C-139 Basin Phosphorus Water Quality and Hydrology Analysis

Deliverable 10.4 –Final Water Quality Improvement Projects Report

(Work Order No. CN040912-WO07-A2)

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EXECUTIVE SUMMARY

Background

Florida's 1994 Everglades Forever Act (EFA), F.S. 373.4592, establishes long-term water quality goals designed to restore and protect the Everglades Protection Area (EPA). The EFA mandates that landowners within the C-139 Basin should not collectively exceed the average annual historic total phosphorus (TP) load adjusted for rainfall. In 2002, the C-139 Basin Best Management Practices (BMPs) Regulatory Program was adopted to ensure that TP load requirements would be met. This BMP program is defined in Chapter 40E-63, F.A.C. ("Rule 40E-63").

During the 2003 legislative session, a Long-Term Plan objective was adopted for the C-139 Basin to identify urban and agricultural discharges within the basin that are candidates for cost-effective implementation of source controls. After two years of implementing the mandatory BMP program, the C-139 Basin has not been able to meet the historic TP load required by Rule 40E-63. Both the South Florida Water Management District (District) and permittees are interested in additional TP load reduction programs within the basin that will be prioritized and addressed in future BMP program optimization plans, as necessary to meet rule requirements.

In order to address TP load reduction, the Everglades Regulation Division of the District contracted ADA through Work Order CN040912-WO07 to implement the C-139 Basin Phosphorus Water Quality and Hydrology Analysis. The objective of the C-139 Basin Phosphorus Water Quality and Hydrology Analysis is to assess the current hydrologic and water quality conditions of the basin, identify locations where additional water quality and flow data is required, and identify and evaluate opportunities for water quality improvement. The project is to be completed in two phases.

Phase I

Phase I was finalized in February 2006 and included four main tasks;

1. Records Review and Action Plan
2. Field Review and Data Collection
3. Subwatershed Segmentation and Screening Level TP Assessment
4. Location of Monitoring Stations

As a result of the Phase I report, four additional monitoring stations were constructed within the C-139 Basin at locations representative of subwatershed outlets. The results are summarized in the February 1, 2006 submission of C-139 Basin Phosphorus Water Quality and Hydrology Analysis Deliverable 5.4 – Phase I Report.

Phase II

Phase II consists of developing a hydrologic and water quality model and to evaluating the technical and regulatory feasibility of water quality improvement projects. The following objectives define the scope of Phase II.

1. Develop a calibrated hydrologic and water quality modeling tool to analyze flows and phosphorus loads in the C-139 Basin. Everglades Regulatory Program staff shall be able to use the model as a tool for prioritizing resources and tailoring Best Management Practice strategies towards achieving compliance with Everglades Forever Act-mandated water quality levels. The simulation results of the calibrated WAM simulation will be visually and statistically compared to all available measured data within the basin to provide an estimate of the modeling uncertainties. The water quality model shall be user-friendly and compatible with District applications. The Consultant shall train District staff in the use of this application.
2. Identify and evaluate a maximum of five water quality improvement projects (selected projects). The recommendations/needs or project types described by C-139 Basin landowners shall be considered.
3. Describe regulatory constraints that may affect implementation of water quality improvement projects within the C-139 Basin. Provide recommendations for pursuing viable rule or policy changes.
4. Identify technical issues, cost and schedule considerations for the selected projects. Evaluating site-specific technical issues, cost and schedule does not apply to farm-level projects.
5. Note uncertainties and limitations associated with project implementation. Along with any other unidentified issues that are uncovered as the contract progresses [e.g., results of the EAA Regional Feasibility Study, Phase 2 (CN040912-WO04)].

This report documents a component of Phase II, referred to in the scope of work as Task 10, which serves to evaluate five (5) regional water quality improvement projects using the calibrated, validated model described in Deliverable 6.4.

Task 10: Water Quality Improvement Projects Analysis

The model selected for Phase II implementation is the Watershed Assessment Model (WAM), which has been used in numerous agricultural basins throughout Florida. In April 2007, ADA submitted Deliverable 6.4 which contains the development, calibration and validation of the existing condition WAM simulation. This document presents an evaluation of the five selected regional water quality improvement projects described below:

Project 1 - Reduce TP loads from the C-139 Basin by providing additional treatment and storage for the Deer Fence and S&M Canals.

Project 2 - Reduce TP loads from the C-139 Basin using the Dinner Island basin (26-310-04) for water storage and treatment of internal runoff.

Project 2A - Reduce TP loads from the C-139 Basin using the Dinner Island basin (26-310-04) for water storage and treatment of runoff from adjacent basins to the east.

Project 2B - Reduce TP loads from the C-139 Basin using the Dinner Island Basin (26-310-04) for water storage and treatment of runoff from adjacent basins to the north.

Project 3 - Reduce TP loads from the C-139 Basin by creating centrally located impoundments to provide storage and additional treatment to specific areas with the greatest water use needs and TP loading in discharges.

The calibrated WAM representation was used to evaluate these five regional water quality improvement projects because it is primarily suited for agricultural basins and is very capable of simulating land-surface processes in locations with high water table elevations. Deliverable 6.4 describes the calibration, validation and limitations of the WAM model developed as part of Phase II for the C-139 Basin.

The Baseline Simulation is an execution of the calibrated model with the existing land-use for a 36-year rainfall period of record (1965 to 2000), or "Baseline Period". The purpose of this methodology is to provide an assessment of the long-term effects of the existing conditions over a wide range of climactic conditions, and as part of a subsequent task determine relative benefits of proposed regional alternatives compared to the baseline condition. The results of the baseline simulation demonstrate how the C-139 WAM simulation can be used as a planning tool in the evaluation of the five selected regional water quality improvement projects.

Evaluation Results for Proposed Regional Projects

The results of the evaluation demonstrate that each project provides an improvement over the existing condition, however none of the projects implemented individually allow the C-139 Basin to discharge TP loads lower than the rainfall adjusted target TP loads outlined in the South Florida Environmental Report (SFER). **Figure ES.1** illustrates a comparison of the existing condition and project implementation scenarios with respect to annual TP load.

Table ES.1 compares the average annual reduction associated with each project as well as a planning-level estimate of construction cost and schedule. The average annual reduction in TP is based on the difference between the existing condition simulation and each proposed project for Projects 1, 2 and 3. The average annual TP reduction for Projects 2a and 2b are the difference between the Project 2 simulation and the proposed conditions. Based on the total cost per pound of TP removed, the most effective projects appear to be Project 2b and Project 2. Of note, Project 2b requires the implementation of Project 2, which has a significantly higher cost per pound TP removed ratio. Project 3 provides a large reduction in basin-wide TP discharges and the median cost per pound TP removed ratio. However Project 3 is also estimated to have the largest total cost and longer schedule duration associated with construction.

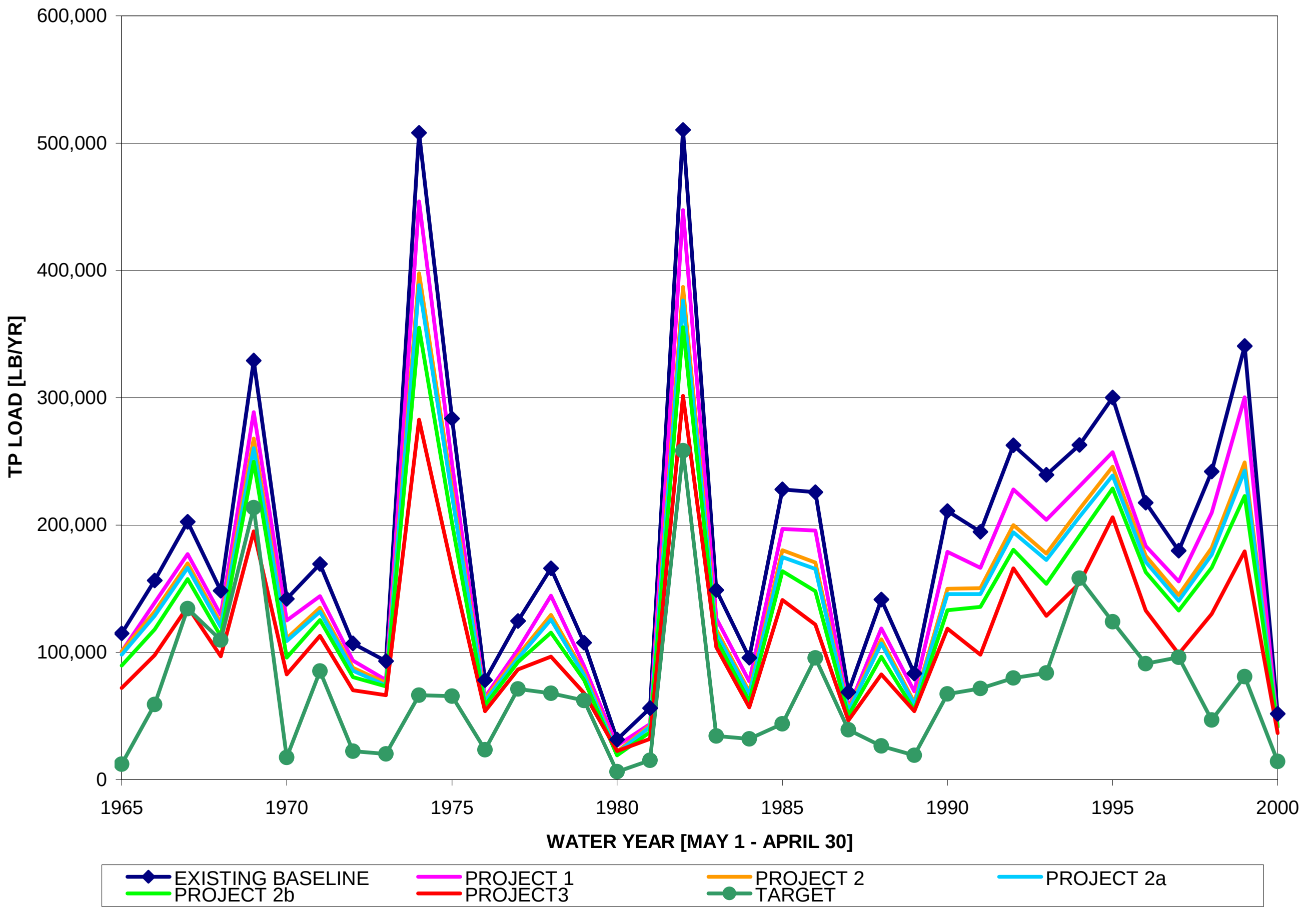

Figure ES.1: Comparison of Existing and Project Condition Simulations

Table ES.1: Comparison of Regional Water Quality Improvement Projects

REGIONAL PROJECT	AVERAGE ANNUAL TP REDUCTION [LB]	PLANNING-LEVEL COST ESTIMATE [$]	SCHEDULE [MONTHS]	COST PER AVG ANNUAL TP REDUCTION [$ / LB REMOVED]
PROJECT 1	12,197	$6,391,000	26	$524.00
PROJECT 2	27,695	$7,539,000	14	$272.00
PROJECT 2a	4,389	$20,844,000	26	$4,749.00
PROJECT 2b	13,892	$1,198,000	19	$86.00
PROJECT 3	74,923	$41,101,000	61	$549.00

Recommendations

The evaluation of the potential benefits and limitations of the five selected regional water quality improvement projects contained within this document are meant to be utilized for planning-level decision support. The following recommendations are based on the results of the evaluation and in consideration of the assumptions and limitations of the evaluation methodology.

1. Based on a comparison of the benefits and limitations of each project the recommended implementation priority for the regional improvement projects would be Project 3, Project 2, Project 2b, Project 1, and Project 2a.

2. The implementation of Projects 2a and 2b should be delayed considering the current timeframe associated with Project 2. As described in Section 11.2, the benefits of Project 2 could require between 15 and 30 years to be fully realized

3. The District should consider the timeline of the current phased restoration efforts on behalf of the FFWCC. If there is a priority in providing the benefits associated with Project 2 at an accelerated timeframe, negotiation with the FFWCC to accelerate the schedule may be required.

4. Once the proposed course of action has been selected it is recommended that a model with greater sophistication with respect to hydraulics and the surface water-groundwater interface be utilized to evaluate construction needs.

5. As described in Section 2.4 and in Deliverable 6.4, there are components of the hydrogeology that are not well understood in the C-139 Basin. Further investigation into the hydrogeologic conditions might include installation of clustered groundwater monitoring sites to get a more complete picture of the horizontal gradients of both the Surficial and Lower Tamiami aquifers and the inter-action between surface and ground waters.

6. The WAM simulations provided within Deliverable 6.4 and as part of the regional project evaluation within this document are best suited for use at the basin-scale. Due to the lumped parameter technique employed by WAM, the addition of increased detail into local or farm-scale hydrology and hydraulics may not provide significant changes in the simulated flows and loads.

1.0 INTRODUCTION

1.1 Background

Florida's 1994 Everglades Forever Act (EFA), F.S. 373.4592, establishes long-term water quality goals designed to restore and protect the Everglades Protection Area (EPA). The C-139 Basin is an approximately 170,000-acre tributary to the EPA. **Figure 1.1** depicts the C-139 Basin and other tributary basins to the EPA. The EFA mandates that landowners within the C-139 Basin should not collectively exceed average annual historic total phosphorus (TP) load adjusted for rainfall. In 2002, the C-139 Basin Best Management Practices (BMPs) Regulatory Program was adopted to ensure that TP load requirements would be met. This BMP program is defined in Chapter 40E-63, F.A.C. ("Rule 40E-63").

During the 2003 legislative session, the 1994 EFA was amended to include reference to the March 17, 2003, Conceptual Plan for Achieving Long-term Water Quality Goals (Long-Term Plan), which includes the C-139 Basin. A Long-Term Plan objective for the C-139 Basin is to identify urban and agricultural discharges within the basin that are candidates for cost-effective implementation of source controls.

After four years of implementing the mandatory BMP program, the C-139 Basin has not been able to meet the historic TP load required by Rule 40E-63. In accordance with the EFA, if the basin is determined to be out of compliance in a given year, remedial action shall be based on the landowners' proportional share of the total TP load. Rule 40E-63, requires that all permittees within the basin uniformly increase the level of BMP implementation in response to an out of compliance determination. In addition, some permittees have expressed interest in TP load reduction programs that can be implemented economically or with funding assistance, such that the basin has the best overall opportunity to comply with the rule. Rule 40E-63 also provides that, should the basin exceed the compliance requirements in four consecutive years more than four times, the rule can be revised to address compliance. Both the South Florida Water Management District (District) and permittees are interested in additional TP load reduction programs within the basin that will be prioritized and addressed in future BMP program optimization plans, as necessary to meet rule requirements.

To date, permittees in the C-139 Basin have elected not to participate in an optional farm-level monitoring program. The rationale for this non-participation may be because the type of monitoring required may not be feasible considering the hydrology of the farm basins and economic considerations. As such, recorded TP concentrations and flow data within the basin are limited.

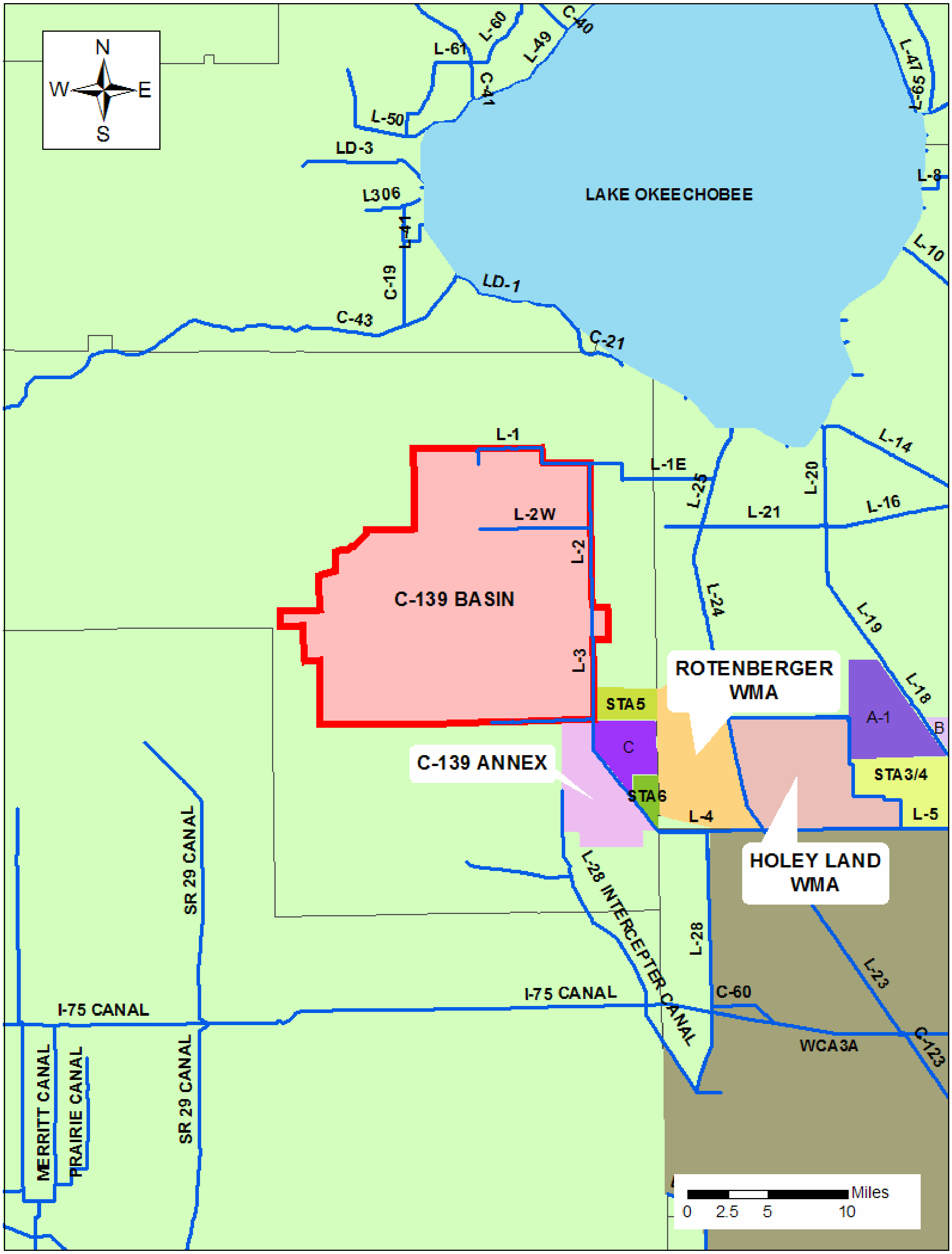

Figure 1.1: C-139 Basin Location Map

1.2 Phase I Summary

The District contracted A.D.A. Engineering, Inc. (ADA) under the General Engineering Services Contract (CN04912), between the District and ADA, to complete the work items associated with the C-139 Basin Phosphorus Water Quality and Hydrology Analysis. The objective of the analysis is to assess the current hydrologic and water quality conditions of the basin, identify locations where additional water quality and flow data is required, and identify and evaluate opportunities for water quality improvement. The project is to be completed in two phases. The Phase I report submitted in February 2006 covered the following tasks and objectives:

Task	Objective
1. Records Review and Action Plan	Review and evaluate relevant and available documentation, and prepare data collection action plan.
2. Field Review and Data Collection	Characterize flow along main C-139 canals, including direction of flow, flow rates and contributing tributaries, and District structures operation and its influence on basin hydrology.
3. Sub-basin Segmentation and Screening Level TP Assessment	Segment the C-139 Basin into drainage sub-basins based on existing hydrologic conditions and the reasons for the sub-basin delineation. Provide screening level assessment of the spatial distribution of potential TP loads within the C-139 Basin.
4. Location of Monitoring Stations	Identify feasible locations for the installation of permanent flow and TP monitoring stations to be representative of the sub-basins identified above.

The information collected as part of Phase I was used in the development of the water quality model described in this report.

1.3 Phase II Objectives

The objectives of this report (Deliverable 10.4) is to evaluate five regional water quality improvement projects utilizing the calibrated and verified hydrologic and water quality monitoring tool described in Deliverable 6.4. This report also identifies technical issues, planning-level cost and schedule considerations for the five selected regional water quality improvement projects.

1.4 Phase II Scope

Under the General Engineering Services Contract (CN04912) the District contracted ADA to complete the work items associated with Phase II (Work Order No. CN040912-WO07-A2). The scope of work for Phase II divided the project into several tasks including:

- Development of evaluation tools (Task 6),
- Identification of potential regional water quality improvement projects (Task 9) and

- Evaluation of five selected projects (Task10).

On April 6, 2007, ADA submitted Deliverable 6.4 (Final Calibration, Validation and Baseline Condition WAM Hydrologic and Phosphorus Water Quality Modeling Report), which describes the WAM representation of the C-139 Basin that is used as the project evaluation tool. Parallel to this effort ADA prepared Deliverable 9.1, submitted on February 24, 2007, which was a memorandum outlining eight proposed regional water quality improvement projects. On March 1, 2007, ADA met with District staff to review the memorandum and determine which regional water quality improvement projects warranted further analysis. ADA submitted Deliverable 9.2 on March 16, 2007 describing various scenarios for five regional water quality improvement projects. Deliverable 9.2 included preliminary planning-level cost estimates and conceptual implementation plans for each scenario. On April 20, 2007, the Distict submitted a letter authorizing five conceptual regional water quality improvement projects for detailed evaluation utilizing WAM. On April 30, 2007, ADA submitted a technical letter (Deliverable 10.1) describing the WAM specific implementation plan for each of the five authorized regional water quality improvement projects. After submission of Deliverable 10.1, the District provided additional informal comments describing modifications to each project's proposed implementation. All formal and informal District comments were combined to describe the final proposed implementation for the five projects evaluated within this report.

For the purposes of preparing Deliverable 10.4, ADA assembled a team comprised of professional staff knowledgeable in hydraulics and hydrology, water quality, and Everglades Restoration to provide a thorough evaluation of the potential impacts associated with the selected regional water quality projects outlined in the District's April 20, 2007 letter.

2.0 EVALUATION METHODOLOGY

2.1 Watershed Assessment Model (WAM)

WAM version 1.3 was developed by Soil and Water Engineering Technologies (SWET) to simulate the hydrologic, hydraulic and nutrient transport processes of watersheds with significant agricultural land uses. There have been many WAM simulations developed throughout Florida including in the Suwannee River Water Management District, St. Johns River Water Management District and South Florida Water Management District. In the region of the C-139 Basin, there have been simulations developed of the C-43 Basin to the north, the Lake Okeechobee Watershed to the northeast and the EAA to the east. WAM was selected to simulate all of these watersheds because it is primarily suited for agricultural basins and is very capable of simulating land-surface processes in locations with high water table elevations. Deliverable 6.4 describes the calibration, validation and limitations of the WAM model developed as part of Phase II for the C-139 Basin.

During the process of developing WAM representations of the selected projects, there were components of the calibrated model that were re-evaluated based on new information and new conceptualization of surface water management features. Specifically, refinements were made to the parameters defining the operation and attenuation of the Central County Drainage District (CCDD) impoundment. Two of the proposed projects incorporate impoundments used for treatment of stormwater runoff. Discussions within the ADA team and iterations of the proposed impoundments led to the following refinements to the WAM representation of the CCDD impoundment.

1. The calibrated simulation documented within Deliverable 6.4 describes the CCDD impoundment as a “Reservoir” reach type, with attenuation factors consistent with open-water bodies. However the CCDD impoundment is an above ground impoundment that is dry for portions of the year and contains significant species of wetland vegetation. Therefore the reach-type determined to have the most representative attenuation factors was “Slough”.
2. The Deliverable 6.4 calibration simulation includes assumed control elevations for the structure operations at the downstream outlet of the impoundment. Based on conversations with staff at the Clewiston Field Station it was determined that the current control strategy practiced by the CCDD is more conservative, therefore the simulated control strategy was refined to match the existing condition.
3. Above ground impoundments simulated within WAM do not assume any seepage through the surrounding levees. However, levees within the C-139 Basin experience significant amount of seepage due to the types of soils and construction methods implemented in this region of the District. Therefore, the calibrated model was modified to include seepage by including additional outfalls with weirs sized to re-create the stage-flow relationship shown in seepage situations. Further details describing the representation of levee seepage within WAM can be found in Section 4.2. The geometry of the reach cross-sections was modified to incorporate the

seepage refinements and appropriately represent available storage within the impoundment.

These refinements affected the calibration and Baseline Simulation results documented in Deliverable 6.4. In most cases the affect is an improvement on the calibration provided in Deliverable 6.4. However, the affect is not large with respect to the overall TP loads of the C-139 Basin. The scope of this document is an evaluation of the proposed regional water quality improvement projects, therefore further details and analyses of the calibration refinements will not be included herein. The complete details of the calibration refinement will be included within the Phase II Consolidated Report (Deliverable 14.1).

2.2 Baseline Simulation Evaluation

The Baseline Simulation is an execution of the calibrated model with the existing land-use for a 36-year rainfall period of record (1965 to 2000), or "Baseline Period". The purpose of this methodology is to provide an assessment of the long-term effects of the existing conditions over a wide range of climactic conditions, and as part of a subsequent task determine relative benefits of proposed regional alternatives compared to the baseline condition. The temporal results presented within Deliverable 6.4 are compared with the refined calibration results in **Figure 2.1.** There is a reduction in annual TP load that can be attributed to the refinements to the CCDD impoundment, as evidenced in the comparison. This reduction is attributable to the increased TP attenuation consistent with the CCDD impoundment being re-classified as a slough.

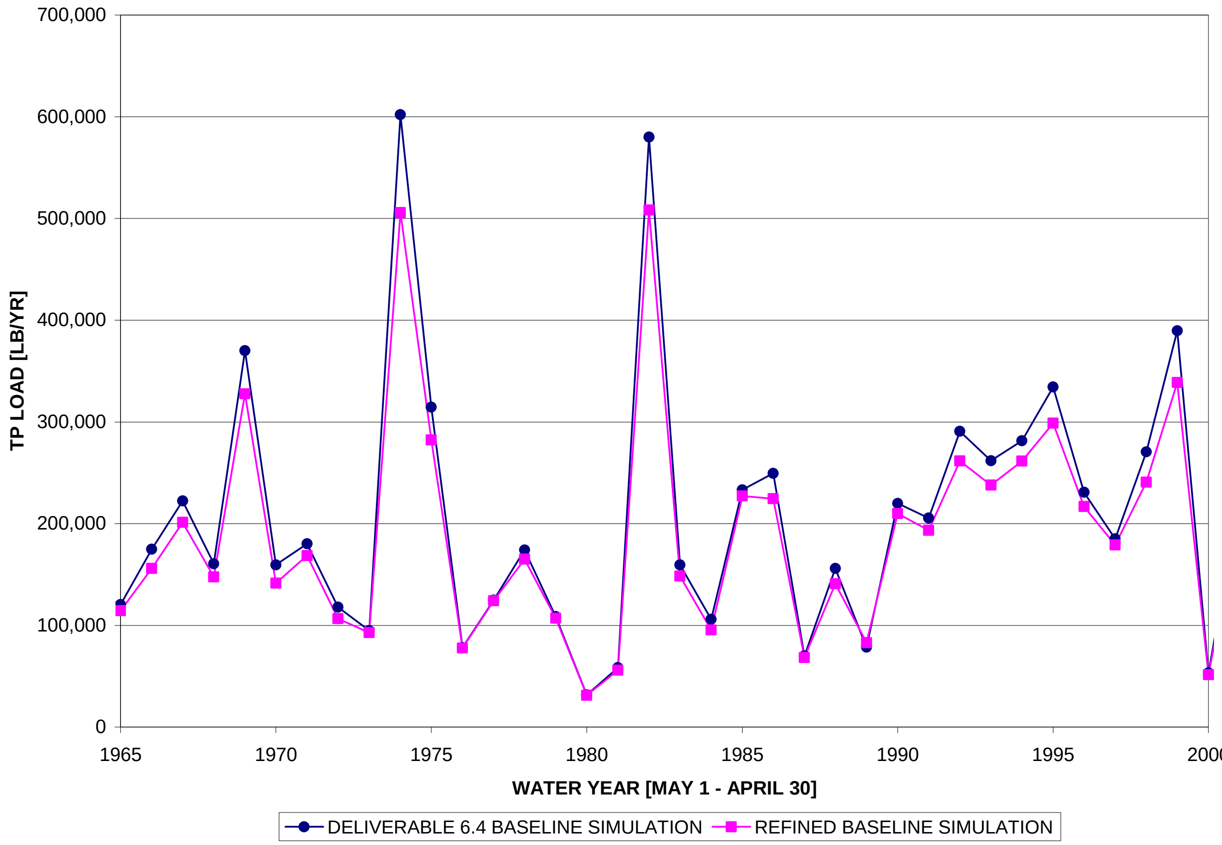

Figure 2.1: Annual Simulated TP Loads for the C-139 Basin

The results of the baseline simulation presented above demonstrate how the C-139 WAM simulation can be used as a planning tool in the evaluation of the five selected regional water quality improvement projects. In the sections that follow, each proposed improvement project is represented within WAM, and the temporal results of that scenario are presented in comparison with the existing condition baseline results shown above. The relative reduction in flows and loads demonstrated by this comparison indicate the benefits associated with the improvement project.

2.3 Compliance Target TP Load Evaluation

As described in Section 1.1, the EFA mandates that landowners within the C-139 Basin not collectively exceed the annual average TP load adjusted for rainfall. The rainfall adjustment is based on observed rainfall and calculated loads during the period extending from October 1, 1978 to September 30, 1988. The annual average TP load calculated for this period is 36.8 metric tons or 81,130 pounds per year for the entire C-139 Basin. An exponential regression of paired sets of annual rainfall and TP load data calculated during the baseline period is used to determine the target annual TP load for the basin. The function defined by the exponential regression is illustrated in **Figure 2.3**. This equation applies for rainfall amounts within certain ranges and calculates a target load (equal to 50% of the confidence interval) and a maximum limit load (equal to 90% of the confidence interval). If the calculated load is less than the target load, the C-139 Basin is in compliance. Utilizing this equation and examining the average annual rainfall for the gages used in the baseline simulation, a time series of annual target loads for each year was generated. This target load time series provides a metric for evaluation of each regional water quality improvement project. In order to illustrate the disparity between the target load and the existing condition WAM Baseline Simulation, **Figure 2.4** presents the simulated, calculated and target annual TP loads for the Baseline Period.

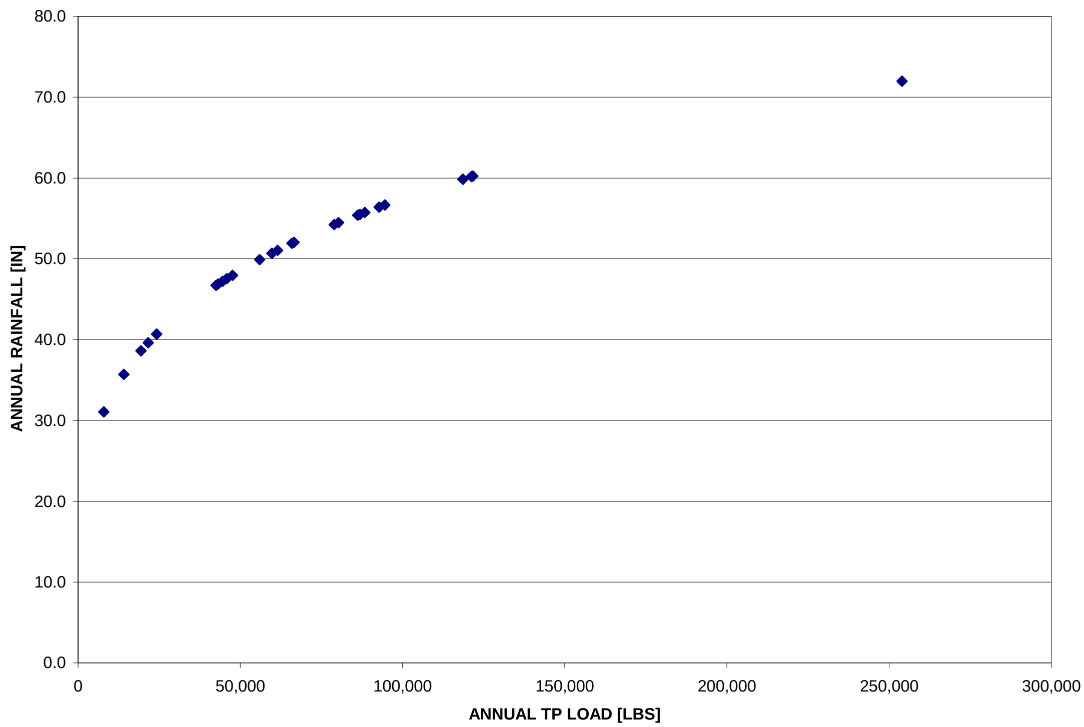

Figure 2.3: Exponential Regression defining C-139 Basin Compliance Targets

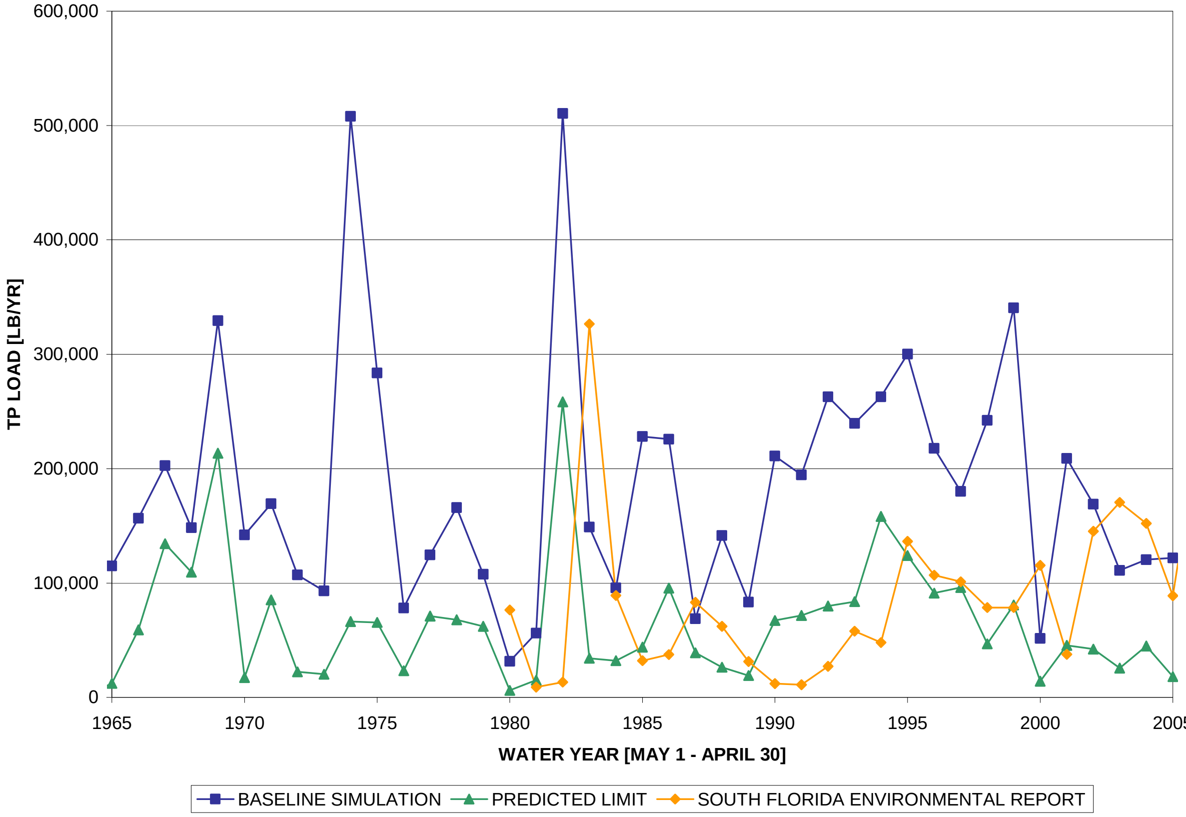

Figure 2.4: Comparison of Simulated, Measured and Targeted TP Load

As **Figure 2.4** illustrates there is a significant difference in magnitude between the simulated and observed for most of the observation period. This is expected since the simulation predicts what the existing conditions watershed response would be to historic meteorological forcings. The variability in the magnitude decreases during the 2000 to 2005 period which is expected since the existing condition of the watershed more closely represents the land uses and management practices observed. Understanding these differences is essential to interpreting the baseline simulation results for each proposed regional water quality improvement project. TP load reductions will be examined in terms of annual reduction percentage, and also in terms of absolute quantities to provide a clear comparison with simulated and targeted conditions. **Table 2.1** below describes the average annual TP load in pounds for each of the time series illustrated in **Figure 2.4**.

Table 2.1: Average TP Load for Targeted Observed and Simulated Conditions

DATA SERIES	AVERAGE ANNUAL TP LOAD
Average Target Limit (1965 – 2005)	64,108
Average Observed (1980 – 2005)	81,868
Average Simulated (1965 – 2005)	188,715

2.4 Comparison of Annual Compliance Target and Estimated TP Loads

During the process of evaluating the proposed projects in comparison with the rainfall-adjusted target TP load, the ADA team investigated additional relationships between simulated and measured parameters within the C-139 Basin. When comparing the historical data of estimated TP loads (based on field measurement) with target TP loads there is a consistent correlation until year 2000, after which the measured estimate TP loads begin to consistently exceed the target TP load. The divergence of calculated and target TP loads is more clearly demonstrated for the cumulative annual TP load, as shown in **Figure 2.5**, which also includes a plot of the difference in elevation between the surficial aquifer (water table) and the Lower Tamiami aquifer potentiometric head. The cumulative annual TP load divergence appears to occur approximately four years after the Lower Tamiami aquifer elevation becomes significantly lower than the surficial aquifer. One potential hypothesis for this change is that increased loads in recent years are due to the importation of Lower Tamiami aquifer water to the C-139 Basin as irrigation for which demand increases as land-use intensifies.

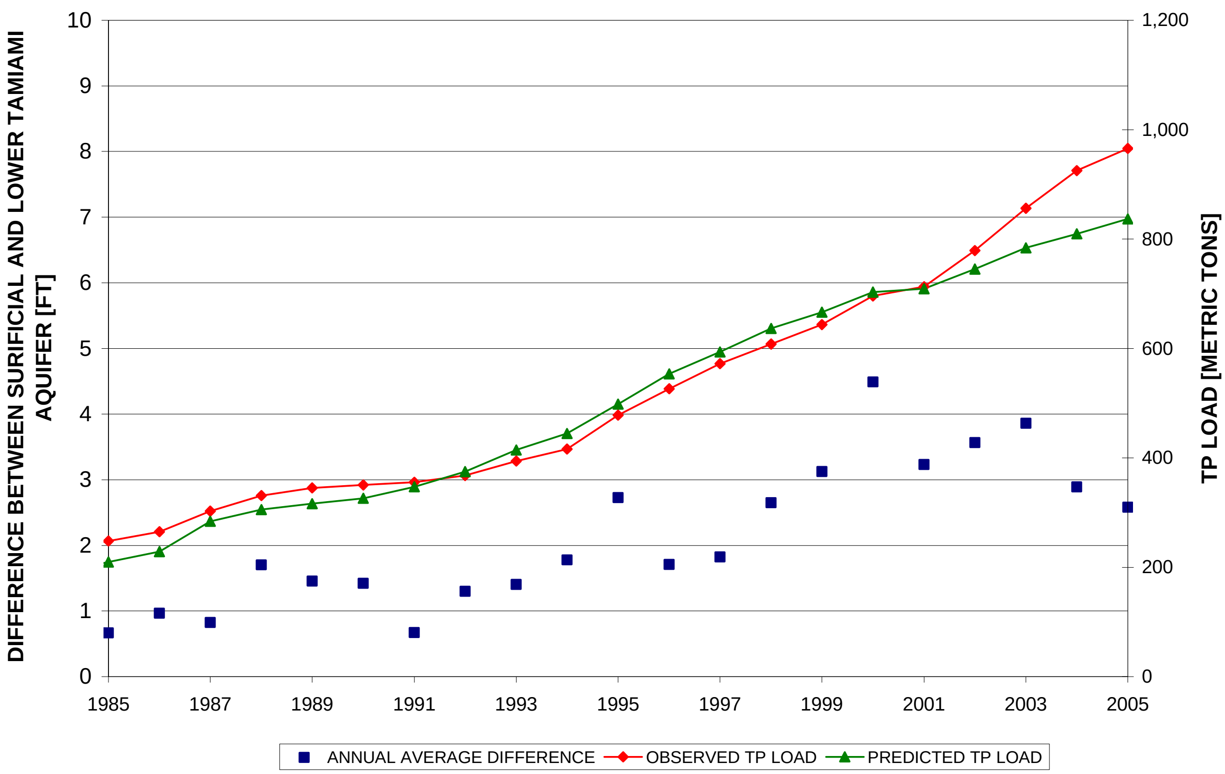

Figure 2.5: Estimated and Target Loads and the Disparity between the WT and LTA

Another potential reason for the divergence is the modification of the downstream surface water management infrastructure associated with the operation of STA 5 and STA 6, which began at approximately the same time. In addition, there is a potential that the quality of historic and current TP load measurement are not congruent, and the disparity may be manifested in the divergence shown. Therefore the relationship illustrated in **Figure 2.5** is insufficient to provide a conclusive relationship between the management of subsurface withdrawals and basin-wide TP loads. However the potential impacts of understanding this relationship warrant further investigation into the hydrogeology of the region.

3.0 REGIONAL WATER QUALITY IMPROVEMENT PROJECTS

3.1 General Project Descriptions

It is apparent from data collected and stakeholders' input that water management and storage are key concerns associated with achieving the TP loading goals required by the Everglades Forever Act. Therefore, each selected project involves the increased water storage via modified management schemes or through the construction of one or several impoundments within the C-139 Basin. The variation between each selected project is primarily dependent on the location of the proposed improvement, and on the contributing area which the impoundment is intended to serve. The five selected projects as defined in the District April 20, 2007 letter are as follows:

- Project 1 - Reduce TP loads from the C-139 Basin by providing additional treatment and storage for the Deer Fence and S&M Canals.
- Project 2 - Reduce TP loads from the C-139 Basin using the Dinner Island Basin (26-310-04) for water storage and treatment of internal runoff.
- Project 2A - Reduce TP loads from the C-139 Basin using the Dinner Island Basin (26-310-04) for water storage and treatment of runoff from adjacent basins to the east.
- Project 2B - Reduce TP loads from the C-139 Basin using the Dinner Island Basin (26-310-04) for water storage and treatment of runoff from adjacent basins to the north.
- Project 3 - Reduce TP loads from the C-139 Basin by creating centrally located impoundments to provide storage and additional treatment to specific areas with the greatest water use needs and TP loading in discharges.

Figure 3.1 below illustrates the general locations of each proposed project within the C-139 Basin. Considering that the construction of a regional impoundment requires due diligence with respect to economic and regulatory concerns, the locations illustrated in **Figure 3.1** are not representative of locations for proposed construction projects, but instead represent conceptual locations for proposed impoundments.

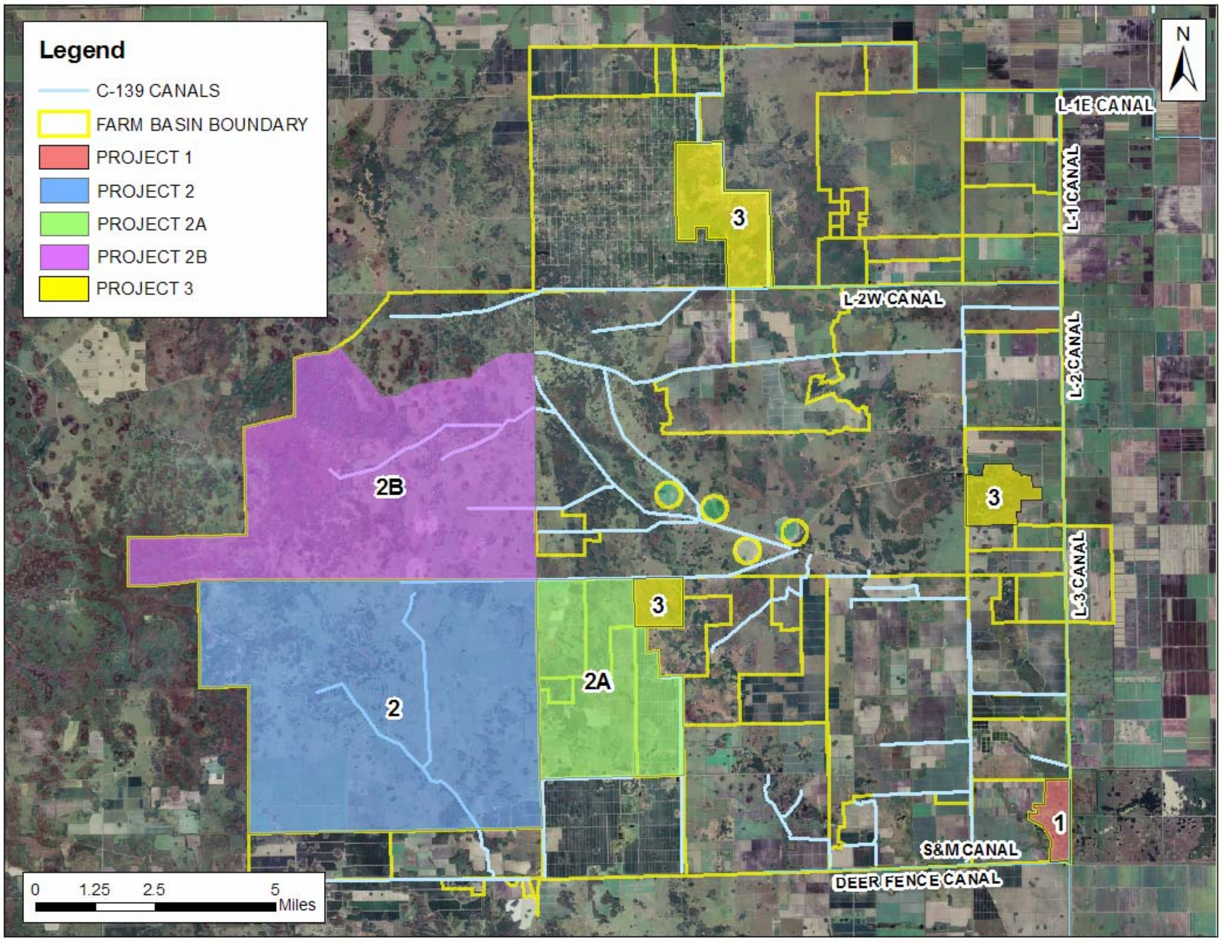

Figure 3.1: Location Map for Regional Water Quality Improvement Projects

3.2 Preliminary Optimization

This report contains a planning-level evaluation of the implementation of each regional water quality improvement project described above. This evaluation consists of a comparison of the impacts of the proposed condition over the Baseline Period with the existing condition Baseline Simulation. Since the Baseline Period is a 36 year period of record, the computational time associated with a single run of the hydraulics submodel (Blasroute) can be greater than 12 hours. Therefore prior to evaluating the impact of each project over the course of the Baseline Period, the first step is to develop a short-term simulation within WAM to determine the effectiveness of the proposed size, location and associated infrastructure modifications.

If the short-term simulation indicates that the size, location or associated infrastructure is insufficient to meet the intended project's goal, the components of each project are modified until the project is effectively represented within WAM. The calibration and validation simulation included 2003, 2004 and 2005 comparisons assuming two years of hydrologic "spin-up" (2000-2001) and one year of hydraulic "spin-up" (2002). A "spin-up" period is utilized to allow the model to negate the error associated with poorly representative initial conditions. Therefore the period selected for each short-term simulation was 2002 through 2003. Each short-term simulation includes two years of hydrologic “spin-up” (2000-2001) and a two year hydraulic period (2002-2003), where 2002 can be viewed as a hydraulic “spin-up” year. With regards to precipitation totals, 2003 was the median rainfall year of the calibration and validation period with an annual total of 48.5 inches. The configuration that demonstrates the optimal proficiency at accomplishing the objectives of each regional project during the short-term simulation is then simulated for the Baseline Period in order to provide a complete evaluation of the project’s effectiveness.

