Interim Draft Report on the

S-4 Basin Feasibility Study

Prepared for

Everglades Agricultural Area Environmental Protection District

and

South Florida Water Management District

Burns & McDonnell
SINCE 1898

S-4 BASIN FEASIBILITY STUDY
INTERIM DRAFT REPORT

prepared for

Everglades Agricultural Area
Environmental Protection District
Palm Beach, Florida

and

South Florida Water Management District
West Palm Beach, Florida

prepared by

Burns & McDonnell Engineering Company, Inc.
Kansas City, Missouri

Project 45130

March 31, 2008

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Everglades Agricultural Area Environmental Protection District
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Everglades Agricultural Area Environmental Protection District
Interim Draft Report for S-4 Basin Feasibility Study
BMCD Project 45130

Dear Mr. Wilson:

Presented herein is our interim draft report on the S-4 Basin Feasibility Study. This interim draft report is a partial draft of the final report for this study. This draft contains all of the background material for the study and analyses of baseline conditions; however, this draft does not contain our analyses of potential diversion alternatives or our recommendations for a preferred alternative. We are making steady progress on our analysis of these project alternatives and plan to have a complete draft report prepared within the next few weeks.

As we complete this study, we may make minor revisions to the enclosed material but believe this information to be largely complete. We encourage you and any other individuals you may designate to review this interim draft report and provide us with your comments because the assumptions documented in this interim draft report will form the basis for our subsequent analyses and recommendations.

Please contact me if you should have any questions.

Sincerely,

Gene L. Foster

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INTRODUCTION

1.0 INTRODUCTION

Burns & McDonnell Engineering Company, Inc. (Burns & McDonnell) was retained by the Everglades Agricultural Area (EAA) Environmental Protection District (EPD) to evaluate the feasibility of diverting stormwater runoff from the S-4 Basin, which is currently discharged to Lake Okeechobee, to an alternate destination. This feasibility study was cosponsored by the South Florida Water Management District (SFWMD). The current study builds on the prior S-4 Basin Diversion study, which was also completed by Burns & McDonnell (2006). This earlier study looked into conceptual routes for these basin diversions, and examined the more desirable alternatives described therein in more detail. This introduction section presents a discussion of the project background, objectives, and study area for the current study.

1.1 PROJECT BACKGROUND

The Florida Department of Environmental Protection (FDEP) and SFWMD have established policies that seek to limit the discharge of phosphorus contained in stormwater runoff to Lake Okeechobee. In Florida's Everglades Forever Act (373.4592, Florida Statutes), several of the Chapter 298 drainage districts that are located along the southern shore of Lake Okeechobee — East Beach Water Control District, South Shore Drainage District, South Florida Conservancy District, East Shore Water Control District, and Cloister or 715 Farms (Agricultural Lease No. 3240) — are specifically required to install conveyance systems that would divert drainage away from Lake Okeechobee. The goal of these diversion systems is typically to divert 80 percent or more of this drainage into the SFWMD's primary conveyance canals so it can be delivered to one of the stormwater treatment areas (STA) for treatment prior to its discharge to the Everglades Protection Area (EPA). These diversion systems are now operational. It is anticipated that at some point, as an element of the Lake Okeechobee Protection Project and in response to the establishment of Total Maximum Daily Load (TMDL) limitations on discharge to the lake, it will be necessary to reduce discharges to Lake Okeechobee from the S-4 Basin as well, particularly for total phosphorus (TP) loads. This reduction in allowable TP discharges to Lake Okeechobee may make full or partial diversion, or pretreatment, of S-4 Basin discharges a necessity.

1.2 S-4 BASIN DIVERSION STUDY RESULTS

To investigate options available to the EPD and SFWMD for diversion of S-4 Basin drainage, Burns & McDonnell completed the S-4 Basin Diversion Study in 2005 (Burns & McDonnell, 2006). This study (hereinafter the Diversion Study) quantified historic S-4 Basin discharges to Lake Okeechobee, analyzed alternatives for diversion of these discharges away from Lake Okeechobee, and presented a high-level plan of the facilities required to affect this diversion. The scope of work for this S-4 Basin Feasibility Study (hereinafter the Feasibility Study) was based on the principal conclusions of the Diversion Study, which are discussed briefly below:

- The historic discharge data analyzed in the Diversion Study indicates that only about half (approximately 52 percent on average) of S-4 Basin drainage is discharged to Lake Okeechobee. The balance of the S-4 Basin drainage is currently discharged to the C-43 Canal (Caloosahatchee River) at S-235. Diverting that portion of the basin drainage that is now discharged to the C-43 Canal will not be a project goal.

