsboro Canal, Ch 54697, U/S of New Gate | 675 | -29 | -30 | |
| Hillsboro Canal, Just U/S of S-6 | 2728 | 1,754 | 1,747 | |
| STA 1W Inflow Ch 100 | 3,208 | 2,775 | 2,492 | 3,250 |
| G-311, to STA 1E | 0 | -970 | -650 | -1,000 |
| STA 1E S-319 Inflow Pump Station | 1,800 | 2,000 | 1,999 | |
| STA 2 Inflow Canal Ch 1100 | 3,300 | 5,383 | 4,372 | 4,720 |
| Comp B NNR Pump Station | 0 | 0 | 0 | |
| A-1 Pump NE Pump Station | 0 | 3,000 | 4,480 | |
| STA 3/4 G-370 East Pump Station | 2,742 | 2,493 | 1,809 | |
| STA 3/4 G-372 West Pump Station | 3662 | 3,640 | 3,641 | |
| STA 5 (Cells 1-2, Compartment C cells) | 1,724 | 2,805 | 2,767 | |
| STA 6, Cells 3 and 5 (EX incl. Section 2) | 1,285 | 208 | 238 | |
| Sum of STA Inflows | 20,449 | 23,271 | 22,448 | |
| C-51W at S-155A (bypass flow) | 830 | 1,347 | 1351 | |
| **STAGES, ft NGVD** | | | | |
| **Station** | **2006 EX** | **Run 51** | **Run 55** | |
| NNR at A-1 Pump NE Pump Station | 12.37 | 11.88 | 9.81 | |
| NNR at Compartment B Pump Station | 11.40 | 11.78 | 9.73 | |
| NNR at G-370 | 9.78 | 11.66 | 9.65 | |
| Cross Ch 200, West End | 13.09 | 12.79 | 11.41 | |
| Cross Ch 23622, Middle | 14.32 | 12.90 | 11.60 | |
| Cross Ch 43983, East End | 15.14 | 12.97 | 11.76 | |
| Ocean Ch 6800, halfway betw Hills & bend | 14.62 | 12.96 | 11.81 | |
| Ocean Ch 46400, at Gladeview Canal | 14.68 | 13.00 | 12.31 | |
| Bolles Canal Ch 422, West End | 12.91 | 12.93 | 12.35 | |
| Target | 12.50 | 12.50 | 12.50 | |

*Note: U/S and D/S: Upstream and Downstream, Ch: chainage (canal location, feet from Lake Okeechobee). Highlighted cells exceed target canal stages.*

The dimensions of canal enlargements for L-7, Cross Canal, and NNR and the gate operations for the L-7 G-338A gate were selected to accomplish diversion of flows to achieve a balanced distribution of flows to the STAs. The diversions were based on an analysis of projected water quality conditions using DMSTA (see **Attachment 1**). The target flows to be diverted and the Alternative 2 diversion flows are presented below in **Table 3-7**. **Table 3-7** illustrates that Alternative 2 Run 55 comes close to meeting the diversion requirements to balance flows and loads to the STAs.

**Table 3-6 – Alternative 2 Target Flows to Achieve Balanced Inflows and Predicted Flows from Hydraulic Modeling**

| Location | Target Flow cfs | MIKE 11 Flows, cfs | |
|---|---|---|---|
| | | Run 51 | Run 55 |
| STA 1W Inflow | 3,250 | 2,775 | 2,492 |
| G-311 Inflow to STA 1E | 1,000 | 970 | 650 |
| STA 2/Compartment B Inflow | 4,720 | 5,132 | 5,200 |
| Flow from Hillsboro to NNR | 1,460 | 1,614 | 2,065 |

Alternative 1 inflows to STA 2 and Compartment B are compared to Alternative 2 Run 51 and 55 flows in **Figure 3-10**. **Figure 3-10** illustrates that the maximum inflow to STA 2/Compartment B for Alternative 2 Run 51 and Run 55 are above recommended levels. **Figure 3-11** illustrates that peak stages in Compartment B North are less for Alternative 2 because a portion of the STA 2 inflow is diverted around Compartment B North (see **Figure 3-12**). Numerous runs were conducted in an attempt to decrease the Hillsboro Canal peak inflow rate. This alternative was able to meet both the canal stage targets and STA flow distribution targets with the A-1 NE pump station capacity equal to 5,000 cfs (note that Alterative 1 A-1 NE pump station capacity is 3,000 cfs). Once it was apparent that Run 55 was providing both flow distribution and peak canal stage targets, it was re-run with narrow, deep cross sections in the Cross and North New River Canals (dimensions the same as shown for Alternative 1 Run 37). Higher stages were observed in the Cross and Ocean Canals with 550 cfs of easterly flow through G-341. Using the wider cross sections for Run 55 did not have flows through G-341.

**Figure 3-13** illustrates flows out of STA 2 and Compartment B through G-335 and the new Compartment B outflow pump. It can be seen that G-335 does not begin discharges until the third day of the simulation, and the Compartment B pump station does not begin flow until G-335 is close to full capacity nine days after the beginning of the simulation. Figure 3-14 illustrates that stages in the Ocean and Cross Canals are within target maximum stages and are lower than existing stages.