Each short-term preliminary evaluation is described with respect to percentage of reduction in TP loads from the existing condition simulation. Each project is regional in nature and none of the proposed 5 projects addresses the entire C-139 Basin individually. As such, the preliminary comparisons are not made at the watershed scale but are instead performed at the local scale by comparing TP loads at the most upstream reach(es) that capture the effects of each project. A percentage of load reduction at these effected reaches is utilized in the preliminary evaluation. However these comparisons only describe the proposed projects benefits at the subwatershed or catchment scale and do not describe the effects of the load reduction on the overall watershed discharge.

4.0 PROJECT 1

4.1 Project 1 Objectives and Components

The April 20th, 2007 letter from the District describes the objective of Project 1 as “Reduce TP loads from the C-139 Basin by providing additional treatment and storage utilizing the Deer Fence and S&M Canals”. Additional detail from this recommendation describes the project as a combination of the following:

1. utilizing the District-owned property at the south-east corner (O’Bern’s pasture Basin 26-319-01) as a storage area serving both the Deer Fence and S&M Canals,
2. interconnection of the eastern-most sections of the Deer Fence and S&M Canals, and
3. potentially increasing the storage capacity of the Deer Fence and S&M Canals.

Identified in **Figure 4.1** below, the Basin 26-319-01 is a 520-acre parcel in the southeast corner of the basin. This parcel is owned and maintained by the District. The primary objective of utilizing this parcel as part of Project 1 implementation is to utilize the identified parcel for water quality treatment. Secondary and tertiary benefits of implementing Project 1 may include water supply storage for future use by private land-owners, and use of the proposed impoundment as a surge-tank to optimize the treatment efficiency of the STA-5/Compartment C/STA-6 complex. The parcel is located within the SM-01 sub-basin which, as described in **Table 2.1,** has a TP loading rate that is comparatively higher than other sub-basins according to the baseline simulation. Additionally, it is alongside the L-3 Canal across from STA-5 and upstream of the bypass structure G-406.

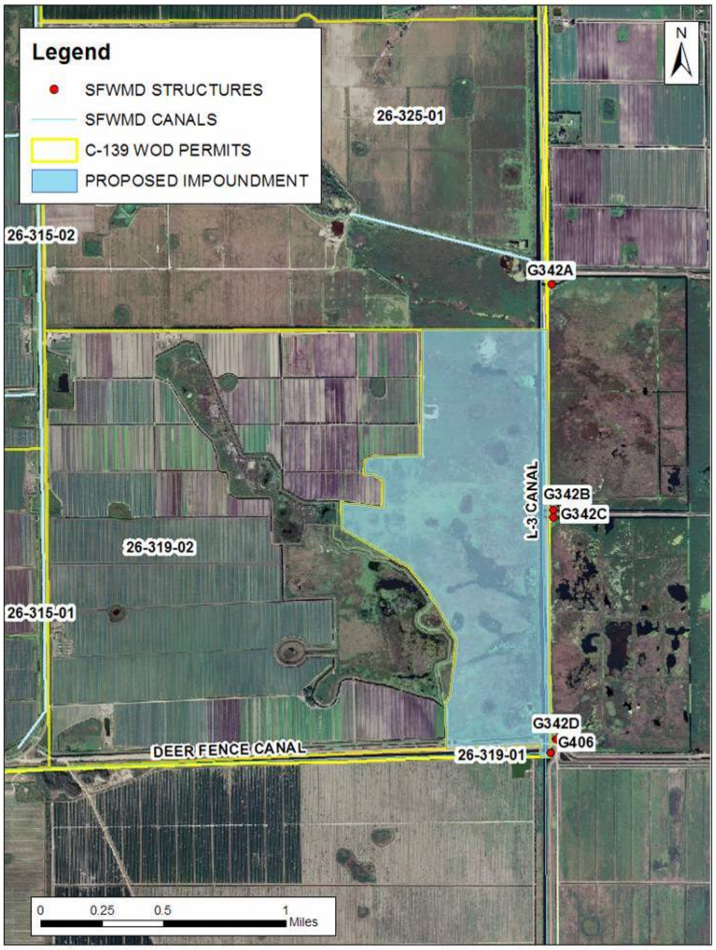

Figure 4.1: Location of the District owned O'Bern Tract

4.2 Project 1 Optimization

As described above, the optimization methodology includes three basic components: impounding water within Basin 26-319-01, interconnecting the Deer Fence and S&M Canals and widening the Deer Fence and S&M Canals. Each of these components includes the potential for multiple configurations of infrastructure. The sub-sections that follow illustrate the components of each iteration and the resulting impacts on TP water quality. At the end of this section, **Table 4-1** describes the potential benefits for each sub-section configuration. Additionally, a comparison of how these benefits compare with the targeted limit is described in Section 2.3 above.

4.2.1. Basin 26-319-01 Impoundment

The Deer Fence and S&M Canals both include existing water control structures that impact the implementation of Project 1. At the end of the S&M Canal, there is an existing sheet-pile weir that will require a retrofit, to allow canal stages within the S&M Canal to be sufficient for inflow into the proposed impoundment, and to prevent inflow from the L-3 Canal. On the Deer Fence Canal, southwest of CR846, is a gated culvert structure (described in Deliverable 6.4). This gate maintains stages upstream at approximately 21.3 feet (ft) relative to the North American Vertical Datum of 1988 (NAVD88) when closed, while downstream stages are reflective of the operational conditions for STA5 and the G-406 structure. **Figure 4.2** illustrates the existing condition reach network for this region.

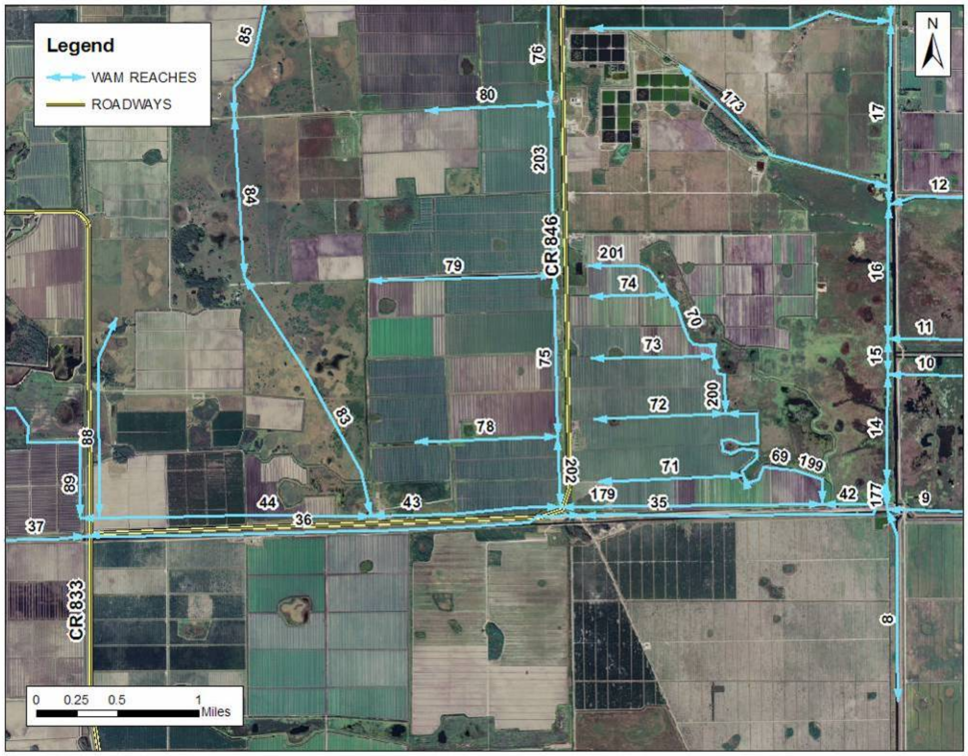

Figure 4.2: Existing Condition WAM Reach Network

For the first iteration, the Deer Fence gated culvert operations were not modified from those defined in the calibration. The S&M Canal weir was assumed to be raised to 14.7 ft NAVD, consistent with the elevation of the nearby O'Bern tailwater recycling project (SFWMD, 2006). The inflow structure for the proposed impoundment was assumed to be a pump with a capacity of 140 cubic feet per second (cfs) based on a flow-duration-curve analysis of simulated S&M Canal flows described within Deliverable 9.2. The outflow structure for the Project 1 impoundment was assumed to be a weir with a crest elevation of 17.0 ft NAVD, or 4 feet above the proposed impoundment base (existing elevation within the O'Bern property). An additional outflow structure is included connecting the proposed impoundment back to the S&M Canal. In the case where canal stages within the Deer Fence and S&M Canals are low and water is stored within the impoundment, this strucure can allow the flexibility necessary to meet that provision. The construction of the proposed impoundment will incorporate a 5.1 mile levee with a top of berm elevation of 21.0 ft NAVD88 in order to provide 4 feet of freeboard that allows for wave-runup and an additional safety factor. The reach network associated with this proposed configuration is illustated in **Figure 4.3**.

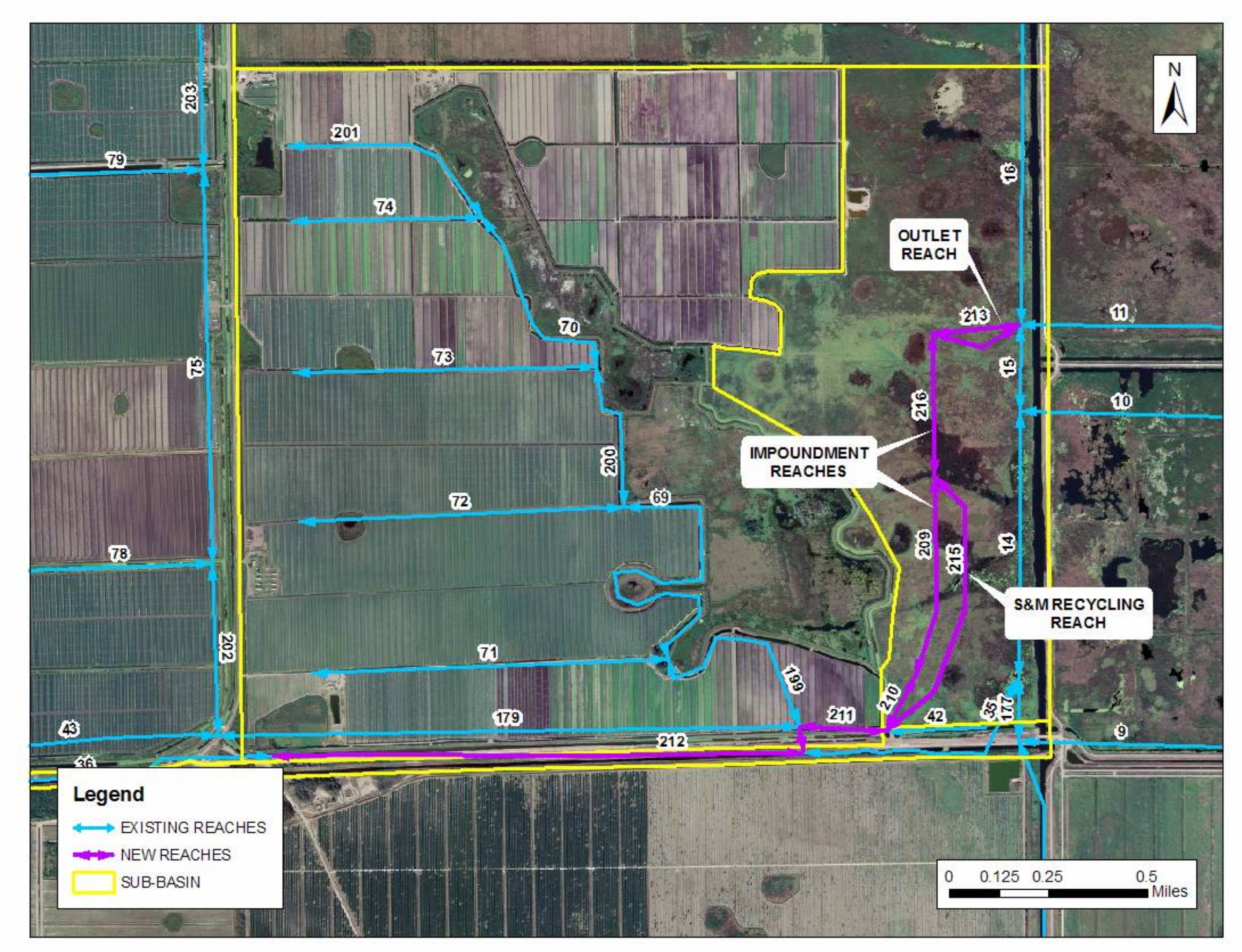

Figure 4.3: Basin 26-319-01 Proposed WAM Reach Network

Assuming this intial configuration, the stages and flows for the proposed impoundment are illustrated in **Figure 4.4**, and **Figure 4.5** illustrates the existing and proposed condition hydrograph for the Deer Fence and S&M Canals The TP load reduction is shown in **Figure 4.6**. The simulated cummulative TP load reduction for 2002 is 7% and for Water Year 2003 is 12%.

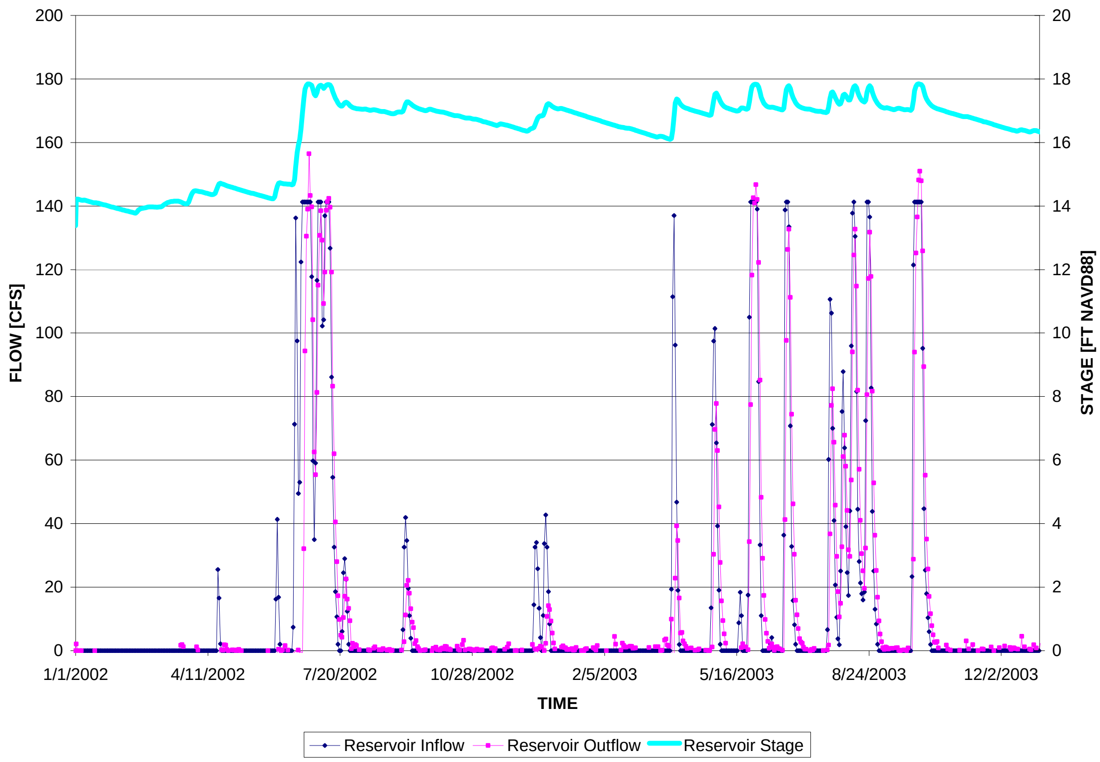

Figure 4.4: Stages and Flows for the Proposed Impoundment

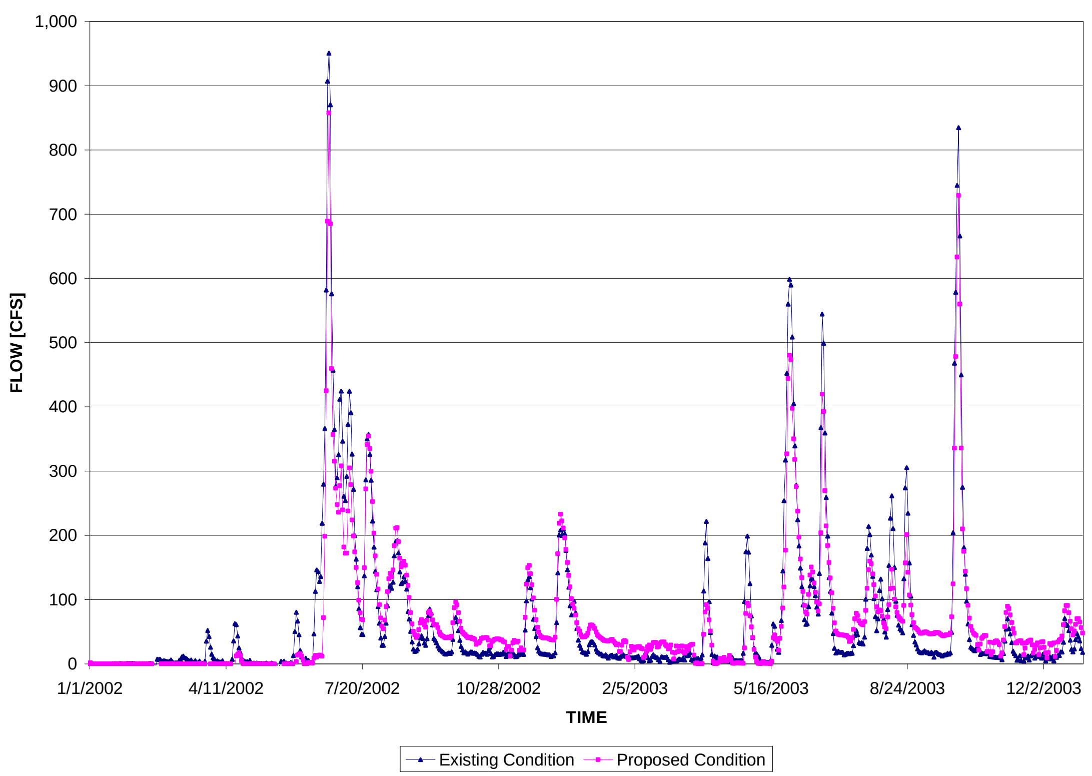

Figure 4.5: Existing and Proposed Flows for Deer Fence and S&M Canals

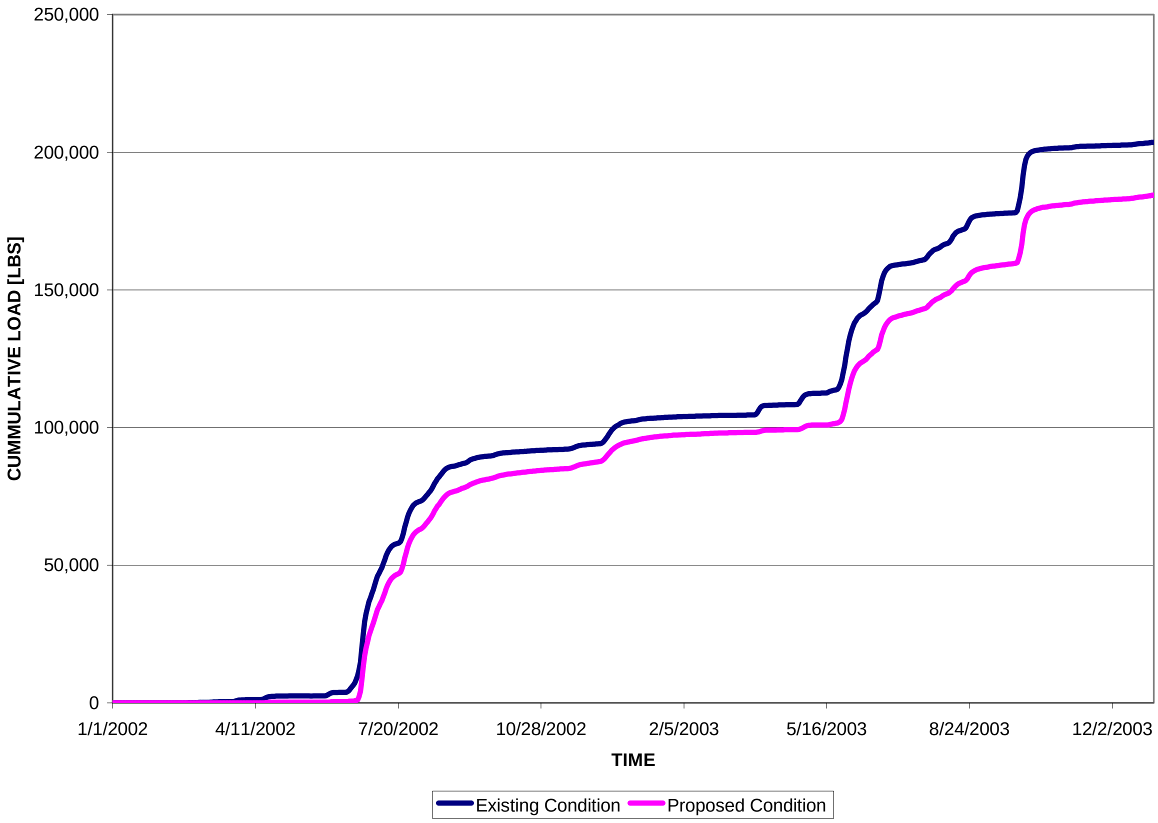

Figure 4.6: Cumulative TP Load for the Existing and Proposed Condition

4.2.2. Impoundment Levee Seepage

For the previous iteration shown in the section above, once the reservoir is full it takes approximately eight to nine months to dry out. This is due to the basic representation of an impoundment in WAM which does not account for seepage losses through the surrounding levees. Therefore, the only losses from the impoundments (in the previous iterations) are from open water evaporation. The proposed impoundment levees, however, shall be considered pervious and, as a result, must account for the losses due to seepage flow through the levees.

Quantifying the amount of seepage through a proposed earthen levee can be a difficult task dependent on the hydraulic conductivity of the material used to construct the levee, the length of the levee and the difference in hydraulic head across the seepage boundary. One methodology for estimating seepage is illustrated in the water balance reports prepared for each of the Stormwater Treatment Areas (STA). This methodology estimates seepage as a function of the hydraulic head, the length of the seepage boundary or levee and a coefficient of seepage. The coefficient of seepage is expressed in cfs per mile of levee per foot of hydraulic head. Because of natural variability in the hydraulic conductivity of the levees, the seepage coefficient was adjusted to minimize the sum of the squared daily water budget error. The coefficients of seepage found in the STA-5 Water Budget Analysis (Tech Pub ERA#427, 2005) were utilized to determine anticipated seepage from the proposed impoundment since STA-5 is located directly across the L-2/L-3 Canal from Basin 26-319-01. Two different coefficients of seepage values were reported for STA-5 Cells 1 and 2, so their average value of 2.15 cfs of seepage loss per foot of head per linear mile of levee was used for this iteration.

It is possible in WAM to set a leakage rate through the wetted perimeter of any specific reach, based on a canal leakance coefficient. However, any volume of water that is conveyed through that boundary leaves the model domain, which would cause a net loss in the mass balance. Therefore, an accurate representation of seepage would be to create an alternate outflow from the impoundment with a control structure that simulates the effects of levee seepage flows. The structures available in WAM to represent the seepage flow through the impoundment are the orifice, pipe and weir structures. The estimated seepage flow was compared with respective equivalent orifice, pipe and weir structures. **Figure 4.7** illustrates a comparison over the range of proposed impoundment stages between the estimated outflow due to seepage and the orifice, pipe and weir equivalent structures. The weir structure was used to simulate seepage since it demonstrated the most conservative approach. The equivalent weir length of 1.76 ft was estimated to be able to accommodate for the seepage flow through the proposed impoundment levee.

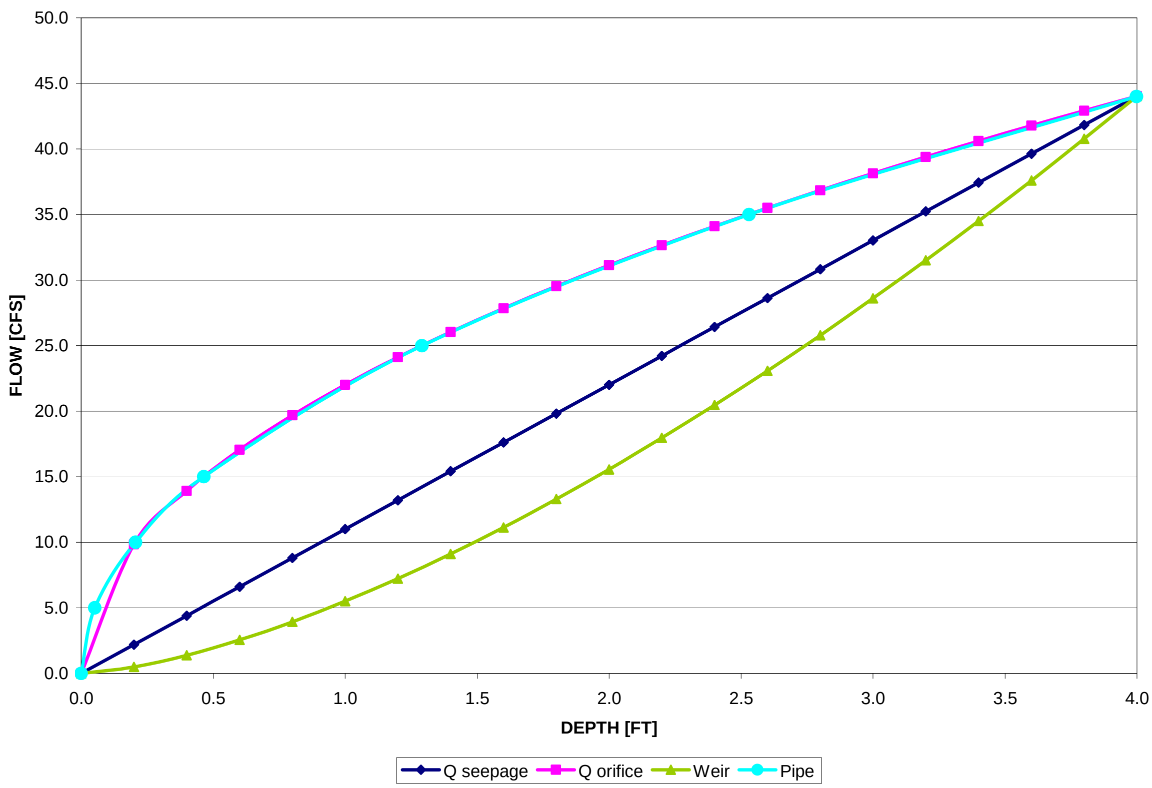

Figure 4.7: Stage-Discharge Comparison of Orifice and Seepage Estimate

The set-up within WAM to implement the seepage scenario involves creating two separate outfalls for the reservoir. The first outfall is a weir with a crest elevation set to maintain four feet of depth within the impoundment. This outfall discharges to a reach connecting the impoundment with the L-3 Canal. The second outfall is a weir sized for the simulation of seepage flow that discharges to a (second) reach connecting the impoundment with the L-3 Canal. Since the water that passes through the second outfall is representative of water that has passed through the levee, it is assumed that the soluble and sedimentary P would be bound up in the soil matrix of the levee as the water flows through. The attenuation coefficients for the downstream reach of the second outfall or "seepage reach" are modified to represent this process. Based on best professional judgement the "a" coefficient of the attenuation equation (described in Deliverable 6.4) was increased to 0.5 for soluble P and 0.11 for sediment P. **Figure 4.8** illustrates the reach schematic associated with this simulation.

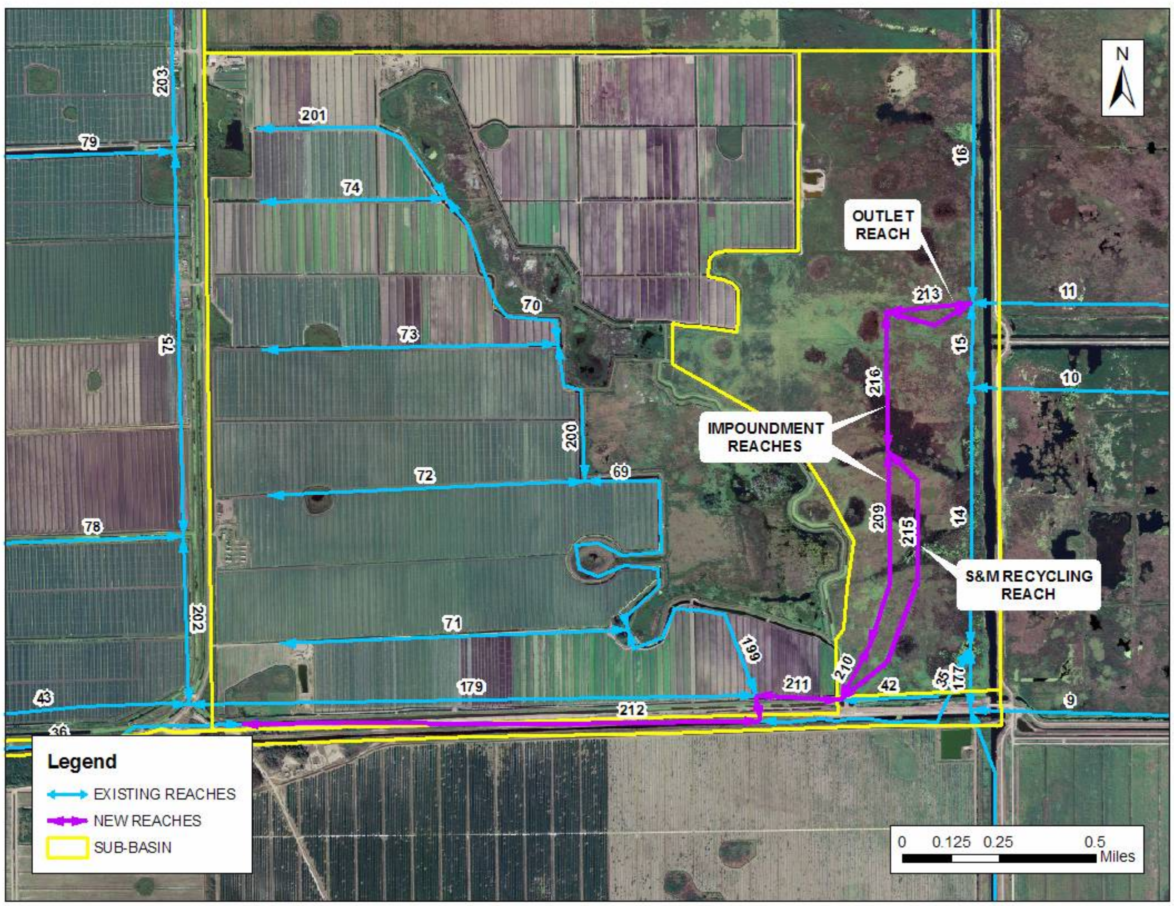

Figure 4.8: Schematic of Seepage Iteration

Figures 4.9, **4.10** and **4.11** illustrate the impact of considering levee seepage to the proposed project's effectiveness. Based on the assumptions described above, the simulated cummulative TP load reduction for Water Year 2002 is 8% and for Water Year 2003 is 13%.

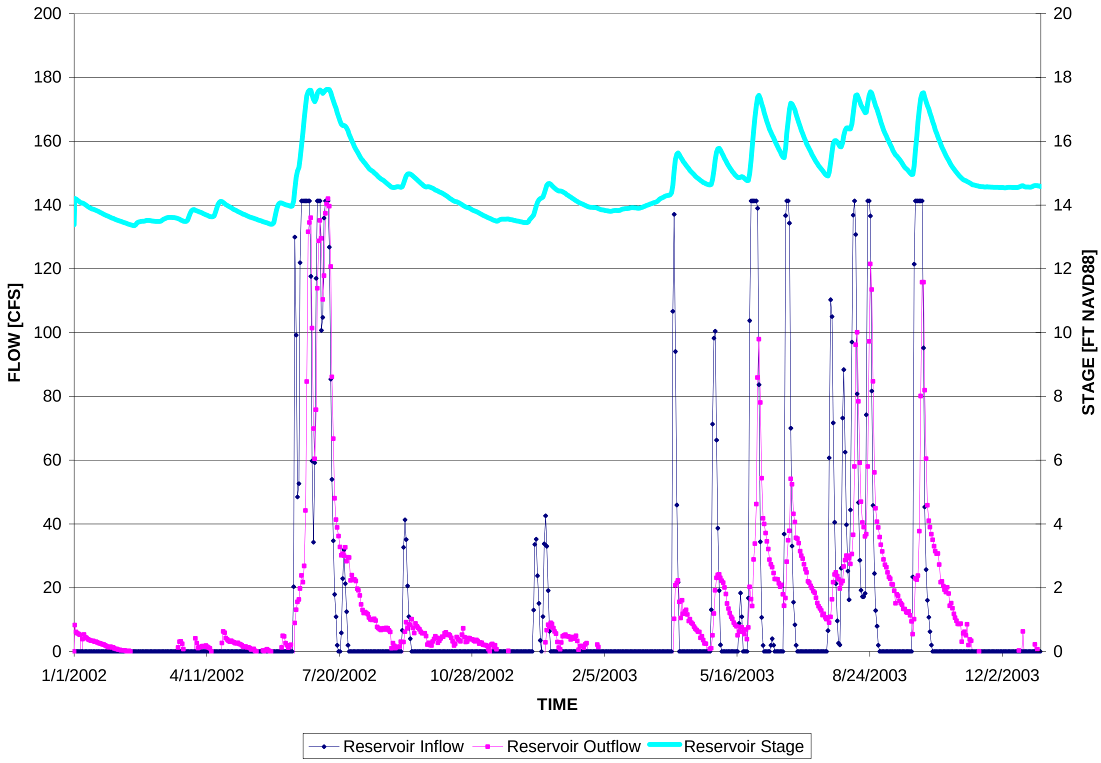

Figure 4.9: Stages and Flows for the Proposed Impoundment

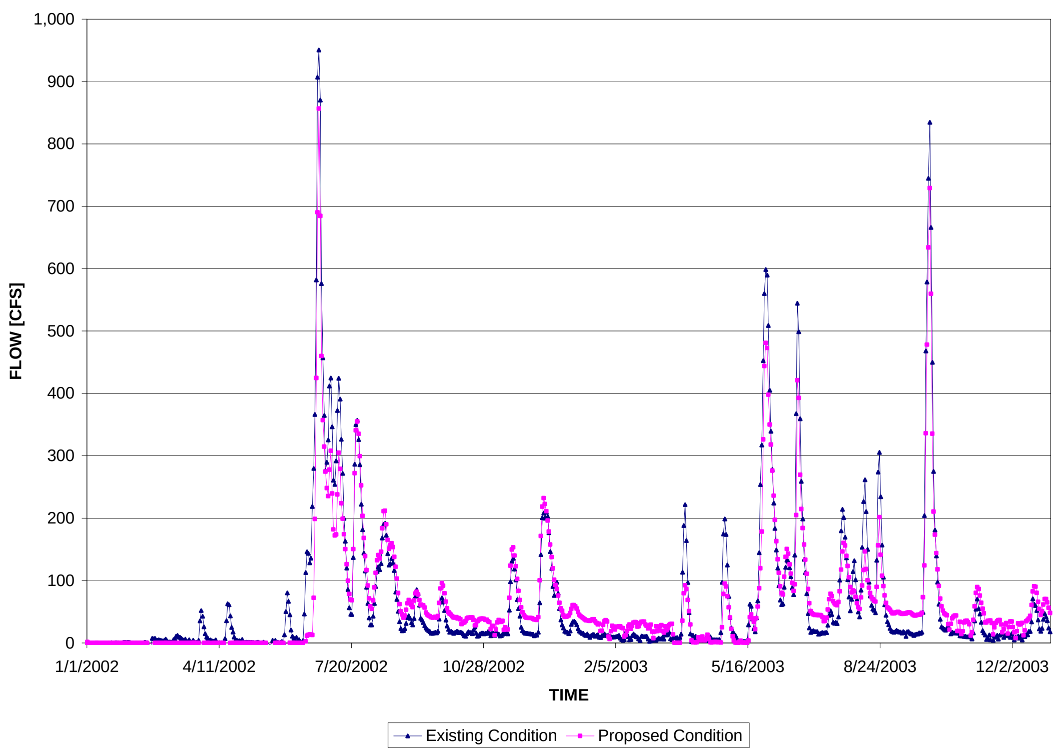

Figure 4.10: Existing and Proposed Flows for Deer Fence and S&M Canals

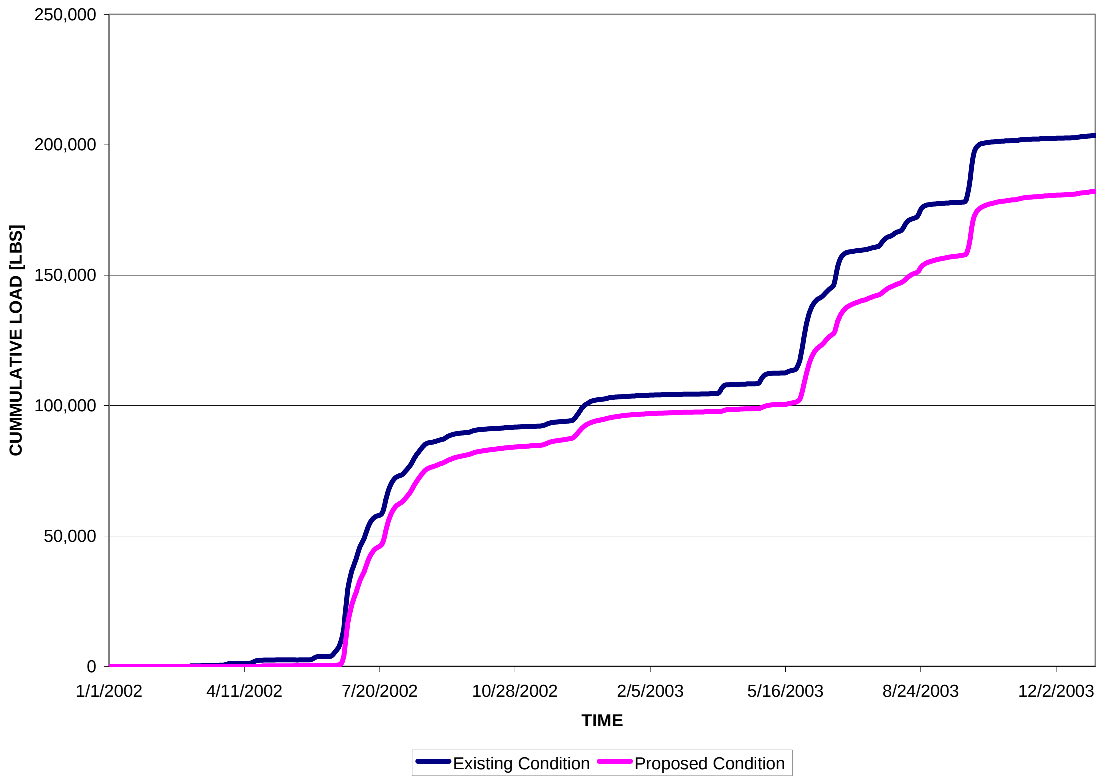

Figure 4.11: Cumulative TP Load for the Existing and Proposed Condition

4.2.3. Eastern Deer Fence and S&M Connection

The District's recommendation (from the letter dated April 20th 2007) describes an investigation into a connection of the "eastern most sections" of the Deer Fence and S&M Canals. In order to facilitate an eastern-most connection, the existing gated culvert would need to be removed while a new water control structure would need to be created downstream of the interconnection. In order to represent this connection within WAM, the structure representing the existing Deer Fence gated culvert structure was moved to the eastern end of the Deer Fence Canal (approximately 2 miles downstream). The location, geometry and assumed operations of all other structures described in the previous impoundment iteration (Impoundment Levee Seepage) were maintained. The geometry of the interconnection is assumed to be a short reach with a weir at the downstream (northern) end. The weir crest elevation was assumed to be equal to the spill crest elevation of the Deer Fence Canal structure, which is at 22.7 ft NGVD. The reach network associated with this proposed configuration is illustated in **Figure 4.12**.

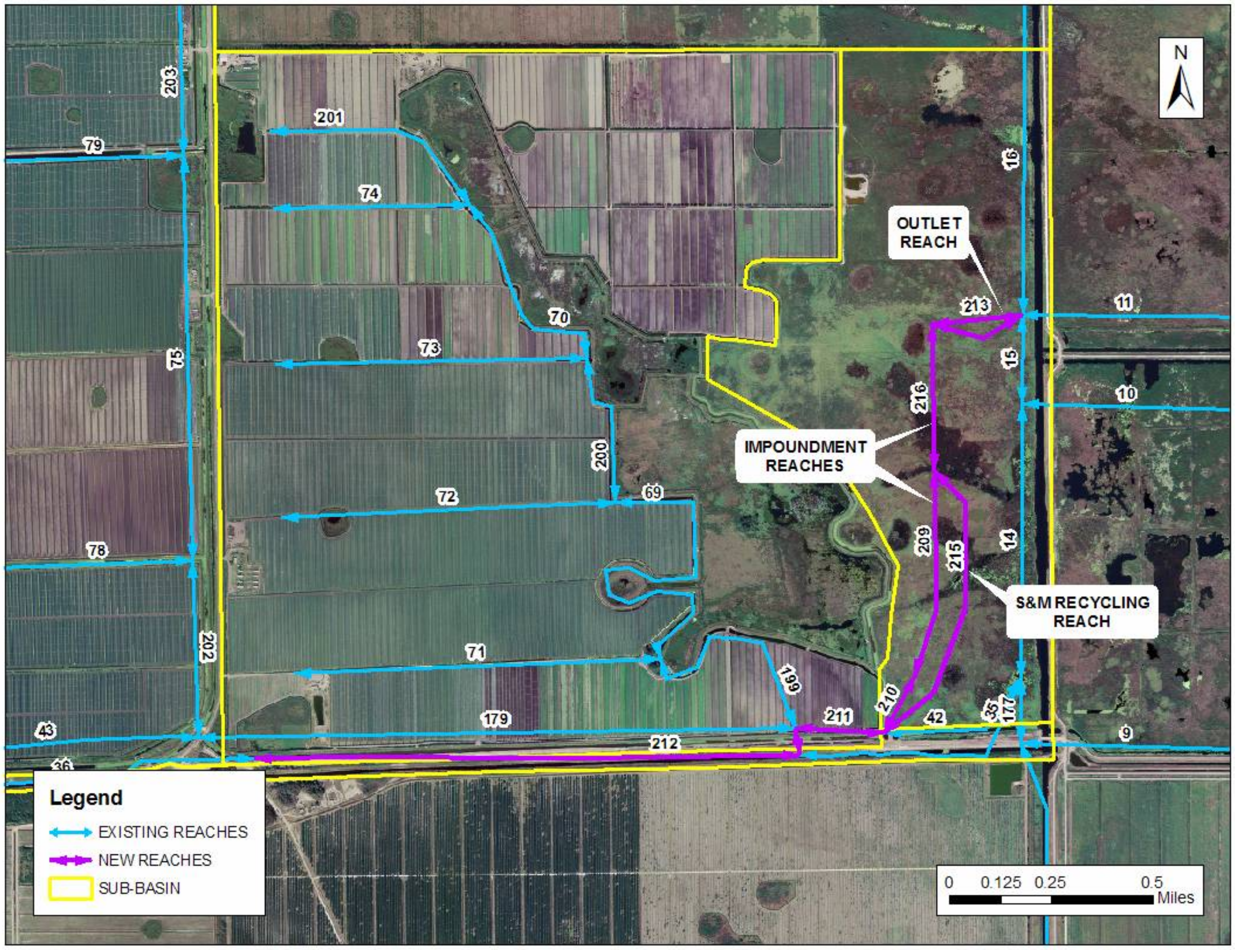

Figure 4.12: Proposed Eastern Interconnection WAM Reach Network

Figures 4.13, **4.14** and **4.15** illustrate that this change had an impact on the performance of the proposed impoundment and show the TP reduction effectiveness of the proposed project. Based on the assumptions described above, the simulated cummulative TP load reduction for 2002 is 11% and for 2003 is 14%. Although the contributing area is significantly increased, the simulated TP reduction increases only slightly. The proposed impoundment is not sufficiently large enough to contain peak runoff events from the S&M Canal contributing area. Therefore minimal additional treatment can be expected when the contributing area is increased by diverting Deer Fence Canal flows, while the impoundment area remains the same. Another consideration for the interconnection between the Deer Fence and S&M Canals would be to make the connection upstream of the gated culvert on the Deer Fence Canal. This location would not require the cost associated with the construction of an additional structure at the downstream end of the Deer Fence Canal. Preliminary investigation, not documented within this report, indicates that the impact on Project 1 TP removal efficiency is similar for either interconnection location.