- The northern end of the Clewiston (Industrial) Canal (generally that part between the S-310 lock and U.S. Highway 27) and the C-21 Canal east of S-169 are lined with marinas and boat docks. To the extent practicable, the proposed diversion system should avoid changes in the operation of these canal segments and S-310 to limit potential recreational impacts within Clewiston.

- Two options for pretreating portions of the S-4 Basin drainage and continuing to discharge this pretreated drainage to Lake Okeechobee were investigated in the Diversion Study. Pretreating this drainage would require development of a large STA within the northern part of the S-4 Basin. The economic and other impacts associated with development of this STA within the basin are considered excessive so these pretreatment options were not recommended for continued consideration in the Feasibility Study.

  Subsequent to completion of the Diversion Study, a proposed project to store S-4 Basin drainage on adjacent public land was initiated (Powell Kugler; MacVicar, Federico & Lamb; 2007). This water storage project represents a possible new pretreatment option for Clewiston Drainage District discharge. This new pretreatment option was added to the alternatives analyzed in the Feasibility Study.

- In the Diversion Study, the C-139 Basin and STA-5 were identified as the most likely destinations for S-4 Basin diversions. However, the potential for delivery of S-4 Basin diversions to the S-3/S-8 Basin should be retained as an option and considered in the detailed planning and analyses for Compartment A-2 of the EAA Storage Reservoir. If STA inflows could be redistributed or additional treatment area developed to accommodate these diversions from the S-4 Basin, the S-3/S-8 Basin could become an attractive destination for these diversions. Both the C-139 and S-3/S-8 basins will be considered as alternative designations for S-4 Basin diversions in the Feasibility Study.

1.3 FEASIBILITY STUDY OBJECTIVES

With these initial assumptions and constraints in mind, the objectives of the Feasibility Study are to further analyze the remaining diversion alternatives and develop the high-level diversion plan presented in the Diversion Study into a conceptual design and corresponding opinion of probable cost.

The principle objectives of the Feasibility Study are as follows:

- Collect data from existing sources (i.e. agencies) on the hydraulic structures:
  - Operating policies, guidelines, and permit restrictions
  - Status of planning studies within the project area
  - Monitoring data (discharge and TP concentrations)
  - Existing geometric data, including surveying S-4 Basin canals/structures

- Analyze the specific diversion system alternatives to develop design discharge rates which include the following:
  - A system for interception of the S-4 Basin drainage for export into one of the SFWMD canals
  - A plan that utilizes the SFWMD canal system to convey these flows to an appropriate STA

- For each diversion alternative, develop a hydraulic model to determine allowable discharge rates under existing conditions, and determine design improvements to convey the design discharge rates

- Develop a conceptual design for each diversion system alternative to include the following:
  - Design discharge rates and required cross sections of canals
  - Design discharge rates, recommended operating criteria and locations for hydraulic structures

- Develop an opinion of probable cost for each alternative including excavation and earthwork, structure construction, land rights, engineering, administration, and inspection

1.4 STUDY AREA

The study area for the Feasibility Study includes the area of the S-4 Basin itself and that of the adjacent drainage basins that may receive the stormwater diverted from the S-4 Basin. These adjacent drainage basins are listed below along with their locations relative to the S-4 Basin:

- C-139 Basin – located immediately south
- S-3/S-8 Basin – located immediately southeast

A map showing the locations of these drainage basins within south Florida is included as Figure 1-1.

The S-4 Basin is located on the southwestern shore of Lake Okeechobee in Glades and Hendry counties, between Clewiston and Moore Haven. This basin occupies an area of approximately 73.4 square miles (Cooper, 1989). The S-4 Basin includes all or portions of five special drainage districts:

- Clewiston Drainage District
- Disston Island Conservancy District
- East Hendry County Drainage District
- South Florida Conservancy District, Unit No. 5
- Sugarland Drainage District

Also included in the S-4 Basin, but not part of one the above drainage districts, is a large farming area owned by the United States Sugar Corporation (U.S. Sugar) that is known as the Townsite area.