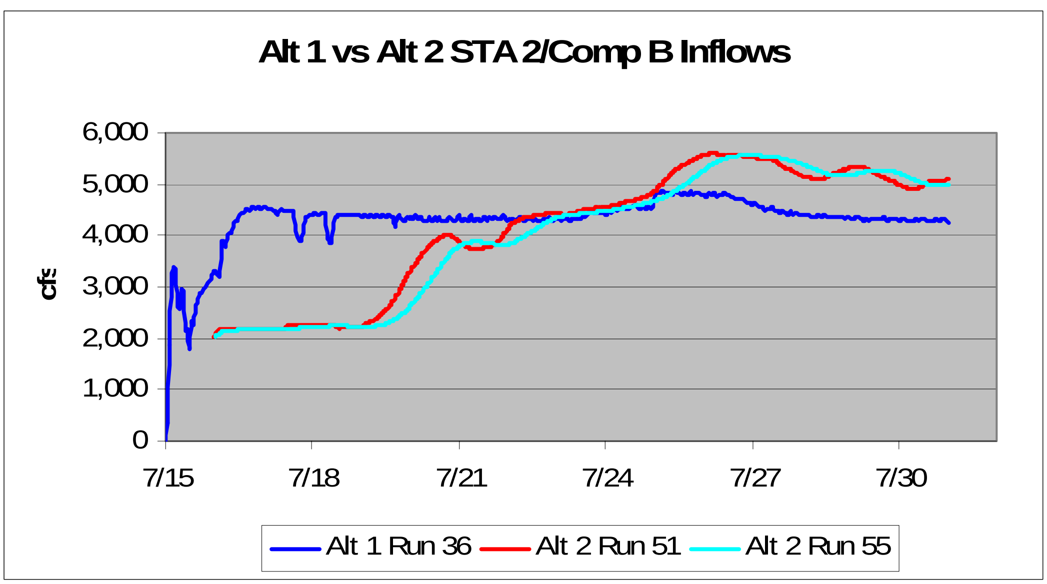



**Figure 3-10 – Alternative 1 and 2 Inflows to STA 2 and Compartment B**

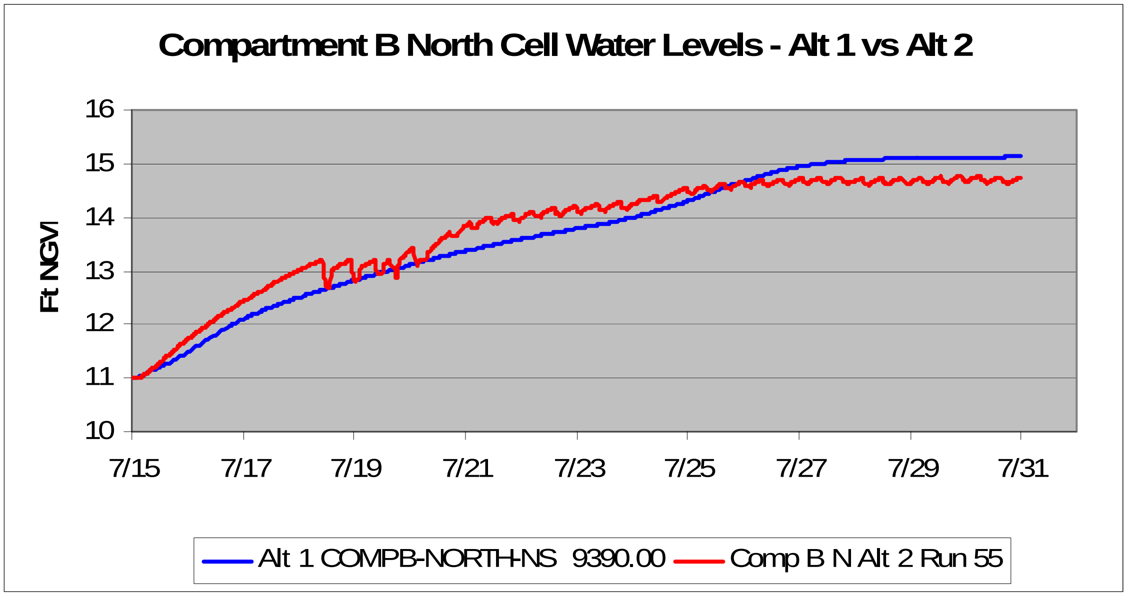



**Figure 3-11 – Water Levels in Compartment B North for Alternatives 1 and 2**

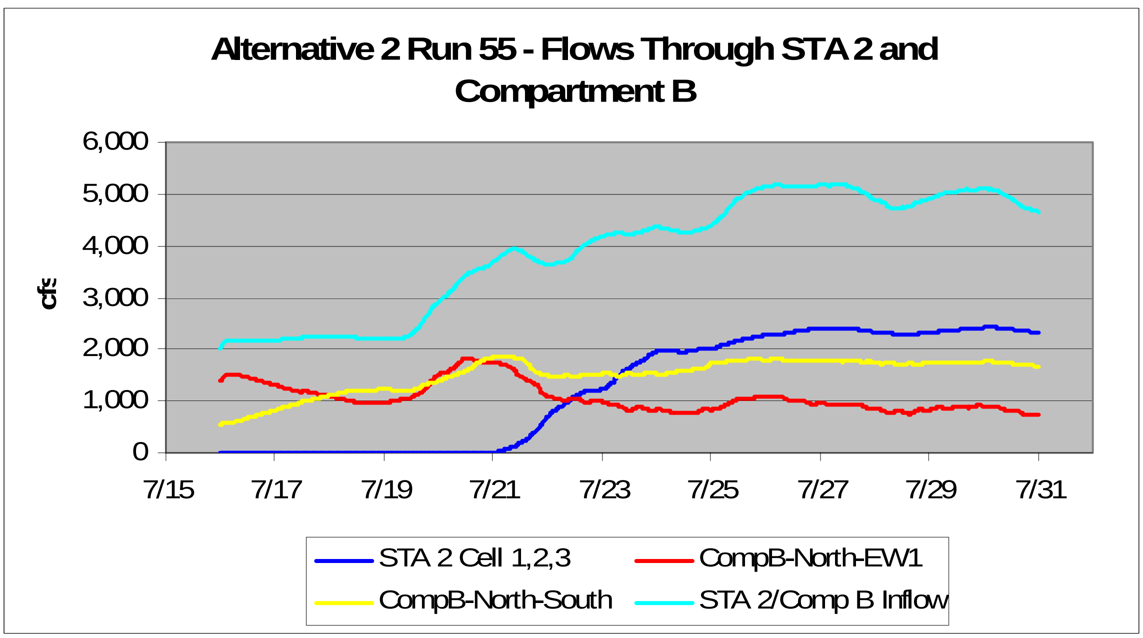



**Figure 3-12 – Flows Through STA 2 and Compartment B**

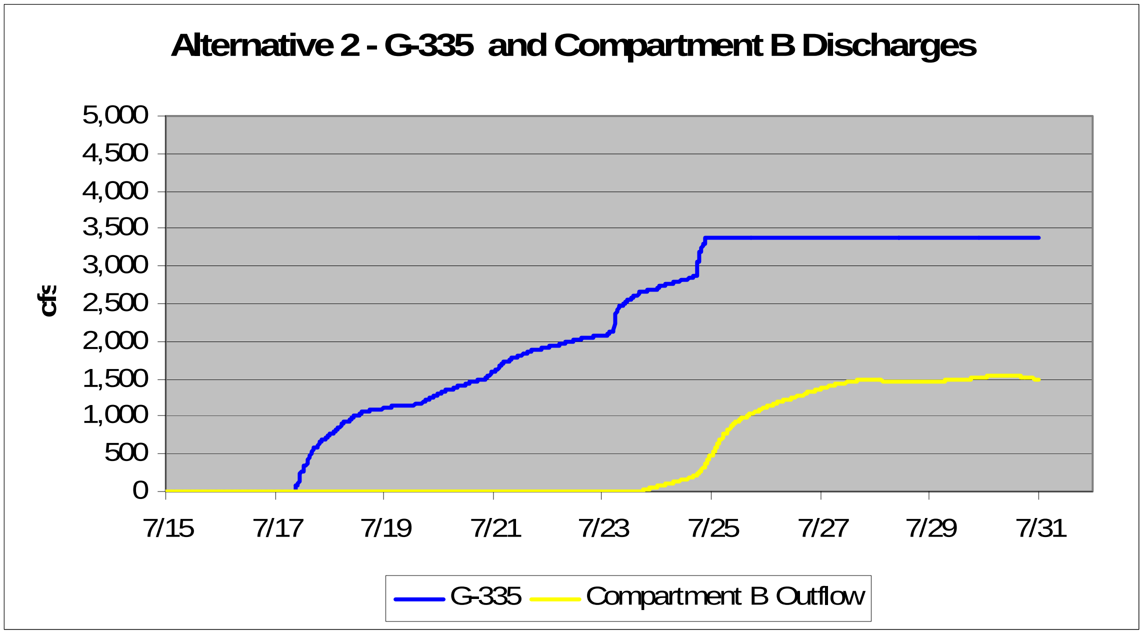



**Figure 3-13 – Flows Leaving STA 2 and Compartment B**

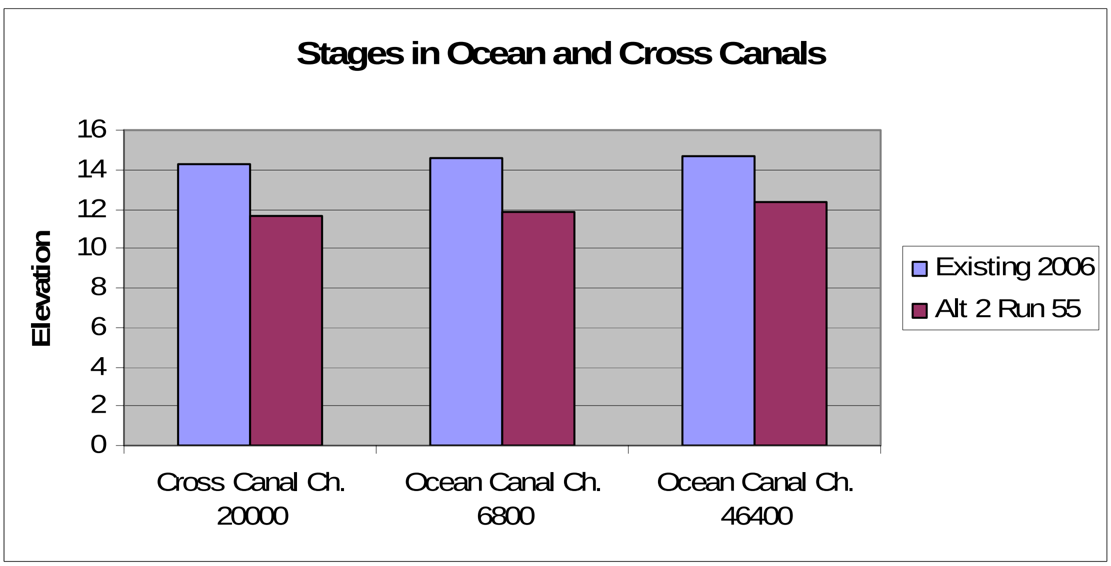



**Figure 3-14 – Water Levels in the Cross and Ocean Canals for Alt 2 Run 55**

A key feature of Alternative 2 is a larger pump station for the A-1 Reservoir. The required A-1 NE pump station capacity is 5,000 cfs, which is 2,000 cfs larger than the capacity assumed for Alternative 1. This larger capacity is needed for the following reasons:

1. In Alternative 2, STA 1W outflows are re-directed through new gate G-338A to the STA 2 Supply Canal via the S-6 Diversion Canal (see **Figure 5.7** shown above).
2. S-6 continues to pump into STA 2, however at **lower** rates than maximum capacity so that the combined inflow from S-6 and STA 1W via G-338A does not exceed 4,700 cfs. This maximum flow is derived from DMSTA 2 analysis of STA 2 and Compartment B. Decreased flows from the Hillsboro Canal to S-6 mean that all runoff from the Hillsboro

Canal upstream of the proposed Hillsboro Gate is diverted to the North New River via the Cross Canal.

3. An inflow pump station to Compartment B on the North New River Canal is not needed due to Items 1 and 2 above. Therefore, G-370 and the A-1 NE Pump Station are the only withdrawals from the North New River. Simulations with a 3,000 cfs pump station resulted in flooding in the Ocean Canal.

**Figure 3.14a** illustrates that the 5,000 cfs pump station will operate for 7 days if the maximum reservoir elevation is 18 ft NGVD and 12 days if the maximum reservoir elevation is 22 ft-NGVD. The modeling conducted for this alternatives analysis assumed that all farms discharged 0.75 inches/day of runoff for 15 days. Historical flows from the EAA indicate that cumulative runoff from the EAA exceeded 0.75 inches/day for five days in November, 1998. No other event from 1990 through 2002 generated runoff in excess of 0.75 inches/day. This analysis indicates that the reservoir will be able to store North New River runoff from the largest 5-day event measured from 1990 through 2002.