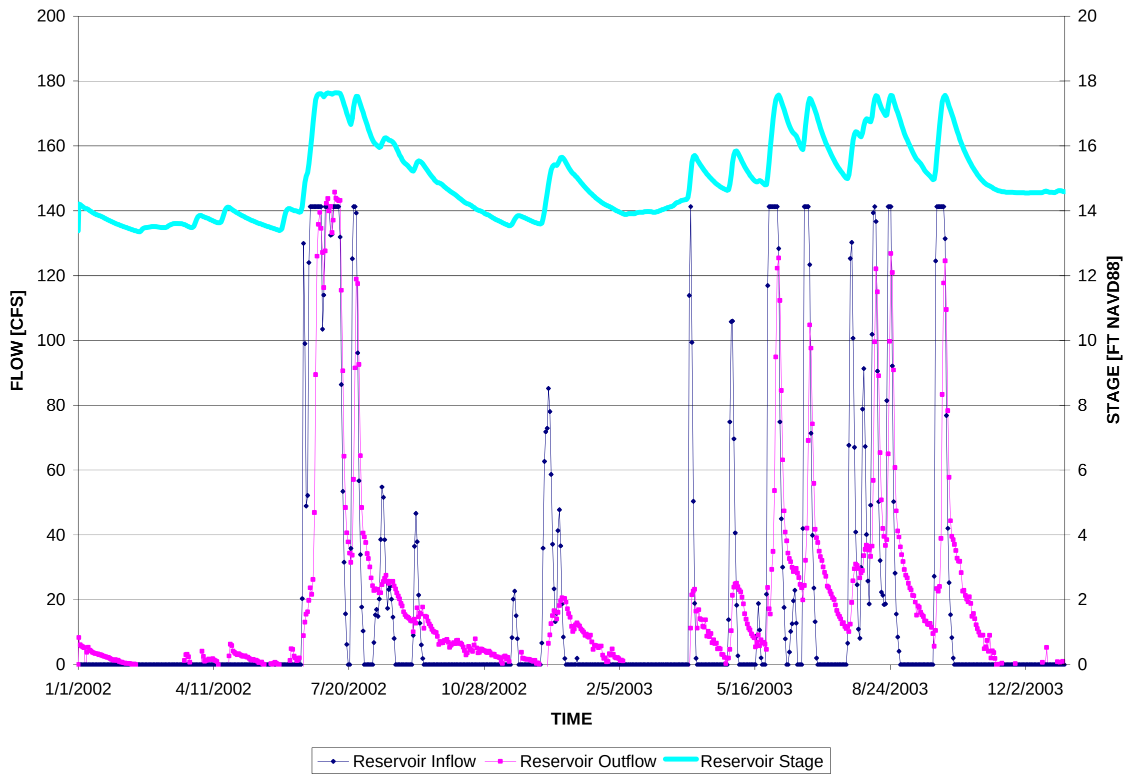

Figure 4.13: Stages and Flows for the Proposed Impoundment

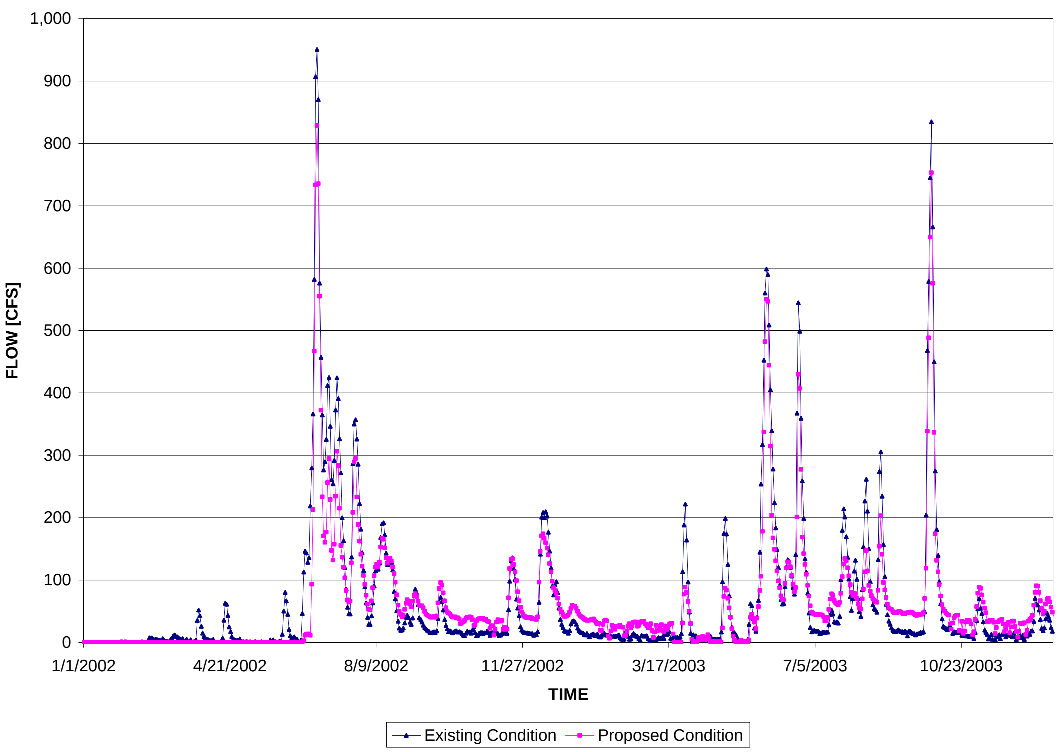

Figure 4.14: Existing and Proposed Flows for Deer Fence and S&M Canals

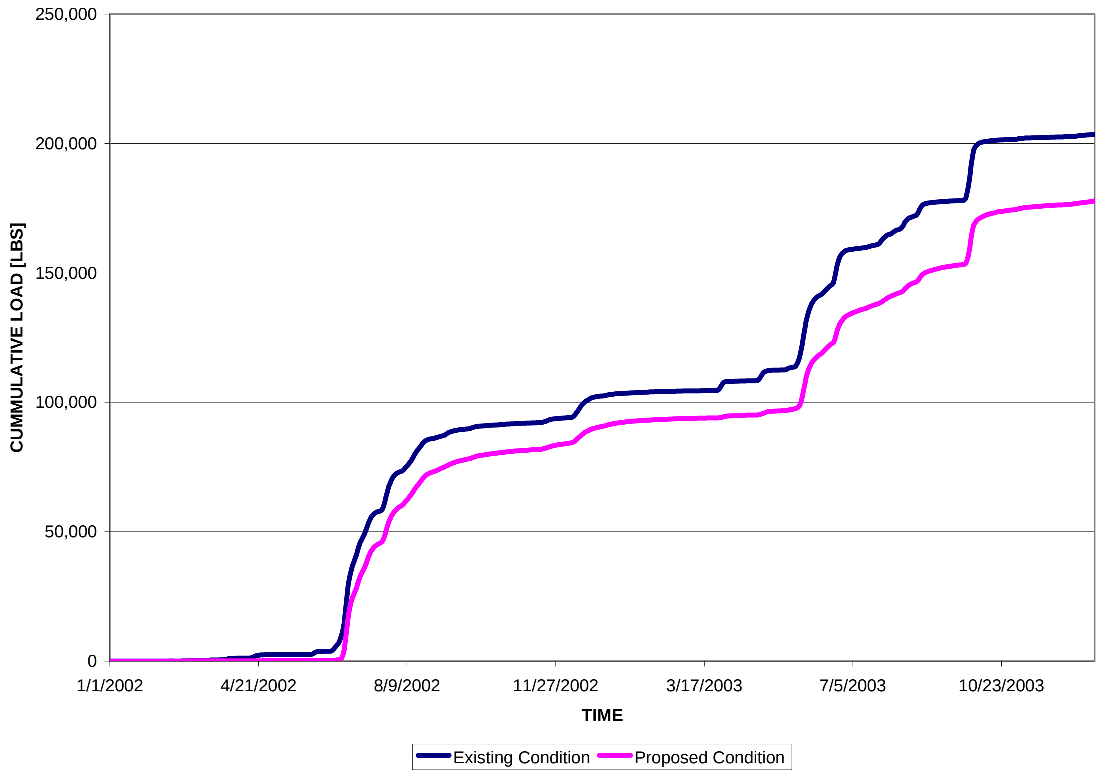

Figure 4.15: Cumulative TP Load for the Existing and Proposed Condition

4.2.4. Widened Canals

Based on the recommendations of District staff increasing the storage capacity of the Deer Fence and S&M Canals is a primary objective. One technique for providing additional in-canal storage is widening the canals. As described in Deliverable 6.4, WAM canal cross-sections are defined for each reach and are based on canal depth and width only. This methodology creates a simplified cross-section characterized as a stack of trapezoids symmetrical about the channel centerline. In the existing condition Baseline Simulation, the Deer Fence Canal and S&M Canal cross-sections for each reach are based on the closest available field survey data. For the canal widening iteration it was assumed that the width of each canal at each depth was doubled at every depth. The infrastructure for this iteration was assumed to be the same as the eastern connection iteration of Section 4.2.3 above. **Figures 4.17**, **4.18** and **4.19** illustrate the change this modification had on the performance of the reservoir and the TP reduction effectiveness on the proposed project. Based on the assumption that the Deer Fence and S&M Canals are widened as described above, the simulated cummulative TP load reduction for 2002 is 10% and for 2003 is 11%.

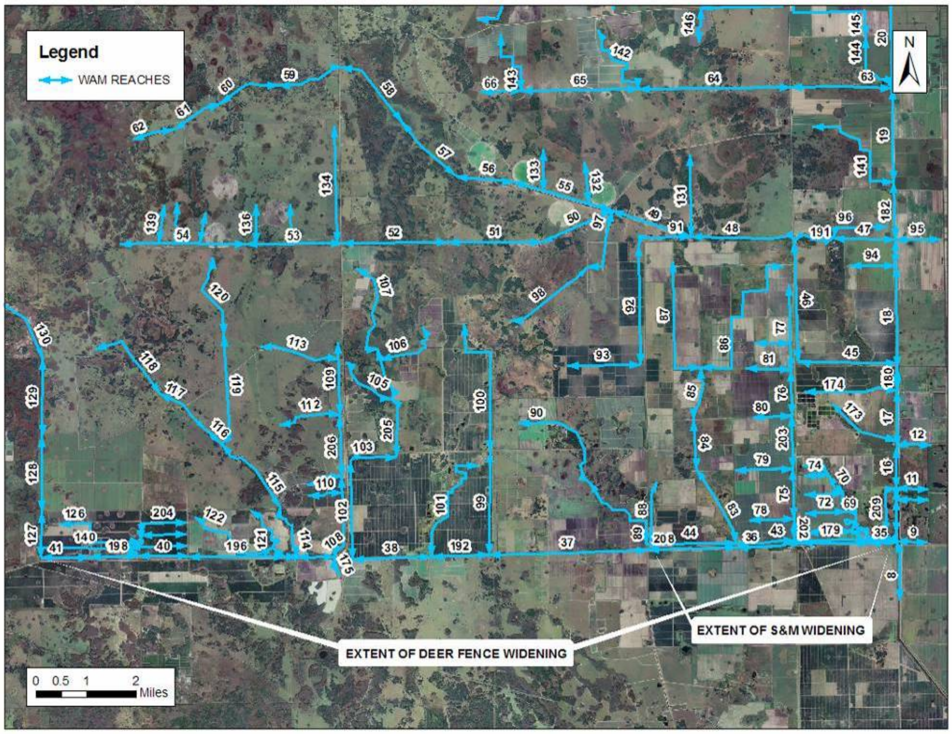

Figure 4.16: Extent of Canal Widening for the Proposed Condition

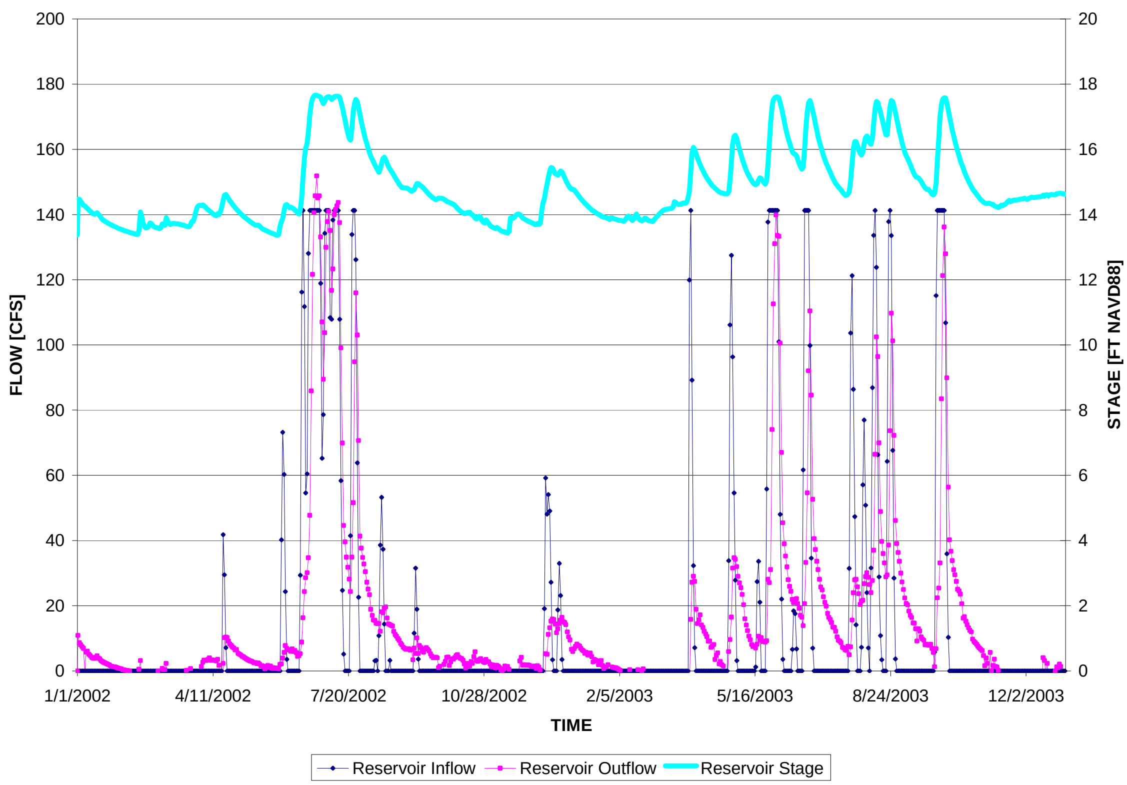

Figure 4.17: Stages and Flows for the Proposed Impoundment

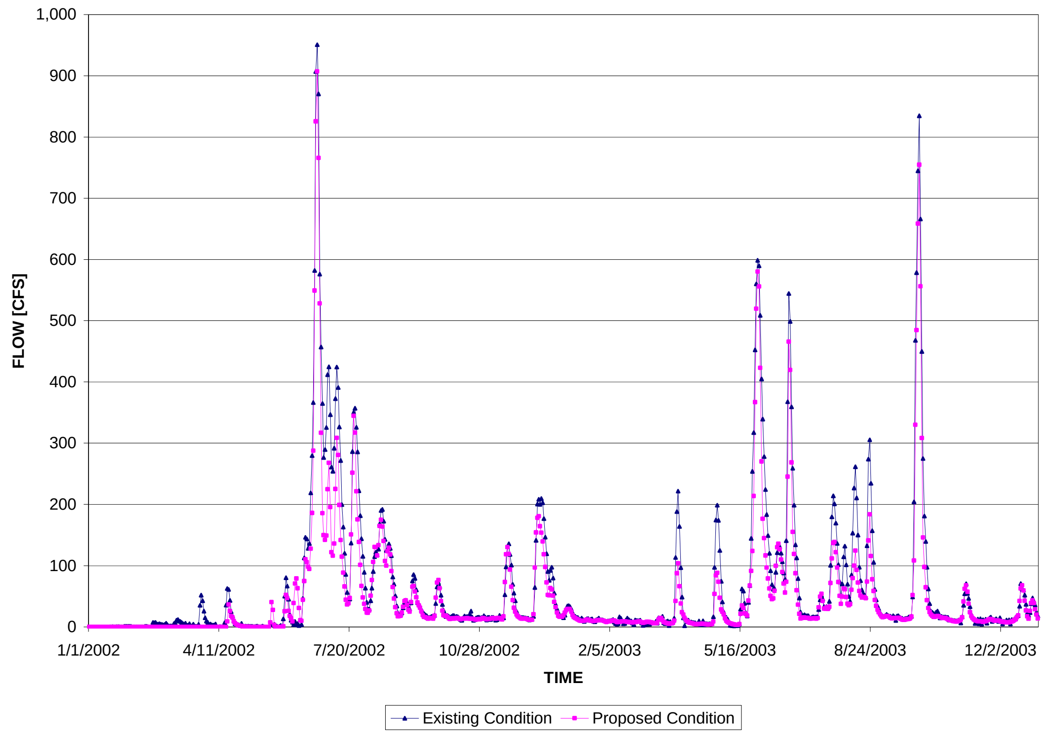

Figure 4.18: Existing and Proposed Flows for Deer Fence and S&M Canals

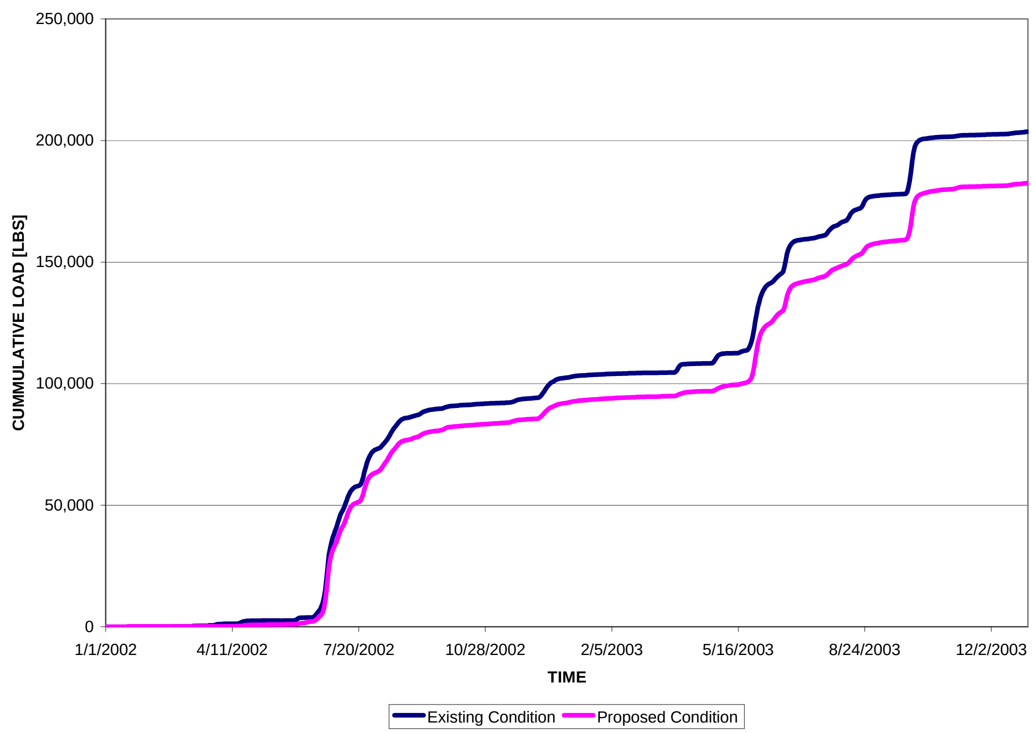

Figure 4.19: Cumulative TP Load for the Existing and Proposed Condition

4.2.5. Elevated Canal Control Elevations

As described above, increasing the storage capacity of the Deer Fence and S&M Canals is a primary objective of the District staff recommendation for Project 1. An alternative to widening the canals is to increase the canal control elevation. The gated culvert structure along the Deer Fence Canal has a gated culvert with a top of the gate spillcrest at elevation 22.7 ft relative to the North America Vertical Datum of 1988 (NAVD88). If the structure were modified such that the gates were still available to be open during high flow events, but would maintain a higher upstream stage when closed, the storage capacity in the canal could be increased. For this iteration the spillcrest elevation was raised one foot to 23.7 feet NAVD88. It should be noted that the use of the WAM model for the purposes of this evaluation is not meant to provide a description of any impacts on flood protection that this iteration may have. The infrastructure for this iteration was assumed to be the same as the eastern connection iteration of Section 4.2.3 above. **Figures 4.20**, **4.21** and **4.22** illustrate the change this modification had on the performance of the reservoir and the TP reduction effectiveness of the proposed project. Based on the assumption to increase the canal control elevation as described above, the simulated cummulative TP load reduction for 2002 is 12% and for 2003 is 14%.

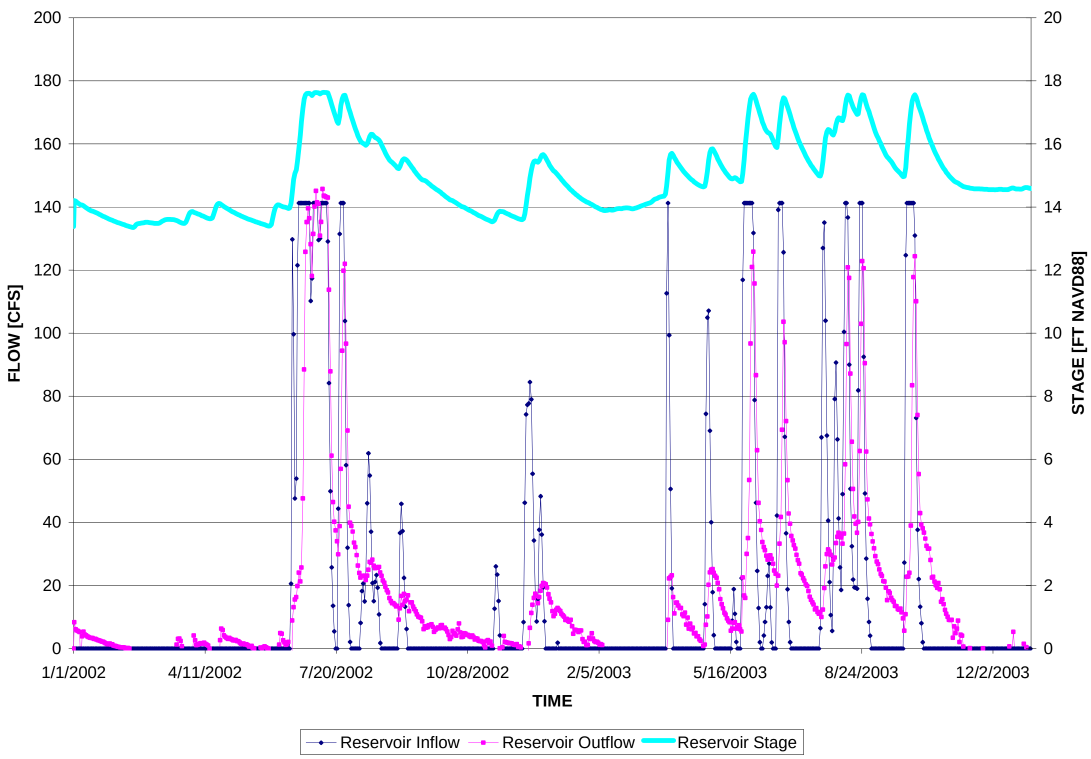

Figure 4.20: Stages and Flows for the Proposed Impoundment

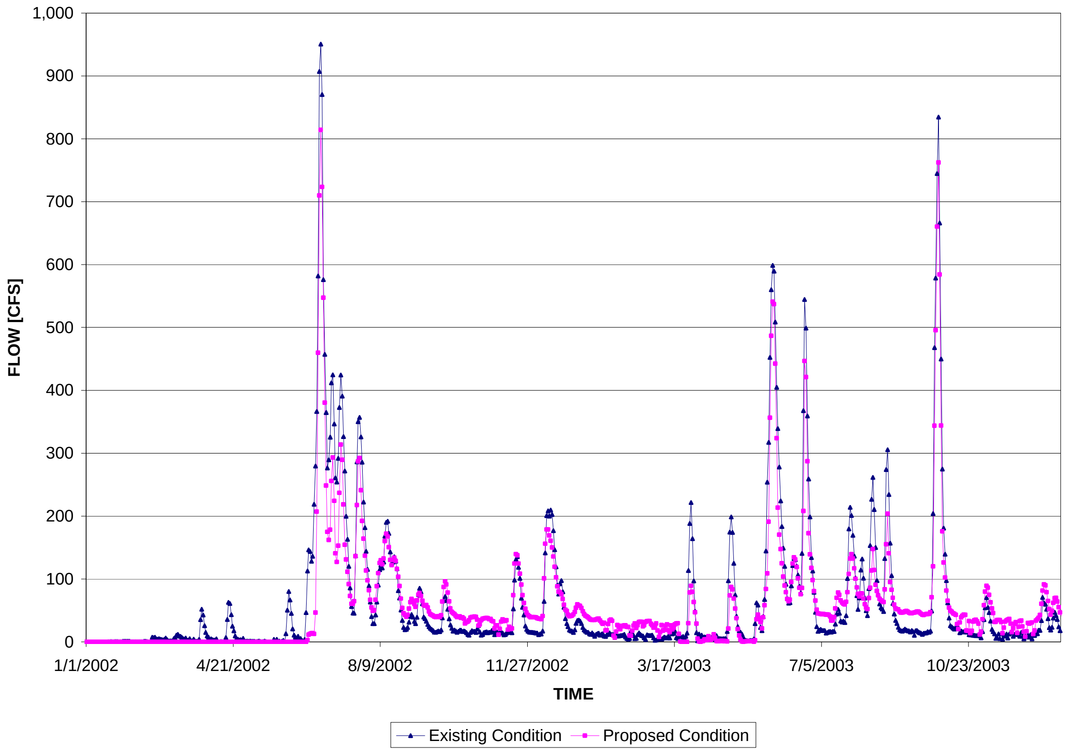

Figure 4.21: Existing and Proposed Flows for Deer Fence and S&M Canals

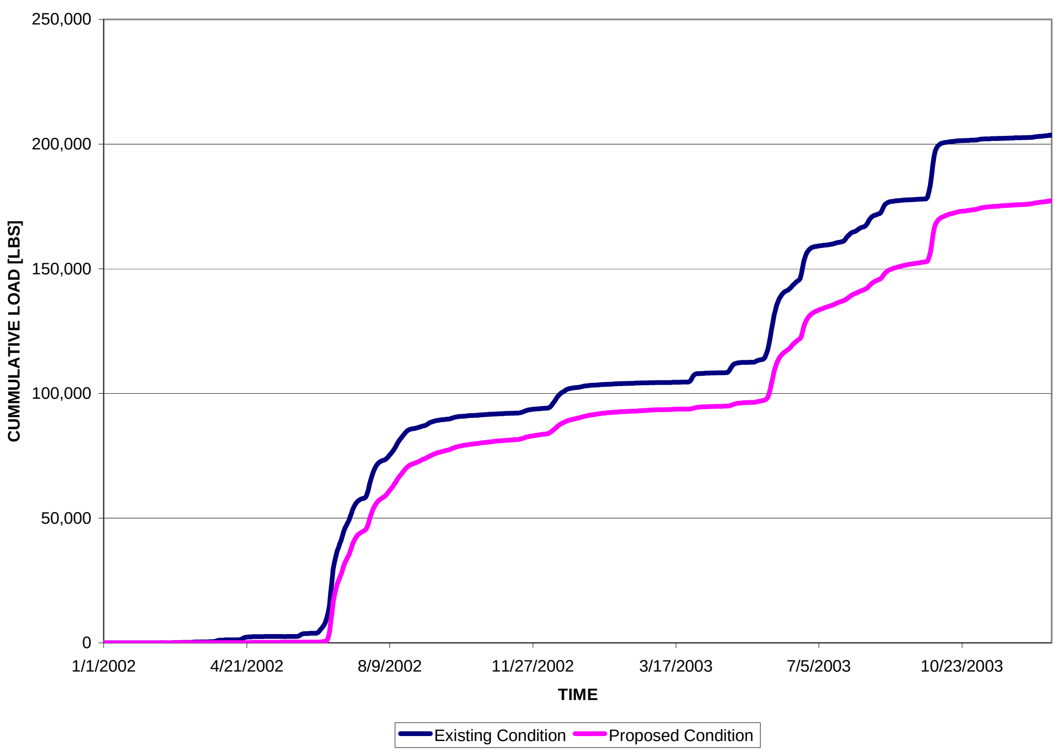

Figure 4.22: Cumulative TP Load for the Existing and Proposed Condition

4.3 Project 1 Short-Term Simulation Summary

Table 4.1 below summarizes the 2002 and 2003 reduction in TP load for the Deer Fence and S&M Canal discharges for each of the various iterations of proposed Project 1 as described by District staff. Based on these results, the optimized configuration of Project 1 is constructing the reservoir and including the eastern connection of the Deer Fence and S&M Canals. The only scenario that provides greater average annual TP load reduction is to increase the control elevation of the gated culvert structure on the Deer Fence Canal. This modification could potentially negatively impact flood protection for all upstream land-owners and does not provide a notable improvement in TP reduction.

Table 4.1: Summary of Short-Term Simulation TP Reduction for each Scenario

PROJECT 1 CONFIGURATION	ANNUAL TP REDUCTION [%]			AVERAGE REDUCTION [LB]
	2002	2003	AVERAGE	
Basin 26-319-01 Impoundment				
Without Seepage	7.0%	12.0%	9.5%	9,671
With Seepage	8.0%	13.0%	10.5%	10,689
Connection of Deer Fence and S&M Canals				
Eastern Connection	11.0%	14.0%	12.5%	12,725
Increasing the Capacity of the Deer Fence and S&M Canals				
Existing Canal Widths Doubled	10.0%	11.0%	10.5%	10,689
Existing Canal Control Elevations Raised 1 foot	12.0%	14.0%	13.0%	13,234

These reductions describe the impacts to the Deer Fence and S&M Canal discharges assuming that the proposed impoundment is empty at the beginning of the simulation and that the impoundment has attenuation effects consistent with the parameters used by WAM to define a slough. Since the reduction applies to only the discharges for a portion of the C-139 Basin, it is useful to compare the 12,725 pound average annual TP reduction for the selected scenario with the annual average TP load from the C-139 Basin described in **Table 2.1.** Extrapolating the comparison of the short-term reduction with the long-term average would show an 8% reduction in the simulated average annual TP load for the entire basin. The Baseline Period evaluation that follows illustrates the simulated long-term impacts of the proposed project.

4.4 Project 1 Baseline Period Evaluation

As described above, Project 1 consists of a 586 acre impoundment with a 5.1 mile perimeter levee constructed in Basin 26-319-01. The scenario that is selected as the optimized configuration includes the eastern connection of the Deer Fence and S&M Canals. A Baseline Period simulation was performed for this configuration to provide an appropriate assessment of the performance of Project 1 over a wide range of hydrologic conditions. **Figure 4.23** illustrates the effects of the optimized Project 1 scenario on basin-wide TP loads over the duration of the Baseline Period. For the short-term simulation, the average annual reduction is 12.5 percent of the Deer Fence and S&M Canal discharges which account for roughly 1/3 of the contributing area of the C-139 Basin. The simulated basin-wide average annual reduction in TP load for the proposed project is 5%. **Figure 4.24** illustrates the cumulative effect on TP loads over the Baseline Period. **Table 4.2** describes the performance of Project 1 with respect to the existing condition and the targeted TP load.

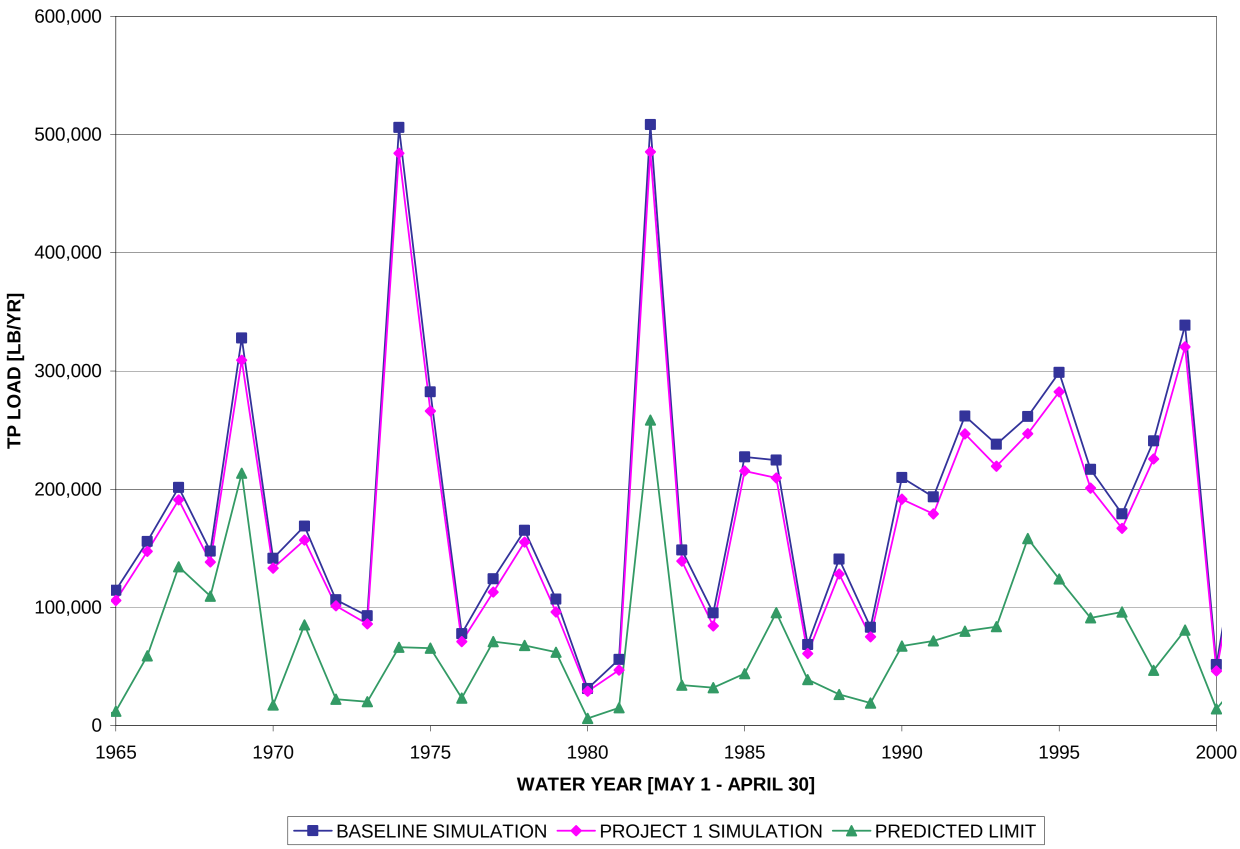

Figure 4.23: Annual TP Load for Project 1 and Existing Condition

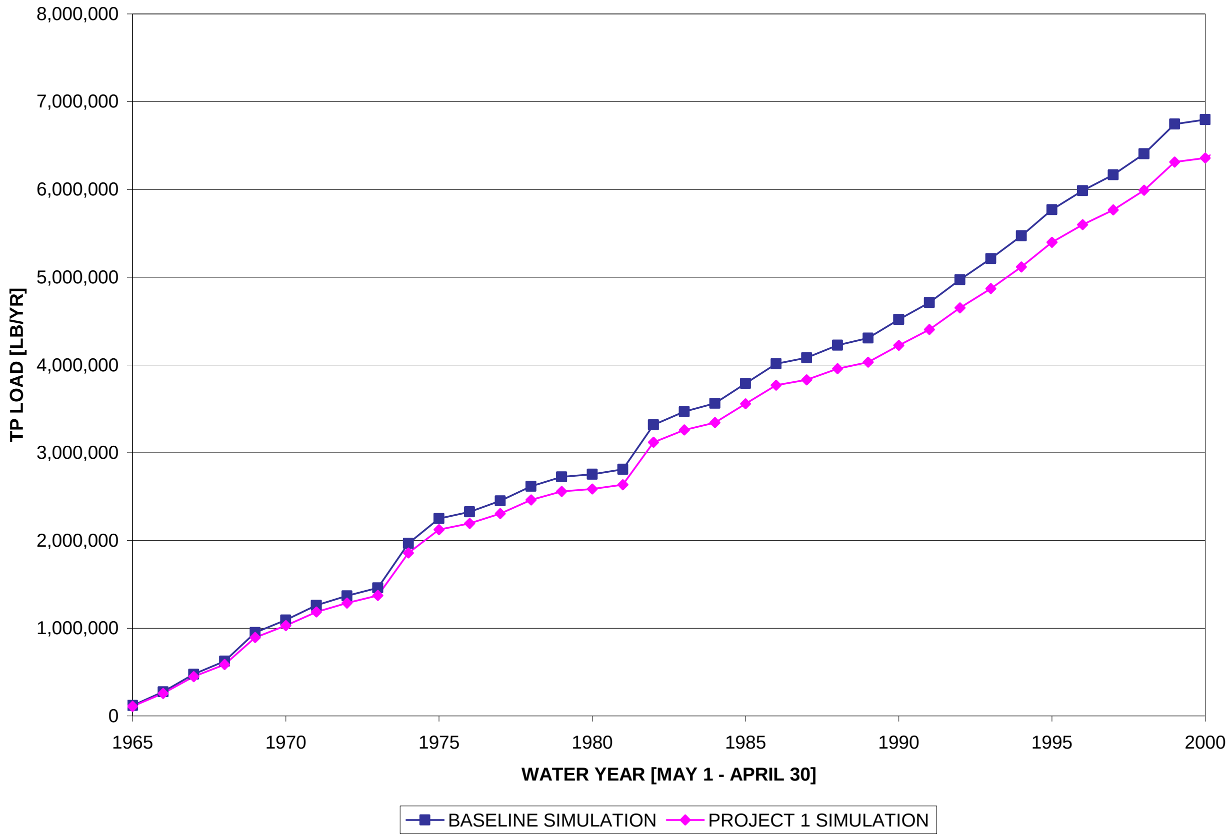

Figure 4.24: Cumulative Effects on TP Load for the Baseline Period

Table 4.2: Comparison of Project 1, Existing and Targeted Annual TP

CONDITION	MIN ANNUAL TP [LB]	MAX ANNUAL TP [LB]	AVG ANNUAL TP [LB]	TOTAL BASELINE PERIOD TP [LB]
PROJECT 1	29,109	485,340	176,518	6,354,652
EXISTING CONDITION	31,421	508,392	188,715	6,793,743
TARGETED LOAD	6,113	258,419	69,852	2,514,680

The improvement in water quality described for Project 1 implementation also assumes that TP attenuation would occur within the impoundment consistent with the type of attenuation seen within a slough. This assumption is based on the best professional judgment of the ADA Team. Additionally, this project assumes a seepage rate that varies solely on the depth of water in the impoundment. If seepage is enhanced by local or regional drawdown of the surficial aquifer due to water supply pumping, than the seepage assumptions could be inaccurate. The proposed improvement project provides only a 6% reduction in simulated annual average TP loads. This reduction is less than the reduction illustrated by the two-year simulation, however the Baseline Period includes significantly larger rainfall events that exceed the treatment capacity of the proposed improvement.

The impoundment is small in comparison with the proposed contributing area therefore large rainfall events produce peak runoff volumes that exceed the capacity of the impoundment. As illustrated in **Figure 4.23** above the project implementation would not provide the significant improvements required to reach the rainfall-adjusted target TP load as described within the SFER. In order to meet the target TP load requirements, Project 1 would need to be coupled with other local and regional improvement projects.

5.0 PROJECT 2

5.1 Project 2 Objectives and Components

As described in the April 20th letter, the second proposed project is using Basin 26-310-04 or the Dinner Island Ranch Wildlife Management Area (DIRWMA) to reduce TP loads from the C-139 Basin. The letter divides Project 2 into three separate projects for evaluation:

Project 2: Assuming agricultural operations are halted and surface water management features are reverted to their native state,

Project 2a: Impounding water from 26-310-04 and water pumped from adjacent farms to the east,

Project 2b: Impounding water from 26-310-04 and gravity-fed from the adjacent farms to the north.

For the purposes of this document each of the three above scenarios will be treated as separate projects with separate optimization schemes.

5.2 Project 2 Optimization

The agricultural operations represented in the Baseline Simulation within the DIRWMA include improved pasture, citrus sugarcane and vegetable. The DIRWMA is owned by the state and managed by the Florida Fish and Wildlife Commission (FFWCC). Since the development of the existing condition for the Baseline Simulation, the sugarcane field has been taken out of production and construction plans and specifications have been prepared to restore the field to natural conditions. An Individual Environmental Resources Permit (ERP) was submitted in May 2007 outlining the restoration of the sugarcane field and a portion of the existing pasture. In consideration of the restoration efforts of the FFWCC, the District recommendation for Project 2 includes a discussion of three primary objectives for evaluation:

- assuming all agricultural operations are halted,
- surface water management features are reverted to a native state and
- modeling the impounding capacity of 26-310-04 without external contributions.

Similarly to the optimization process of Project 1, the three primary objectives outlined for Project 2 are subject to short-term iterations prior to the final 36 year period of record evaluation.

5.2.1. Restoration of Landscape to Native Condition

The first iteration of Project 2 assumes that the DIRWMA is converted from the existing land-uses of pasture, sugarcane, vegetable and citrus to a native use, namely the “Scrub and Brushland” classification within WAM. As described in Deliverable 6.4, within WAM all of the farm-scale surface water management infrastructure is incorporated within the land-use parameterization scheme. Therefore converting the land-uses from agricultural to native within WAM does include aspects of the second objective outlined above as well. **Figure 5.1** below illustrates the WAM reach network and land use modification utilized for this iteration.

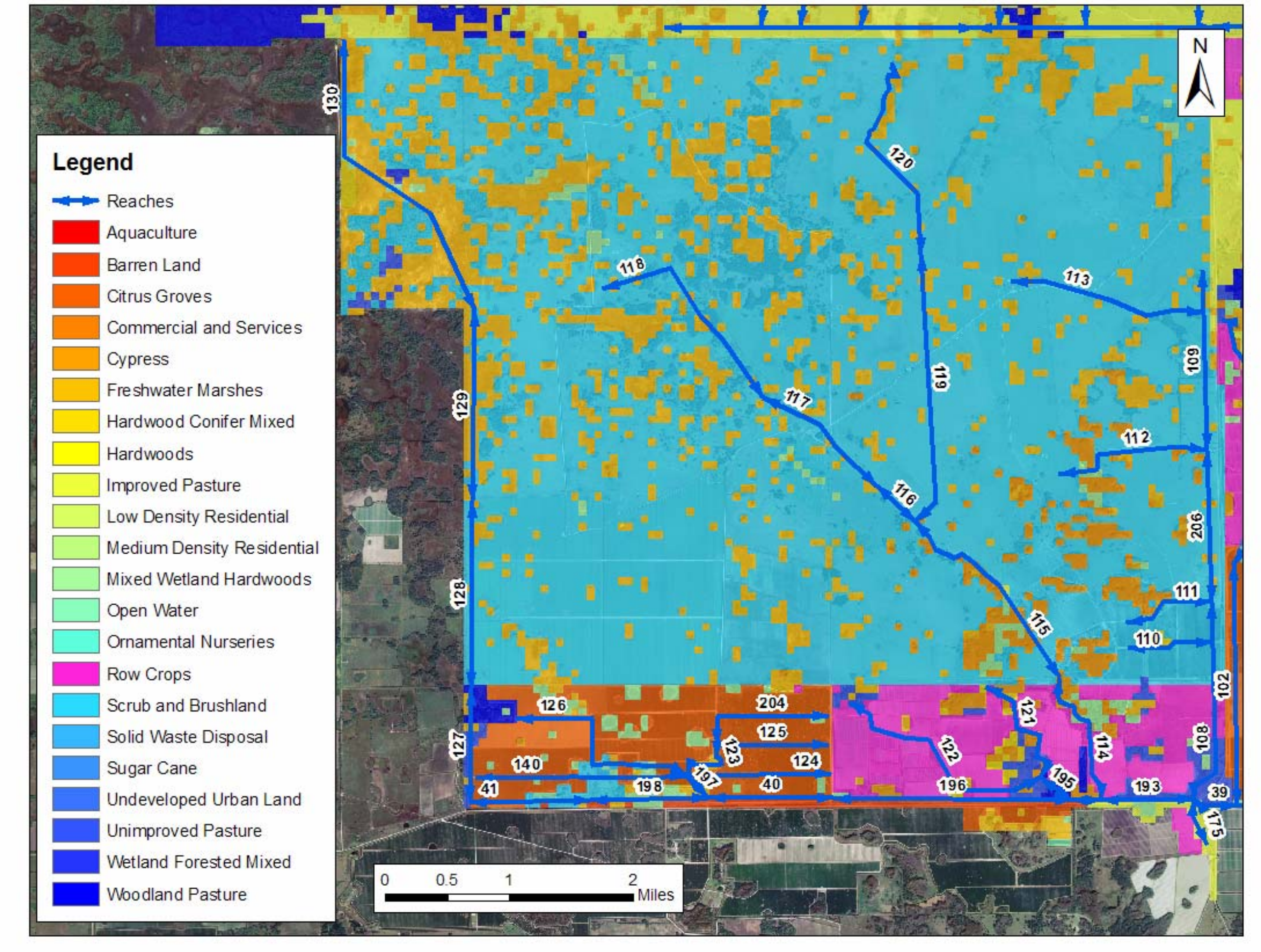

Figure 5.1: DIRWMA Agricultural Lands converted to Scrub and Brushland

The results of this land-use modification are significant with respect to runoff volumes and TP loads. **Figures 5.2** and **5.3** illustrate the runoff existing and proposed condition hydrograph and cummulative TP load simulated at the Duck Curve Bridge on the Deer Fence Canal. The average annual TP load reduction from the existing condition is a significant 43% due to not only reduced nutrient loading by the lack of agricultural use, but also to a reduction in runoff due to the removal of farm-level drainage networks.

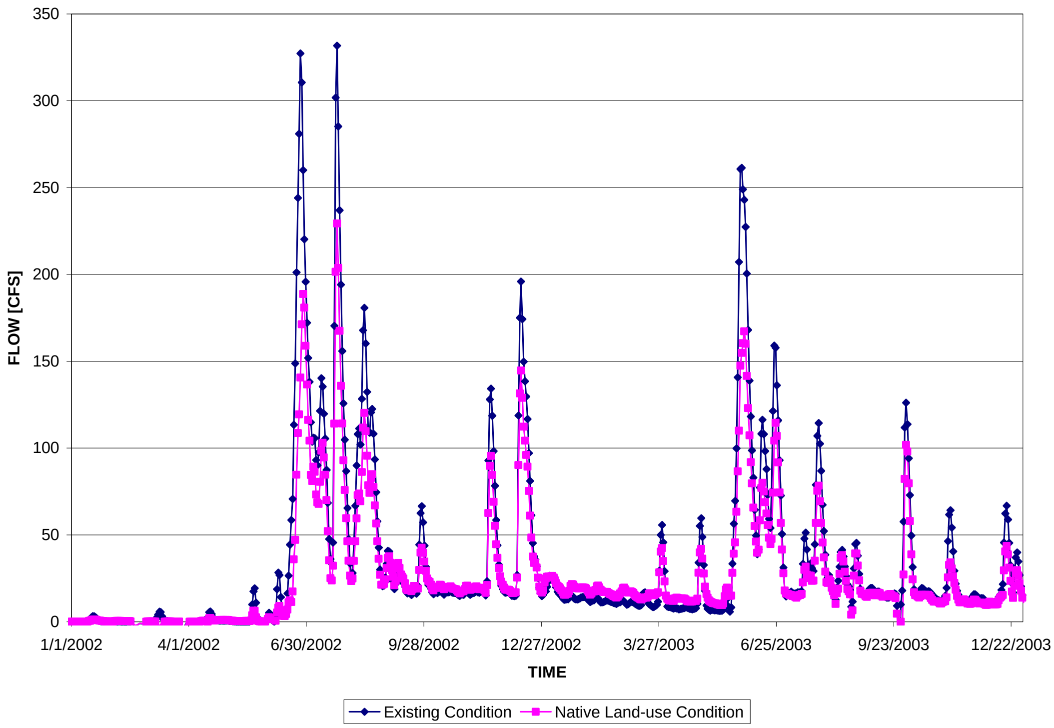

Figure 5.2: Reduction in Runoff at Duck Curve

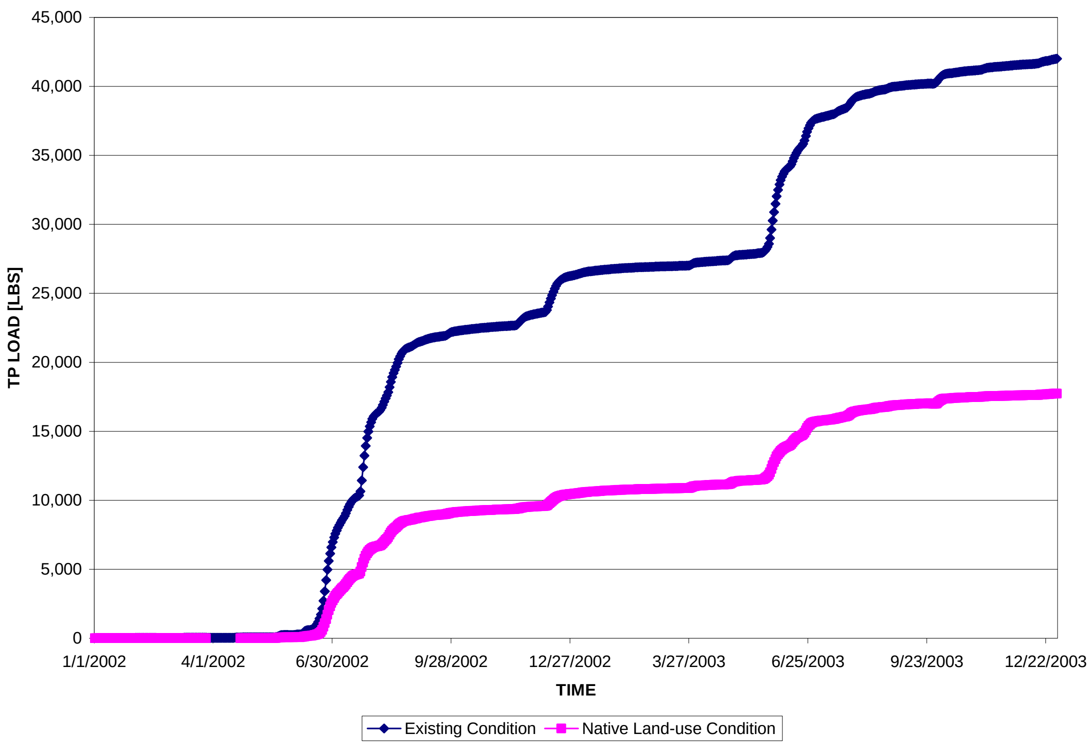

Figure 5.3: Reduction in TP Load at Duck Curve

5.2.2. Removal of Canal Network

The second iteration of Project 2 also assumes that all agricultural operations at DIRWMA are halted, but also assumes that the majority of the reach network has been removed. **Figure 5.4** illustrates the modified WAM schematic.