* * * * *

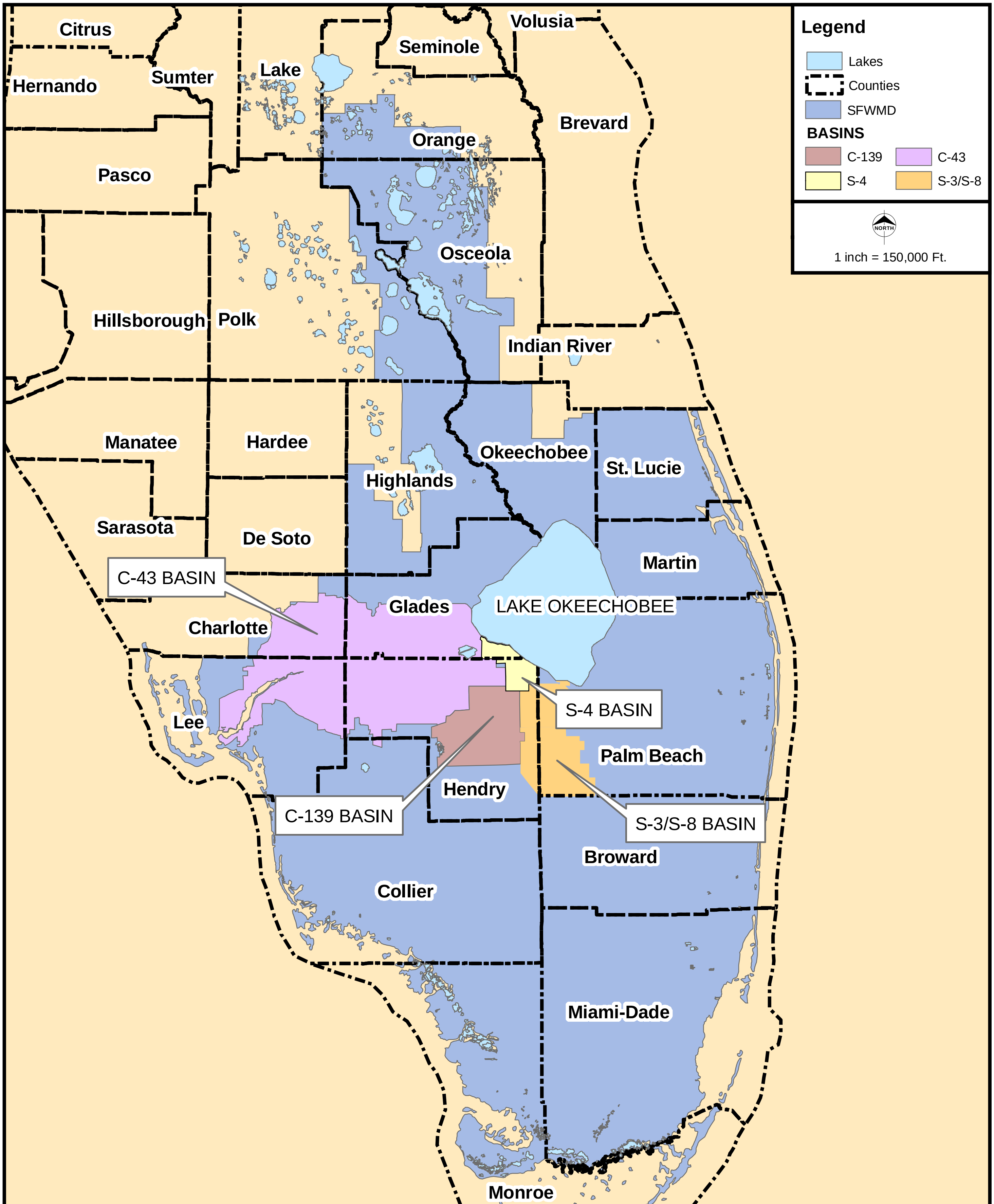

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Figure 1-1
S-4 BASIN FEASIBILITY PROJECT STUDY AREA

Section 2.0

BASIN CHARACTERISTICS

2.0 BASIN CHARACTERISTICS

This report section provides a description of the S-4 and adjacent drainage basins, along with a summary of historic hydrologic data on runoff volumes, stages, TP concentrations and loads, and other relevant data.