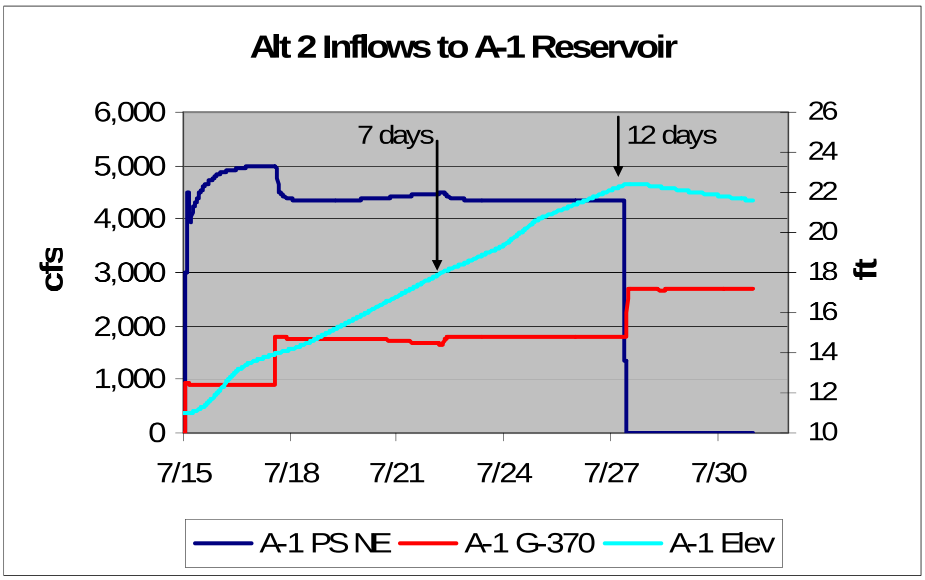



**Figure 3.14a – Alternative 2 Flows to the A-1 Reservoir**

Run 55 of Alternative 2 provided the best set of results for both flow distribution and peak stage reduction objectives. To re-cap, alternative 2 Run 55 consists of;

- expansion of the L-7 Canal and separation of the L-7 from LNWR,
- new gate G-338A
- expansion of the STA 2 Diversion, Supply, and Inflow Canals,
- Compartment B and Cell 4 (currently linked to STA 2) separate from STA 2
- New gate on the Hillsboro Canal south of the junction with the Cross Canal. This gate will be permanently closed, except for special conditions.
- Expansion of the Ocean Canal from the Sam Senter Canal to the Hillsboro Canal
- Expansion of the Hillsboro Canal from the Ocean Canal to the Cross Canal
- Expansion of the Cross and North New River Canals to the same dimensions recommended for Alternative 1
- Addition of 2000 cfs capacity to the A-1 Reservoir NE pump station.

### 3.3 Alternative 3

#### 3.3.1 Detailed Description of Alternative 3

The implementation of this alternative in MIKE 11 is summarized below:

1. In order to handle the new inflow from the Miami Canal, the Manley Ditch Canal was widened. As shown in **Figure 3-15**, it was assumed that Manley Ditch will have a bottom width of 30 ft, invert elevation of 0.0 ft-NGVD, 1.4H:1V side slopes, and a total canal depth of 17 ft from chainage 26,600 to 63,000 ft. The existing cross section was assumed to have a bottom width of 12 ft, invert elevation of 2 ft-NGVD, 1H:V side slopes, and a total canal depth of 10 ft. Note that surveyed cross sections representing existing conditions are **not** available and were **assumed**.
2. Since the objective of this alternative is to direct water from the Miami Canal into Manley Ditch, the boundary file for this alternative was modified to direct runoff from a portion of Miami Canal drainage basin (Farm 50-067-05) to the Manley Ditch. The Miami Canal Lateral Inflow in the existing conditions model and all other alternatives is at chainage 74,627.95 ft. The peak discharge for this farm at ¾" runoff/day is 233 cfs. This inflow was re-directed to MC-B5 (Manley Ditch).
3. The STA 5 N-Seepage Canal was extended to L-3 and the cross section area was assumed to be the same as for the Manley Ditch. As shown below in **Figure 3-16**, 15,400 was not modified, 15,492 is a transition cross section, and 15,992 is set at the same dimensions of the expanded Manley Ditch. The existing Canal ends at 15,992, the extended STA 5 N-Seepage Canal ends at 19,440.
4. An existing 30 cfs pump station on the STA 5 N-Seepage Canal for returning seepage to STA 5 was deleted and replaced by a new 550 cfs pump station at chainage 19,300. The turn-on elevation is 9 ft-NGVD. Flow of 0 cfs at 8 ft-NGVD, 275 cfs at 9 ft-NGVD, and 550 cfs at 12 ft-NGVD.
5. In order to convey additional water to STA 5 and ultimately into Compartment C and STA 6, a new gate (ALT3-Inflow) was included at the junction of Manley Ditch with the Miami Canal. This gate was modeled as two 15-foot gates and will open only if headwater elevations (west of the connection with the Miami Canal) are greater than 12 ft-NGVD.