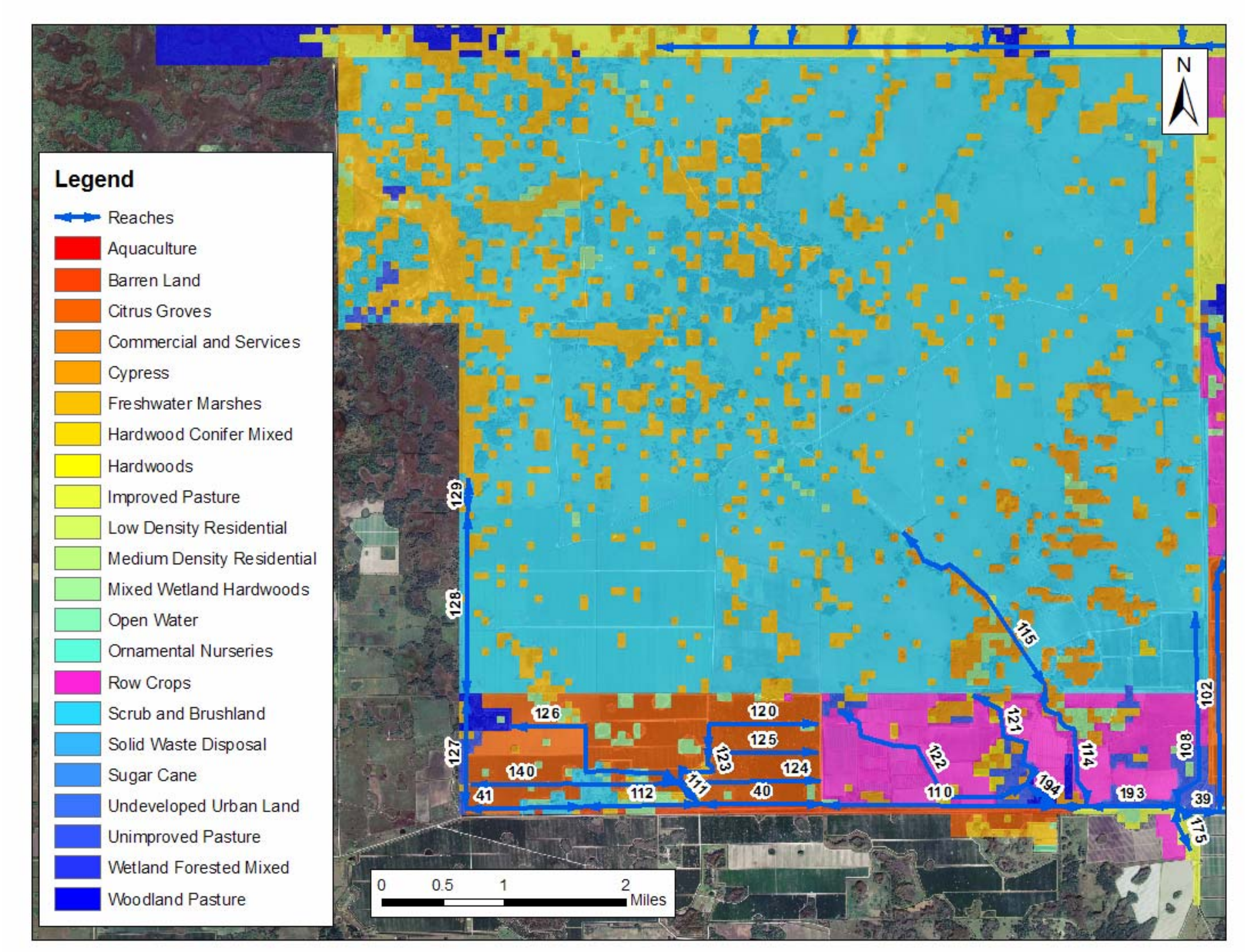

Figure 5.4: DIRWMA Agricultural Lands converted to Scrub and Brushland

The results of this hydrography modification is not nearly as significant with respect to reduction in runoff volumes and TP loads. When the WAM reaches are removed, runoff is forced travel to the outlet via overland flow instead of channel flow, thereby reducing the time of concentration for the catchment. However since the agricultural land-uses were changed to scrub and brushland, the volume of runoff generated is drastically reduced already. **Figures 5.5** and **5.6** illustrate the impact of this modification as simulated at the Duck Curve Bridge. The annual average TP load reduction compared with the existing condition for this iteration is 43%. The similarity with the previous scenario illustrates that the bulk of agricultural surface water management system within WAM is represented as part of the land-use parameterization. This is because agricultural land-uses are notable for significant ditching that would be difficult to represent in detail with the reach network.

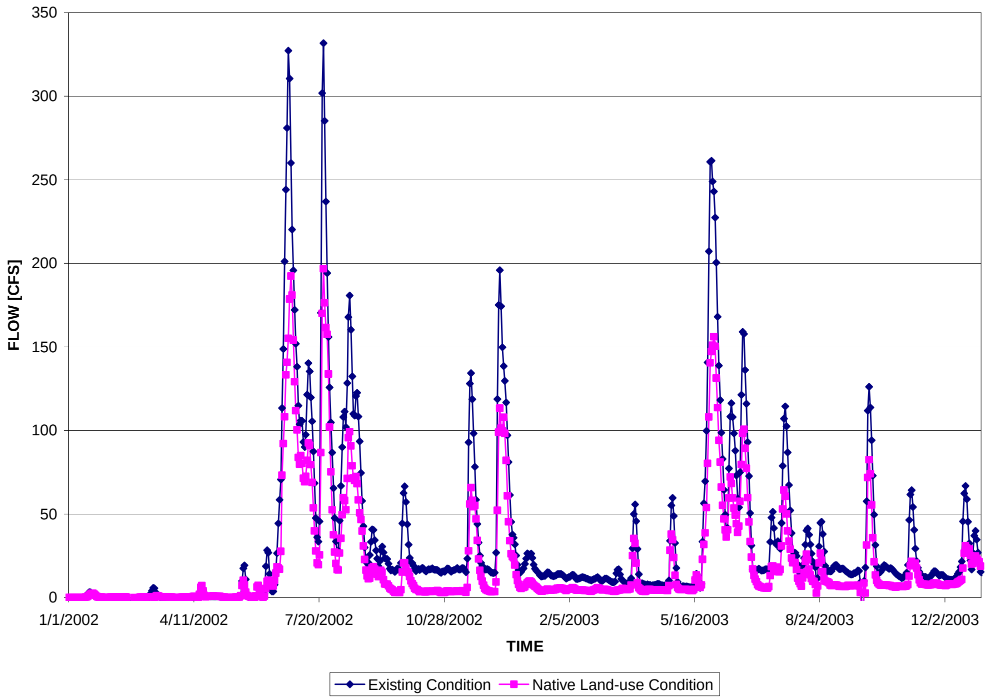

Figure 5.5: Reduction in Runoff at Duck Curve

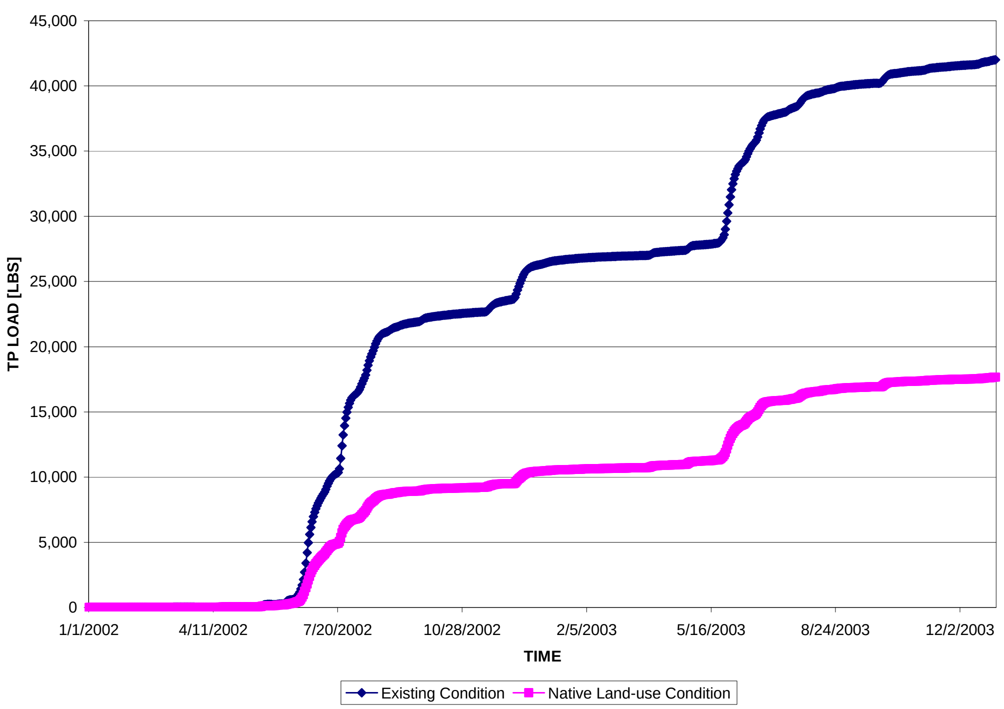

Figure 5.6: Reduction in TP Load at Duck Curve

5.2.3. Proposed Structural Improvements

The May 2007 ERP submitted by FFWCC for restoration of the sugarcane field and a portion of the improved pasture lands describe various changes to the existing surface water management system. Included in these changes are the revised operational criteria of the S-41 and S-40 structures. These structures are assumed to assist in restoring natural hydroperiods to impacted freshwater marshes. The S-41 structure is located at the southwestern corner of the DIRWMA boundary, while the S-40 structure is located on the White Farm Canal. The S-41 structure is assumed to open when the upstream stage exceeds 26.0 ft NAVD88 and close when the stage recedes below 24.0 feet NAVD. The S-40 structure is assumed to open when the stage in the canal exceeds 25.0 ft NAVD88 and close when the stage recedes below 23.0 feet NAVD.

For the purposes of this iteration, the S-40 and S-41 gates were simulated at the bottom of reaches 115 and 128. This is the physical location of the S-41 structure, but not of the S-40 structure. The location of the S-40 described in the DIRWMA ERP (26-00434-S), would be at the upstream end of reach 115, which would have no effect on the previous iteration which assumes that all reaches upstream of 115 have been plugged or filled as part of the restoration efforts. **Figure 5.7** illustrates the proposed WAM reach network for this iteration. **Figures 5.8** and **5.9** illustrate the impact of the proposed structures on the water quantity and quality. Similar to the two previous iterations, the average annual TP load reduction for this iteration at Duck Curve Bridge is 43%.

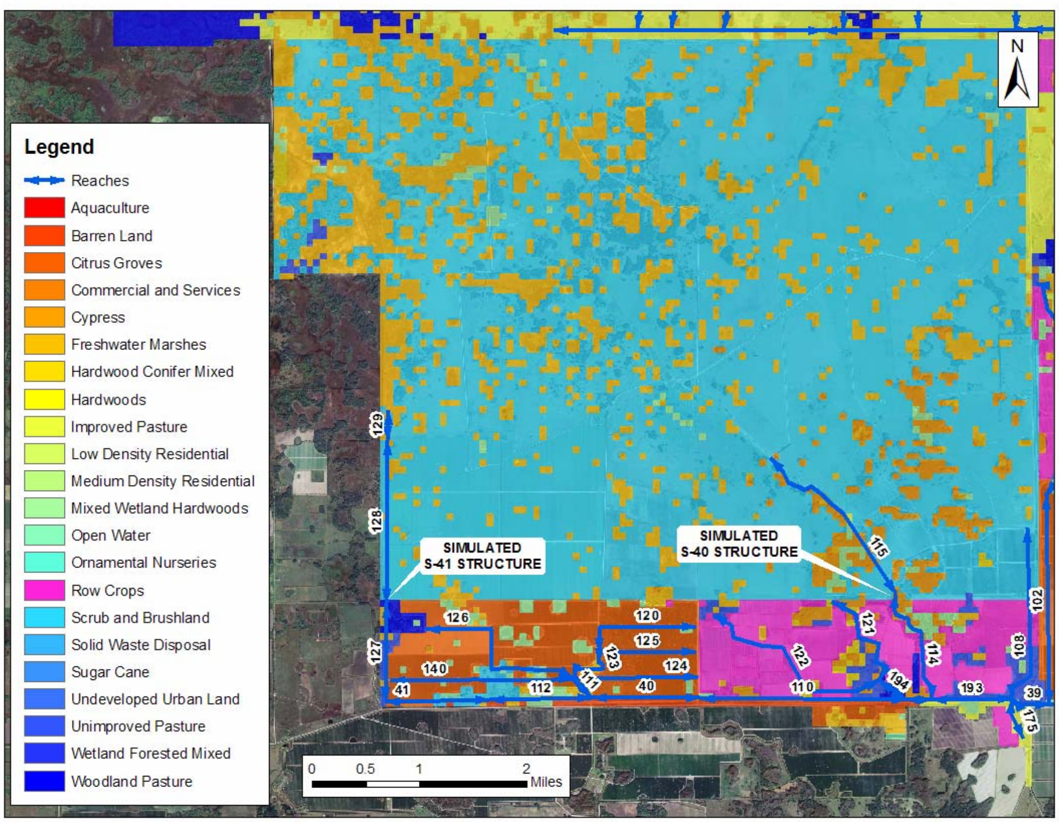

Figure 5.7: Proposed Structural Installations

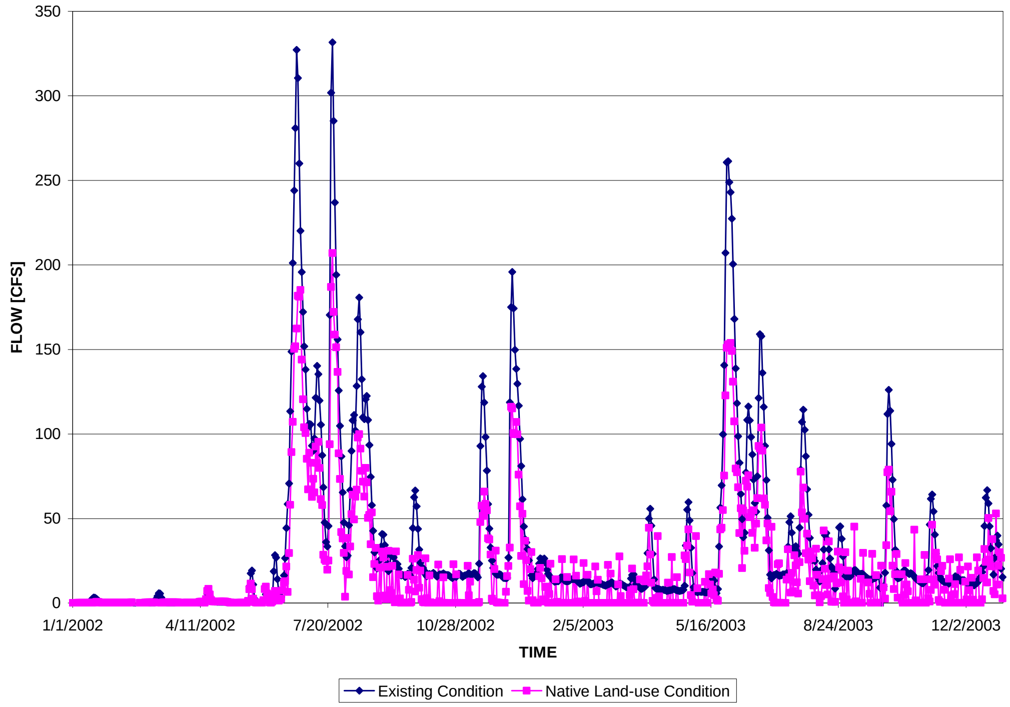

Figure 5.8: Reduction in Runoff at Duck Curve

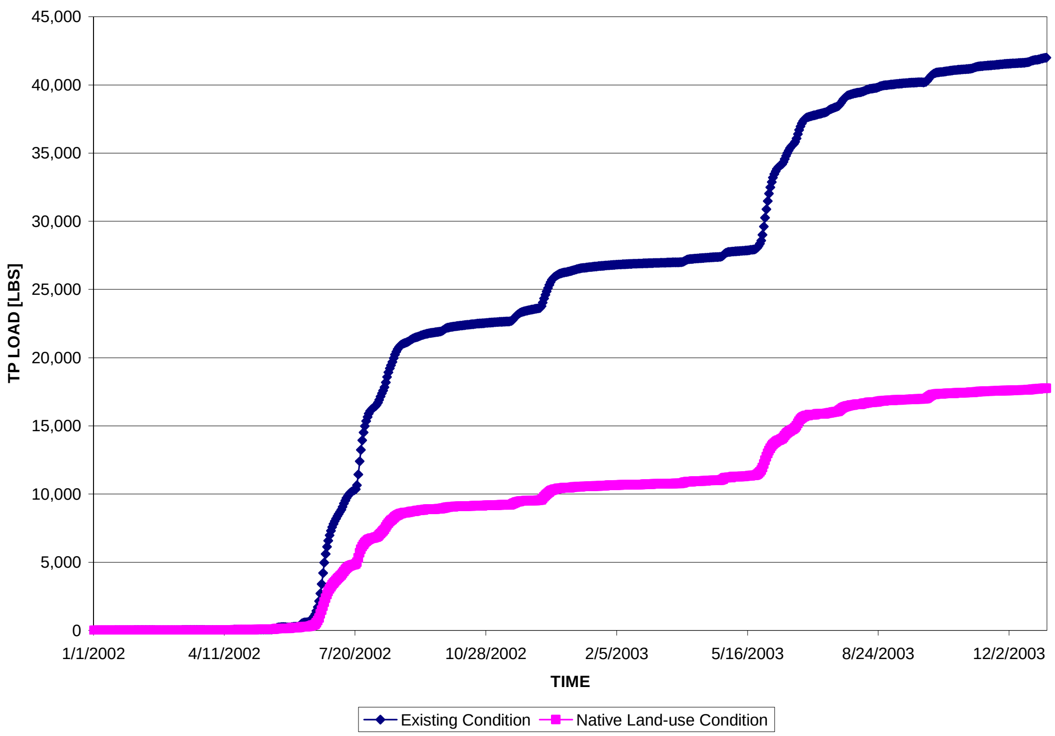

Figure 5.9: Reduction in TP Load at Duck Curve

5.2.4. Retaining All Basin 26-310-04 Runoff

One of the modifications recommended for evaluation within the April 20th letter is "retaining basin 26-310-04's runoff within its existing boundary". There are two structural methods for containing all of the runoff from the basin: constructing a large enough impoundment to hold all of the runoff and constructing a structural impediment at the Basin outlet. In discussions with District staff, constructing an impoundment within DIRWMA should be avoided if possible. An impediment to the basin outlet would potentially consist of a series of gates and levees.

As described within Deliverable 6.4, WAM utilizes separate sub-models to represent the hydrology and the hydraulics of the basin. Because of this, there is no connection that allows high stages within a stream reach to spill over the bank and route water back onto the grid-cells that define the basin hydrology. If a structure is installed that prevents flow from continuing to the next reach downstream, the stage in the impeded reach increases until the entire volume of runoff from upstream is contained within the reach network. Within WAM, the methodology commonly used in circumstances where stages in the reach overtop the banks is to include a wide over-bank area within the cross-section of the stream that is sufficiently large to hold the expected volume. If a series of structures was incorporated within the WAM reach network in the DIRWMA that retained all runoff, the upstream reaches would either stage up to unreasonable levels, or the cross-sections could be widened to handle bank overtopping. These wide reaches would effectively act as impoundments and would be simulated in the same fashion. Therefore, if the intent is to construct an impediment to any outflow from the DIRWMA without the construction of an impoundment, it could not be represented in WAM.

Based on the results of the calibrated baseline, a series of levees and structures that would retain the volume of runoff associated with peak canal stages of 28.9 ft NAVD88 would prevent runoff from leaving the DIRWMA during the 1965 to 2000 baseline period. Assuming all runoff was retained within Basin 26-310-04 the reduction in TP simulated at Duck Curve would be 74%, preventing 44,323 pounds of TP in 2002 and 24,558 pounds of TP in 2003 from discharging from DIRWMA.

5.3 Project 2 Short-Term Simulation Summary

Table 5.1 below describes the 2002 and 2003 reduction in TP load simulated at Duck Curve for the various scenarios of Project 2. The primary reduction of simulated TP load is due to the restoration of the landscape of Basin 26-310-04 from pasture, sugarcane, vegetable and citrus to natural scrub and brushland. The remaining simulated scenario modifications did not yield significant changes during the short-term evaluation period. The most significant reduction in TP load could be achieved if a surface water management system could be constructed that retained all runoff from the DIRWMA. However, it is assumed that this type of surface water management system could not be constructed without regional above ground impoundments, which are not the preference of the District within DIRWMA at this time.

Table 5.1: Summary of Short-Term Simulation TP Reduction for each Scenario

PROJECT 2 CONFIGURATION	ANNUAL TP REDUCTION [%]			AVERAGE REDUCTION [LB]
	2002	2003	AVERAGE	
Native Configuration	40%	46%	43%	19,872
Removal of Canal Network	40%	46%	43%	19,872
Proposed Structural Improvements	40%	47%	44%	20,335
Retaining All Basin 26-310-04 Runoff	76%	71%	73%	34,440

A comparison of the 19,872 pound average annual TP reduction for the selected scenario with the annual average TP load from the C-139 Basin described in **Table 2.1** illustrates that the removal of that quantity would account for a 12% reduction in the simulated average annual TP load for the entire basin. These reductions assume that the restored landscape has similar drainage and water quality parameters consistent with the parameters used by WAM to define a scrub and brushland. This does not take into account the heightened quantities of TP within the soil column expected in abandoned agricultural land. There has been some modeling done in the past for the Lake Okeechobee watershed with the PHANTM II model to determine the impacts of conversion from pasture land to native habitat. Results of that modeling and the expert opinion of Dr. Del Botcher are that the TP levels in the runoff may take between 15 and 30 years to return to near-native concentrations after the pasture land is converted.

5.4 Project 2 Baseline Period Evaluation

As described above Project 2 consists primarily of restoring the landscape within Basin 26-310-04 to native conditions. The various scenarios evaluated above include the additional features of restoration: filling existing surface water management features and installing water control structures. A Baseline Period simulation was performed for this final configuration that includes filled canals and structural improvements to provide an appropriate assessment of the performance of Project 2 over a wide range of hydrologic conditions. **Figure 5.10** illustrates the effects of the optimized Project 2 scenario on basin-wide TP loads over the duration of the baseline period. **Figure 5.11** illustrates the cumulative effect on TP loads over the baseline period.

As is illustrated in **Figure 5.10**, the Project 2 condition reduces annual TP loads for every year. Reductions are more significant in years with large annual loads, this pattern is likely due to the reduction of runoff generated by the native land-uses which is more pronounced in wetter years. **Table 5.2** describes the performance of Project 2 with respect to the existing condition and the targeted TP load.

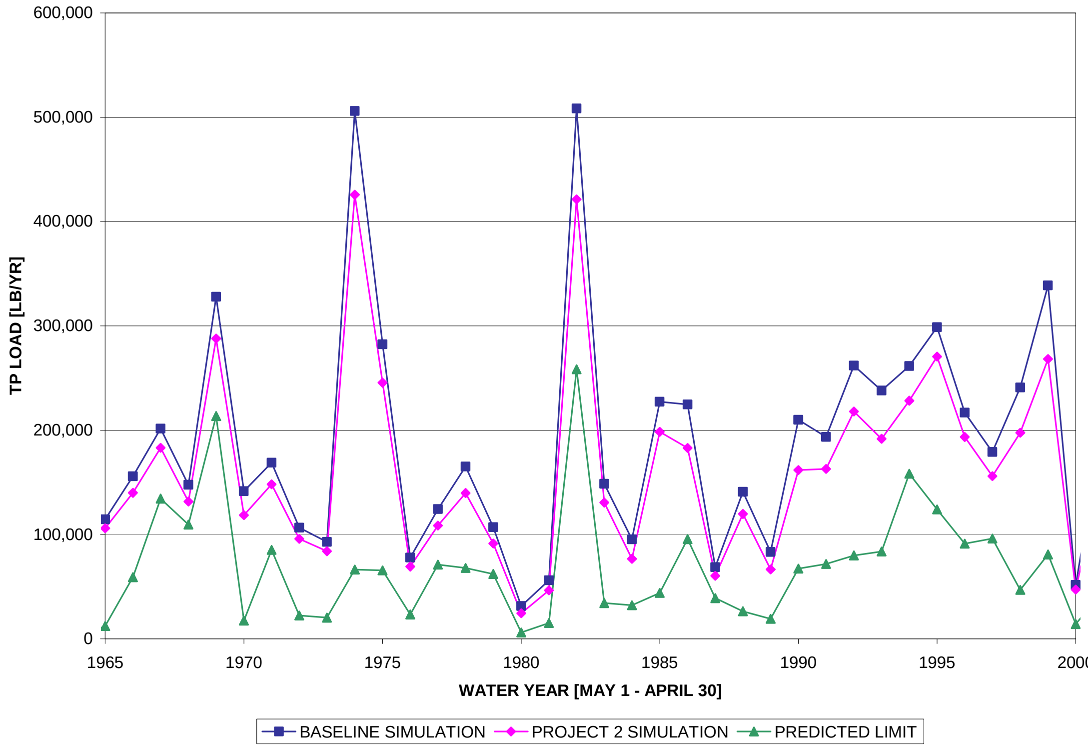

Figure 5.10: Annual TP Load for Project 2 and Existing Condition

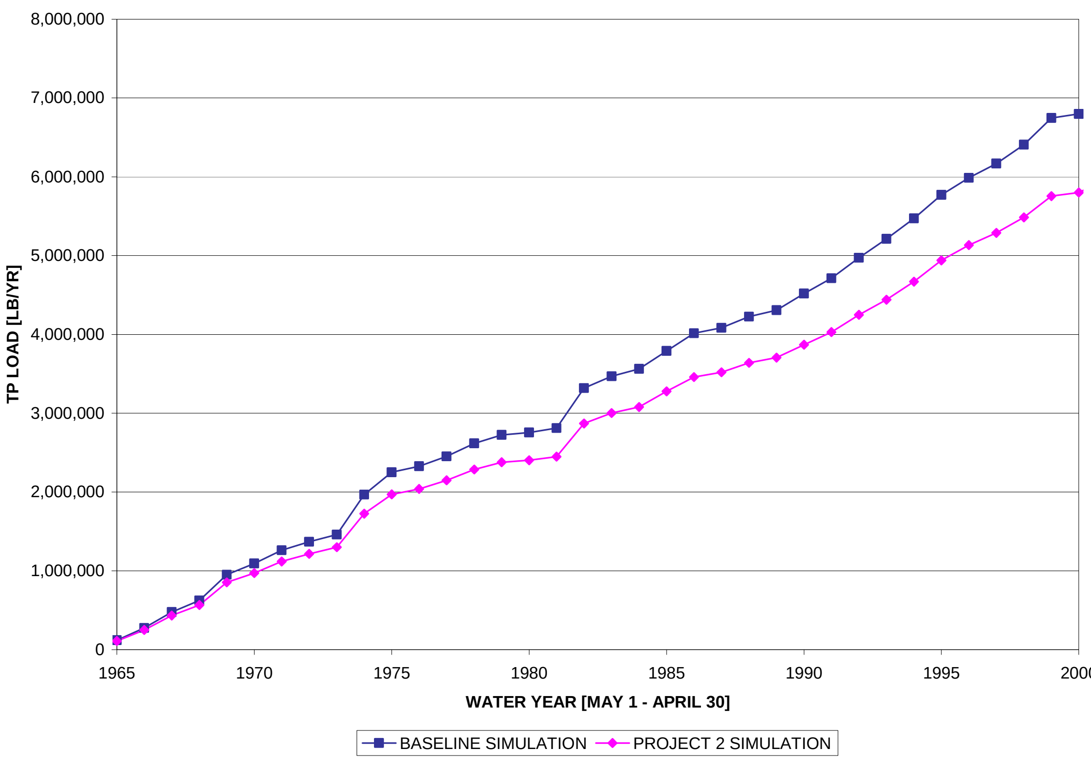

Figure 5.11: Cumulative Effects on TP Load for the Baseline Period

Table 5.2: Comparison of Project 2, Existing and Targeted Annual TP

CONDITION	MIN ANNUAL TP [LB]	MAX ANNUAL TP [LB]	AVG ANNUAL TP [LB]	TOTAL BASELINE PERIOD TP [LB]
PROJECT 2	24,397	425,618	161,020	5,796,721
EXISTING CONDITION	31,421	508,392	188,715	6,793,743
TARGETED LOAD	6,113	258,419	69,852	2,514,680

Although the proposed improvement project does provide moderate reductions with respect to TP loads, the project implementation would not provide the significant improvements required to reach the rainfall-adjusted target TP load as described within the SFER. In order to meet the target TP load requirements, Project 2 would need to be coupled with other local and regional improvement projects.

6.0 PROJECT 2A

6.1 Project 2a Objectives and Components

As described in the April 20th letter, the second proposed project is divided into three separate projects for evaluation. The second scenario is described within this report as Project 2a, and involves impounding water from 26-310-04 and water pumped from adjacent farms to the east. This diversion would not be performed throughout the season, only during peak events. The objective of redirecting these flows is to provide a longer flow-path for agricultural runoff, thereby providing greater treatment prior to discharge from the C-139 Basin. The extent of the adjacent farms to the east from which runoff is to be diverted is illustrated in **Figure 6.1**.

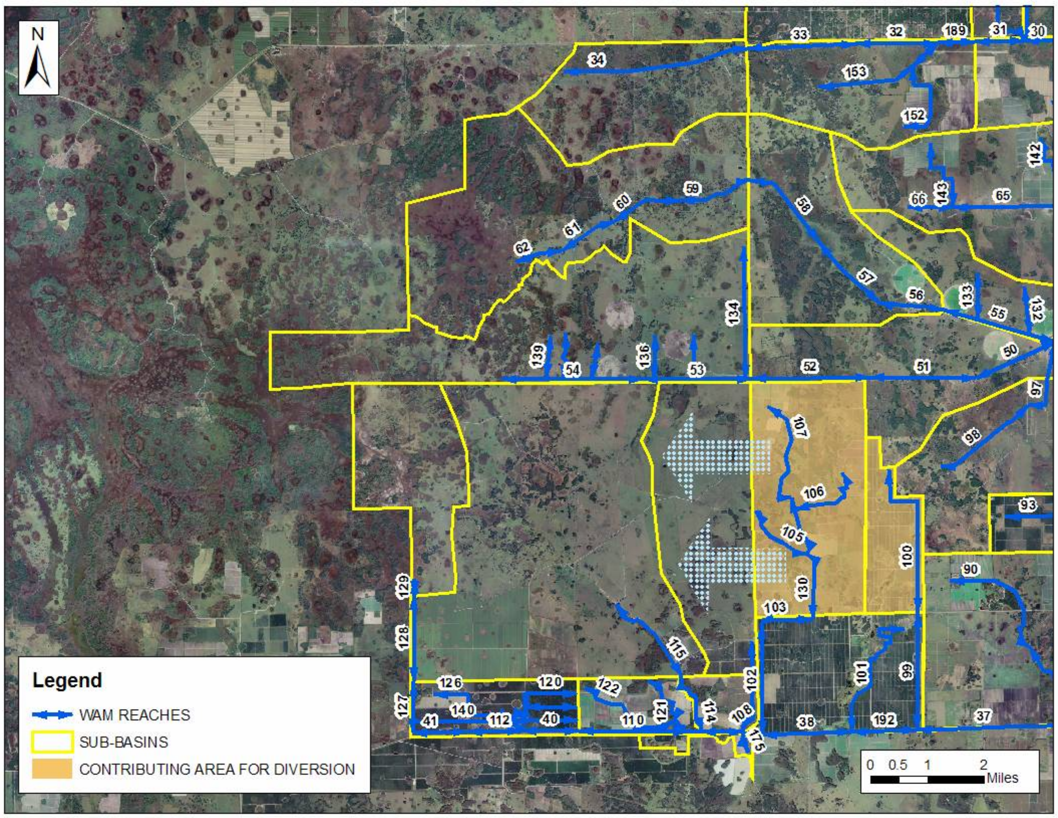

Figure 6.1: Contributing area to be diverted during peak events

6.2 Project 2a Optimization

In order to implement the scenario described above, a series of canals, gates and a pump station will be required. This is because the natural topography of the region does not allow for flow to be diverted west and north via gravity. **Figure 6.2** illustrates the proposed modification.

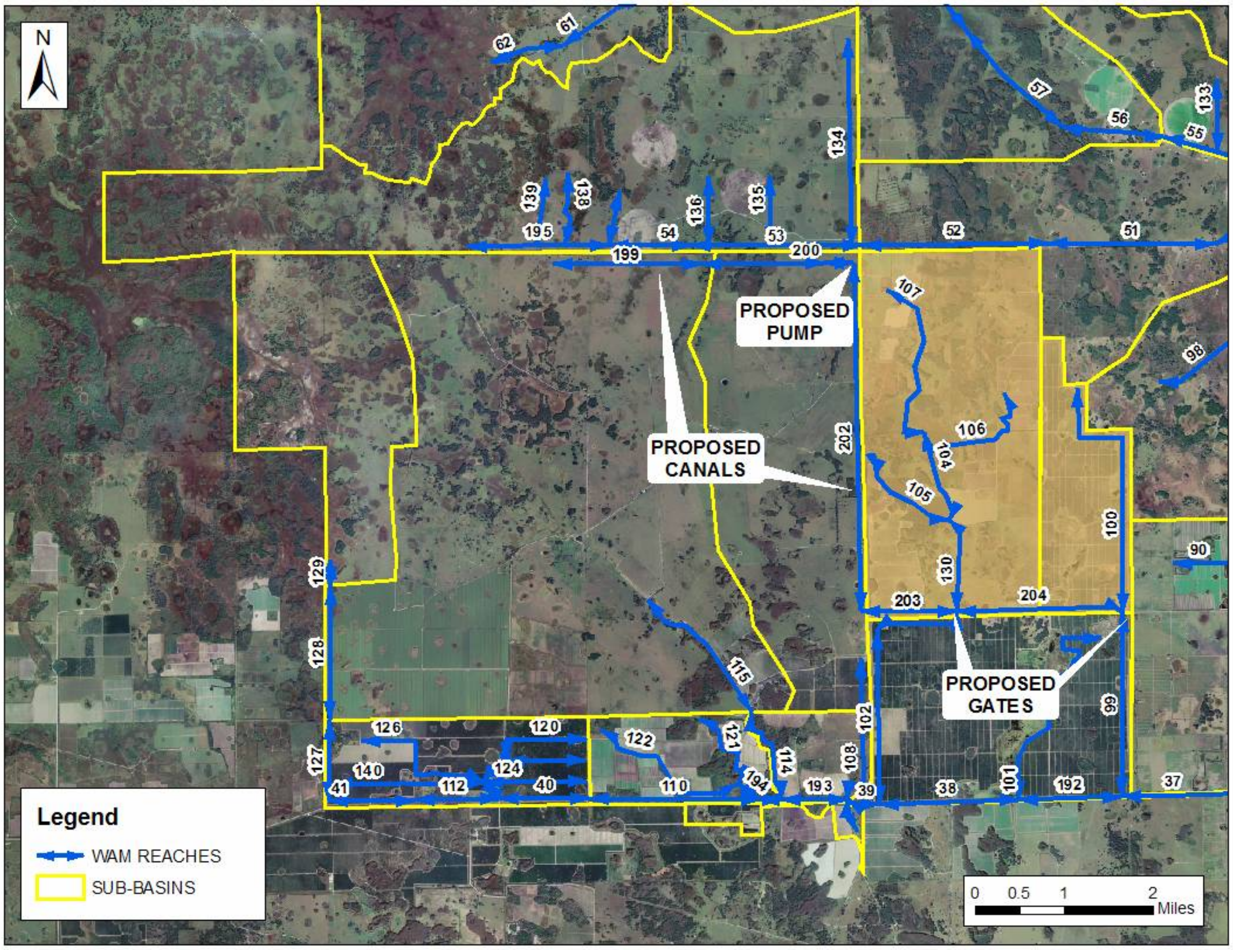

Figure 6.2: Project 2a Proposed Modification

Based on the District's recommendations, as described in Section 1.4 above, the preferred utilization of the DIRWMA is for slough-like overland sheet flow treatment of additional runoff not using any above ground impoundments. Reaches 204, 203 and 202 are diversion reaches with assumed cross-sectional areas equal to the existing connecting canal with Deer Fence, reach 102. The proposed gates divert existing flows from reach 100 to 99 and reach 130 to 102 to reaches 204 and 203 when a major runoff event occurs. For the purposes of the 2002-2003 two year iteration, it is assumed that when the stages in the upper reaches of the diverted contributing area (reach 107) exceed 21.5 feet NAVD, then the gates are closed and the pump is turned on, diverting runoff to the west and through the DIRWMA. Within the model the proposed pump has a capacity of 45 cfs based on a statistical review of simulated flows from the diverted contributing area. A flow-duration-curve analysis of the existing condition Baseline Period simulated flows for the contributing area exceeded 45 cfs only 5% of the time. The east west canal at the northern limit of the DIRWMA represented by reaches 199, 200 and 201 is intended to act as a distribution point allowing pumped runoff from the farms to the east to sheetflow through the restored DIRWMA prior to entering the Deer Fence Canal.

Although hydraulic infrastructure constructed as shown above could provide the effect outlined in District recommendations, using a canal as a headwater for overland flow is a mechanism that can not be represented within WAM. As described in Deliverable 6.4, WAM divides every watershed representation into two main components hydrology and hydraulics. Overland sheet flow is represented as part of the hydrology when runoff from individual grid cells are routed to the nearest hydraulic reach. Once runoff is routed to the reach, WAM represents further transport processes using the hydraulics submodel. If runoff is pumped into a reach with no outlet within WAM, any future runoff will stay within the reach and the stages will rise to compensate for the increase in volume. Regardless of the magnitude of the stage increase, runoff cannot leave the reach and be routed to any other reaches via overland flow. This is a limitation that exists not only in WAM but in most available watershed modeling packages.

In order to represent the effects of the proposal to allow overland flow from the distribution canal down through the DIRWMA, reaches were created that have high inverts, wide, shallow cross-sections and slough-like water quality attenuation parameters. The final WAM configuration of this project is illustrated in **Figure 6.3** below.

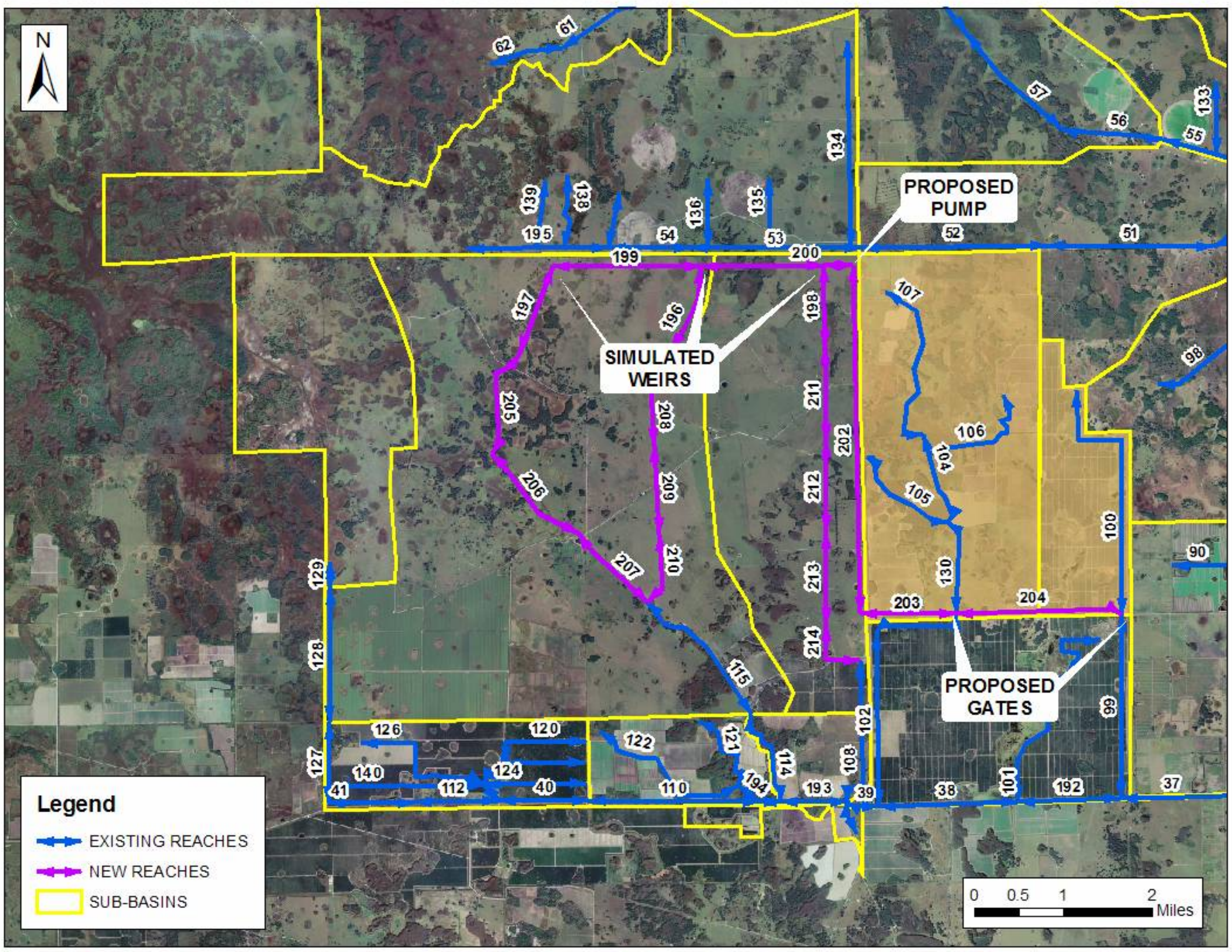

Figure 6.3: Project 2a WAM Representation

The discharge from the proposed pump is stored in reaches 199, 200 and 201 which have high inverts of 23 NAVD88 and act as a distribution point for the intersecting reaches of 196, 197 and 198. Reaches 196, 197 and 198 represent overland flow pathways for the pumped discharges to use to filter south through the DIRWMA. There is a weir at the top of each reach that has a crest elevation determined to equalize the flow into all three reaches. The crest elevations for the weirs at reaches 196, 197 and 198 are 29.5, 28.7 and 30.3, respectively. This is because there is a head loss across the length of reaches 199, 200 and 201, and in order to equalize the outflow, the weir crests need to be at different elevations. The inverts of the overland flow reaches are set equal with the average topographic surface at that location. The overland flow reaches are divided into segments approximately 1 mile in length so that the inverts of the overland flow path can follow the natural topography.

6.2.1. Alternative representation of Project 2a

Since overland sheet flow is represented within the hydrology sub-model of WAM, using several hydrologic and hydraulic iterations is one potential methodology for representing the effects of Project 2a that has been discussed with District staff. The premise of an iterative approach is to calculate the flows and loads generated by the contributing area to be diverted in the initial simulation. The equivalent volume of water with the correct concentration of TP would then be applied to the DIRWMA as rainfall in a second iteration of the model. The rainfall would then be routed to the correct reaches utilizing the hydrologic overland flow routing routines.

There have been cases when the rainfall mechanism within WAM has been used to represent land-use practices and not natural precipitation, such as an agricultural spray field utilizing treated wastewater for irrigation. However there are several distinctions between the parameters of representing Project 2a with an iterative approach and representing treated wastewater spray fields.

One distinction is that the concentration of TP in the runoff from the diverted contributing area varies with time based on land management practices such as fertilization, planting and harvesting. WAM does not have a mechanism for allowing the concentration of background TP in rainfall to vary with time over the simulation period. In order to simulate changes in fertilization with time, the land-use parameters representing the restored pastures in DIRWMA would have to be modified to incorporate the anticipated TP load. Since land-use parameters are defined for one-year recurring cycles, there would also be a difficulty in representing the loads being recreated based on the diverted runoff would as annually cyclical.

Another distinction of simulating the diverted runoff as rainfall using an iterative approach is that rainfall is generally applied uniformly across a defined area (rainzone), such as a Theissen polygon or a NEXRAD pixel. Since the method for diverting water to DIRWMA assumes the use of a distribution canal along the northern boundary, the TP concentrations would be expected to decrease spatially due to attenuation. If the diverted runoff were applied as rainfall to the DIRWMA, the rainzone would need to be defined in a manner appropriate to the anticipated spatial variation of the proposed project.

Another concern in utilizing the iterative approach is the computational time associated with the Baseline Period simulation. Since each simulation requires a 12 hour run time, and the iterative approach would require two simulations at a minimum, this alternative methodology would require extended computational effort.

Due to the complexities involved in any alternative representation of Project 2a, the methodology described in Section 6.2 was used to evaluate the effectiveness of this regional water quality improvement project. The following sections describe the results of this evaluation.

6.2.2. Short-term simulation results

The first determination in evaluating Project 2a is examining the effectiveness of diverting major runoff events, as described in **Figure 6.2** above. **Figures 6.4** and **6.5** below illustrate a comparison of the flow south under normal conditions and the diverted peak flow during elevated stages.

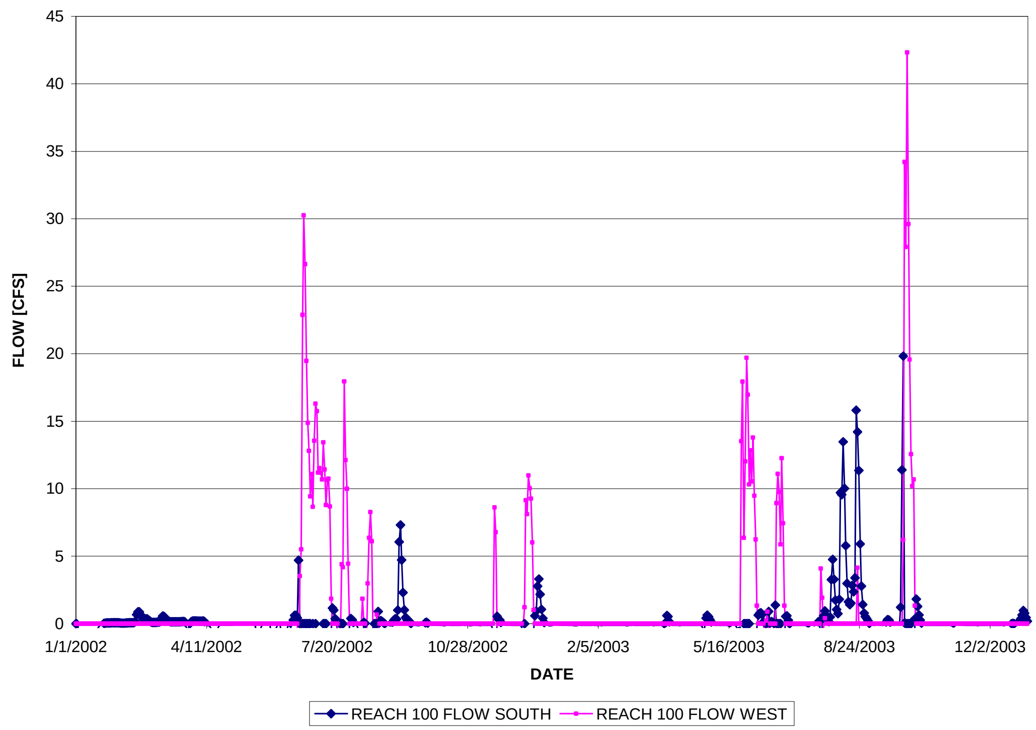

Figure 6.4: Western Flow Diversion at Reach 100

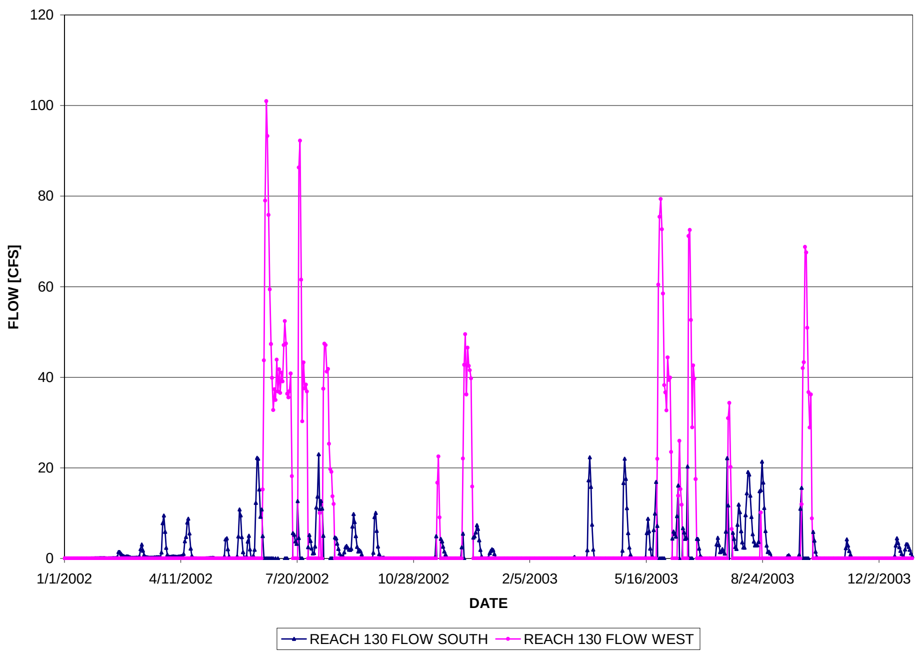

Figure 6.5: Western Flow Diversion at Reach 130

Another important evaluation to determine the effectiveness of the WAM representation of Project 2a is to assess the overland flow reaches in simulating sheet flow across the DIRWMA. **Figure 6.6** illustrates the flow into the headwaters of each reach from the distribution canal, while **Figures 6.7, 6.8** and **6.9** illustrate the depth of flow in each overland flow reach as compared with the width of the channel at that depth.