2.1 DATA SOURCES

The hydrologic data utilized in this study were all obtained from the SFWMD. The data for those structures with the EPD prefix were collected by the Environmental Protection District and then submitted to SFWMD on an annual basis. Therefore, all of the data received from SFWMD for the EPD-monitored structures is simply a compilation of the data provided by the EPD over the years. As stated in the Diversion Study, the period of record for these data is from 5/1/1992 to 4/30/2004. The EPD's monitoring program has been suspended so there are no more current data available for these ten EPD-designated structures. The provided EPD data includes estimates of daily flow, TP loads, and TP concentrations.

Most of the EPD-monitored structures are equipped with auto-samplers that collect composite samples. These composite samples are either flow-weighted (sampling rate proportional to pump discharge rate) or time-weighted (sampled at constant rate whenever pumps are operating). The composite samples were collected and analyzed at intervals ranging from about 14 to 30 days, with more frequent collections during periods of increased pump operation.

For certain control structures and other monitoring stations in the C-139 and S-3/S-8 drainage basins, the flow data and TP concentration and load estimates used in this study were provided directly by SFWMD staff as part of the EAA Regional Feasibility Study (RFS). The structures included in this category are G-136, G-342A – G-342D, G-406, L3DF, G-88, G-89, and G-155. In the C-139 Basin, the period of record for these RFS data is WY 1995–2005. These SFWMD-provided data are summarized in a report prepared by Burns & McDonnell (2005).

Except as noted above, the balance of the data used in this study were downloaded directly from SFWMD's environmental database, DBHYDRO. Data on daily mean flow rates, stages, rainfall, and pan evaporation plus the results of TP sampling and analysis were obtained from DBHYDRO. The periods of record for these data varies by structure or monitoring station but are generally longer than for the EPD data, except for those structures that were placed into service more recently (SFWMD, no date).

TP concentrations and loads at structures or for time periods not provided directly by EPD or SFWMD were estimated by Burns & McDonnell from available sampling data. Where and when available, preference was given to concentration data from flow-weighted composite samples. For structures without composite sample data, grab sample data were used. The available grab sample data were screened to eliminate samples collected on days with no flow or with flow in the wrong direction. Simple linear regression techniques were also used to eliminate outliers — samples with extremely low or high TP concentrations. After screening, concentrations between the remaining grab samples were assumed to vary linearly with time. The methodology used to estimate TP loads from periodic sample data is equivalent to the method used by SFWMD to estimate TP loads from the EAA (Chapter 40E-63, F.A.C.).

The period of record selected for the analyses in this study is water years (WY) 1993–2004 (5/1/1992–4/30/2004). This 12-year period was selected based on the available period of record for the EPD monitoring data.

2.2 S-4 BASIN

Figure 2-1 is a map of the S-4 Basin that shows the boundaries of the individual drainage districts within the basin and the locations of major control structures and canals. A schematic of this basin, which more clearly shows the arrangement of principal control structures and their receiving canals, is included as Figure 2-2. There are a number of discharge structures — pump stations, culverts, spillways and locks — within the S-4 Basin where flow volumes and/or water quality are monitored.

Similar to many of the drainage improvements in South Florida, the original design of the basin's conveyance structures and pump stations was based on a drainage rate of 0.75 inch per day from agricultural areas and 4.0 inches per day from the City of Clewiston. Within the S-4 Basin, there are 112 square miles of agricultural lands and the City of Clewiston covers approximately 4.5 square miles.