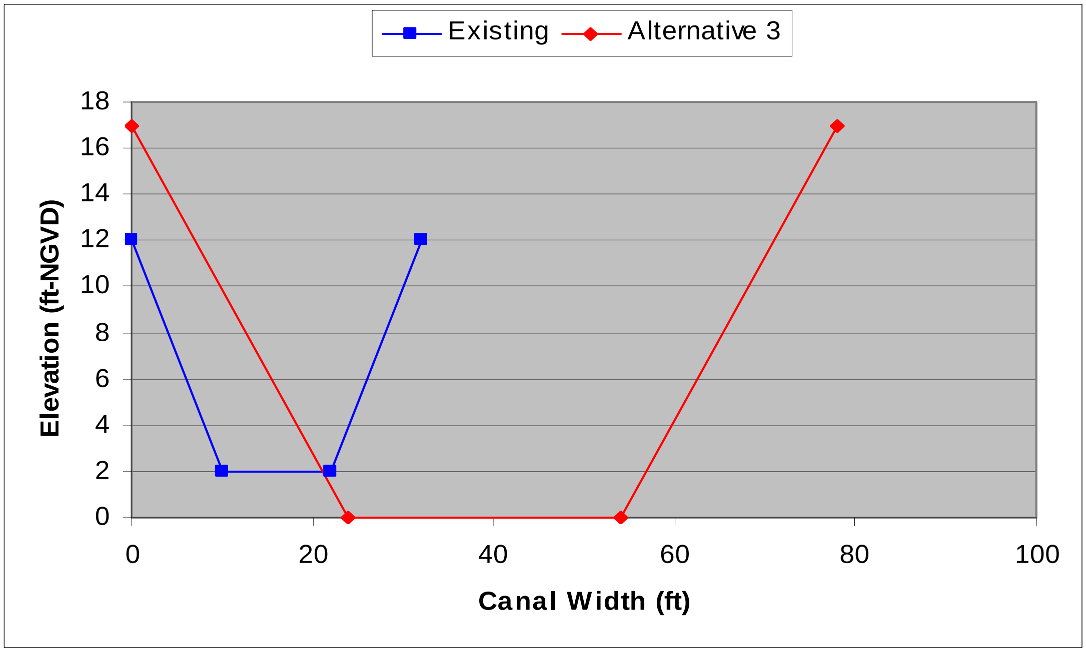



**Figure 3-15 - Manley Ditch Widening Typical Section**

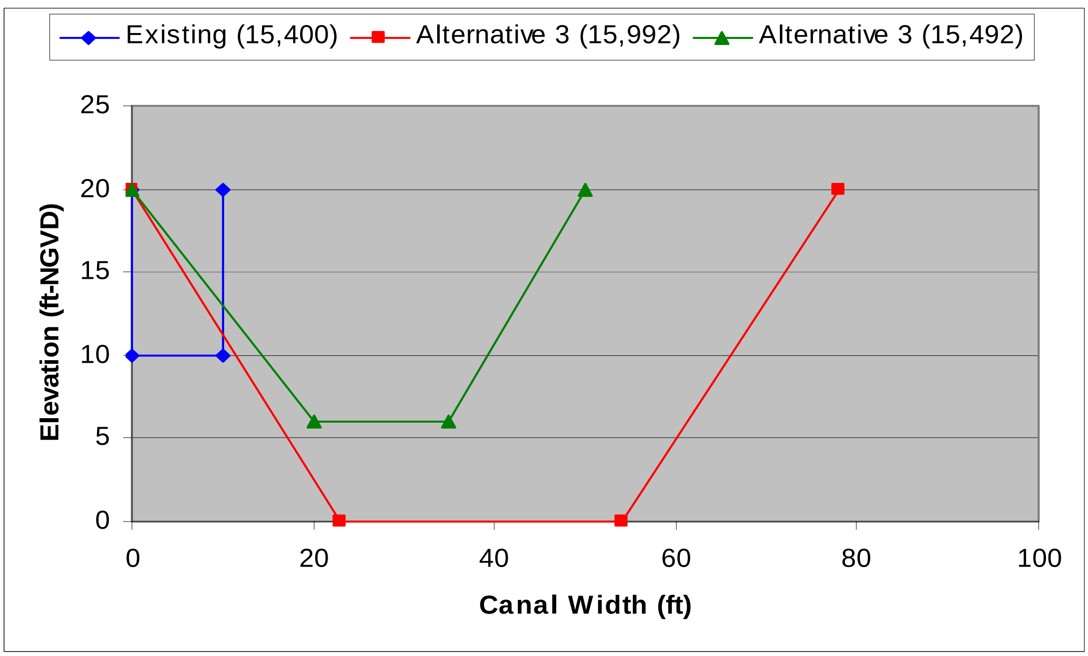



**Figure 3-16 - STA 5 North Seepage Canal Modifications**

### 3.3.2 Hydraulic Analysis

Alternative 3 is a variant of Alternative 1. The only difference between the two alternatives is the destination of runoff from the Manley Ditch. Alternative 3 directs most of the Manley Ditch runoff to the L-2 Canal for subsequent treatment in STA 5/Compartment C/STA 6 (hereinafter called STA 5/6). This alternative would only be implemented if current scientific investigations being conducted by SFWMD indicate that TP removal capacity can be improved beyond historical levels. Should these investigations indicate that STA 5 removal rates will improve, then there will be sufficient capacity in STA 5/STA 6 for treatment of additional runoff. **Figure 3-17** indicates that Alternative 3 is effective in delivering additional flows to STA 5/6. **Figure 3-18** indicates that Alternative 3 results in lower discharges to STA 3/4 through existing pump station G-372. This reduction will improve efficiency of TP removal in STA 3/4. The one negative effect of Alternative 3 is higher stages in L-2 north of existing gate G-406, as shown in **Figure 3-19**.

Additionally, a pump station was added at the outflow of STA 6 per prior conceptual descriptions of Compartment C and STA 6. There was a lower stage in the STA 6 discharge canal. However, the elevation in L-2 north of G-406 was not affected. It is expected that an inflow pump North of G-406 would have a more positive effect in reducing stages in L-2.