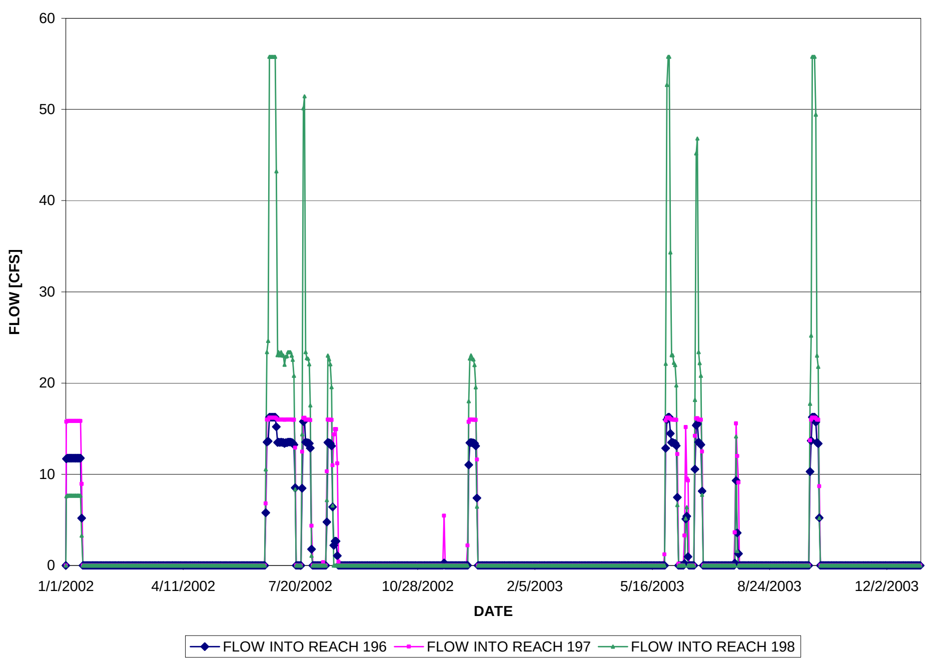

Figure 6.6: Distribution of Pumped Runoff to Overland Reaches

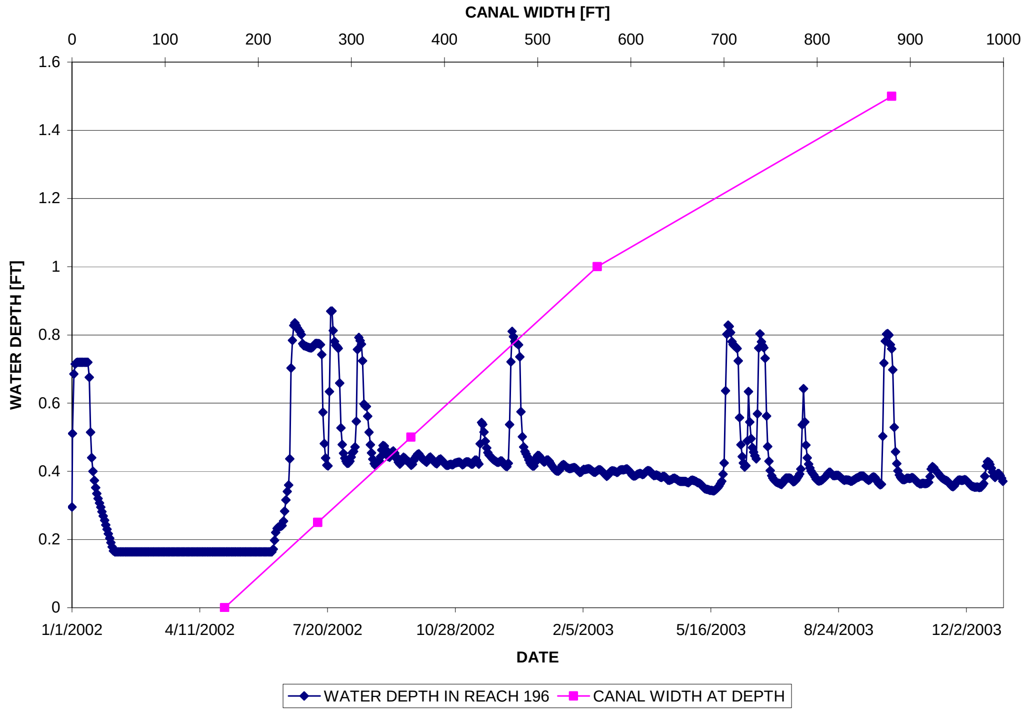

Figure 6.7: Water Depth and Channel Width for Reach 196

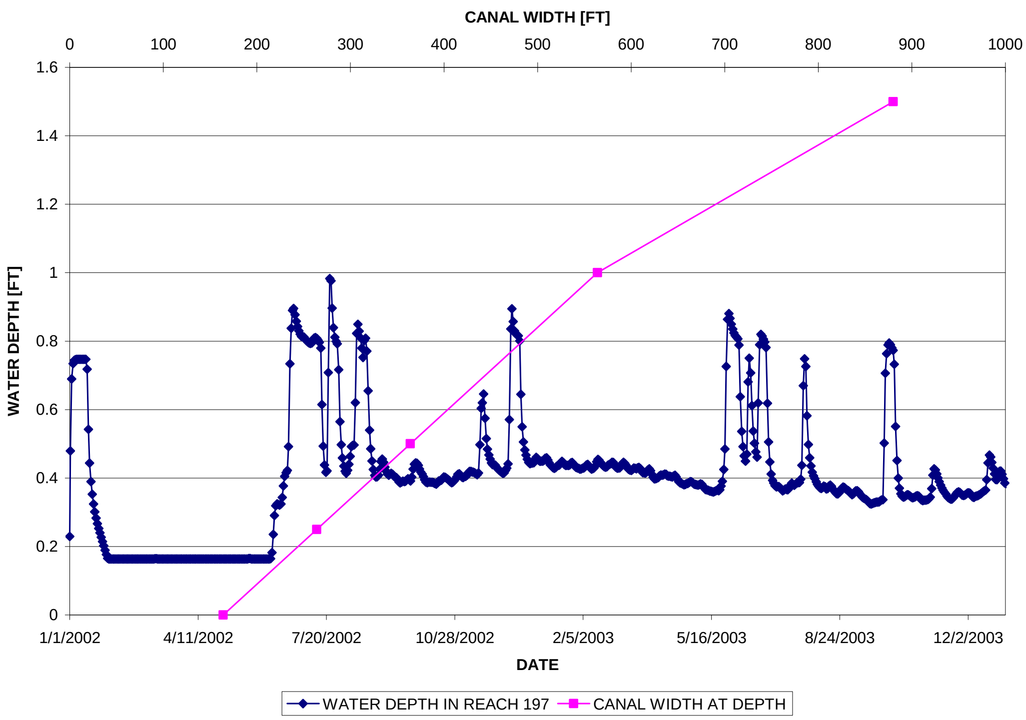

Figure 6.8: Water Depth and Channel Width for Reach 197

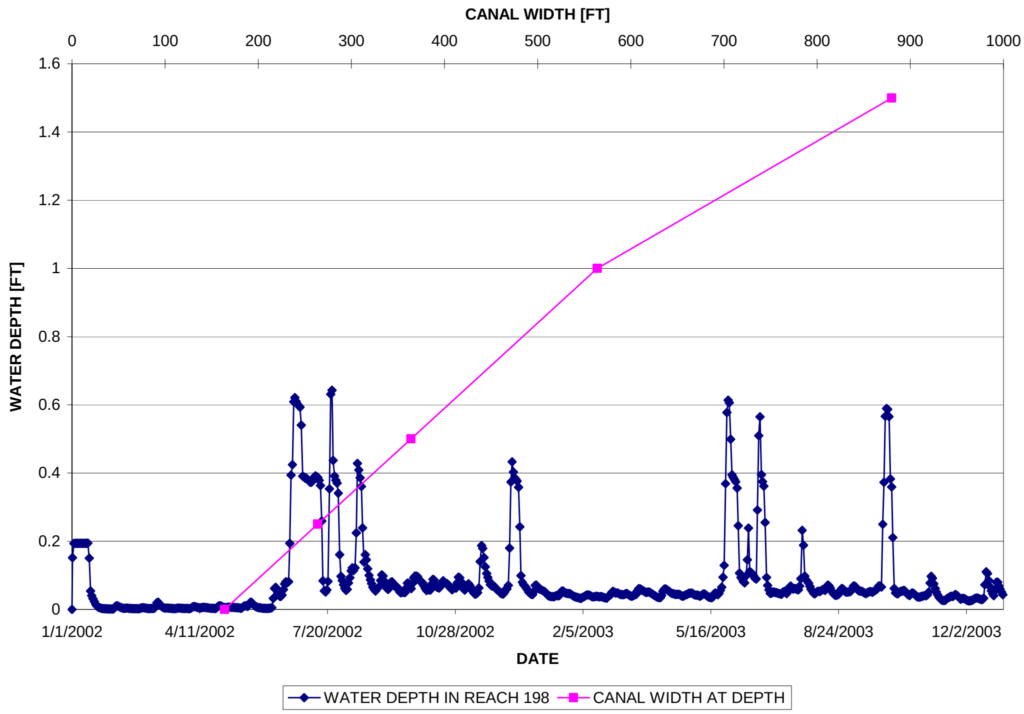

Figure 6.9: Water Depth and Channel Width for Reach 198

Figures 6.7, 6.8 and **6.9** illustrate that during high flow events, during 2002 and 2003, there was insufficient runoff to cover the entire width of the DIRWMA with overland sheet flow. This is consistent with accepted knowledge that overland sheet flow generally occurs for distances of 300 feet at a maximum (SCS TR-55, 1986), after which natural deviations in the land surface or vegetation cause the flow regime to convert to channelized flow in small rivulet like streams.

Since Project 2a is a variation of Project 2, the simulated reduction in TP loading is evaluated not only with respect to the existing condition, but also with respect to the Project 2 results. **Figure 6.10** illustrates the simulated cumulative TP load in pounds for the 2002 and 2003 simulation period. The scenario evaluation for Project 2 was illustrated along the Deer Fence Canal at Duck Curve, because this was the closest downstream reach that captured the effects of all the changes associated with Project 2. For Project 2a the evaluation is performed at a location slightly further downstream at Reach 37, since the scenario includes changes in the tributaries of Deer Fence Canal downstream of Duck Curve. The average annual reduction for Project 2a from the base is 60%. However, the incremental improvement over Project 2 is 12%.

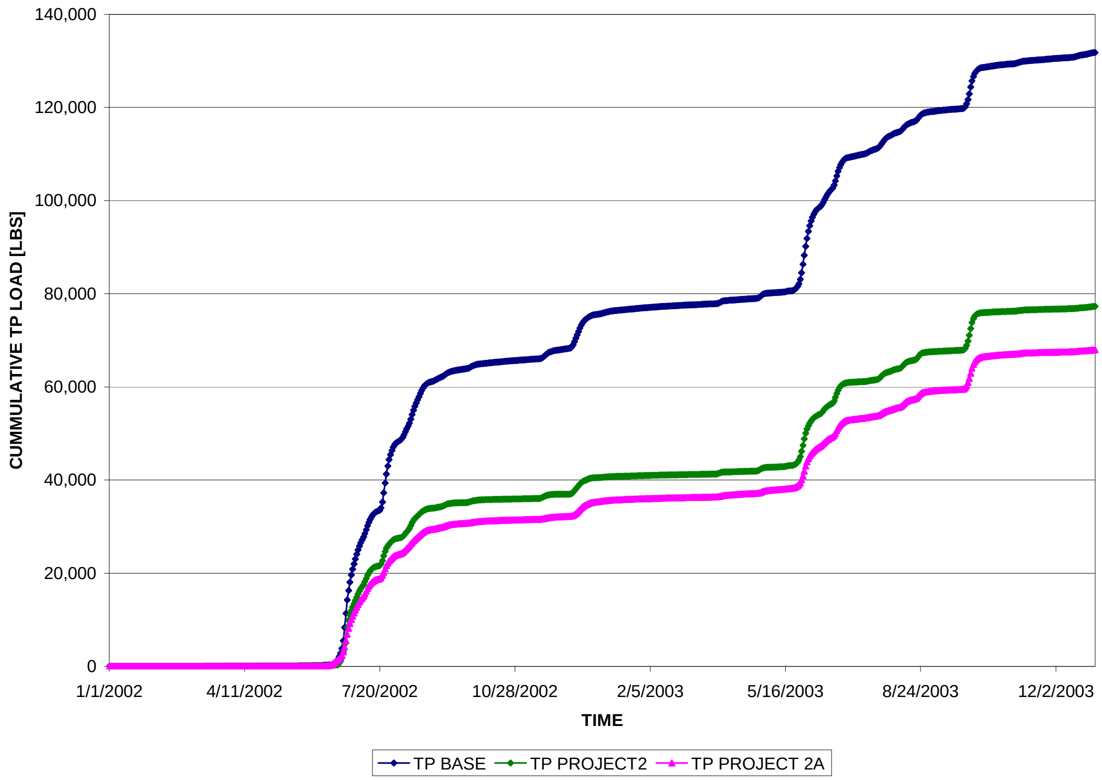

Figure 6.10: Western Flow Diversion at Reach 130

6.3 Project 2a Short-Term Simulation Summary

Table 6.1 below compares the 2002 and 2003 reduction in TP load simulated for Project 2a at Duck Curve with Project 2. The District recommendations for Project 2a describe "water pumped from the adjacent farms to the east (of Basin 26-310-04)". There were no other scenarios described in the recommendation. As part of the efforts of the ADA Team, there were several scenarios developed for Project 2a. These alternate scenarios are not presented in this document since they were not alternatives of Project 2a but were less optimal configurations of the proposed project.

Table 6.1: Comparison of Short-Term Simulation Project 2a with Project 2

PROJECT SCENARIO	ANNUAL TP REDUCTION [%]			AVERAGE REDUCTION [LB]
	2002	2003	AVERAGE	
Project 2	40%	47%	44%	20,335
Project 2a	46%	35%	41%	27,262

The reductions shown in Project 2a incorporate all of the modifications inherent to Project 2. Therefore Project 2a also assumes that the restored landscape has drainage and water quality parameters consistent with the parameters used by WAM to define a scrub and brushland. This does not take into account the heightened quantities of TP within the soil column expected in abandoned agricultural land. A comparison of the 27,262 pound average annual TP reduction for the selected scenario with the annual average TP load from the C-139 Basin described in **Table 2.1** illustrates that the removal of that quantity would account for a 41% reduction in the simulated average annual TP load for the entire basin.

6.4 Project 2a Baseline Period Evaluation

As described above Project 2a consists of a modification of existing surface water management infrastructure that diverts water west from farms adjacent to Basin 26-310-04 to the east during high runoff events. Project 2a incorporates all of the changes proposed in Project 2. A Baseline Period simulation was performed for this configuration to provide an appropriate assessment of the performance of Project 2a over a wide range of hydrologic conditions. **Figure 6.11** illustrates the effects of the optimized Project 2a scenario on basin-wide TP loads over the duration of the baseline period. **Figure 6.12** illustrates the cumulative effect on TP loads over the baseline period. **Figure 6.11** illustrates that results of the Project 2a simulation reflect the improvements shown in Project 2. **Figures 6.11** and **6.12** demonstrate that Project 2a provides an incremental benefit to the impact of Project 2. **Table 6.2** describes the performance of Project 2a with respect to Project 2, the existing condition and the targeted TP load.

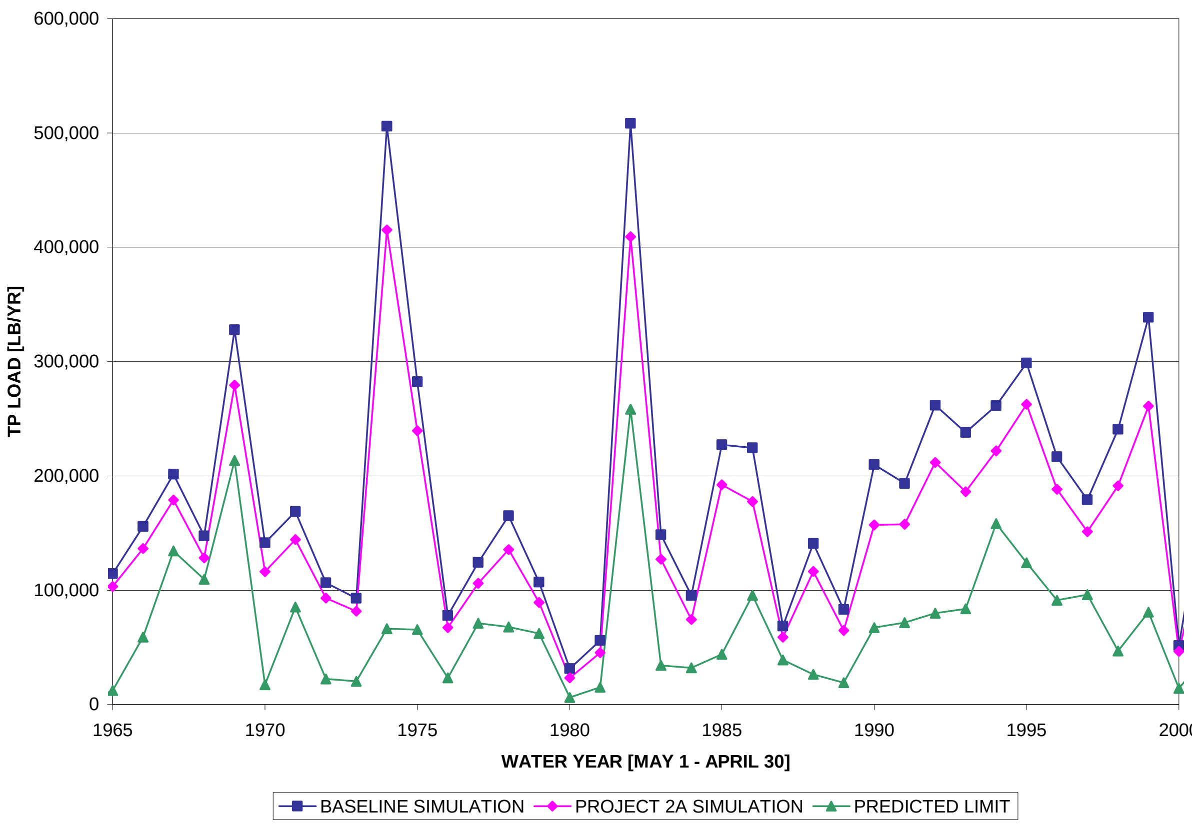

Figure 6.11: Annual TP Load for Project 2a and Existing Condition

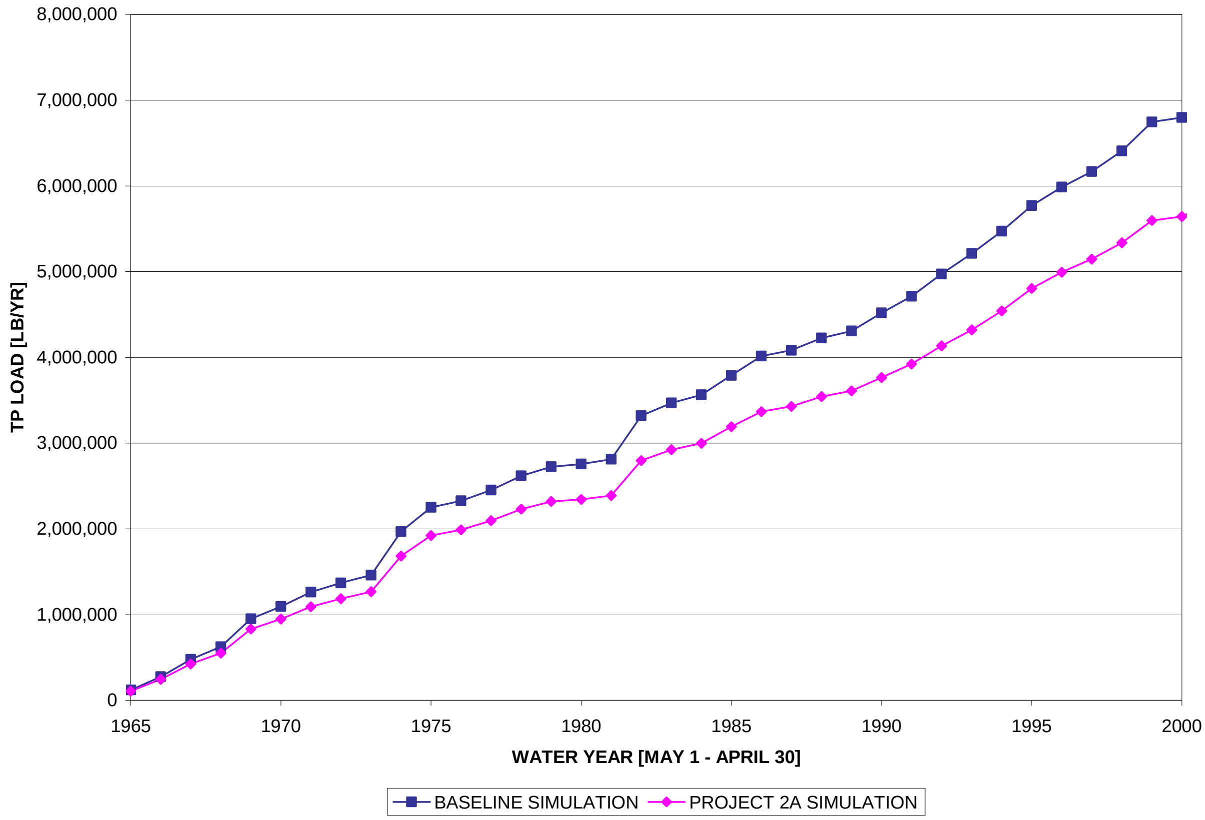

Figure 6.12: Cumulative Effects on TP Load for the Baseline Period

Table 6.2: Comparison of Project 2a, Project 2, Existing and Targeted Annual TP

CONDITION	MIN ANNUAL TP [LB]	MAX ANNUAL TP [LB]	AVG ANNUAL TP [LB]	TOTAL BASELINE PERIOD TP [LB]
PROJECT 2a	23,391	415,103	156,631	5,638,739
PROJECT 2	24,397	425,618	161,020	5,796,721
EXISTING CONDITION	31,421	508,392	188,715	6,793,743
TARGETED LOAD	6,113	258,419	69,852	2,514,680

Although the proposed improvement project does provide increased TP load reductions with respect to Project 2, the combined project implementation of Projects 2 and 2a would not provide the significant improvements required to reach the rainfall-adjusted target TP load as described within the SFER. In order to meet the target TP load requirements, Projects 2 and 2a would need to be coupled with other local and regional improvement projects.

7.0 PROJECT 2B

7.1 Project 2b Objectives and Components

As described in the April 20th letter, the second proposed project is divided into three separate projects for evaluation. The third scenario is described within this report as Project 2b, and involves impounding water from 26-310-04 and gravity-fed from the adjacent farms to the north. The objective of redirecting flows from adjacent farms to the north, as described in the District's recommendation, is to restore the historic drainage patterns for the sub-region by "interconnecting" the areas.

Based on the topography and field reconnaissance described in C-139 Phase I Report, the historic drainage patterns of the sub-region include a slough of connected freshwater marsh habitats. Prior to the construction of the ALICO South Boundary Canal and the Deer Fence Canal, the western sub-basins of the C-139 Basin drained south as part of a larger marsh system. **Figure 7.1** illustrates the topography of the region and demonstrates the natural drainage pathways of the western sub-region.

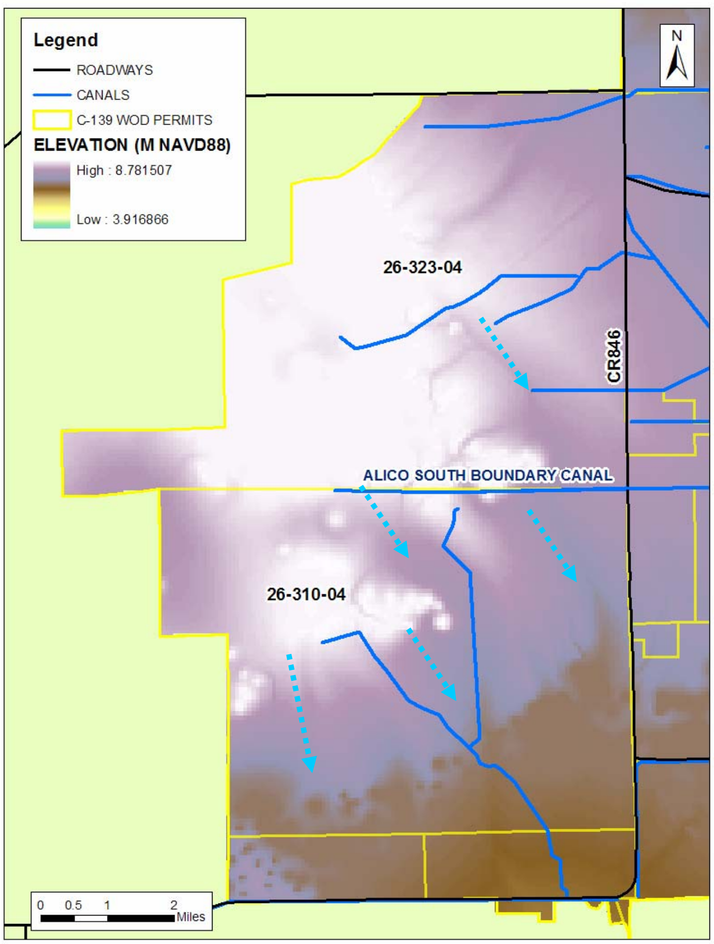

Figure 7.1: Topography of the Western C-139 Basin

The objective of Project 2b is to achieve additional water quality treatment by restoring the historic pathways, and providing an alternate, slower and less-channelized pathway for the runoff from western Basin 26-323-04. In consideration of the restoration efforts of the FFWCC, the District's recommendation for Project 2b is to assume that all agricultural operations within the DIRWMA are halted, the surface water management features are reverted to a native state and the basin's natural impoundment capacity is optimized.

7.2 Project 2b Optimization

The scenario for treatment described by the District recommendations involves closing off the reaches connecting the western portion of Basin 26-323-04 with the eastern stormwater management system, allowing the stages to increase until the excess runoff would travel south into DIRWMA via overland sheet flow. **Figure 7.2** illustrates the conceptual scenario.

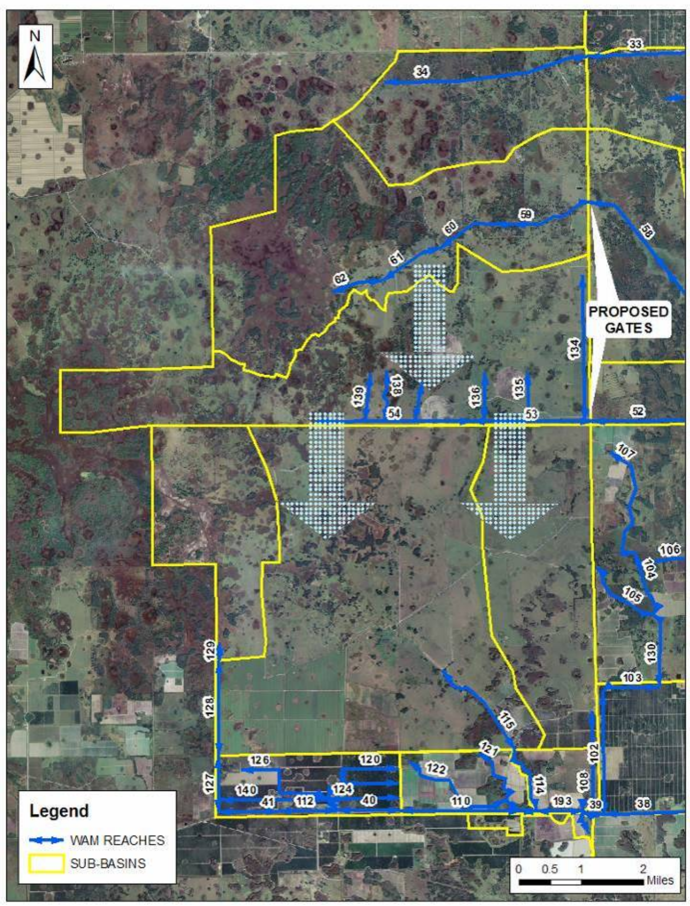

Figure 7.2: Conceptual Project 2b Implementation

As described as part of Project 2a optimization, it is not possible within the framework of WAM for water to be routed out of a canal reach and onto the land surface. One potential modification would be to remove the reaches and the sub-basin boundaries within the western portion of Basin 26-323-04. WAM would then represent overland flow processes from the northern portion of the sub-region through the DIRWMA. However, this would be temporally permanent and could not be changed with the season. The expectation described by the District's recommendation is that water could be diverted seasonally, and not on a permanent basis.

The most effective way to represent this scenario is to add wide, shallow reaches connecting the northern portions of the sub-region with the existing reaches in the DIRWMA similar to the Project 2a methodology. **Figure 7.3** illustrates the modified implementation plan for Project 2b. Similarly to the Project 2a configuration each overland segment is approximately 1 mile in length and the invert elevation is equal to the average topographic surface elevation. In order to represent even distribution of flow through the overland flow reaches, weirs were included at the top of the 196, 197 and 198 reaches with a crest elevation of 28.7 feet NAVD.

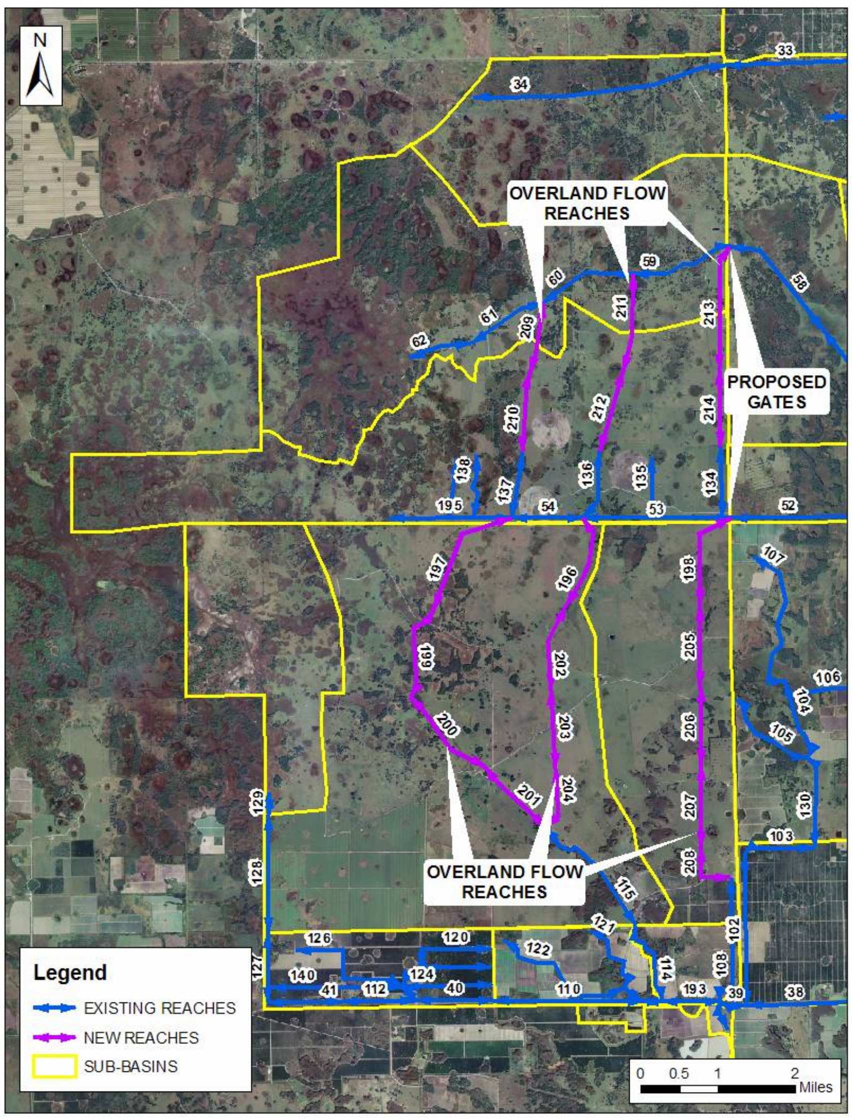

Figure 7.3: Project 2b Implementation Schematic

The fundamental mechanics of Project 2b include gates that can be closed to divert water from channelized flow east to overland flow south. The only variable for the simulation is the timing of closing the gates and duration the gates stay closed. The two scenarios evaluated below are: event-based diversion and seasonal diversion. For event-based diversion the gates are closed during large runoff events, which are defined as periods when the simulated stages in Reach 34 exceed 21.6 feet NAVD. For seasonal diversion the gates are closed on June 1st of each year and remain closed until the start of the next calendar year on January 1st.

7.2.1. Event-based diversion scenario results

In order to verify that the runoff is being diverted during high runoff events, **Figures 7.4** and **7.5** illustrate the flow east from basins L2-01-11 and L2-01-12 being diverted south through the DIRWMA.

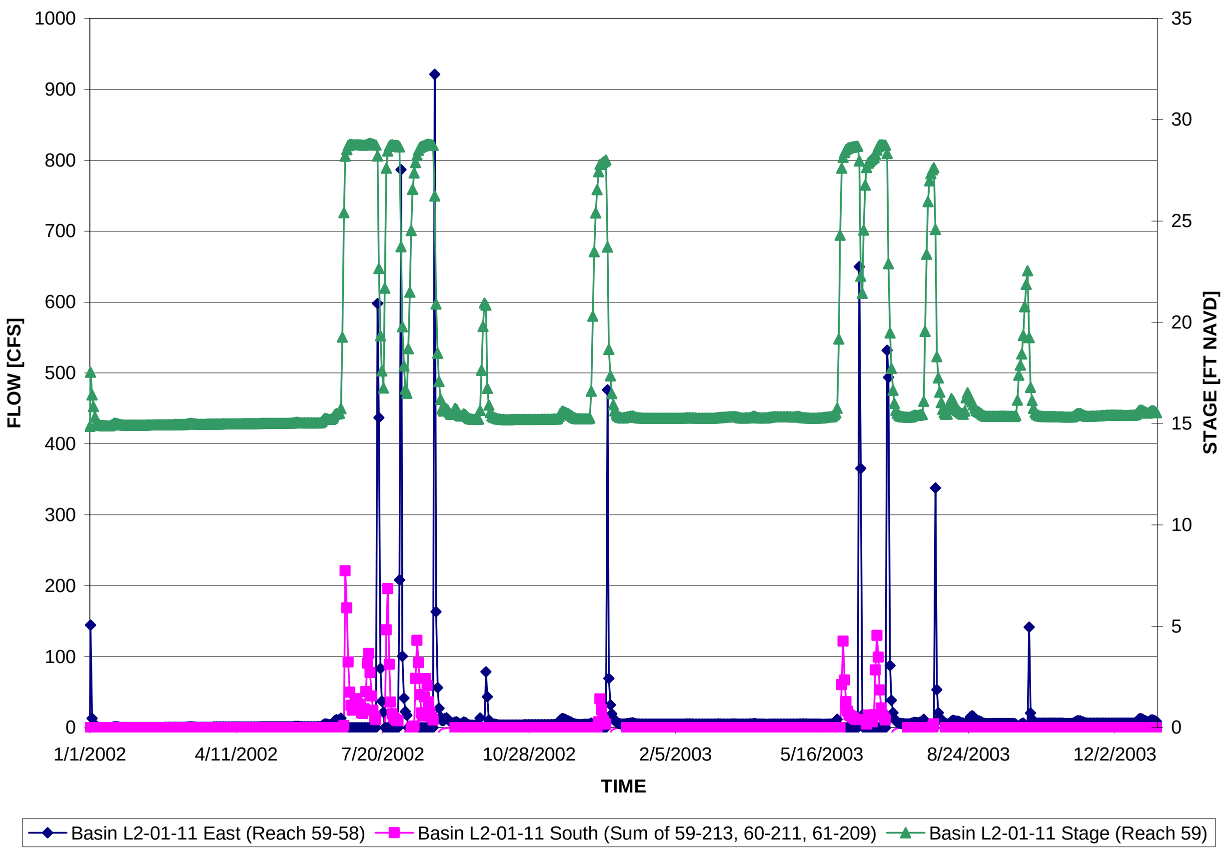

Figure 7.4: Diverted Flow and Canal Stages for Basin L2-01-11

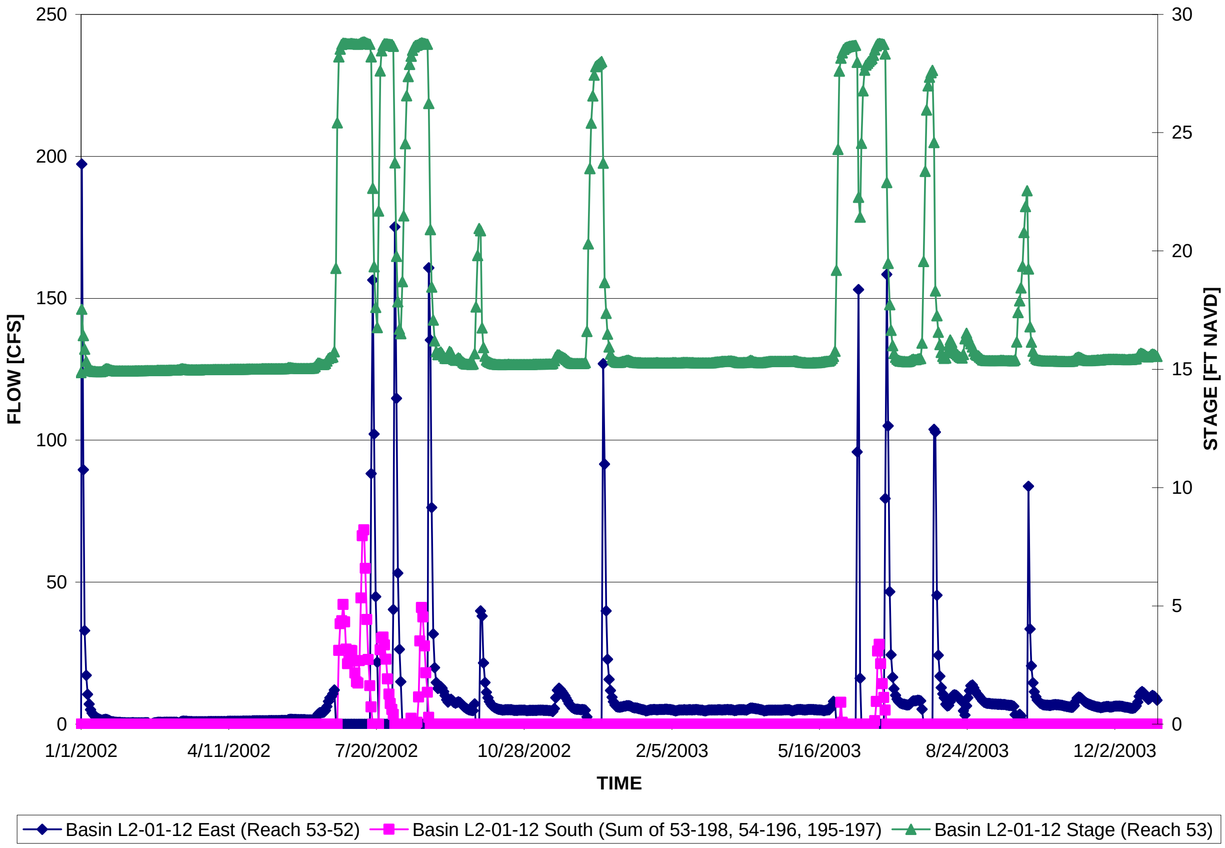

Figure 7.5: Diverted Flow and Canal Stages for Basin L2-01-12

Figures 7.6, 7.7 and **7.8** illustrate the flow rate and depth of flow in the overland flow reaches 196, 197 and 198. The figures also illustrate the canal width which is notably wider than the proposed width for Project 2a. Since the runoff diverted in Project 2b is significantly greater than the runoff diverted in Project 2a, the overland reaches were made wider to insure stages were not unreasonably high.

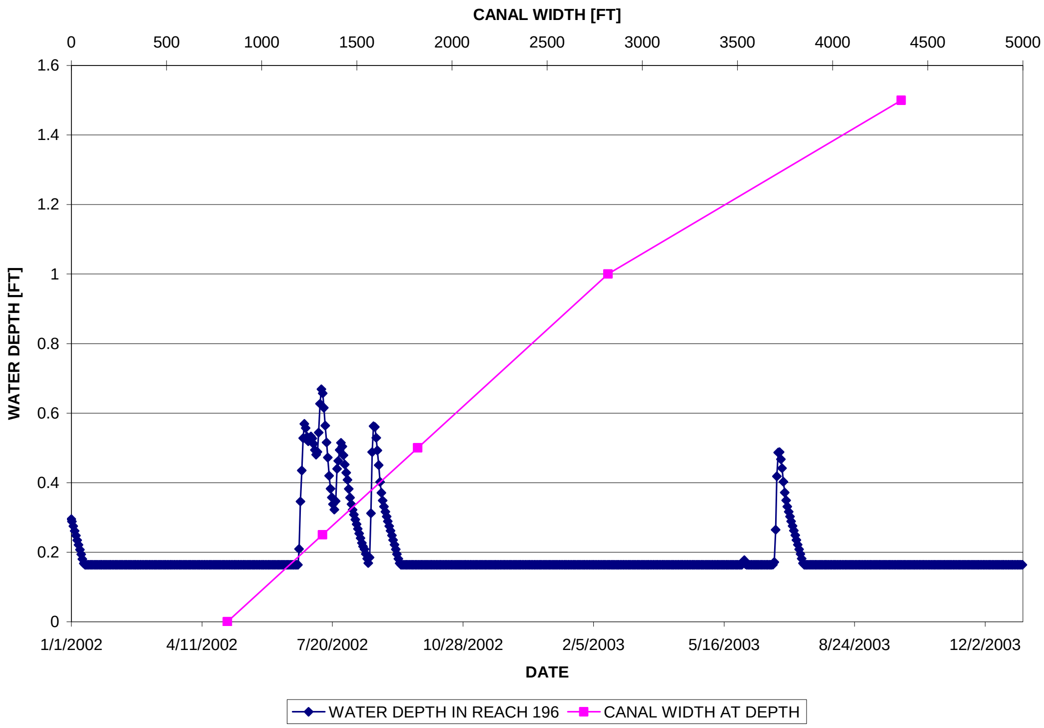

Figure 7.6: Overland Flow Reach Stage and Width in Reach 196

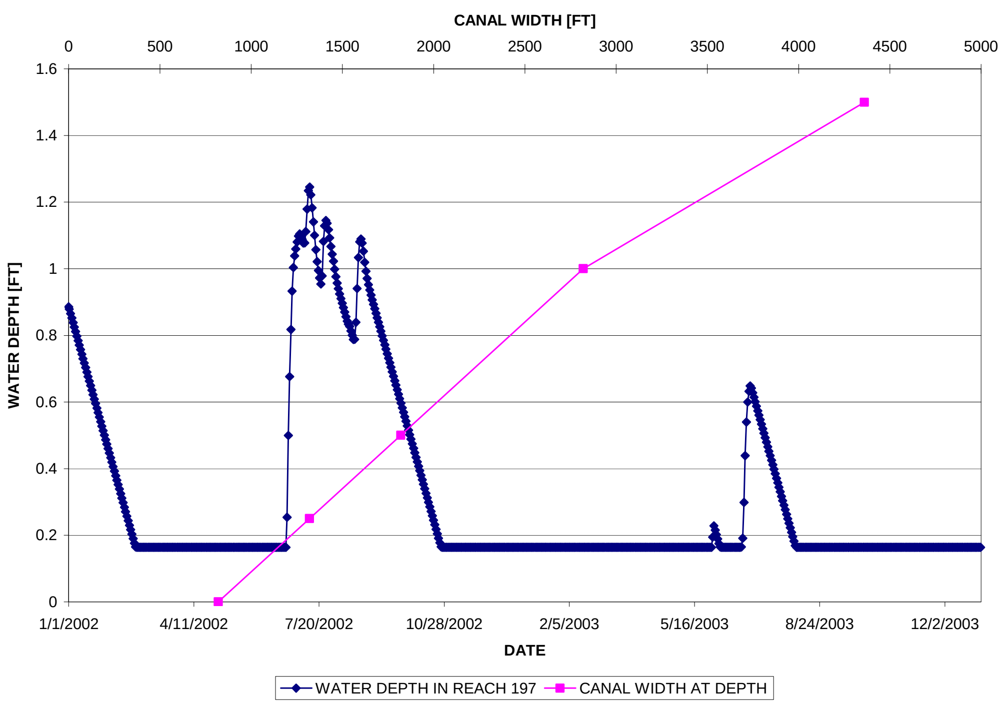

Figure 7.7: Overland Flow Reach Stage and Width in Reach 197

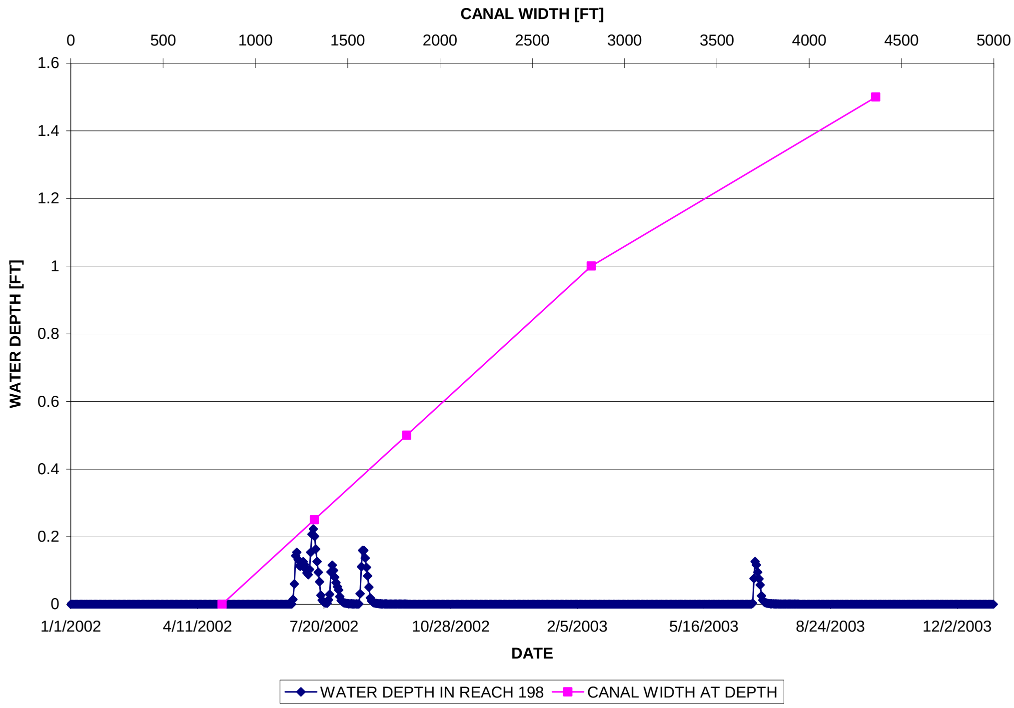

Figure 7.8: Overland Flow Reach Stage and Width in Reach 198

For Project 2 the Deer Fence Canal at the Duck Curve Bridge was the location where the impacts were measured because it was the downstream most location that included all changing reaches as tributaries. For the purposes of Project 2b's effectiveness, the only location where both Basins 26-323-04 and 26-310-04 are tributary is the C-139 Basin outflow which incorporates a substantial amount of unmodified contributing area that would dilute the affects of the proposed project. Therefore, the flows and TP loads of the three pertinent downstream reaches (39, 52 and 58) are added together in the output comparisons. The impact of this flow diversion on total flow and cumulative TP load, in comparison with the base condition and as compared to Project 2 (native DIRWMA), is illustrated on **Figures 7.9** and **7.10**. The annual TP reduction for Project 2b as compared with Project 2 is 27% for 2002 and 16% for 2003.