2.2.1 Clewiston Drainage District

The Clewiston Drainage District is located in the northeast corner of the S-4 Basin in the City of Clewiston. This drainage district is bounded on the east by the Clewiston Canal (also known as the Industrial Canal) and on the north by Canal C-21. The Sugarland Drainage District abuts this district on the west and south. Drainage from this district is discharged at two primary locations:

Figure 2-1: S-4 Basin Map

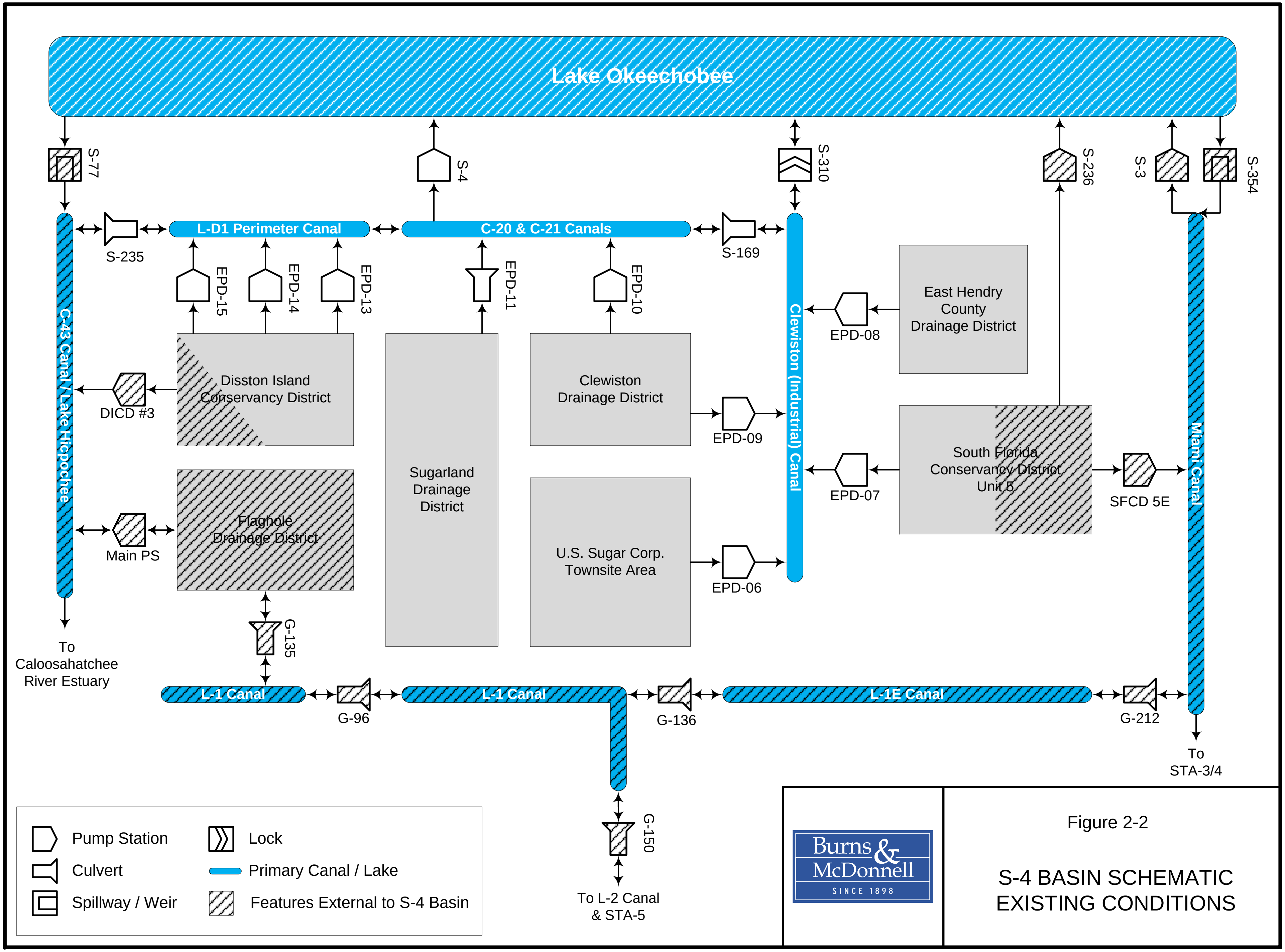


Figure 2-2

S-4 BASIN SCHEMATIC
EXISTING CONDITIONS

- Clewiston Pump Station No. 2 (EPD-10): Pump Station No. 2 is the largest capacity pump station in the Clewiston Drainage District and is located in the north-central part of the district. This station includes three pumps, each rated at 42,000 gallons per minute (gpm), that discharge to the C-21 Canal (Roberts, no date-a).