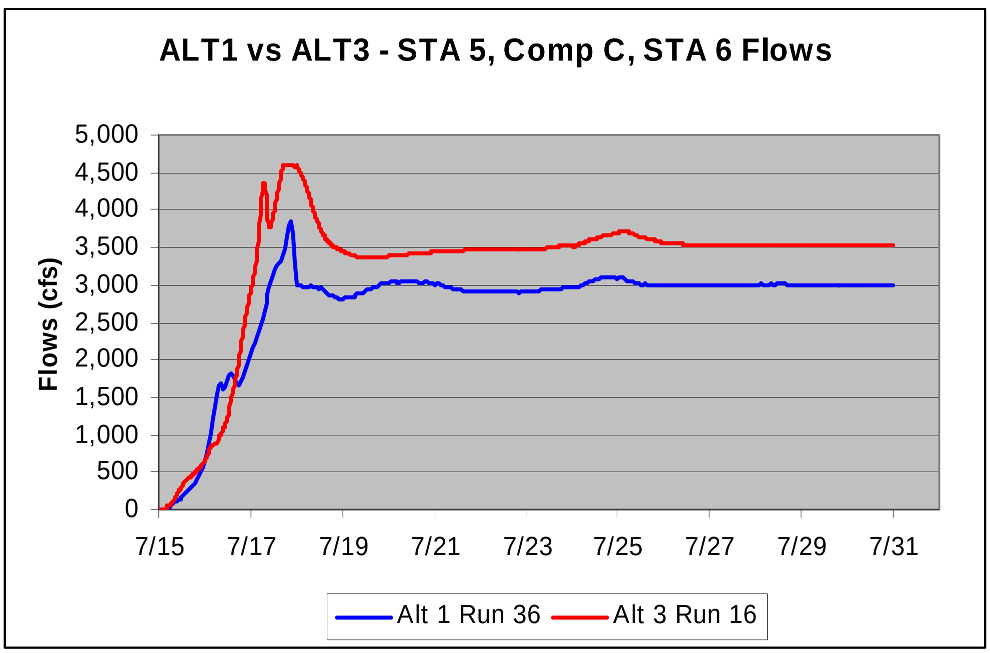



**Figure 3-17: Alternative 1 vs Alternative 3 Flows through STA 5, Compartment C, & STA 6**

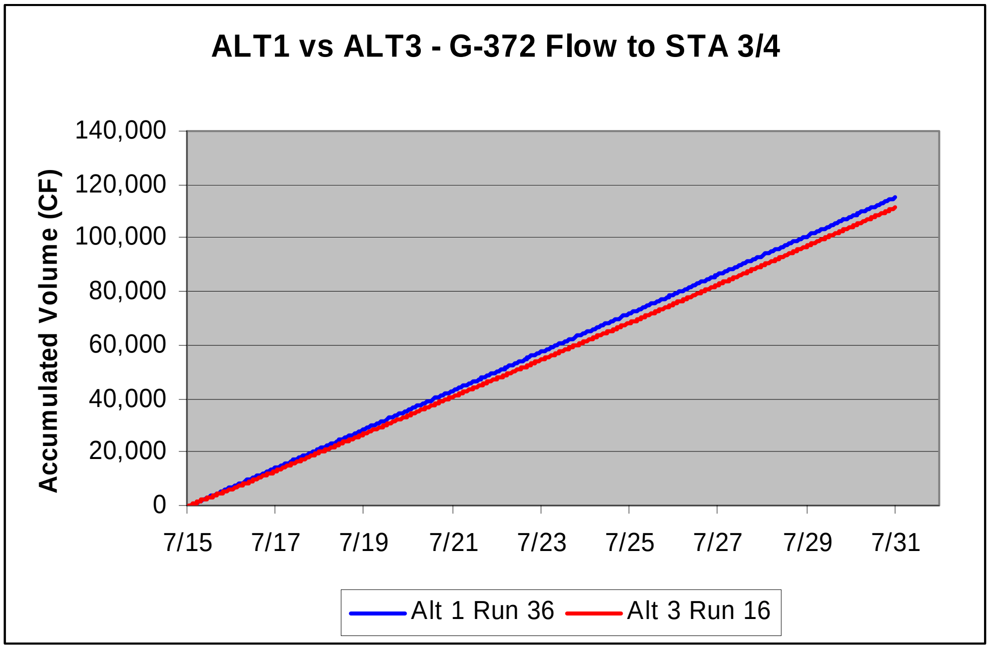



**Figure 3-18: Alternative 1 vs Alternative 3 Accumulated Volume to STA 3/4**

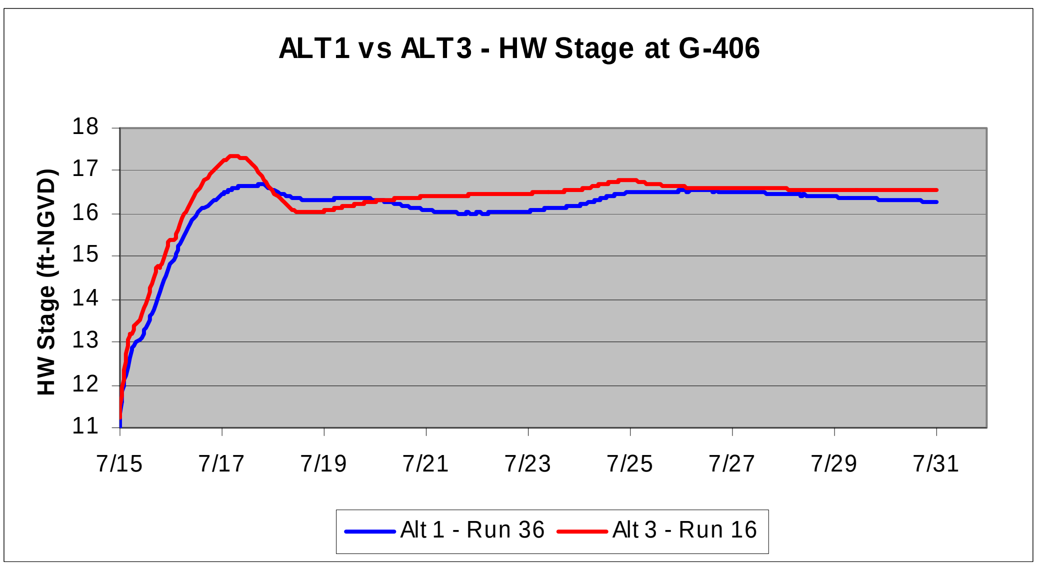



**Figure 3-19: Alternative 1 vs Alternative 3 Stages in L-2, North of G-406**

## 3.4 Alternative 4

### 3.4.1 Detailed Description of Alternative 4

Alternative 4 was intended to utilize the best features of Alternatives 1, 2, and 3 to meet project objectives while minimizing cost. No combination of alternatives was found that would reduce project cost, therefore further assessment of Alternative 4 was abandoned.

### 3.4.2 Hydraulic Analysis

Six different combinations of features from Alternatives 1-3 were tested. No combination provided an improvement over Alternatives 1 – 3.

## 3.5 Alternative 5

### 3.5.1 Detailed Description of Alternative 5

Alternative 5 is a derivative of Alternative 1, except that the Compartment B internal flow pattern will operate separate from STA 2 Cells 1, 2, and 3. In order to achieve this, inverted siphons were placed below the STA 2 Inflow Canal and the STA 2 Cell 4 Discharge Canal. **Figure 3-20** below illustrates the location of these siphons and the Compartment B layout for this Alternative.