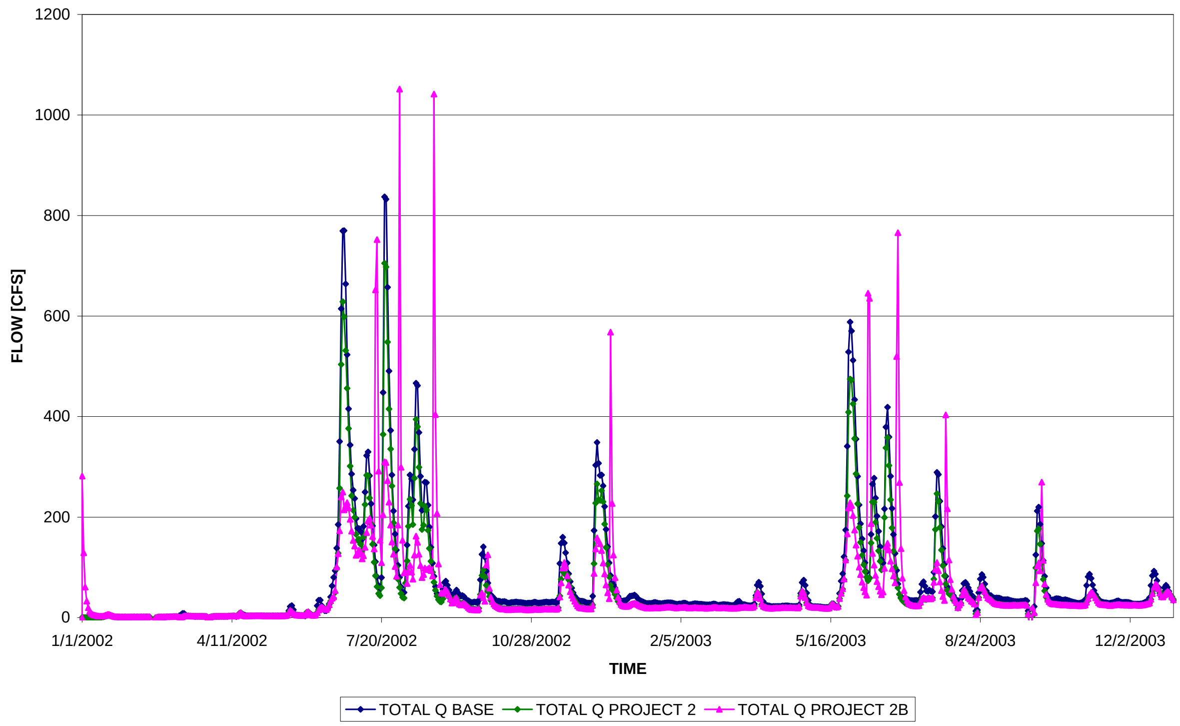

Figure 7.9: Reduction in Runoff for Western 26-323-04 and at Duck Curve

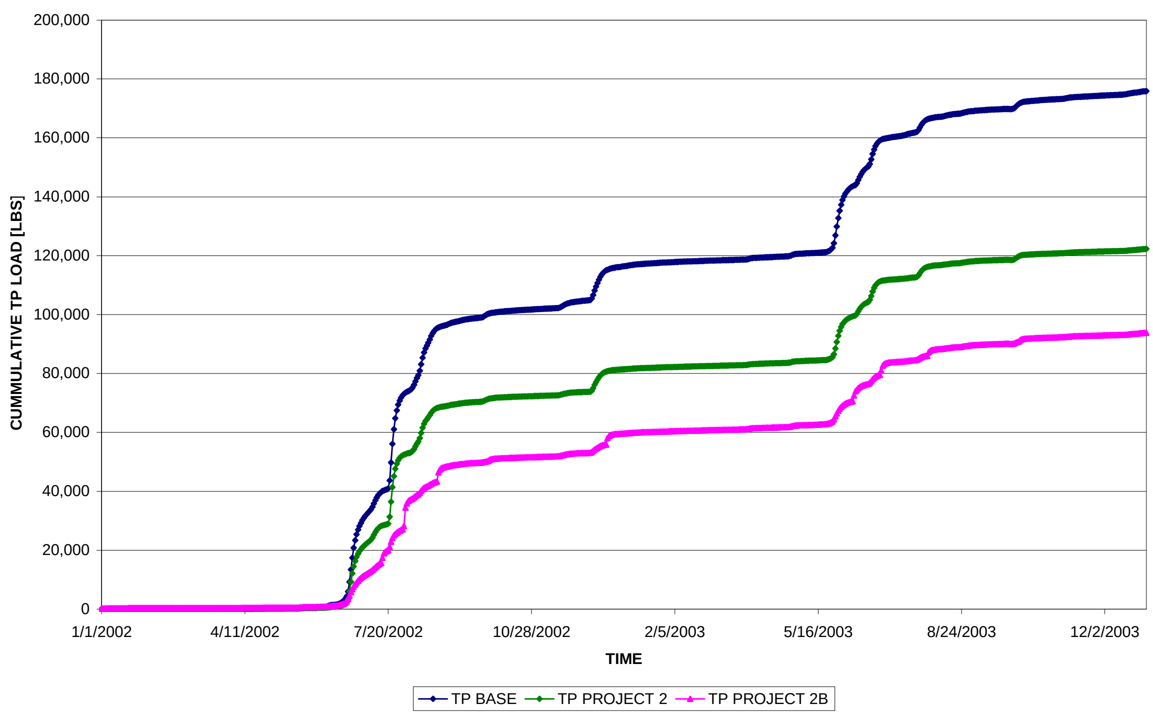

Figure 7.10: Reduction in TP for Western 26-323-04 and at Duck Curve

7.2.2. Seasonal diversion scenario results

In order to verify that the runoff is being diverted during the wet season, **Figures 7.11** and **7.12** illustrate the flow east from basins L2-01-11 and L2-01-12 being diverted south through the DIRWMA.

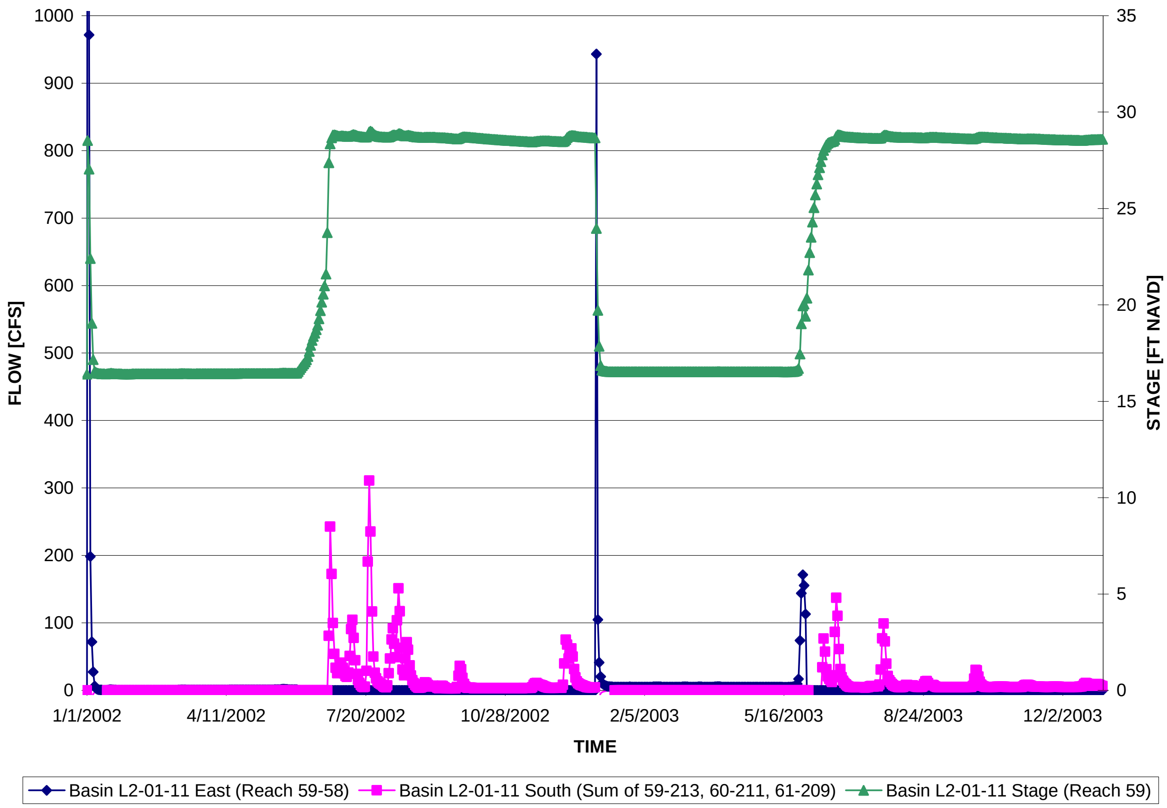

Figure 7.11: Diverted Flow and Canal Stages for Basin L2-01-11

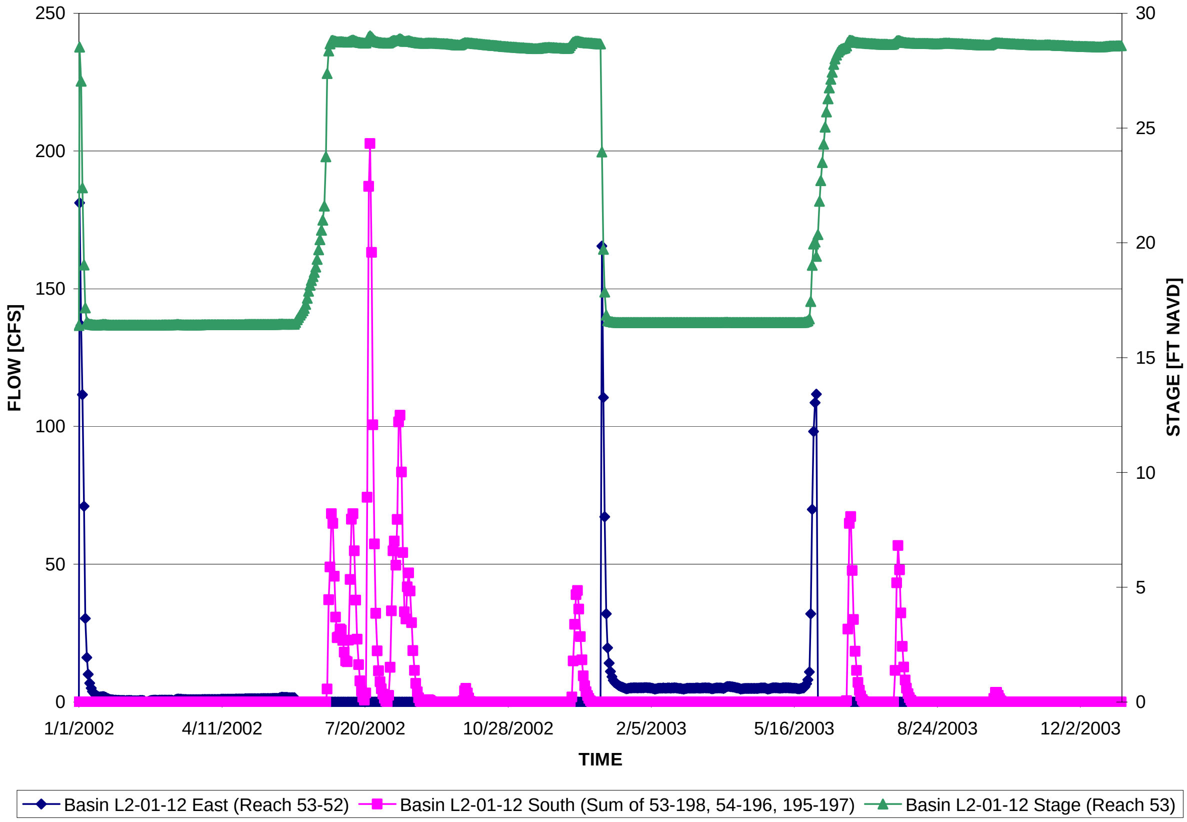

Figure 7.12: Diverted Flow and Canal Stages for Basin L2-01-12

Figures 7.13, 7.14 and **7.15** illustrate the flow rate and depth of flow in the overland flow reaches 196, 197 and 198.

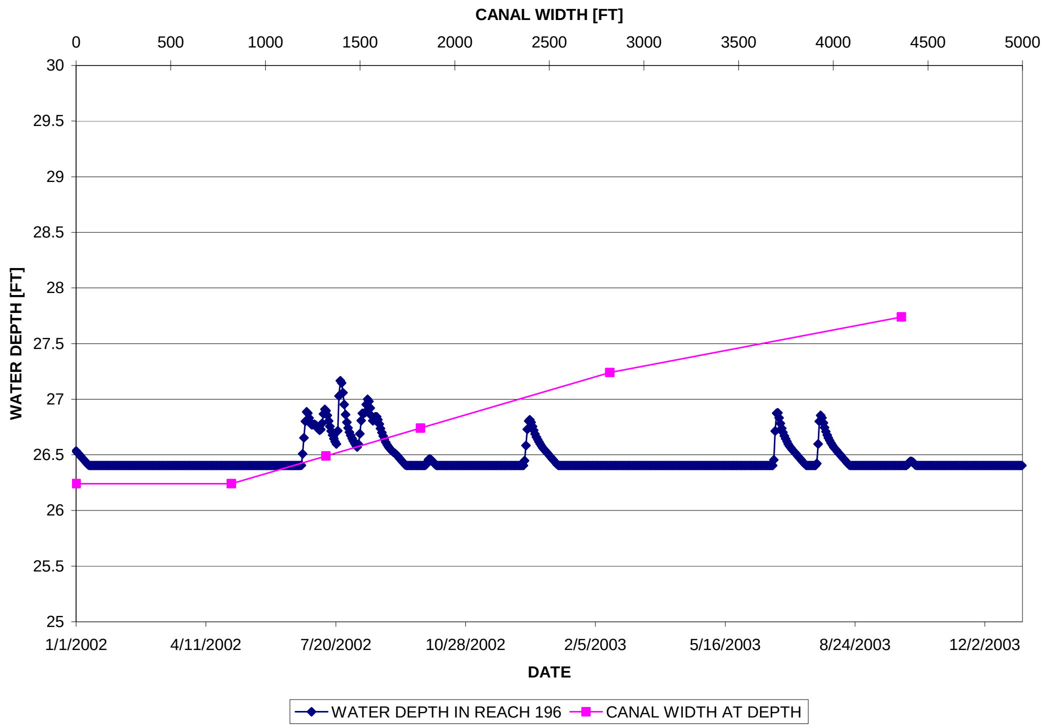

Figure 7.13: Overland Flow Reach Stage and Width in Reach 196

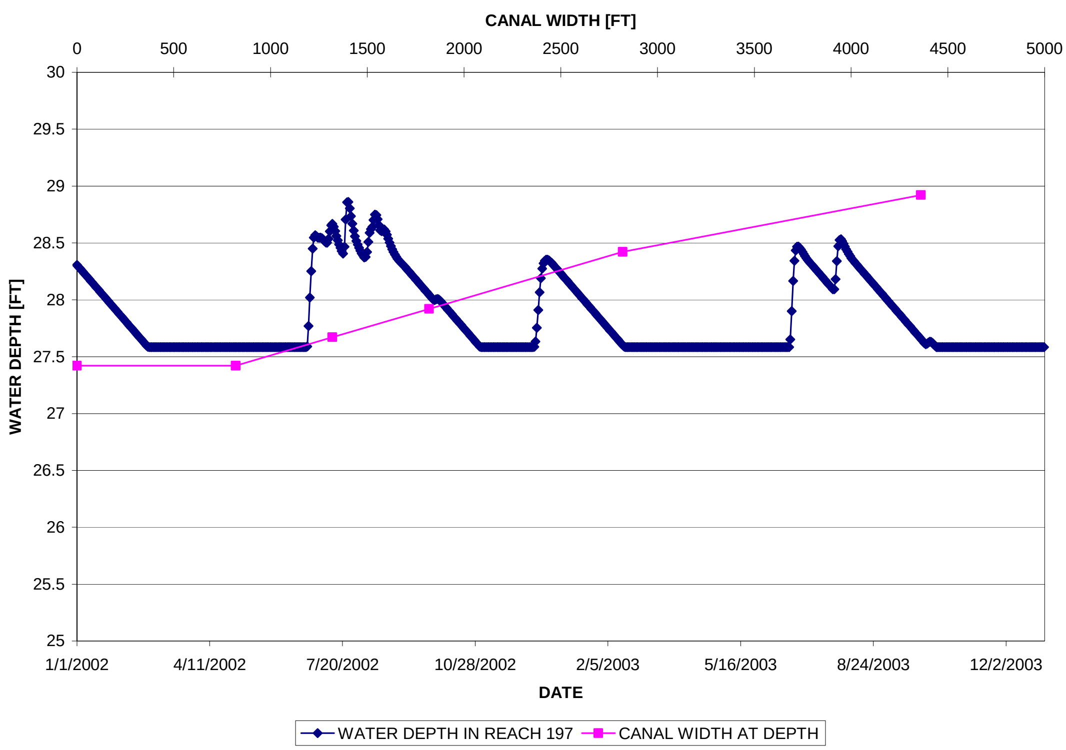

Figure 7.14: Overland Flow Reach Stage and Width in Reach 197

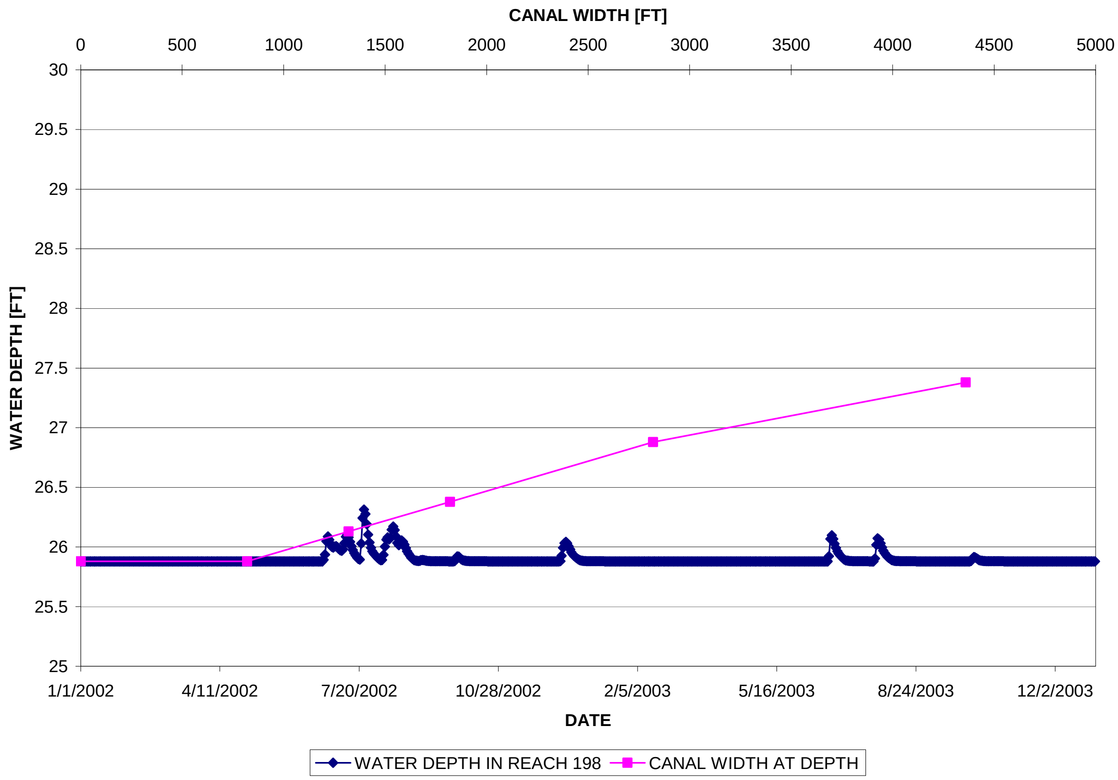

Figure 7.15: Overland Flow Reach Stage and Width in Reach 198

The impact of seasonal diversion on total flow and cumulative TP load, in comparison with the base condition and as compared to Project 2 (native DIRWMA), is illustrated on **Figures 7.16** and **7.17**. The annual TP reduction for Project 2b as compared with Project 2 is 43% for 2002 and 25% for 2003.

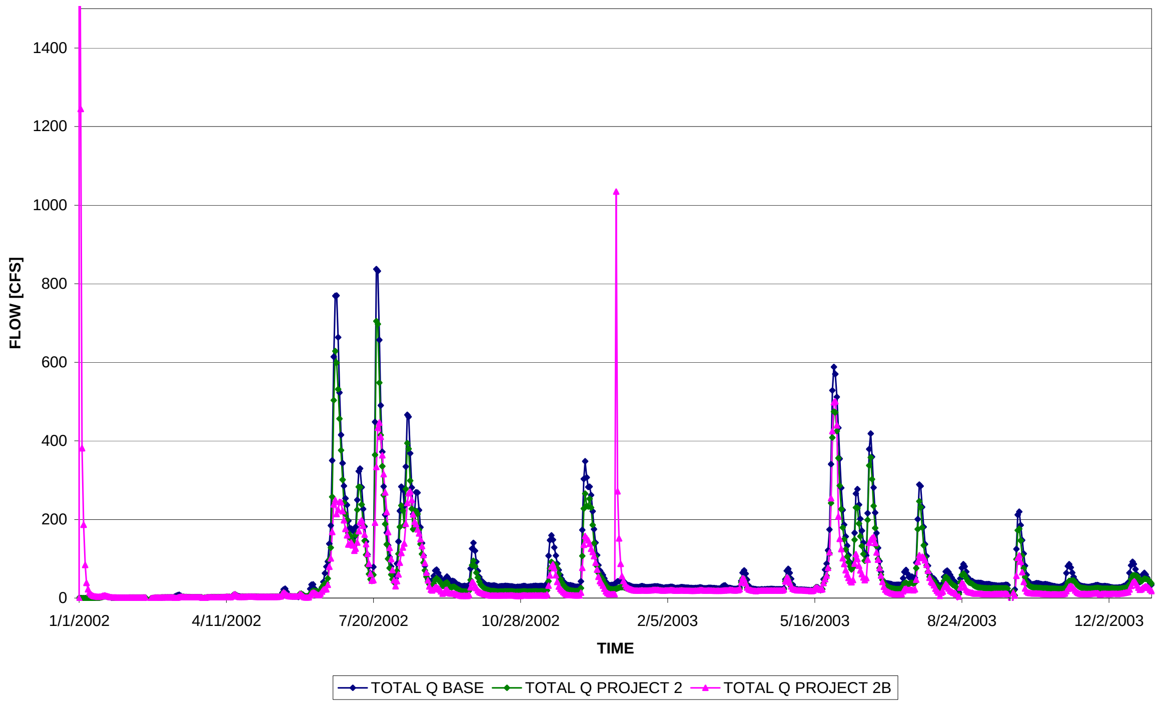

Figure 7.16: Reduction in Runoff for Western 26-323-04 and at Duck Curve

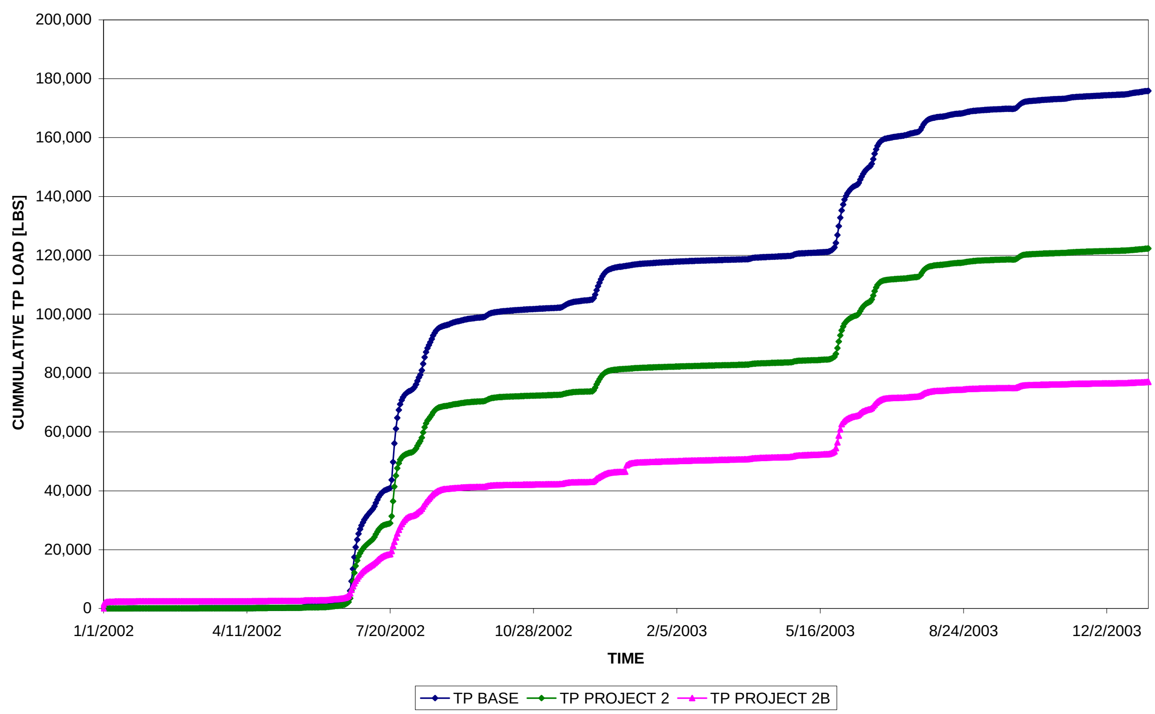

Figure 7.17: Reduction in TP for Western 26-323-04 and at Duck Curve

7.3 Project 2b Short-Term Simulation Summary

Table 7.1 below compares the 2002 and 2003 reduction in TP load at Duck Curve simulated for Project 2 and Project 2b for both event-based and seasonal diversion. The District recommendations for Project 2b describe “water gravity fed from the adjacent farms to the north (of Basin 26-310-04)”. There were no other scenarios described in the recommendation. As part of the efforts of the ADA Team, there were several scenarios developed for Project 2b, however the two variants of operational strategy are the only alternatives presented.

Table 7.1: Comparison of Short-Term Simulation Project 2b with Project 2

PROJECT SCENARIO	ANNUAL TP REDUCTION [%]			AVERAGE REDUCTION [LB]
	2002	2003	AVERAGE	
Project 2	40%	47%	44%	20,335
Project 2b Event Based Controls	49%	42%	46%	41,002
Project 2b Seasonal Controls	60%	49%	55%	49,447

The reductions shown in each scenario of Project 2b incorporate all of the modifications inherent to Project 2. Therefore each scenario of Project 2b assumes that the restored landscape has drainage and water quality parameters consistent with the parameters used by WAM to define a scrub and brushland. This does not take into account the heightened quantities of TP within the soil column expected in abandoned agricultural land.

Based on the results presented above the seasonal control scenario provides a significantly improved reduction in average annual TP load at 41,002 pounds as compared with the event based methodology. A comparison of the reduction for the seasonal scenario with the annual average TP load from the C-139 Basin described in **Table 2.1** illustrates that the removal of that quantity would account for a 55% reduction in the simulated average annual TP load for the entire basin. This is an incremental improvement of 11% over the 44% provided by Project 2.

7.4 Project 2b Baseline Period Evaluation

As described above, Project 2b consists of a modification of existing surface water management infrastructure that diverts water west from farms adjacent to Basin 26-310-04 to the north during the wet season. Project 2b incorporates all of the changes proposed in Project 2. A Baseline Period simulation was performed for this configuration to provide an appropriate assessment of the performance of Project 2b over a wide range of hydrologic conditions. **Figure 7.18** illustrates the effects of the optimized Project 2b scenario on basin-wide TP loads over the duration of the baseline period. **Figure 7.19** illustrates the cumulative effect on TP loads over the baseline period. **Figures 7.18** and **7.19** demonstrate that Project 2b provides an incremental benefit to the impact of Project 2. **Table 7.2** describes the performance of Project 2b with respect to Project 2, the existing condition and the targeted TP load.

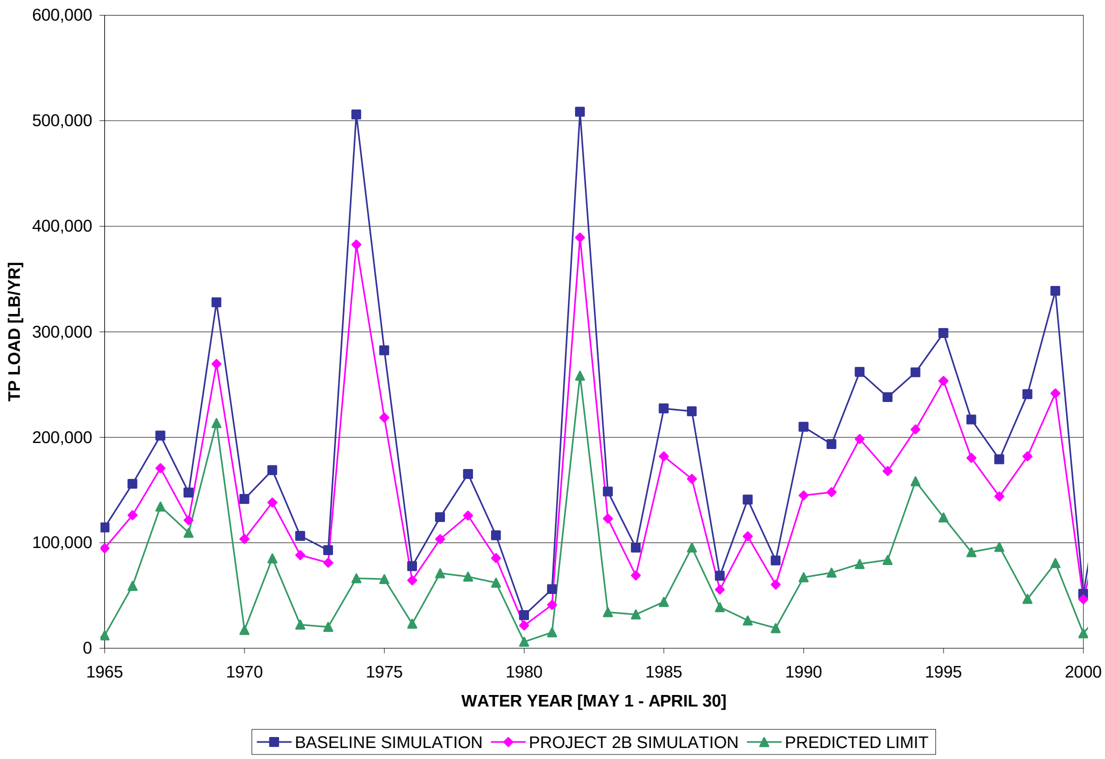

Figure 7.18: Annual TP Load for Project 2b and Existing Condition

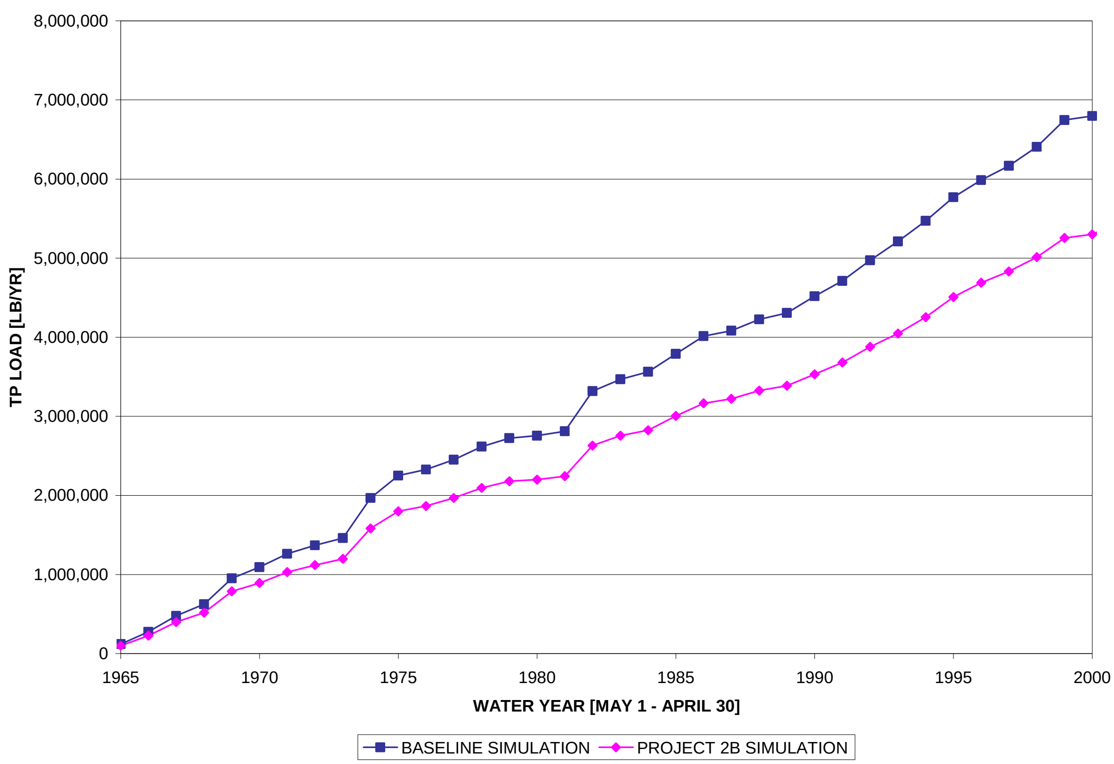

Figure 7.19: Cumulative Effects on TP Load for the Baseline Period

Table 7.2: Comparison of Project 2b, Project 2, Existing and Targeted Annual TP

CONDITION	MIN ANNUAL TP [LB]	MAX ANNUAL TP [LB]	AVG ANNUAL TP [LB]	TOTAL BASELINE PERIOD TP [LB]
PROJECT 2b	21,550	389,394	147,128	5,296,609
PROJECT 2	24,397	425,618	161,020	5,796,721
EXISTING CONDITION	31,421	508,392	188,715	6,793,743
TARGETED LOAD	6,113	258,419	69,852	2,514,680

Although the proposed improvement project does provide increased TP load reductions with respect to Project 2, the combined project implementation of Projects 2 and 2b would not provide the significant improvements required to reach the rainfall-adjusted target TP load as described within the SFER. In order to meet the target TP load requirements, Projects 2 and 2b would need to be coupled with other local and regional improvement projects.

8.0 PROJECT 3

8.1 Project 3 Objectives and Components

The April 20th, 2007 letter from the District describes the objective of Project 3 as "Reduce TP loads from the C-139 Basin by creating centrally located impoundments to provide storage and additional treatment to specific areas with the greatest water use needs and TP loading in discharges". Additional direction from the recommendation describes the use of "no more than three centrally located conceptual reservoirs (including an optimized CCDD impoundment) to reduce flows and loads". The reservoirs described herein are conceptual in nature and do not represent lands that are identified for the construction of publicly-funded impoundments.

Figure 8.1 identifies the location of the three conceptual impoundments utilized in the analysis of Project 3. The CCDD impoundment is currently used for stormwater retention of the adjacent Montura Ranches subdivision. The reason it is listed as a proposed location for a reservoir is because the current condition of the western levee prevents the CCDD from utilizing the reservoir to store water at a stage greater than 22.6 ft NAVD88 or two feet of depth. The eastern and southern levees are sufficiently constructed to impound water to a stage of 24.6 ft NAVD88. Assuming as a part of this project the western levee was upgraded to allow the additional two feet of depth, the CCDD impoundment would have approximately 5,000 additional acre-feet of storage.

The other two proposed impoundments are conceptual in nature and were located within the model in locations that were suitable for treatment of agricultural runoff. This would displace land-uses that provided the least significant change to the TP loading shown in the existing condition simulation. As described in Section 3.1 above, the construction of a regional impoundment requires due diligence with respect to economic and regulatory concerns. Therefore the locations illustrated in **Figure 8.1** are not representative of locations for proposed construction projects, but instead represent conceptual locations for proposed impoundments. The most conservative assumption is to displace land-uses with the lowest simulated TP loading rates since any existing agricultural uses within the foot-print of the proposed impoundment will not be simulated in the proposed condition simulation.

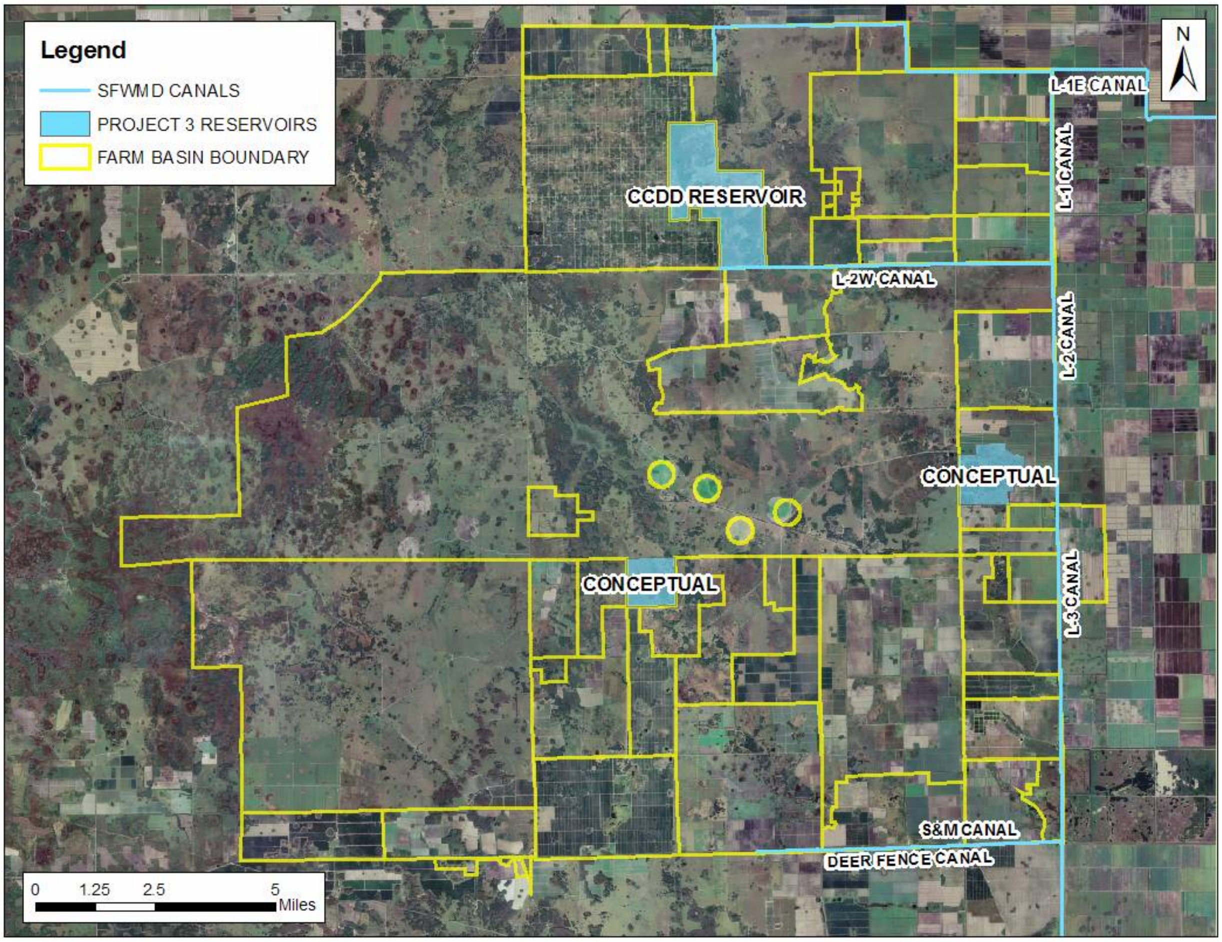

Figure 8.1: Location of Proposed Project 3 Impoundments

8.2 Project 3 Optimization

As illustrated within Project 1 above, impoundments are simulated within WAM as reaches with a geometry appropriate to hold the anticipated volume of water at stages consistent with available topographic and survey data where it may or may not be available. The modifications made to the existing conditions model for each impoundment is described below.

8.2.1. CCDD Impoundment Modification

The only significant modification to the existing condition model to represent increased storage in the CCDD impoundment was to modify the downstream control structure. The configuration of the CCDD impoundment within WAM is illustrated in **Figure 8.2**. The new reaches illustrated in Figure 8.2 were added to represent seepage from the CCDD impoundment to the neighboring lands, there were no reaches added for the representing Project 3 implementation. There is an inflow structure at the bottom of Reach 178, and a gate at the bottom of Reach 154. Reaches 155 and 154 have lengths and cross-sections that are consistent with the stage-volume relationship expected for the impoundment.

Figure 8.3 illustrates the simulated stages within the CCDD impoundment for the existing operation condition and the Project 3 proposed condition. The simulated outflow gate opens when stages within the impoundment exceed 24.6 ft NAVD88 and closes when stages within the impoundment are below 20.7 ft NAVD. This operational strategy allows the impoundment to double the existing capacity from roughly 4,910 acre-ft to approximately 9,820 acre-ft.

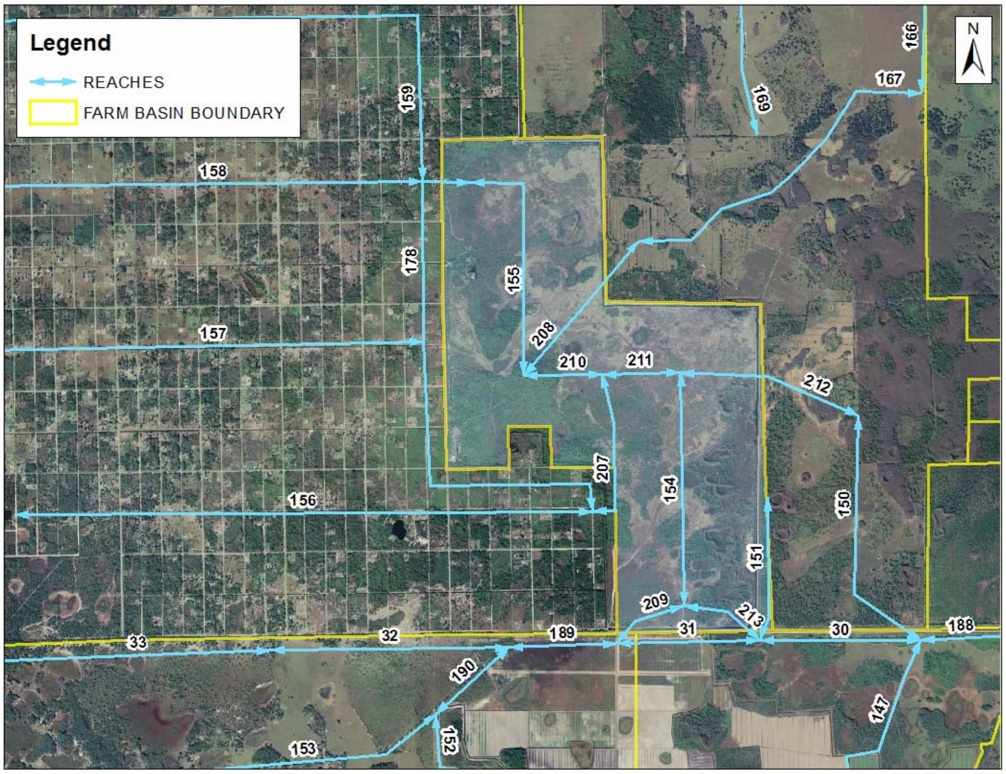

Figure 8.3: WAM Representation of the CCDD Impoundment

Figure 8.4 demonstrates the increased capacity of the CCDD impoundment by showing the inflow, outflow and stage of the retrofitted impoundment. This simulation does provide potentially unrealistic treatment efficiency during 2002, since the impoundment is essentially dry at the beginning of the simulation. Because of the initial condition and the expanded storage provided by the proposed project, there is no outflow from the CCDD impoundment over the short-term simulation period. This would indicate that there is available volume to redirect additional contributing area to the impoundment by changing the regional surface water management infrastructure, however the long term Baseline Simulation provides a clearer picture of the utility of the available storage as is presented in Section 8.3 below. **Figure 8.5** illustrates the effect of the increased storage on flows simulated within the L-2W Canal downstream of the G-152 structure. The existing condition flows show two periods of elevated flow in the L-2W Canal that are not shown in the proposed condition. These are time periods when the CCDD impoundment outflow gate is opened for a significant duration. **Figure 8.6** illustrates the reduction in cumulative simulated TP load at the same location.