- Clewiston Pump Station No. 3 (EPD-09): This pump station is located in the southeastern part of this drainage district and discharges to the Clewiston Canal. There are two 36-inch pumps, both rated at 36,000 gpm, in this station (Roberts, no date-a).

Pump Station No. 1 is located along the Clewiston Canal in the northeast portion of the Clewiston Drainage District but is no longer in service.

2.2.2 Disston Island Conservancy District

The Disston Island Conservancy District (DICD) is located in the northwestern corner of the S-4 Basin. This district is bounded on the north by the L-D1 Borrow Canal and Lake Okeechobee; on the east by the C-20 Canal; on the south by the Nine Mile Canal, and Sugarland and Flaghole drainage districts; and on the west by the C-43 Canal and Lake Hicpochee. The part of this district that is generally west of U.S. Highway 27 is not considered to be part of the S-4 Basin but instead part of the C-43 Basin. Drainage from this conservancy district is discharged at four primary pump stations:

- DICD Pump Station No. 1 (EPD-15): Pump Station No. 1 is located in the northwestern portion of the DICD and discharges to the L-D1 Borrow Canal. This pump station consists of two pumps and a gated culvert. One of the pumps is rated at 56,000 gpm and the other at 28,000 gpm (Roberts, no date-a). The available monitoring data for this pump station show it is operated only occasionally, with little or no discharge in some years.

- DICD Pump Station No. 2 (EPD-14): This pump station is located near the middle of the northern border of this drainage district and discharges to the L-D1 Borrow Canal. There are three pumps in this station. The nominal capacity of the first two pumps is 70,000 gpm but the capacity of the newer third pump is unknown (Roberts, no date-a).

- DICD Pump Station No. 3: Pump Station No. 3 is located at the western end of the DICD and discharges to Lake Hicpochee and the C-43 Basin. This pump station, which has a rated capacity of

80,000 gpm (DICD, no date), is not located within the S-4 Basin and is not monitored by the EPD. Therefore, no discharge records for this station are available in the EPD data obtained from SFWMD.

- DICD Pump Station No. 4 (EPD-13): Pump Station No. 4 is located in the eastern portion of the DICD, at the eastern end of DICD's Main Canal. This canal runs east-west through the district, with Pump Station No. 3 located at its western terminus. This pump station consists of three pumps, each rated at 30,000 gpm (Roberts, no date-a). The stormwater discharged at this pump station flows into the L-D1 Borrow Canal, just over a half mile west of the C-20 Canal and S-4 Pump Station.

2.2.3 East Hendry County Drainage District

The East Hendry County Drainage District is located on the shore of Lake Okeechobee, just east of Clewiston. This district lies between the Hendry-Palm Beach County line and the Clewiston Canal, and is bordered on the south by the South Florida Conservancy District.

There is a single discharge structure within this district: Pump Station No. 1 (EPD-08). This station consists of two pumps, an electric pump with a rated capacity of 12,000 gpm and a diesel pump of unknown capacity (Roberts, no date-a).

2.2.4 South Florida Conservancy District

The South Florida Conservancy District, Unit No. 5 (SFCD) is located along of the shore of Lake Okeechobee, except for that portion that is within Hendry County and south of the East Hendry County Drainage District. It is this portion of the basin in Hendry County that is considered to be part of the S-4 Basin. The balance of the SFCD drains directly to Lake Okeechobee at Pump Station S-236 (P-5-N) or to the Miami Canal at Pump Station P-5-E.

There is a single SFCD pump station that discharges into the Clewiston Canal and the S-4 Basin. This pump station is known as P-5-W or EPD-07. This station consists of four pumps, two with a nominal capacity of 48,000 gpm and two with a capacity of 38,000 gpm (Roberts, no date-a).

The SFCD is one of the Chapter 298 drainage districts that is required under the Everglades Forever Act to divert at least 80 percent of its drainage away from Lake Okeechobee. This new conveyance system, which consists of a new P-5-E pump station and canal improvements, began operation is 2006. Since that time the available discharge data for the S-236 and P-5-E pump stations indicate that most of the district's drainage has been discharged through this new pump station into the Miami Canal.