The inverted siphons were modeled in MIKE 11 as five 5-ft diameter reinforced concrete culverts at each crossing, meaning 10 culverts total. Culvert dimensions were modified so that head loss at each location was maintained below 0.5-feet. The culverts were 230 ft in length with an invert equal to -13 ft-NGVD, 9 ft below the canal crossing.

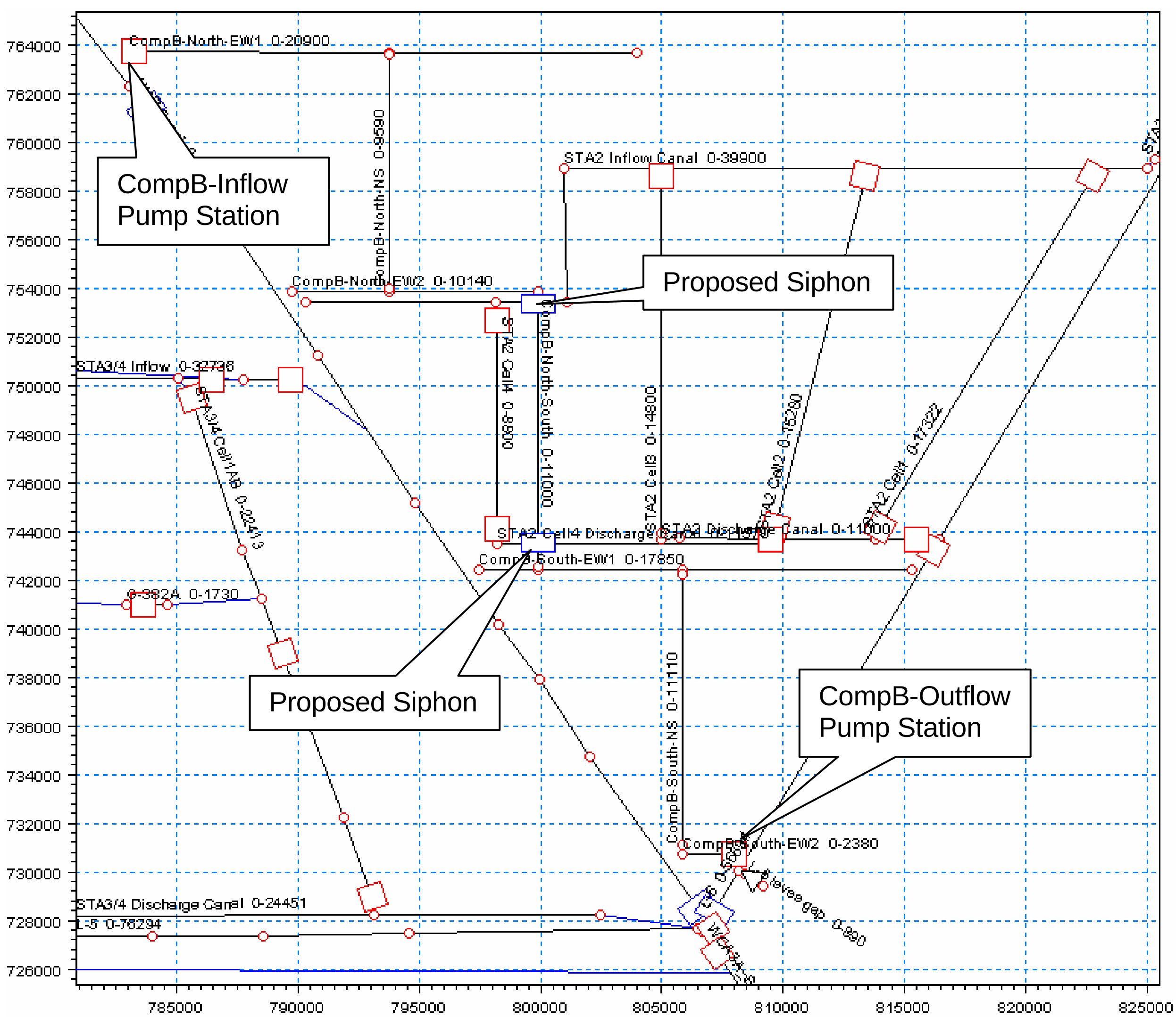



**Figure 3-20 - Alternative 5 Compartment B Layout**

### 3.5.2 Hydraulic Analysis

Head loss in the siphons was modeled using a specialty computer program from LMNO Engineering, Research, and Software, Ltd. (www.lmnoeng.com). The culverts were added to the MIKE 11 network and entrance coefficients were selected so that the head loss of the culverts in MIKE 11 was the same as simulated using the siphon program. The addition of the siphons for Alternative 5 resulted in Compartment B stages of 14 ft-NGVD on the North cell. The STA 2 Cells 1, 2, and 3 never exceeded water depths above 4 ft. However, the Alternative 1 accumulated volume in the North cell of Compartment B held approximately twice the amount as Alternative 5. **Figure 3-21** below illustrates the pumped volume into Compartment B from the NNR Canal. Inflows and water levels in the North cell of Compartment B are presented in **Figure 3-22**. It can be seen that the inflows to Compartment B begin at approximately 1300 cfs and quickly drop to 400 cfs. There are significant instabilities in inflow pump rates to Compartment B with the siphons. These instabilities were not reduced because:

- the overall inflow rate to Compartment B was significantly less than for Alternative 1, and
- maintenance concerns are significant for inverted siphons.