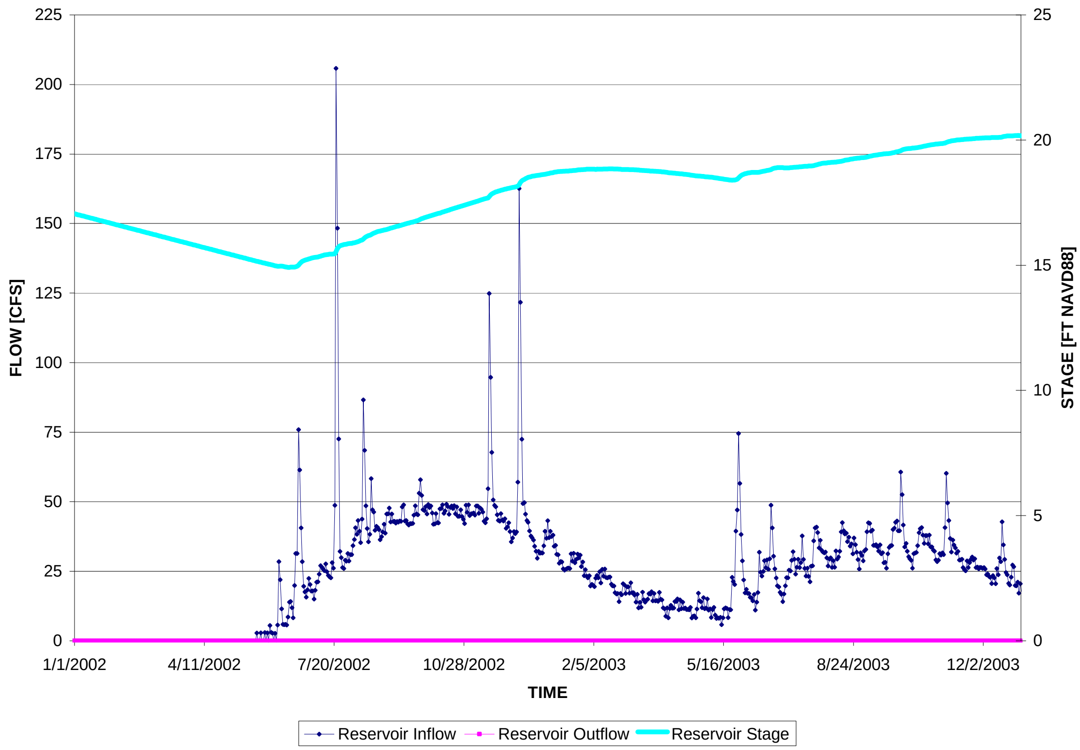

Figure 8.4: Simulated Stages and Flows for the Retrofitted CCDD

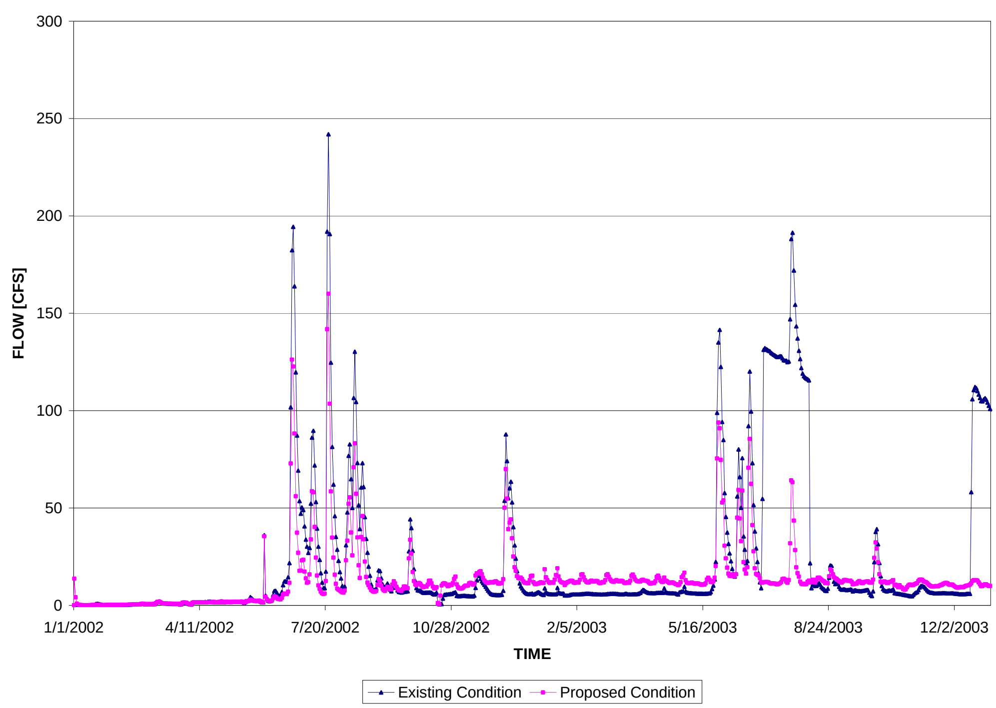

Figure 8.5: Reduction in Simulated flows in L-2W Canal

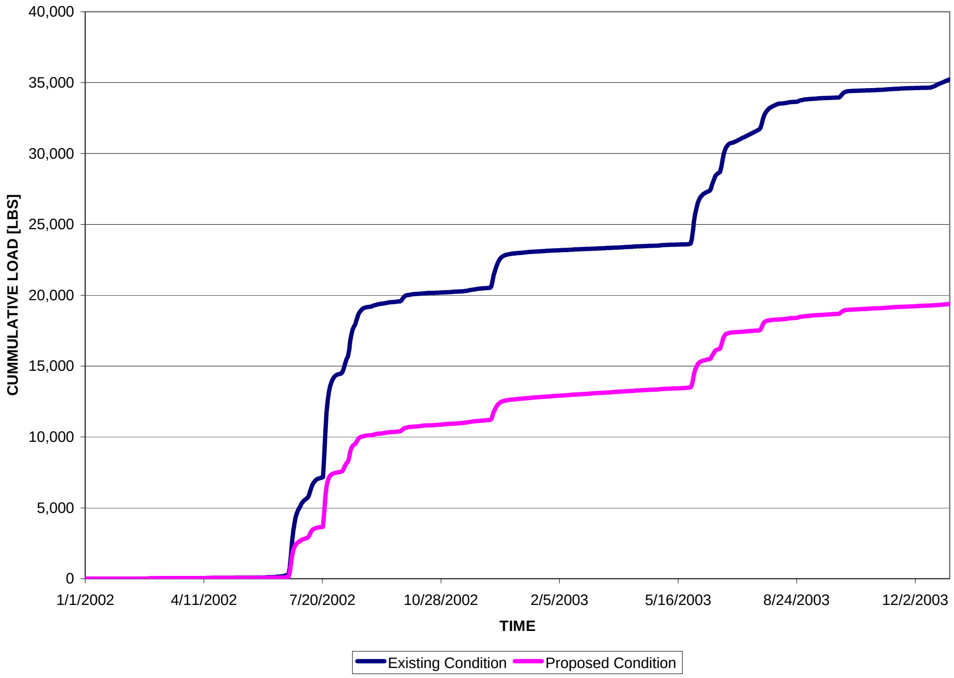

Figure 8.6: Reduction in Simulated Cumulative TP Load in L-2W Canal

8.2.2. Eastern Conceptual Impoundment

As illustrated in **Figure 8.1** there are two conceptual impoundment locations evaluated as a part of Project 3 in addition to the improved CCDD impoundment. The eastern impoundment is approximately 780 acres in size and would have proposed inflow and outflow canals connecting the impoundment to the L-2 Canal. **Figure 8.7** illustrates the configuration simulated within WAM. Reach 209 represents the impoundment. Inflow is provided by a variable-flow pump station with a capacity of 220 cfs based on the 5% exceedance probability of the flow in the L-2 Canal adjacent to the impoundment location. The pump station is controlled based on stages in the L-2 Canal. The pump station is controlled based on stages upstream in reach 57. When stages in reach 57 exceed 16.0 ft NAVD the proposed flow rate is 115 cfs, at 16.7 ft NAVD the proposed flow rate is 165 cfs and with stages exceeding 17.3 ft NAVD the proposed flow rate is 220 cfs. Similar to Project 1, the WAM representation of the proposed impoundment simulates levee seepage using a reach (215) and a weir sized to reflect a stage-discharge relationship consistent with published seepage values in the region. The outflow structure is a broad-crested weir with a crest elevation of 21.0 ft NAVD88 which is 4 feet above the average natural ground elevation within the levee.

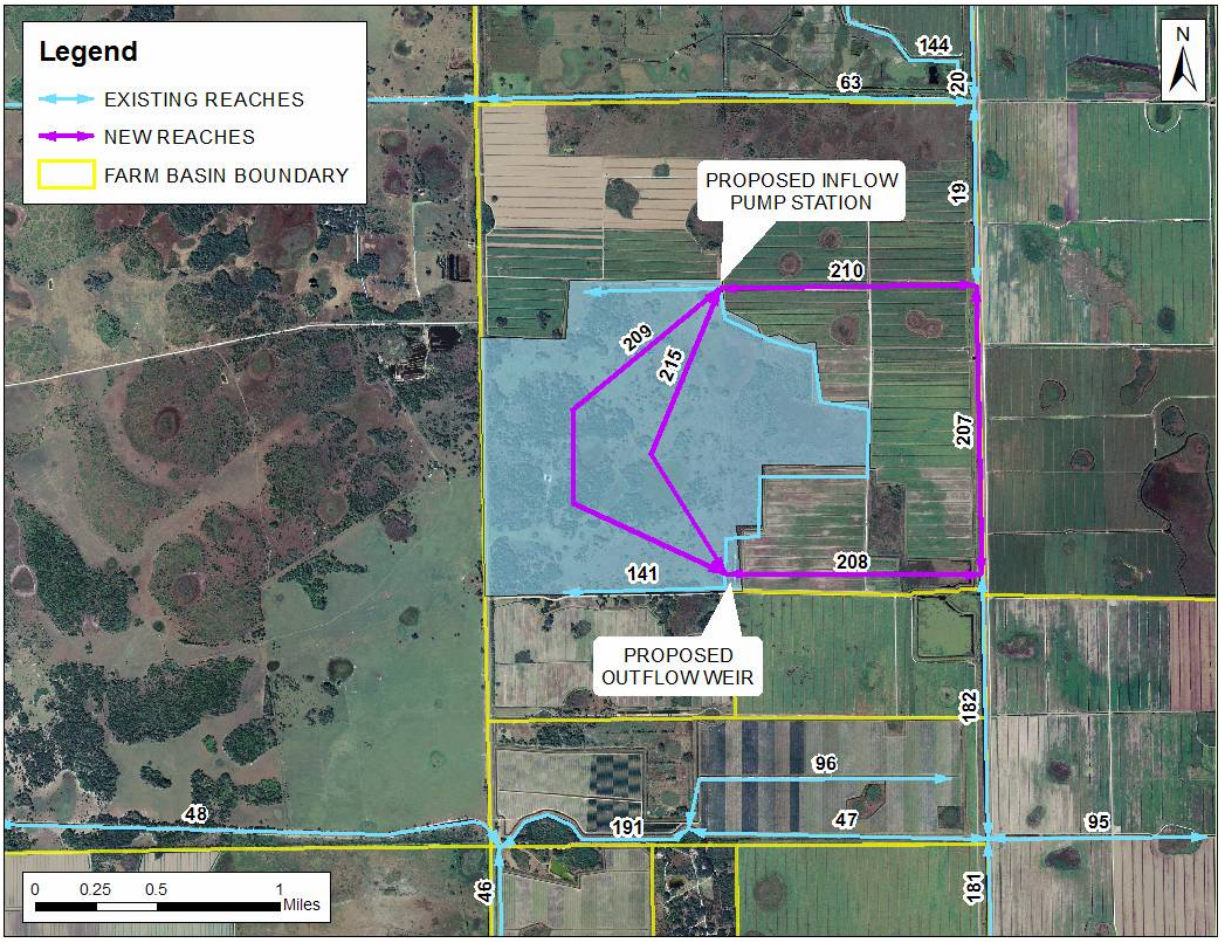

Figure 8.7: Eastern Conceptual Impoundment WAM Schematic

Figure 8.8 describes the inflow, outflow and stage for the proposed impoundment. As shown in **Figure 8.8**, the proposed impoundment is not filled to the 4 foot maximum depth during the short-term two year period simulation. However, **Figure 8.9** illustrates the effect of the increased storage on flows simulated within the L-2 Canal. This location also incorporates the effects of the CCDD impoundment modification. All of the proposed improvements of Project 3 were incorporated within the same simulation. The impoundment provides storage for the peak flows, and redistributes the peak flow volume since the only outflows are seepage. Even though the impoundment configuration is too large for 2002 and 2003, the size was not decreased and operations were not modified as part of the optimization process. There are several years during the Baseline Period with higher volumes of annual runoff than 2002 or 2003. **Figure 8.10** illustrates the reduction in cumulative simulated TP load at the same location.

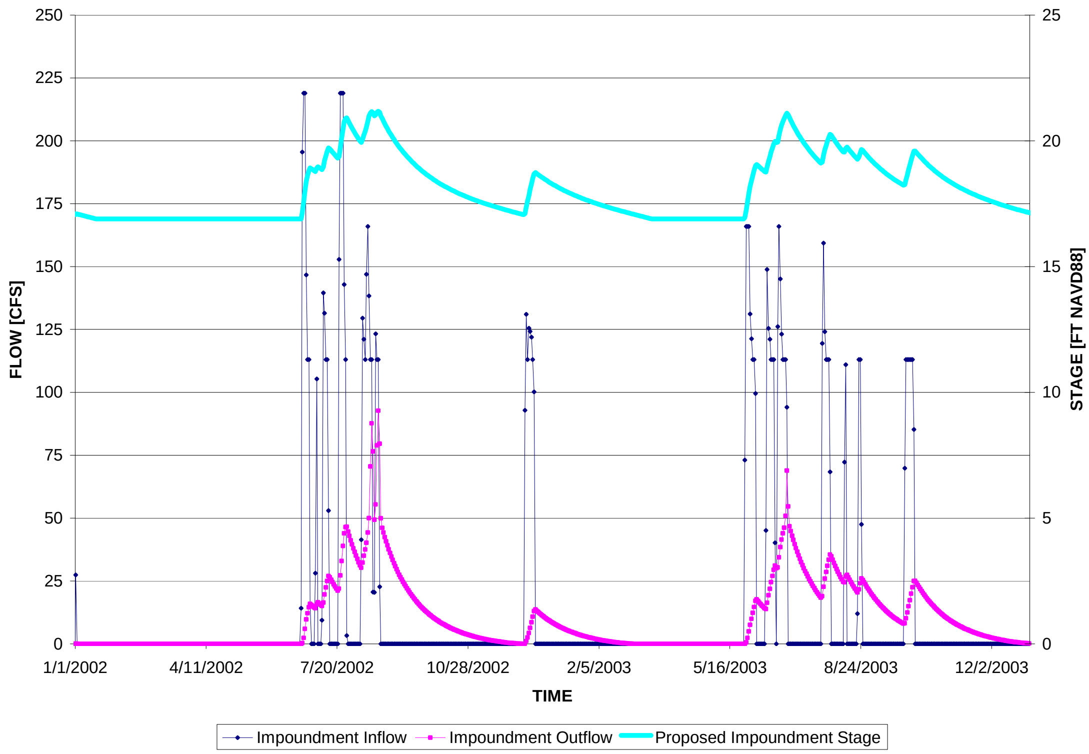

Figure 8.8: Simulated Stages and Flows for the Eastern Conceptual Impoundment

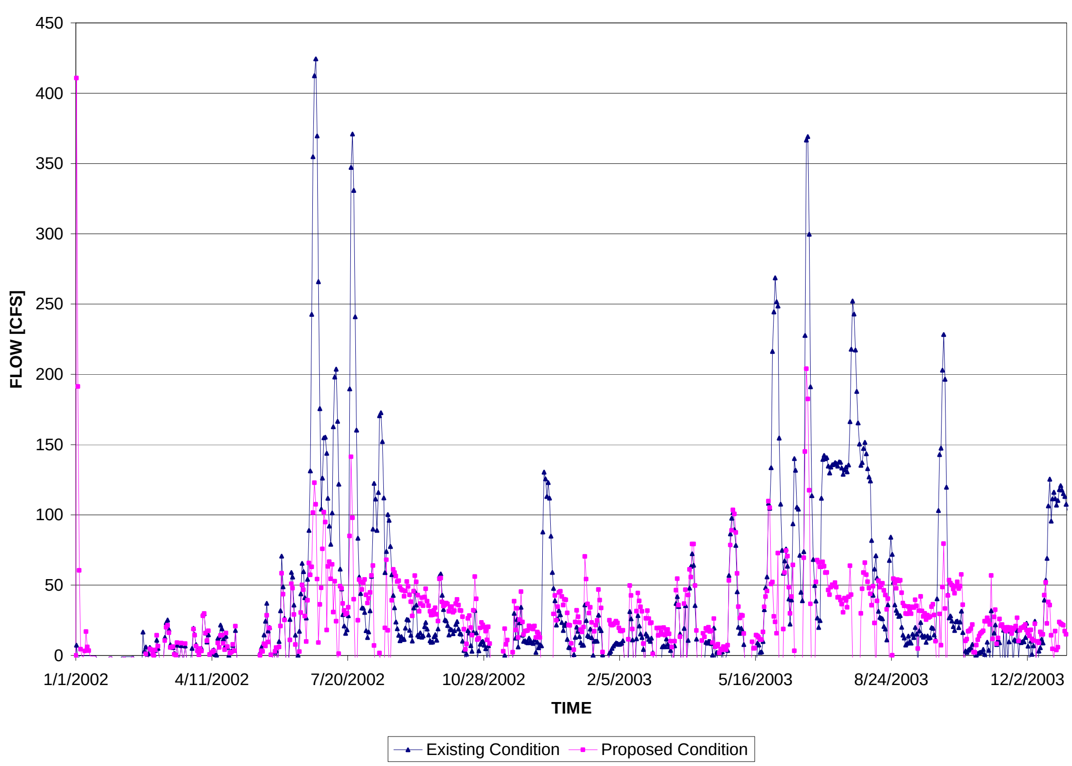

Figure 8.9: Reduction in Simulated flows in L-2 Canal

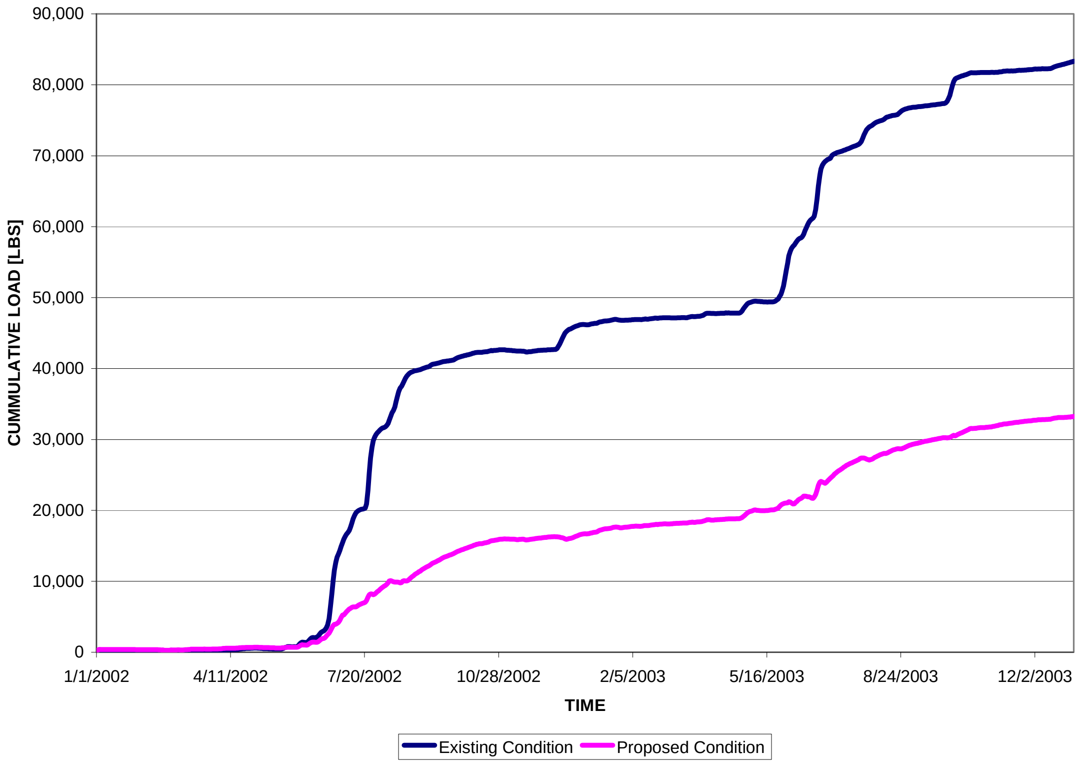

Figure 8.10: Reduction in Simulated Cumulative TP Load in L-2 Canal

8.2.3. Southern Conceptual Impoundment

As illustrated in **Figure 8.1** there are two conceptual impoundment locations evaluated as a part of Project 3 in addition to the improved CCDD impoundment. In Deliverable 10.1, the southern impoundment is shown as approximately 1,189 acres, however during the process of optimization it was determined that the initial size estimate was too large since it was never filled during the short-term simulation. Subsequently, the size was revised to approximately 656 acres. The proposed depth for the southern impoundment is 4.0 feet similar to the other proposed impoundments, therefore the proposed capacity is 2,624 acre-feet. The southern impoundment **Figure 8.11** illustrates the configuration simulated within WAM. Reaches 211 and 214 represent the impoundment. Inflow is provided by two variable-flow pump stations with a total capacity of 230 cfs. The north pump station is controlled based on stages upstream in reach 57. When stages in reach 57 exceed 16.0 ft NAVD the proposed flow rate is 50 cfs, at 16.7 ft NAVD the proposed flow rate is 175 cfs and with stages exceeding 17.3 ft NAVD the proposed flow rate is 210 cfs. The south pump station is controlled based on stages upstream of the pump in reach 213. When stages in reach 213 exceed 18.0 ft NAVD, the proposed flow rate is 10 cfs, at 18.7 ft NAVD the proposed flow rate is 15 cfs and with stages exceeding 21.0 ft NAVD the proposed flow rate is 20 cfs. Similar to Project 1, the WAM representation of the proposed impoundment simulates levee seepage using a reach (216) and a weir sized at 0.42 ft wide to reflect a stage-discharge relationship consistent with published seepage values in the region. The outflow structure is a broad-crested weir with a crest elevation of 28.5 ft NAVD which is 4 feet above the average natural ground elevation within the levee.

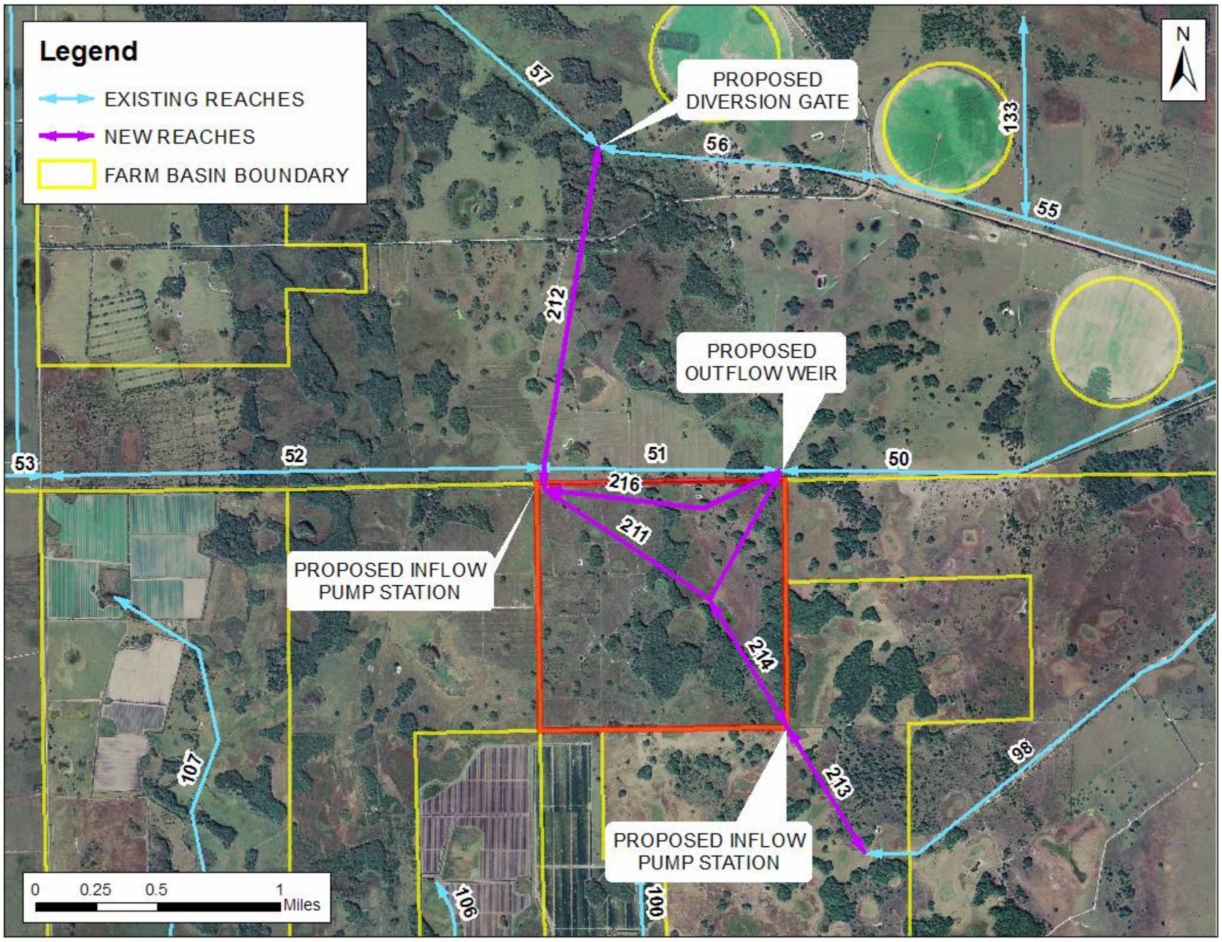

Figure 8.11: Southern Conceptual Impoundment WAM Schematic

Figure 8.12 describes the inflow, outflow and stage for the proposed impoundment. Similar to the eastern conceptual impoundment, the southern conceptual impoundment is rarely filled to capacity during the two-year, short-term simulation. As described above the original conceptualization of Project 3 included a significantly larger southern impoundment, however initial simulations showed that the impoundment was oversized when compared with the contributing area. **Figure 8.13** illustrates the effect of the increased storage on flows simulated within the ALICO South Boundary Canal at a location represented within the model directly downstream of reaches 55 and 50. The proposed impoundment provides storage for all of the peak volumes. **Figure 8.14** illustrates the reduction in cumulative simulated TP load in the ALICO South Boundary Canal. This location is unaffected by the implementation of other projects within the Project 3 simulation.

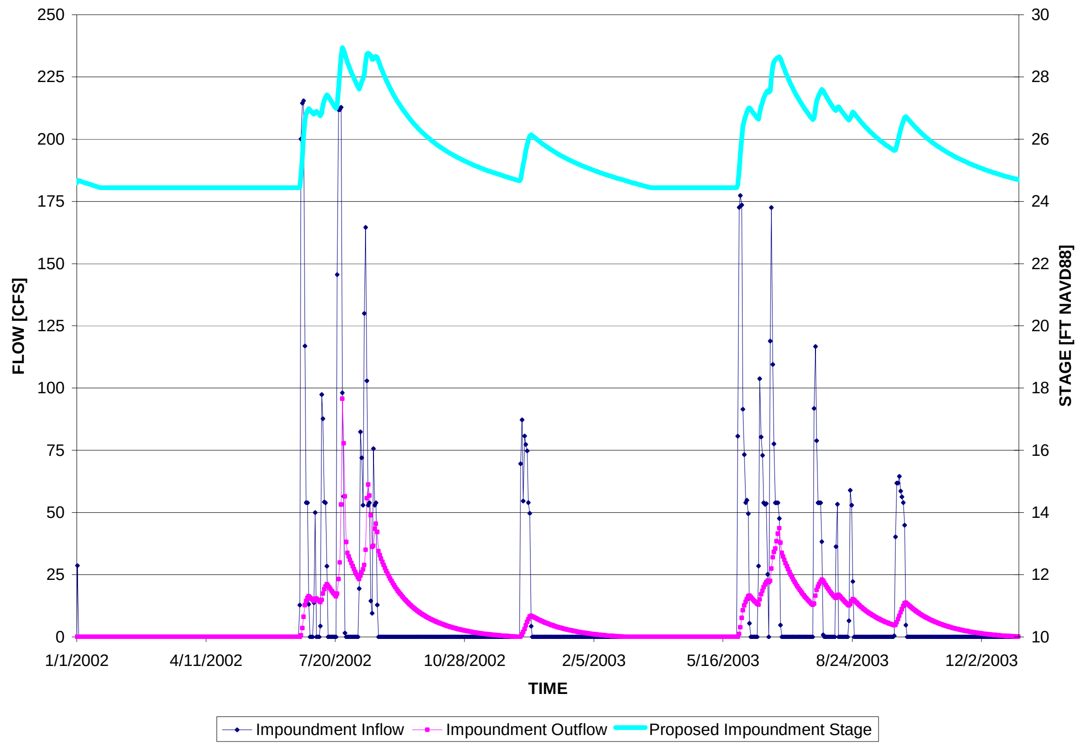

Figure 8.12: Simulated Stages and Flows for the Southern Conceptual Impoundment

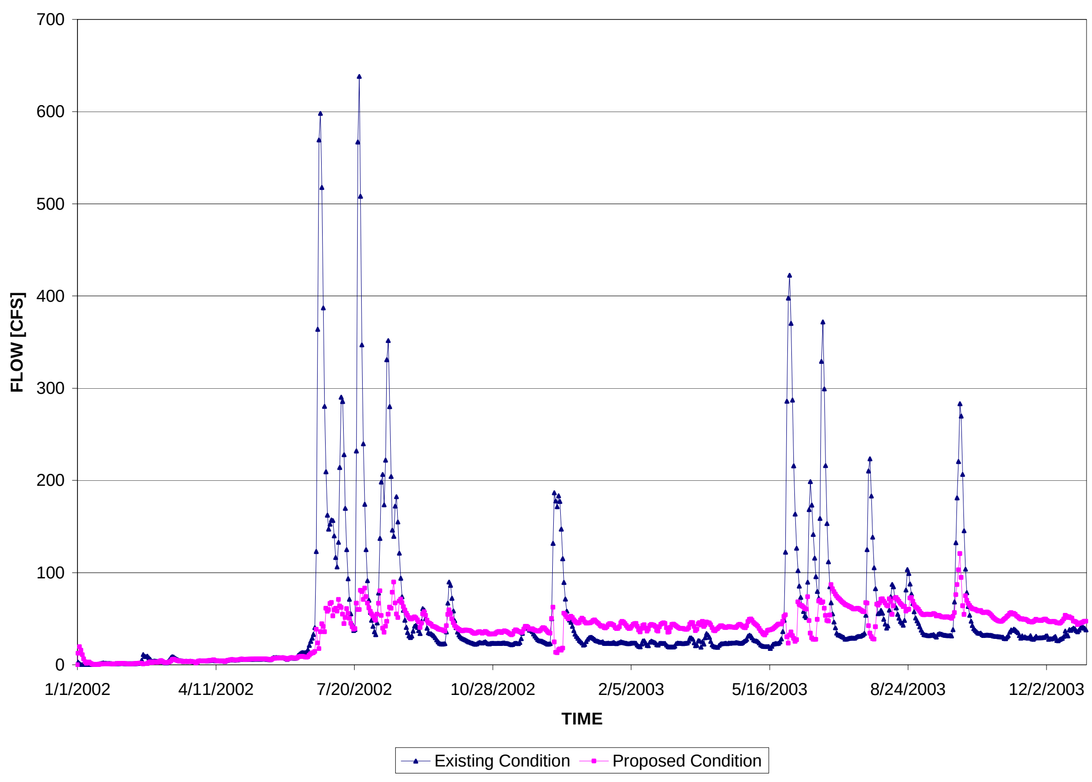

Figure 8.13: Reduction in Simulated flows in ALICO South Boundary Canal

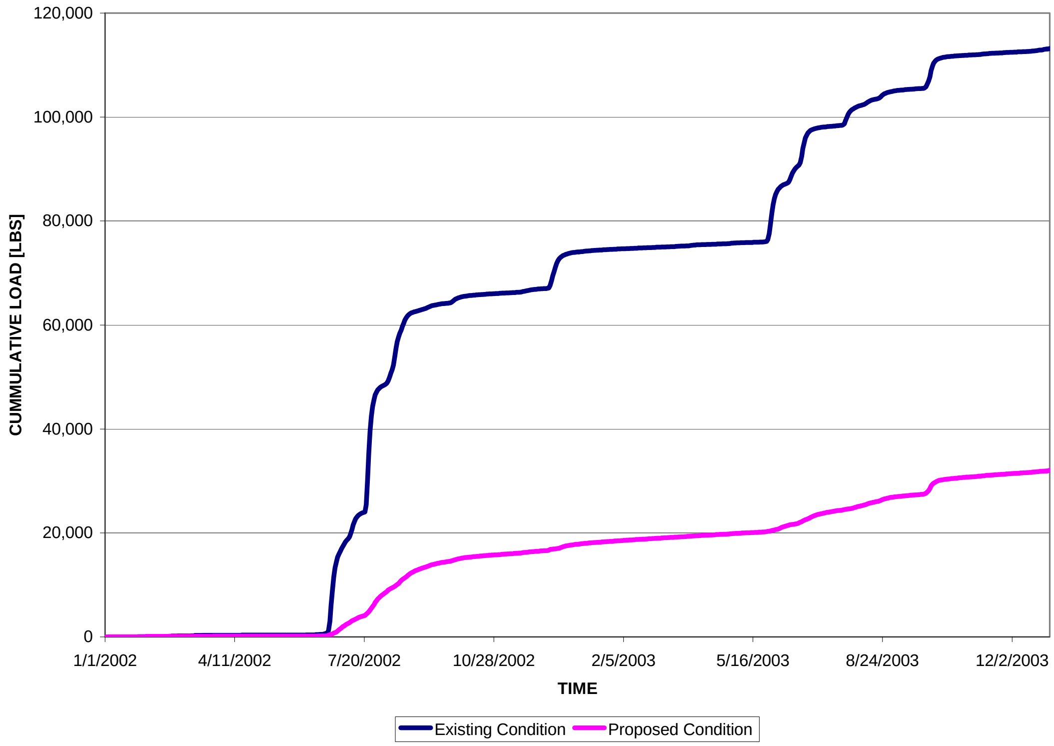

Figure 8.14: Reduced Cumulative TP Load in ALICO South Boundary Canal

8.3 Project 3 Short-Term Simulation Summary

The District recommendations for Project 3 describe a reduction in TP loads from the C-139 Basin by creating centrally located impoundments to provide storage and additional treatment. There were no other scenarios described in the recommendation. As part of the efforts of the ADA Team, there were several scenarios developed for Project 3 in order to determine the optimum infrastructure for the proposed impoundments. These alternate scenarios are not presented in this document since they were not alternatives of Project 3 but were less optimal configurations of the proposed project.

In Section 8.2 above, the three individual improvements with differing effects on the downstream reaches are presented. Since Project 3 incorporates all three improvements in one simulation the total impact can be measured at a location downstream of all three impoundments, the L-3 Canal. **Figures 8.15** and **8.16** illustrate the reduced flows and loads in the L-3 Canal for the 2002 and 2003 period. **Table 8.1** below compares the 2002 and 2003 reduction in TP load simulated at each individual location that is downstream of the proposed improvement and in the L-3 Canal.

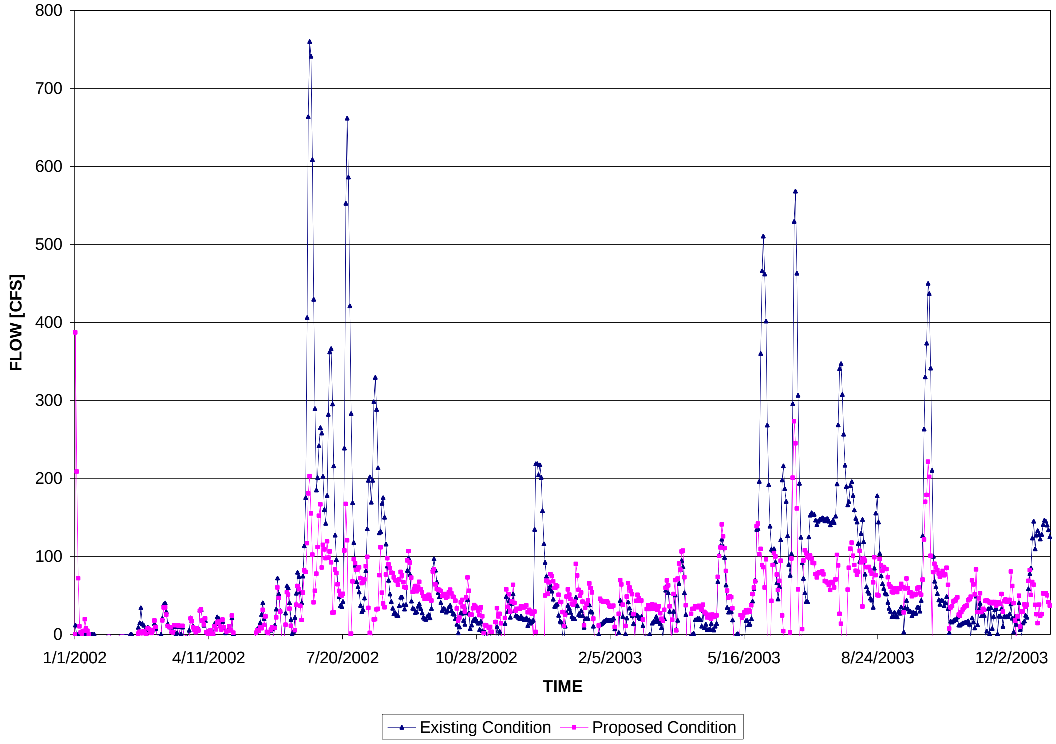

Figure 8.15: Reduction in Simulated Flows in the L-3 Canal

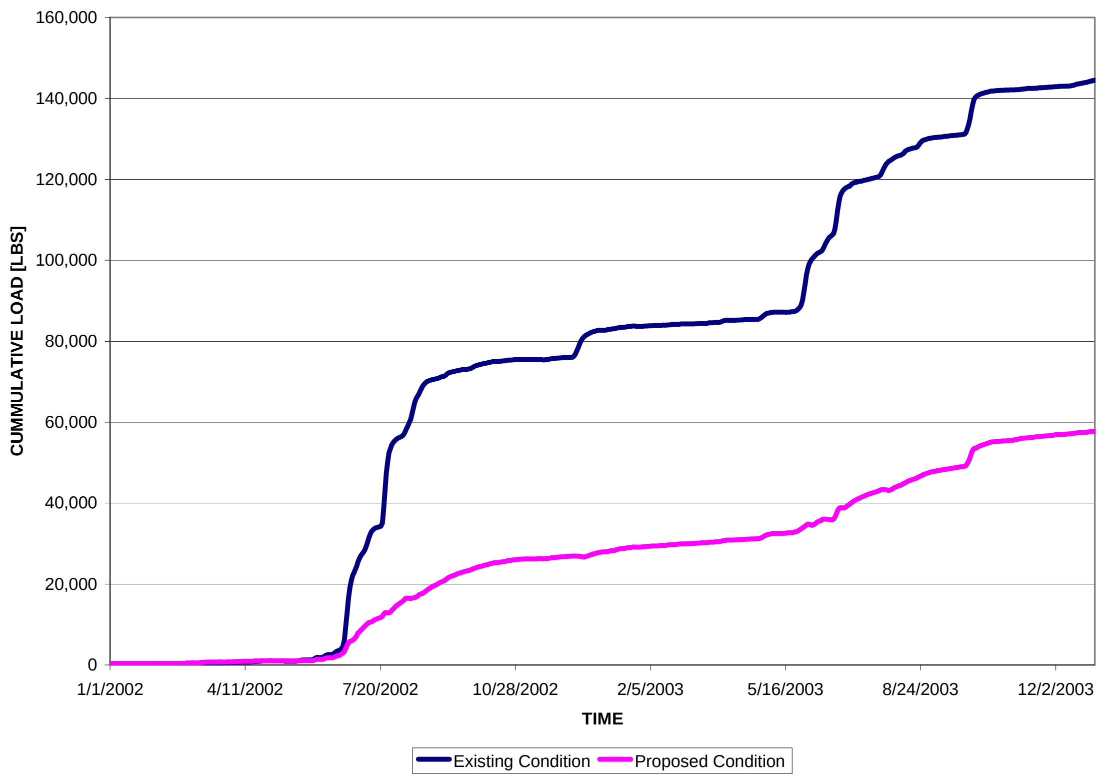

Figure 8.16: Reduced Cumulative TP Load in L-3 Canal

Table 8.1: Project 3 Annual TP Reduction by Component

PROJECT IMPACTS MEASUREMENT LOCATION	ANNUAL TP REDUCTION [%]			AVERAGE REDUCTION [LB]
	2002	2003	AVERAGE	
Project 3 (L-2W Canal)	45%	45%	45%	7,932
Project 3 (L-2 Canal)	64%	56%	60%	25,048
Project 3 (ALICO South Boundary Canal)	76%	64%	70%	40,588
Project 3 (L-2 / L-3 Canal)	66%	52%	59%	43,363

A comparison of the 43,363 pound average annual TP reduction for the selected scenario with the annual average TP load from the C-139 Basin described in **Table 2.1** illustrates that the removal of that quantity would account for a 23% reduction in the simulated average annual TP load for the entire basin.

8.4 Project 3 Baseline Period Evaluation

As described above Project 3 consists of the improvement of one existing impoundment and the construction of two new impoundments. The proposed improvement to the existing CCDD impoundment would double the existing storage. The total proposed increase in basin-wide storage for Project 3 is 10,654 ac-ft. A Baseline Period simulation was performed for this configuration to provide an appropriate assessment of the performance of Project 3 over a wide range of hydrologic conditions. **Figure 8.17** illustrates the effects of the Project 3 scenario on basin-wide TP loads over the duration of the Baseline Period. **Figure 8.18** illustrates the cumulative effect on TP loads over the Baseline Period. **Table 8.2** describes the performance of Project 3 with respect to the existing condition and the targeted TP load.

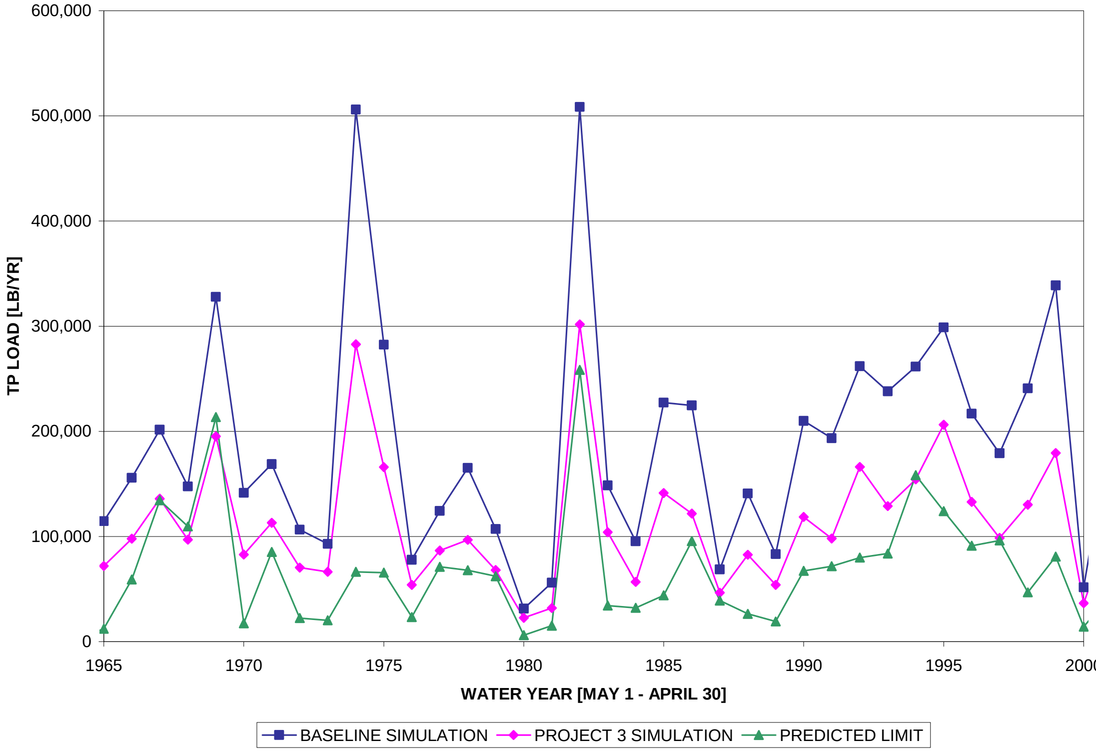

Figure 8.17: Annual TP Load for Project 3 and Existing Condition

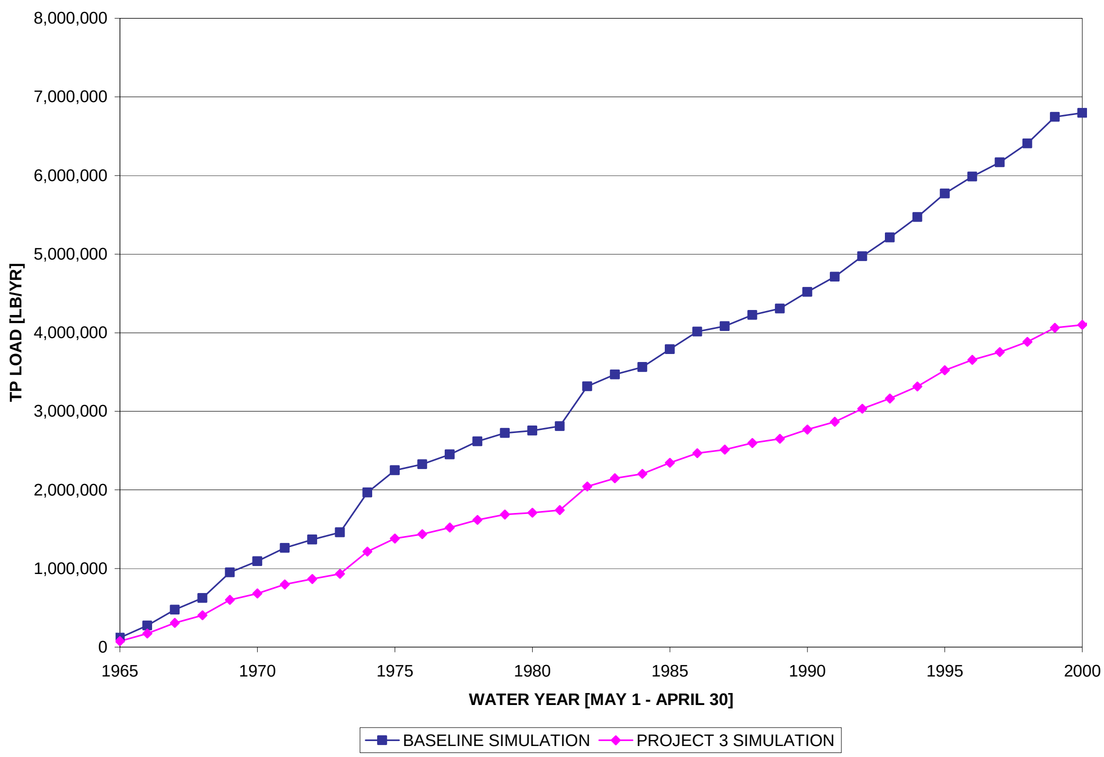

Figure 8.18: Cumulative Effects on TP Load for the Baseline Period

Table 8.2: Comparison of Project 3, Existing and Targeted Annual TP

CONDITION	MIN ANNUAL TP [LB]	MAX ANNUAL TP [LB]	AVG ANNUAL TP [LB]	TOTAL BASELINE PERIOD TP [LB]
PROJECT 3	22,535	301,590	113,792	4,096,502
EXISTING CONDITION	31,421	508,392	188,715	6,793,743
TARGETED LOAD	6,113	258,419	69,852	2,514,680

The improvement in water quality described for Project 3 implementation assumes that TP attenuation would occur within each impoundment consistent with the type of attenuation seen within a slough. This assumption is based on the best professional judgment of the ADA Team for an impoundment that would have wetland vegetation prevalent in habitats that are inundated with shallow waters for portions of the year. Additionally, this project assumes a seepage rate that varies solely on the depth of water in each impoundment. If seepage is enhanced by local or regional drawdown of the surficial aquifer due to water supply pumping, than the seepage assumptions could be inaccurate.

Although the proposed improvement project does provide significant TP load reductions with respect to the existing condition, Project 3 would not provide the improvements required to reach the rainfall-adjusted target TP load as described within the SFER. In order to meet the target TP load requirements, Project 3 would need to be coupled with other local and regional improvement projects.

9.0 PLANNING-LEVEL COST ESTIMATE

9.1 General

The purpose of this section is to document the planning-level cost estimate for each of the regional proposed projects outlined in Section 3.1. The objective of these regional proposed projects is to evaluate identified opportunities for water quality improvements. The planning-level cost estimates were prepared using guidelines outlined in the South Florida Water Management District's Design Criteria Memorandum (DCM-7) dated February 2006 and prior planning studies completed in the vicinity of the C-139 Basin. DCM-7 provides guidance and the required estimate submittal details to be used by design consultants for the development and updating of Opinion of Probable Construction Costs (a.k.a. Engineering Construction Cost Estimates) for Acceler8 Projects. The goal of this guidance document is to provide uniform, consistent procedures for engineering construction cost estimate preparation.

The following sub-sections define the approach implemented in determining the planning-level cost estimate for each regional proposed project. **Appendix A** provides a more detailed description of the components of the cost estimate, as defined by DCM-7.