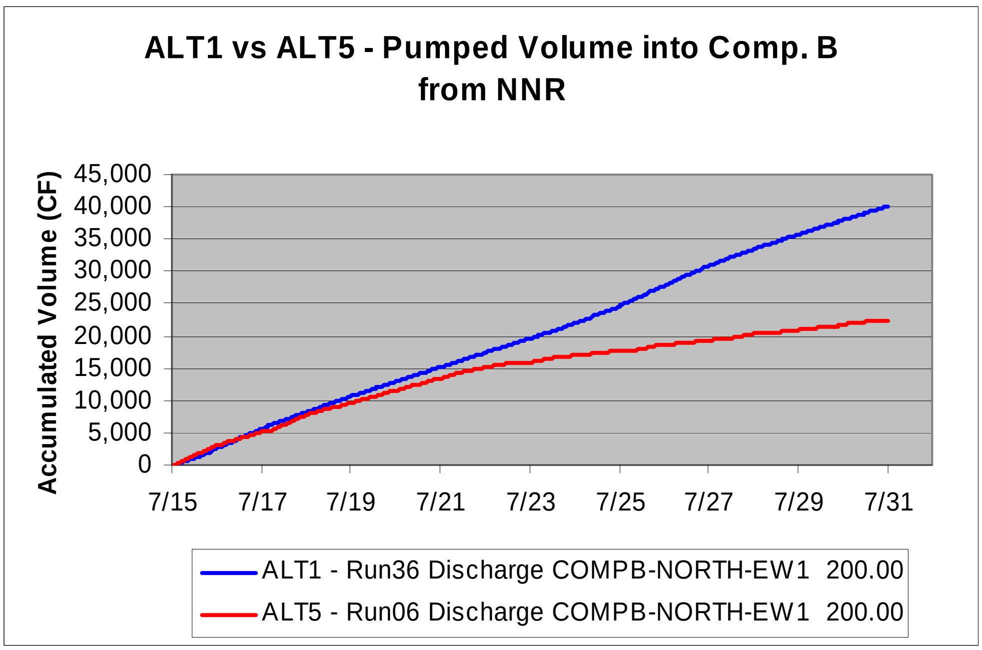



**Figure 3-21 -- Alternative 1 and Alternative 5 Accumulated Volume in Compartment B**

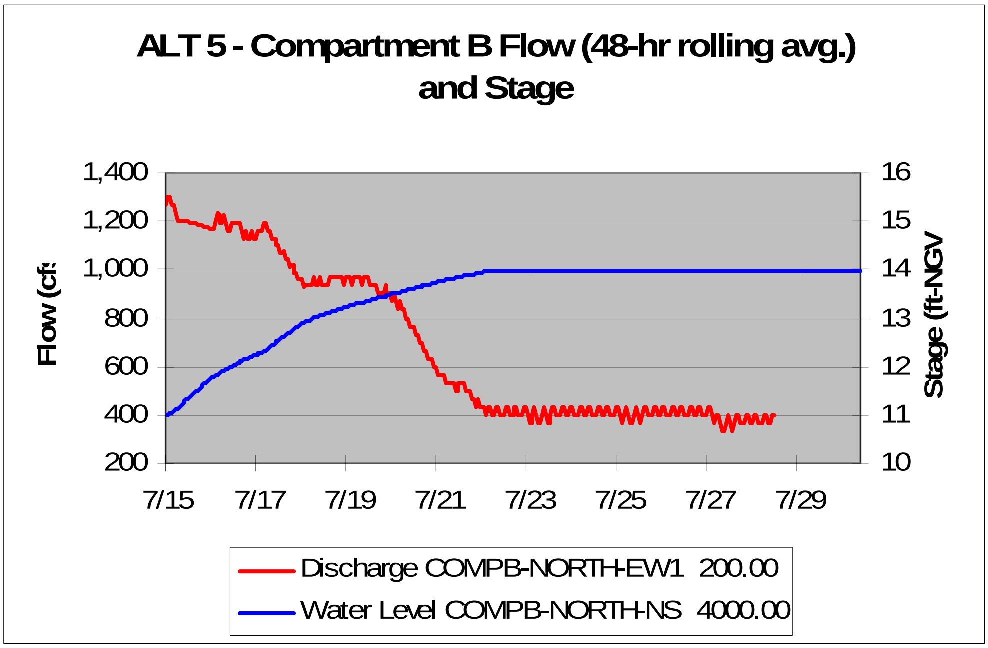



**Figure 3-22 – Alternative 5 Compartment B Inflow Rates and North Cell Water Levels**

# 4.0 PRINCIPAL CONCLUSIONS

The alternatives presented above accomplish, in varying degrees, the basic goals for achieving balanced flows and loads to the WCAs. The alternatives presented above also meet the flood control peak canal stage objectives. The alternatives positive and negative features are summarized below:

| Alternative | Positive Features | Negative Features |
|---|---|---|
| **1** | • Achieved flow redistribution<br>• Achieved flood control objectives<br>• Allows for multiple flow paths for EAA runoff | • Significant canal enlargements<br>• Requires acquisition of agricultural lands to divert runoff from the WPB Canal to the Sam Senter Canal |
| **2** | • Achieved flow redistribution<br>• Achieved flood control objectives<br>• Allows for multiple flow paths for EAA runoff | • Significant canal enlargements<br>• Requires acquisition and dredging of protected wetlands within LNWR<br>• Requires acquisition of agricultural lands to divert runoff from the WPB Canal to the Sam Senter Canal |
| **3** | • Same as Alt 1, but has additional flood control benefits to the Miami Canal through diversion of Manley Ditch runoff to the L-2 Canal | • Same as Alt 1, but requires even more acquisition of agricultural lands to divert runoff from the Manley Ditch to L-2 |
| **4** | • No additional benefits beyond the benefits for Alt 1 and 2 | • Has the largest number of new project features |
| **5** | • Same as Alt 1 | • Reduced flows through Compartment B<br>• Requires use of large inverted siphons |

# 5.0 RECOMMENDATIONS

Recommendations for further study are presented below:

1. If Alternative 3 is considered further, Manley Ditch should be surveyed so that the veracity of assumed cross sections can be confirmed or modified.

2. If Alternative 5 is considered further, the suitability of utilizing inverted siphons should be carefully evaluated. Inverted siphons in low-velocity environments that are prevalent in south Florida are prone to clogging. This assessment only assessed velocity of the siphons for extreme events.

3. Simulations should be conducted for lower runoff volumes than 3/4" to verify that the diversions are firm diversions for runoff events less than the events considered during the simulations described herein.

4. Simulations should be performed to refine structure operations during extreme events. The simulations described herein come close to but do not exactly match the diversions recommended by Burns & MacDonnell.

5. Long-term simulations should be conducted for 1-2 years to verify the annual volumes delivered to the STAs.

6. An assessment should be conducted of the WCAs to determine the regional impact of inter-basin transfers on hydroperiods of WCA 1A, WCA 2A, and WCA 3A.