9.2 Assumptions

Recent Hendry County Property Appraiser data shows that in May 2006, agricultural land within the C-139 Basin was purchased at a price of approximately $3,500 an acre. All land acquisition costs were assumed to have a unit cost of $5,000 per acre, to provide a level of contingency due to escalating land costs. Costs for labor and materials were based on unit pricing data used in the development of the EAA Regional Feasibility Study (EAA RFS). Since the EAA RFS was completed in 2005, the unit prices were converted to present day values assuming a 5% inflation rate for 3 years.

Overhead was calculated by itemizing the components identified in DCM-7. The individual components of the overhead cost used were 3% for jobsite supervision, 2% for small tools and supplies, and 1% for contractor's quality control. The total itemized overhead cost is 6%. A mobilization cost of 12% was assumed for all proposed projects. These percentages were applied to the subtotal of direct costs for each proposed project.

After the above costs were incorporated into the second subtotal, the following percentages were applied to the second subtotal: 1.5% for contingency, 10% for contractor's profit, and 30% contingency which is consistent with a Class 4 Basis of Design Report (BODR) Estimate. Sales tax was assumed to be incorporated into the cost-per-unit-flow cost for pump stations and to no portion of the canal widening estimate, because there were no material and equipment costs to be accounted.

9.3 Project 1

This project proposes to reduce TP loads in the southeastern corner of the C-139 Basin by implementing identified water quality improvements that provide treatment and storage to the Deer Fence and S&M Canals. The water quality treatment is achieved through a proposed 520-acre above-ground impoundment located on District-owned property called the O'Bern's pasture. The significant impoundment features proposed include a 5.12 mile perimeter levee, an inflow pump structure (with a capacity of 140 cfs) and an outfall weir structure with a crest elevation that is 4 feet above the proposed impoundment base. The outfall weir discharges to the L-3 Canal. Along the eastern end of the S&M Canal there is an existing sheet pile weir to be retrofit and a proposed gate in order to allow the stages within the S&M Canal to be sufficient for inflow into the proposed impoundment and to prevent inflow from the L-3 Canal.

Other recommended surface water management infrastructure is also proposed for Project 1 to optimize the operation and flexibility of this project's water quality improvements. One of the proposed improvements includes a proposed gated culvert structure to connect the Deer Fence and S&M Canals. In order to facilitate this connection, a new gated structure is also proposed downstream of the interconnection, at the confluence of the Deer Fence and L-3 Canals. **Figure 9.1** illustrates the locations and specifications associated with these improvements.

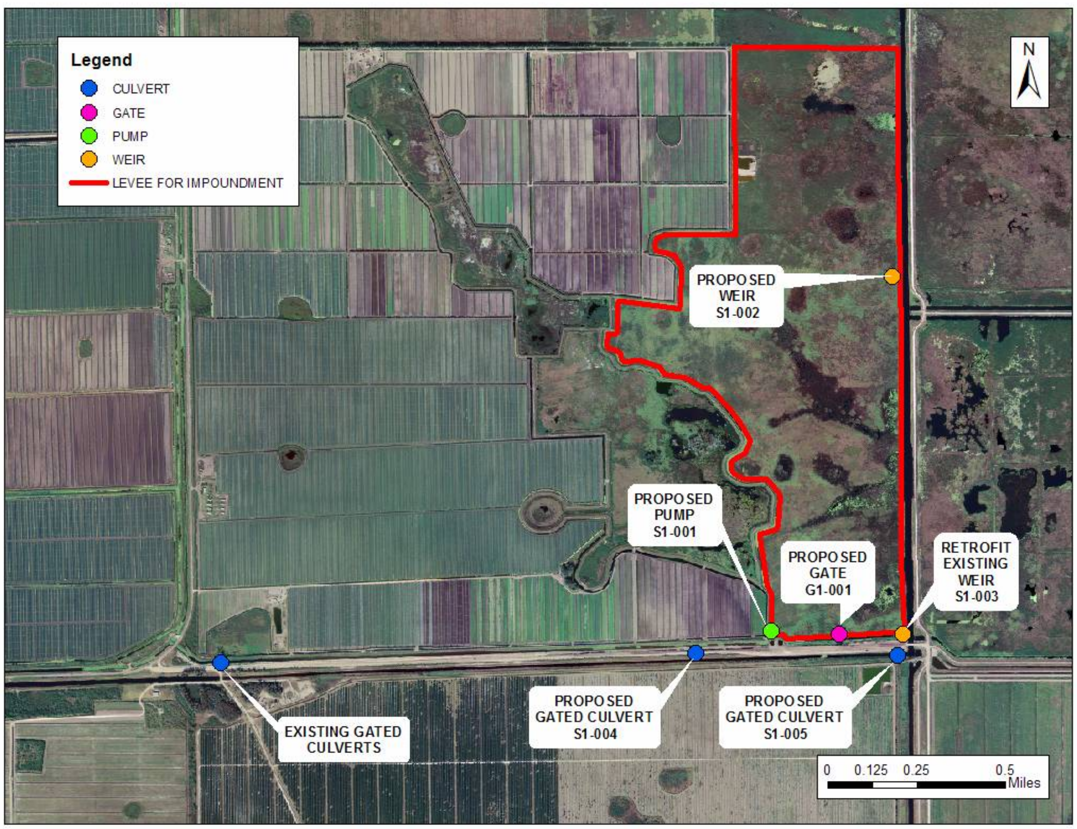

Figure 9.1: Project 1 Configuration

The total proposed planning-level cost associated with Project 1 is $6,391,000. All details describing associated water control structures, earthwork and mobilization can be found in the detailed cost estimate in **Appendix A**.

9.4 Project 2

This project proposes to reduce TP loads in the DIRWMA by proposing to change the existing land use and convert it back to its native condition. This project is based on the assumption of halting all agricultural operations in DIRWMA and proposes to change the existing agricultural land uses from 18,340 acres of pasture, 2,020 acres of sugarcane, 570 acres of vegetable and 815 acres of citrus to a native land use, scrub and brushland. The conversion to native land-use will require the removal of citrus trees, sugarcane, vegetable plants, canals and surface water management features. Some additional clearing and grubbing will be required for removal of furrows. It is also proposed that approximately 18.3 miles of existing canal system be plugged or filled in such a way as to prevent future drainage or flow short-circuiting. Canal filling will be accomplished by degrading adjacent canal spoil areas or accesses roads and using the available fill from these sources to fill the canal. If adequate spoil material is not available to fill the canals, a 10-foot plug will be constructed in the canals at a minimum of 2,500 foot spacing. **Figure 9.2** illustrates the locations and specifications associated with these improvements.

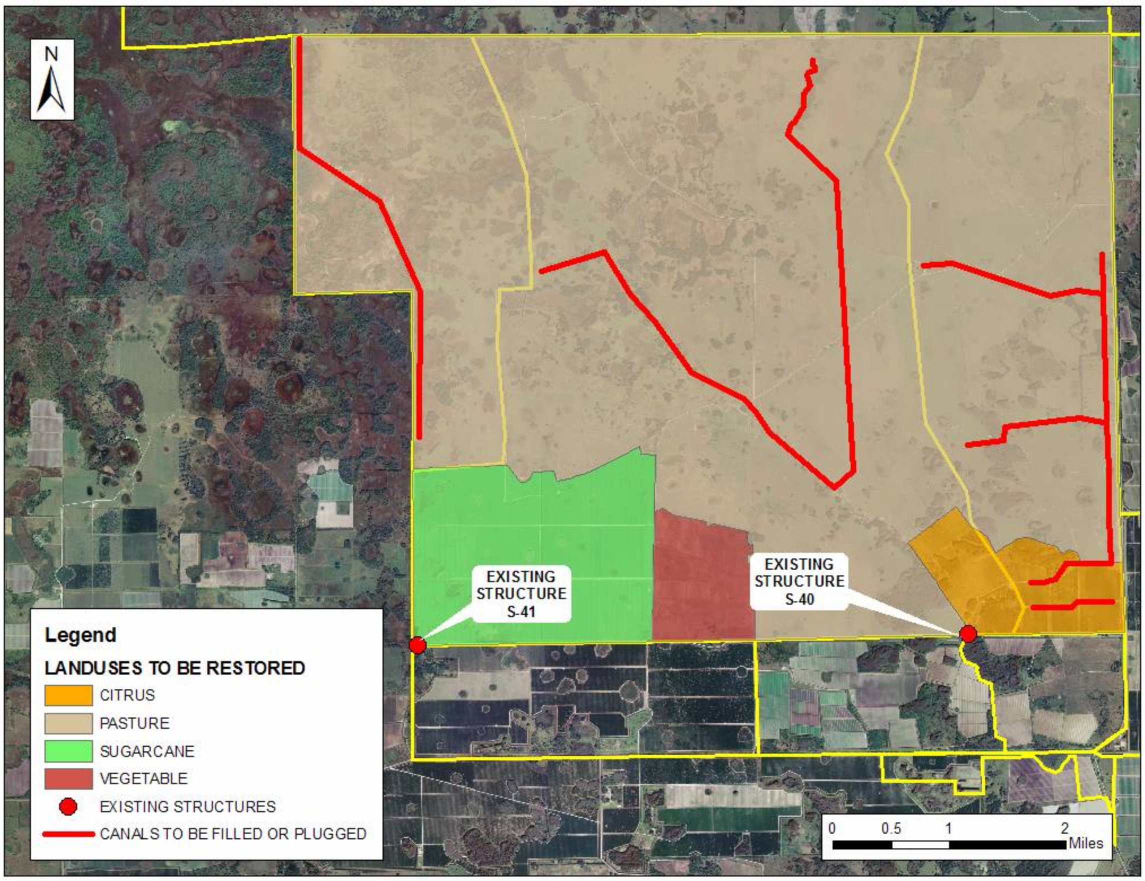

Figure 9.2: Project 2 Configuration

The total proposed planning-level cost associated with Project 2 is $7,539,000. All details describing associated landscape restoration can be found in the detailed cost estimate in **Appendix A**.

The planning-level costs described above include the restoration of the entire DIRWMA. Even though a portion of these efforts are currently underway as part of the FFWCC's efforts to restore DIRWMA. The FFWCC is currently contracting with the engineering consulting firm (Engineering & Applied Science, Inc.) to restore all 2,020 acres of sugarcane and a 2,300 acre portion of the pasture. The long term plan for the FFWCC is to restore all of the agricultural uses within the DIRWMA to provide habitat for the Florida panther. However the timelines for these efforts is unclear, given that the current cattle grazing use agreement is due to expire on April 9, 2009 and upon mutual agreement between the FFWCC and the lessees, a one-time, five-year renewal may be granted. Therefore the planning-level costs described in **Appendix A** would apply if the District's project evaluation deems this regional project a priority such that the FFWCC's timeline for restoration must be accelerated.

9.5 Project 2a

This project follows the assumptions and modifications proposed in Project 2 (the restoration of land use from pasture, sugarcane, vegetable and citrus to native scrub and brushland, clearing and grubbing, and filling or plugging canals). Additionally, Project 2a considers the diversion of agricultural runoff from the adjacent farms located to the east through a longer flow path for treatment. The longer flow path, through proposed new and widened canals, provides additional treatment during peak events. In order for the agricultural runoff from the eastern farmlands to be delivered westward, four new gated structures and a pump station are proposed to facilitate conveyance. **Figure 9.3** illustrates the locations and specifications associated with these improvements.

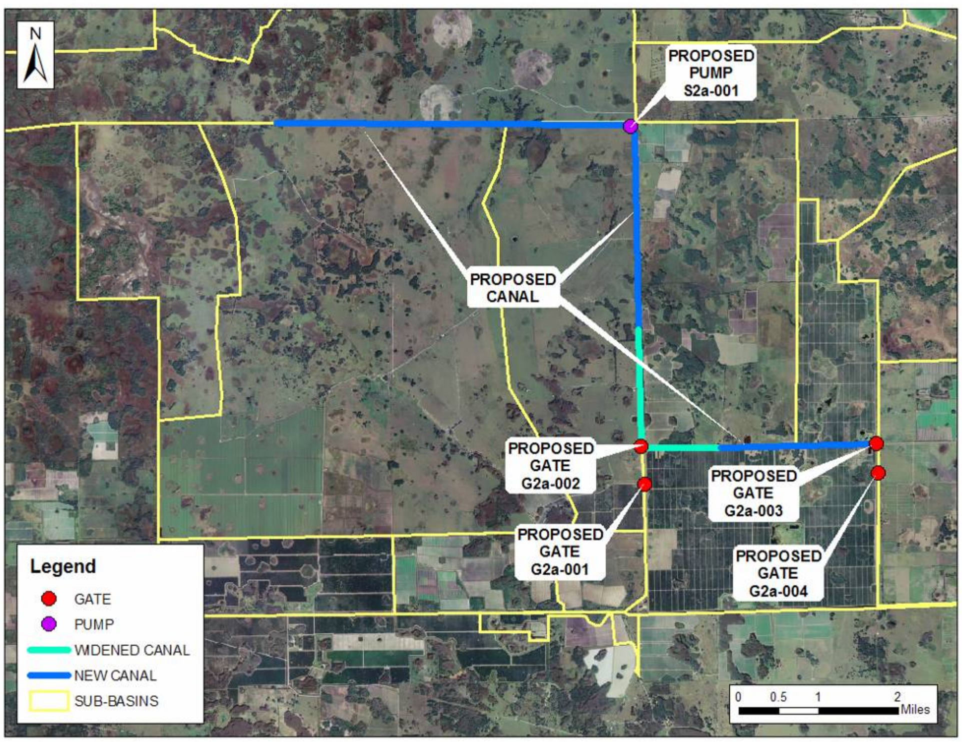

Figure 9.3: Project 2a Configuration

The total proposed planning-level cost associated with Project 2A is $20,844,000. All details describing associated landscape restoration can be found in the detailed cost estimate in Appendix A. As described above, in order for the benefits of Project 2a to be realized, Project 2 must be completed. Therefore the total implementation cost of Project 2a will include the $7,539,000 cost of Project 2, making the final cost $28,383,000.

9.6 Project 2b

This project follows the assumptions and modifications proposed in Project 2 (the restoration of land use from pasture, sugarcane, vegetable and citrus to native scrub and brushland, clearing and grubbing, and filling of canals). Similarly to Project 2a, Project 2b also considers the diversion of agricultural runoff from the adjacent farms located to the north to restore the historic drainage patterns and interconnect the surrounding areas. In order for the northern farmlands to convey the agricultural runoff southward, six new gated structures are proposed to facilitate conveyance. **Figure 9.4** illustrates the locations and specifications associated with these improvements.

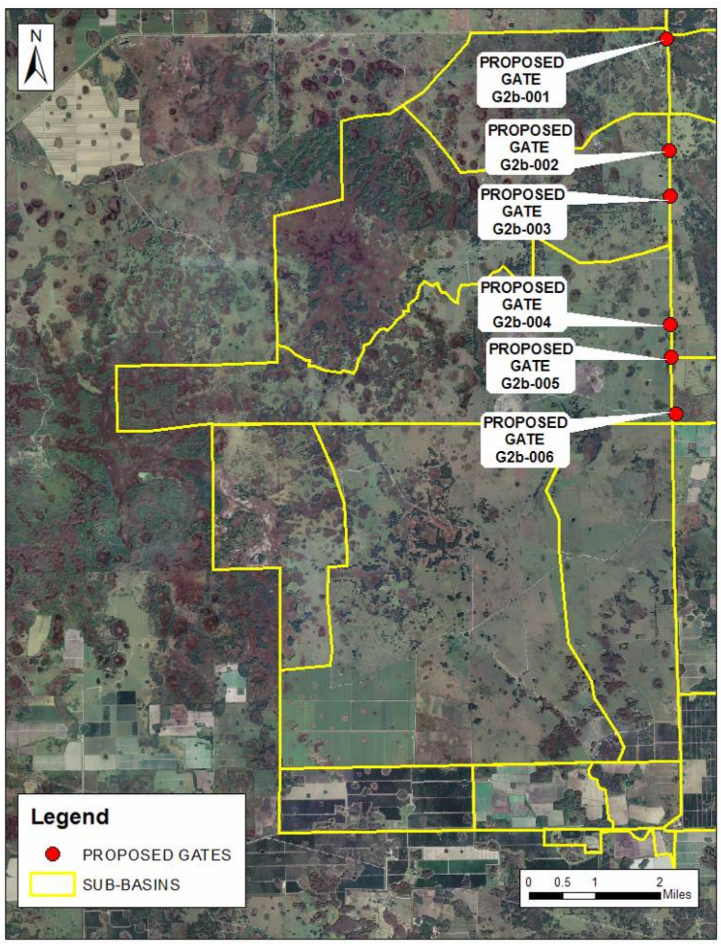

Figure 9.4: Project 2b Configuration

The total planning-level cost associated with Project 2b is $1,198,000. All details describing associated landscape restoration can be found in the detailed cost estimate in Appendix A. As described above, in order for the benefits of Project 2b to be realized, Project 2 must be completed. Therefore the total implementation cost of Project 2b will include the $7,539,000 cost of Project 2, making the final cost $8,737,000.

9.7 Project 3

The implementation of Project 3 as presented in Section 8.0 above requires the construction of three significant components: the west levee of the CCDD impoundment, the eastern conceptual impoundment and the southern conceptual impoundment. The following sub-sections describe the costs associated with the construction of all of these components.

9.7.1. CCDD Impoundment

Project 3 proposes to reduce TP loads in the C-139 Basin by implementing three separate improvements. The first is the rehabilitation of the existing CCDD impoundment to provide additional storage and treatment. The centrally located reservoir proposes increased storage in the existing 2,455-acre above-ground by reconstructing and improving 4.8 miles of perimeter levee on the western bank. For the purposes of this cost estimate the rehabilitated levee is assumed to be constructed to match the eastern levee. **Figure 9.5** illustrates the cross-section of the eastern levee. This cost estimate assumes that the five existing inflow pumps owned and operated by the CCDD are sufficient to continue providing flood protection to the Montura Ranches subdivision and acting as the only inflow to the impoundment. The reconstructed berm will allow for additional storage up to 24.6 feet NAVD88. This matches the operational conditions described in the existing permit. Therefore for the purposes of this cost estimate, it is assumed that the existing outfall structure has sufficient capacity. **Figure 9.6** illustrates the extent of the reconstructed impoundment levee.

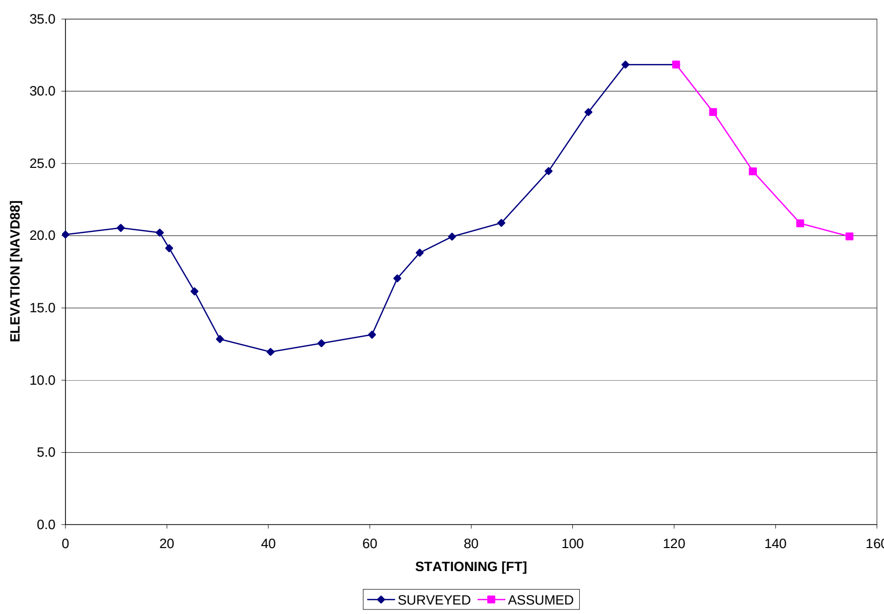

Figure 9.5: Project 3 CCDD Impoundment Rehabilitation

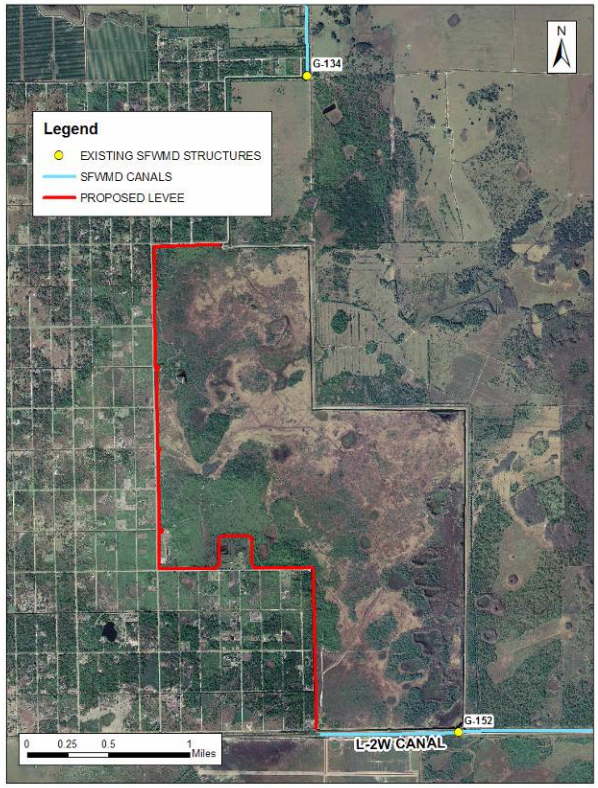

Figure 9.6: Project 3 CCDD Impoundment Rehabilitation

9.7.2. Eastern Conceptual Impoundment

Another individual component of Project 3 proposes to reduce TP loads in the C-139 Basin by constructing an impoundment adjacent to the L-2 Canal to provide storage and treatment. The centrally located reservoir proposes water quality treatment through a 967-acre above-ground impoundment. As described in Section 8.1 above, the proposed location for this impoundment is conceptual and does not reflect a location where a proposed impoundment will be constructed.

In order to construct this conceptual impoundment simulated within the Project 3 evaluation, 967 acres of private land would need to be acquired. The significant impoundment features proposed include a 5.7 mile perimeter levee, an inflow pump structure (with a capacity of 220 cfs) and an outfall weir structure with a crest elevation that is 4 feet above the proposed impoundment base at 21.0 feet NAVD88. For consistency it was assumed that the cross-section of the proposed 5.7 mile perimeter levee matches the CCDD levee cross-section shown in **Figure 9.5**, adjusted for regional changes in natural ground elevation. Based on the conceptual location presented herein, the inflow pump station and the outfall weir are connected to the L-2 Canal via two proposed canals with a total length of approximately 10,700 feet. **Figure 9.7** illustrates the location of these water quality improvements.

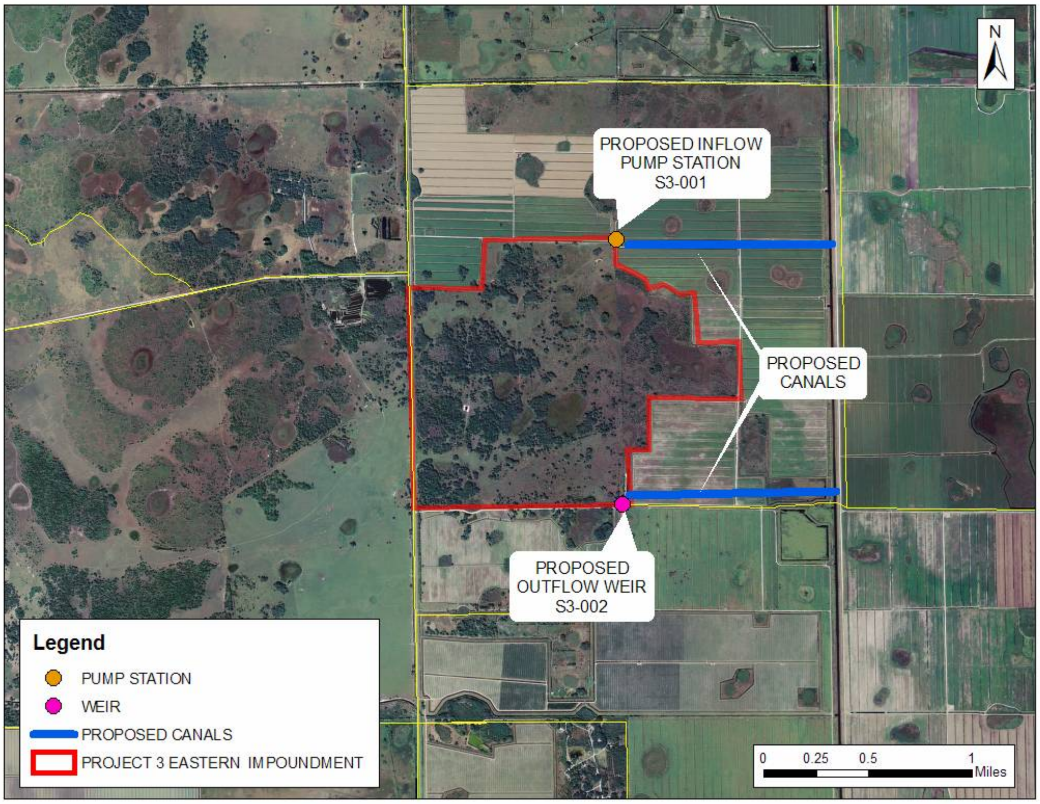

Figure 9.7: Project 3 East Impoundment Configuration

9.7.3. Southern Conceptual Impoundment

This project proposes to reduce TP loads in the C-139 Basin by constructing an impoundment adjacent to the ALICO South Boundary Canal to provide storage and treatment. The centrally located reservoir proposes water quality treatment through a 656-acre above-ground impoundment. As described in Section 8.1 above, the proposed location for this impoundment is conceptual and does not reflect a location where a proposed impoundment will be constructed.

In order to construct this conceptual impoundment simulated within the Project 3 evaluation, 656 acres of private land would need to be acquired. The significant impoundment features proposed include a 4.1 mile perimeter levee, two inflow pump structures (with a capacity of 210 and 20 cfs), two inflow canals (with a length of 10,820 feet) and an outfall weir structure with a crest elevation that is 4 feet above the proposed impoundment base or 28.5 feet NAVD88. For consistency it was assumed that the cross-section of the proposed 4.1 mile perimeter levee matches the CCDD levee cross-section shown in **Figure 9.5**, adjusted for regional changes in natural ground elevation. In order to divert water to the inflow pumps during high runoff events, two proposed diversion gates will need to be constructed on the Devil's Garden Canal to the north and at the eastern edge of Basin 26-308-01. The outfall weir discharges to the ALICO South Boundary Canal. **Figure 9.8** illustrates the location of these water quality improvements.

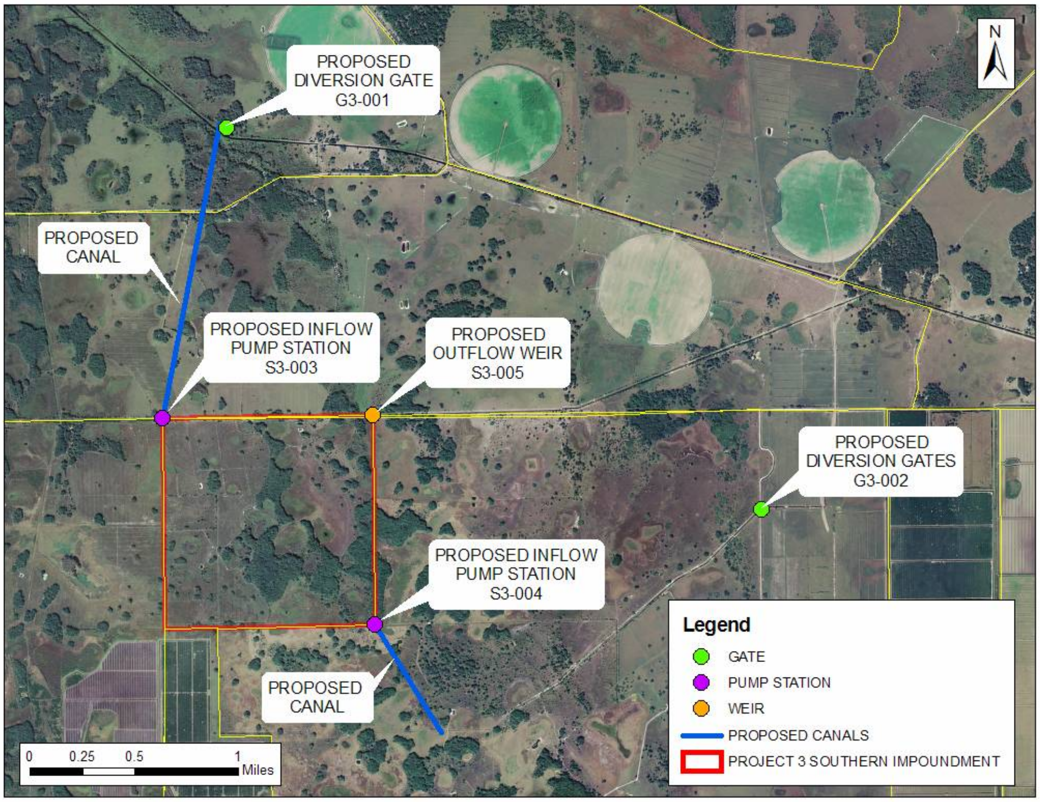

Figure 9.8: Project 3 South Impoundment Configuration

The total proposed planning-level cost associated with Project 3 is $41,101,000. All details describing associated land acquisition and construction costs can be found in the detailed cost estimate in **Appendix A**.

9.8 Associated Construction Costs Summary

The construction cost associated each regional project is a limitation to project implementation. In order to determine the overall benefits associated with the construction of each project **Table 9.1** illustrates a comparison of the planning-level estimate construction costs with the average annual reduction in TP load. By dividing the associated costs by the average reduction a cost per pound of TP removed is determined. This technique allows for a comparison of each project as it relates

Table 9.1: Comparison of Proposed Project Benefits and Limitations

REGIONAL PROJECT	AVERAGE ANNUAL TP REDUCTION [LB]	PLANNING-LEVEL COST ESTIMATE [$]	COST PER AVG ANNUAL TP REDUCTION [$ / LB REMOVED]
PROJECT 1	12,197	$6,391,000	$524.00
PROJECT 2	27,695	$7,539,000	$272.00
PROJECT 2a	4,389	$20,844,000	$4,749.00
PROJECT 2b	13,892	$1,198,000	$86.00
PROJECT 3	74,923	$41,101,000	$549.00

10.0 PLANNING-LEVEL SCHEDULE ESTIMATE

10.1 General

This section provides a planning-level estimate of the duration for implementing each regional proposed project. The schedule components considered as part of the implementation processes are as follows:

- Development of Construction Plans and Specifications
- Land Acquisition
- Acquisition of Required Permits
- Contract Procurement
- Construction

10.2 Assumptions

The construction plans and specifications component involves the development of construction contract documents (plans and specifications) by a licensed Professional Engineer. The duration of this component is assumed to be between 3 and 12 months depending on the project complexity. Although not all projects require land acquisition, for those that do, the process is assumed to last approximately 12 to 24 months. Several of the components require regulatory changes that are more complex than an average construction project. Changes to District rules and intra-basin transfer are some of the regulatory complexities presented by these scenarios. Therefore the duration of permit acquisition is difficult to estimate, but was assumed to be within the range of 3 to 16 months. The revised scope of this project does not include a detailed regulatory feasibility analysis to be performed for the proposed projects. Further details concerning permit acquisition will be based on the recommendation of District staff. For the purposes of this document the procurement process includes requesting bids and awarding a contract and should last approximately 3 months. The duration of the project's construction is assumed to be within a range of 12 to 24 months depending on the complexity of the project. These assumptions are based on standard design-bid-construct methodologies; innovative contracting methods such as design-build or construction manager at risk could accelerate the proposed schedules.

The components of the proposed schedule are not necessarily linear and as such the total duration of the project is not the sum of the duration of each individual component. For planning purposes it is assumed that the land acquisition process can begin at 30% design (plans and specifications) completion and the permitting process can begin once the project has reached 60% design. Permitting cannot be completed any sooner than 90 days after the completion of land acquisition. The procurement process begins at the end of the land acquisition and permitting process.

10.3 Implementation

The assumptions described above were incorporated into a Gantt chart for each proposed project which is included in **Appendix B**. **Table 10.1** summarizes the planning-level schedule estimate for each proposed project.

Table 10.1: Summary of the Estimated Implementation Duration

REGIONAL PROJECT	PROJECT DESCRIPTION	ESTIMATED IMPLEMENTATION SCHEDULE [MONTHS]
PROJECT 1	Providing treatment and storage in Basin 26-319-01	26
PROJECT 2	Using the DIRWA for storage and treatment	14
PROJECT 2a	Including treatment of runoff from the East to Project 2	26
PROJECT 2b	Including treatment of runoff from the North to Project 2	19
PROJECT 3	Creating centrally located impoundments for treatment	61

11.0 CONCLUSIONS AND RECOMMENDATIONS

11.1 Phase II Objectives

Based on the scope of work, the objectives of Phase II of the C139 Basin Phosphorus Water Quality and Hydrology Analysis are as follows:

1. Develop a calibrated hydrologic and water quality modeling tool to analyze flows and phosphorus loads in the C-139 Basin. Everglades Regulatory Program staff shall be able to use the model as a tool for prioritizing resources and tailoring Best Management Practice strategies towards achieving compliance with Everglades Forever Act-mandated water quality levels. The simulation results of the calibrated WAM model will be visually and statistically compared to all available measured data within the basin to provide an estimate of the modeling uncertainties. The water quality model shall be user-friendly and compatible with District applications. The Consultant shall train District staff in the use of this application.
2. Identify and evaluate a maximum of five water quality improvement projects (selected projects). The recommendations/needs or project types described by C-139 Basin landowners shall be considered.
3. Describe regulatory constraints that may affect implementation of water quality improvement projects within the C-139 Basin. Evaluate the regulatory feasibility of the selected water quality improvement projects or types of projects. Provide recommendations for pursuing viable rule or policy changes.
4. Identify technical issues, cost and schedule considerations for the selected projects. Evaluating site-specific technical issues, cost and schedule does not apply to farm-level projects.
5. Note uncertainties and limitations associated with project implementation along with any other unidentified issues that are uncovered as the contract progresses [e.g., results of the EAA Regional Feasibility Study, Phase 2 (CN040912-WO04)].

11.2 Significant Assumptions

As with any representation of a natural system, there are several assumptions inherent in the evaluation of each proposed regional project. The five proposed improvement projects can be categorized into two attenuation oriented treatment techniques: impoundment and overland sheet flow.

The benefits associated with Projects 1 and 3 are based on the use of impoundments to provide treatment. Within WAM each stream reach is categorized by type and each reach type has associated attenuation parameters. There are reach types within WAM commonly used to define reservoirs that describe open water impoundments that are filled with water year round and generally support only aquatic vegetation. Due to the substrate material and the operation of related structures, the impoundments within C-139 can be dry for substantial periods of the year. The assumption used within the evaluation of Projects 1 and 3 is that the vegetation and hydroperiods of the impoundment could be better categorized as a wetland or slough, which has distinctly higher attenuation parameters within WAM.

The benefits associated with Projects 2, 2a and 2b depend on the restoration of existing agricultural land-uses within DIRWMA. Projects 2a and 2b then further depend on the assumption that water delivered to the north end of a restored DIRWMA via pump or increased backwater stages will force an overland, sheet-flow like flow path across the restored lands. As described in Section 5.3, based on expert opinion, the benefits to water quality may not be fully realized for 15 to 30 years after the restoration efforts have been completed. In addition, it may not be realistic to expect that sheet-flow will occur in an area that is traditionally an upland habitat. Standard engineering practices assume that overland sheet-flow changes flow regimes and converts to shallow concentrated flow after a maximum of 300 feet. The likely outcome is that natural rivulets will be created by the increased stages and flows, similar to the methodology used to represent Projects 2a and 2b within WAM.

This regional project evaluation is provided utilizing WAM which is the best available tool to evaluate basin-scale nutrient loads within agricultural basins in Florida. However, as described within Deliverable 6.4 and within this document, WAM does not provide a complete integration of surface water and groundwater processes which may be required prior to construction designs are created for the proposed regional projects. The WAM tool simplifies the levee seepage and the sub-surface infiltration processes that are prevalent for impoundments within the C-139 region.

11.3 Quality Control

Each regional water quality improvement project required modification of the hydrography, land-use and parameterization schemes of the calibrated existing condition model. ADA performed the model development and internal quality controls required of the evaluation efforts. For example, in the case of a regional water quality project where runoff is diverted into a slough or impoundment, a mass balance analysis was performed on all the input and output mechanisms for the affected region. An example of internal quality control with respect to water quality included the verification of in-stream attenuation of TP for applicable projects. In addition, SWET provided external review and quality control of the input and output datasets associated with each project evaluation.

11.4 Project Evaluation and Conclusions

Due to the scale of regional water quality improvement projects there are many parameters that define the potential benefits and limitations of project implementation. Although the primary function of each proposed project is to reduce TP load discharges from the C-139 Basin, the construction cost and schedule can have a significant influence on the feasibility of a regional project.

11.4.1. Water Quality Improvement

Figure 11.1 illustrates the average annual reduction in TP load of due to the implementation of each project. For Projects 1, 2 and 3 the reduction is as compared with the existing condition, whereas for Projects 2a and 2b the reduction is incremental and so the values represent a comparison with the Project 2 condition.

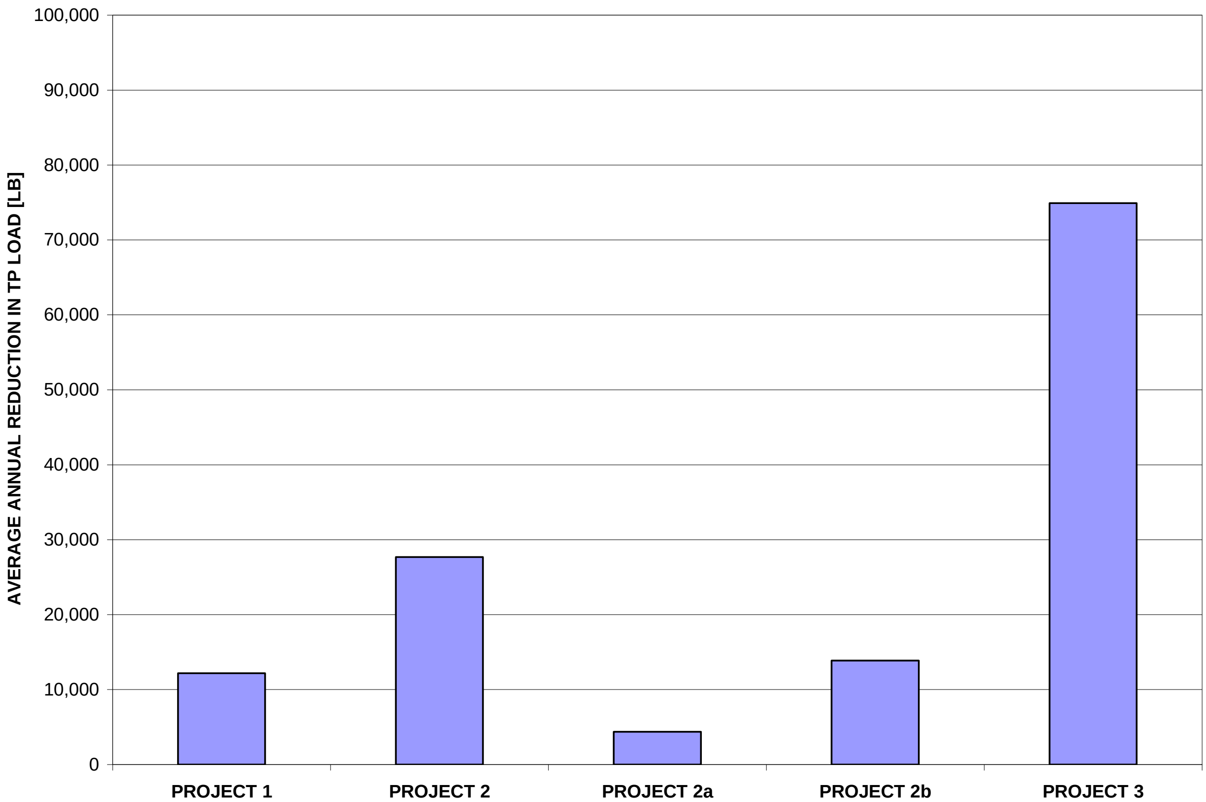

Figure 11.1: Comparison of Annual Average Reduction in TP Load

Figure 11.2 illustrates the temporal reduction of each proposed project as compared with the existing condition and the rainfall-adjusted annual target load. Of note, because of the nature of Projects 2a and 2b the benefits shown represent simulations where the reductions due to Project 2 implementation are incorporated. None of the projects individually are able to meet the rainfall-adjusted annual target load. However there is potential that the implementation of multiple projects could bring the C-139 Basin TP load discharges close to the target levels during average and below average years. Although there is no provision for simulations that incorporate combinations of the individual projects evaluated, the nature of the spatial locations of each project allow for certain combinations to be evaluated assuming the combined benefits would be additive. The changes to the hydrology for Projects 1 and 3 or Projects 2a and 3 are contained in separate sub-basins, and therefore it could be assumed that the benefits could be summed in estimating the benefits of both. Projects 1 and 2 or 2a or 2b would likely provide mutual benefits if implemented simultaneously, however since the DIRWMA contributes runoff to the proposed impoundment of Project 1 the relationship between the two results may not be additive.

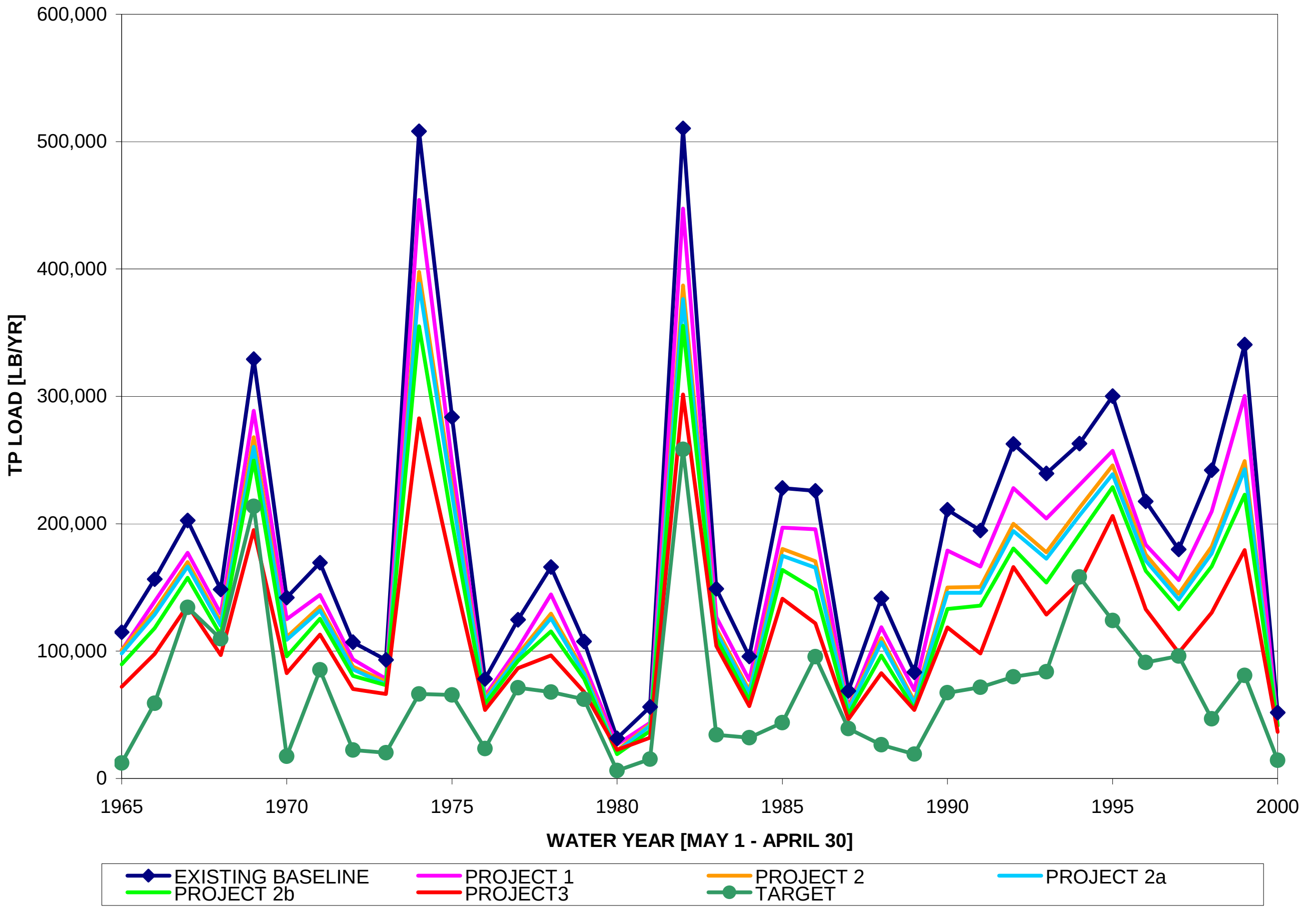

Figure 11.2: Comparison of Simulated Annual TP Load and Targeted Load

11.4.2. Associated Construction Costs and Schedule Duration

Sections 9.0 and 10.0 above present a planning-level estimate of cost and schedule. **Table 11.1** below compares each of these characteristics and also presents a cost per pound of removed TP for each project. The average annual reduction in TP is based on the difference between the existing condition simulation and each proposed project for Projects 1, 2 and 3. The average annual TP reduction for Projects 2a and 2b are the difference between the Project 2 simulation and the proposed conditions. Based on the total cost per pound of TP removed, the most effective projects appear to be Project 2b and Project 2. Of note, Project 2b requires the implementation of Project 2, which has a significantly higher cost per pound TP removed ratio. Project 3 provides a large reduction in basin-wide TP discharges and the median cost per pound TP removed ratio. However Project 3 is also estimated to have the largest total cost and longer schedule duration associated with construction.

Table 11.1: Comparison of Proposed Project Benefits and Limitations

REGIONAL PROJECT	AVERAGE ANNUAL TP REDUCTION [LB]	PLANNING-LEVEL COST ESTIMATE [$]	SCHEDULE [MONTHS]	COST PER AVG ANNUAL TP REDUCTION [$ / LB REMOVED]
PROJECT 1	12,197	$6,391,000	26	$524.00
PROJECT 2	27,695	$7,539,000	14	$272.00
PROJECT 2a	4,389	$20,844,000	26	$4,749.00
PROJECT 2b	13,892	$1,198,000	19	$86.00
PROJECT 3	74,923	$41,101,000	61	$549.00

11.5 Recommendations

The evaluation of the potential benefits and limitations of the five selected regional water quality improvement projects contained above is meant to be utilized for planning-level decision support. The following recommendations are based on the results of the evaluation and in consideration of the assumptions and limitations of the evaluation methodology.

1. Based on a comparison of the benefits and limitations of each project the recommended implementation priority for the regional improvement projects would be Project 3, Project 2, Project 2b, Project 1, and Project 2a.

2. The implementation of Projects 2a and 2b should be delayed considering the current timeframe associated with Project 2. As described in Section 11.2, the benefits of Project 2 could require between 15 and 30 years to be fully realized

3. The District should consider the timeline of the current phased restoration efforts on behalf of the FFWCC. If there is a priority in providing the benefits associated with Project 2 at an accelerated timeframe, negotiation with the FFWCC to accelerate the schedule may be required.

4. Once the proposed course of action has been selected it is recommended that a model with greater sophistication with respect to hydraulics and the surface water-groundwater interface be utilized to evaluate construction needs.

5. As described in Section 2.4 and in Deliverable 6.4, there are components of the hydrogeology that are not well understood in the C-139 Basin. Further investigation into the hydrogeologic conditions might include installation of clustered groundwater monitoring sites to get a more complete picture of the horizontal gradients of both the Surficial and Lower Tamiami aquifers and the inter-action between surface and ground waters.

6. The WAM simulations provided within Deliverable 6.4 and as part of the regional project evaluation within this document are best suited for use at the basin-scale.

Due to the lumped parameter technique employed by WAM, the addition of increased detail into local or farm-scale hydrology and hydraulics may not provide significant changes in the simulated flows and loads.

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