The P-5-W Pump Station is still operational but now used only occasionally during major storm events and for periodic maintenance checks. Unfortunately, the EPD's monitoring program was suspended before the new SFCD diversion system became operational. Therefore, there is no direct measure of how this diversion system will affect discharges into the S-4 Basin at the P-5-W Pump Station. In order to estimate the magnitude of these future pump station discharges, it was assumed that this station would be operated only when historic discharges were high [greater than 250 cubic feet per second (cfs)]. Under this scenario, these high discharges would still occur but all average or lesser discharges would be eliminated. With this assumption the estimated post-diversion (adjusted) discharges at P-5-W (EPD-07) would average less than 5 percent of historic values, with an average annual discharge volume of 493 acre-feet and TP load of 152 kg.

2.2.5 Sugarland Drainage District

The Sugarland Drainage District is located in the center of the S-4 Basin. It shares boundaries with the Clewiston Drainage District, Disston Island Conservancy District, Flaghole Drainage District, and U.S. Sugar Townsite area. Part of the eastern border of this district is formed by the Clewiston Canal.

The Sugarland Drainage District has only one primary discharge structure, the Sugarland Main Pump Station (EPD-11). This pump station is located in the north-central part of this district and discharges to a short stub canal that connects with Canal C-20 and C-21 where these two canals intersect. The drainage pumps in this station are no longer in service, so the only operational discharge structure from this drainage district is a gated culvert that passes through the pump station. The flow through this culvert is regulated manually to maintain a stage in district canals of approximately 16.5 feet NGVD.

A relatively small portion of this drainage district, the area generally located north of U.S. Highway 27 and immediately west of State Highway 720, drains to the Nine Mile Canal. This canal flows into the C-21 Canal downstream of the Sugarland Main Pump Station, so there are no discharge measurements from this portion of the district.

2.2.6 U.S. Sugar Townsite Area

U.S. Sugar's Townsite area is a large tract of farmland located in the southeastern corner of the S-4 Basin. This area, which covers approximately 14 square miles, abuts Levee 1 (L-1) and Levee 1 East (L-1E) along its southern border. The other boundaries of this drainage area are formed by the Clewiston Canal to the east and Sugarland Drainage District to the north and west.

The drainage from this area is discharged at U.S. Sugar's Southline Pump Station (EPD-06). This pump station is located at the southern terminus of the Clewiston Canal and contains seven pumps: five rated at 22,000 gpm each and two rated at 45,000 gpm each.

2.2.7 SFWMD Structures

In addition to the EPD-monitored control structures described above, there are several structures within or adjacent to the S-4 Basin that are operated and monitored by SFWMD. Some of the structures included in this category are operated in cooperation with the U.S. Army Corps of Engineers (COE). Each of these structures is described below.

2.2.7.1 S-310 Lock

S-310 is a navigation lock structure located in L-D2, part of the Herbert Hoover Dike system that rings Lake Okeechobee. The lock chamber is 60 feet long by 50 feet wide. Under normal operation, this lock is left open to allow free passage of boats (and water) between Lake Okeechobee and the northern end of the Clewiston Canal. This lock is normally closed only when lake stages exceed 15.5 feet NGVD and during hurricane alerts to help prevent flooding within the S-4 Basin (Cooper, 1989).

When S-310 is open, drainage from the S-4 Basin is free to flow into Lake Okeechobee. The purpose of the proposed basin diversion project is to limit these flows to the extent practicable. The principal source of this drainage is discharge from one of the four primary pump stations located along the Clewiston Canal: EPD-06, EPD-07, EPD-08 and EPD-09. Though relatively uncommon, water can also flow into the Clewiston Canal and then Lake Okeechobee through the S-169 culverts. Water also flows from the lake into the Clewiston Canal when necessary to meet water supply demands from this canal and the C-20 and C-21 canals via S-169.

Discharges through S-310 were estimated from data for the INDUST monitoring station (DBKEY 15628). In DBHYDRO, the location of this station is reported to be at the U.S. Highway 27 bridges over the Clewiston Canal, which are located about 2,000 feet south of S-310 and south of the canal that connects to S-169. However, comparison of the flow data for the INDUST station with that for the four EPD pump stations on the Clewiston Canal and S-169 would indicate that the data for the INDUST station is actually a measure of flows between Lake Okeechobee and the Clewiston Canal at S-310. Positive flows at this station indicate releases from Lake Okeechobee to the canal and negative flows are discharges to the lake.

There are also stage data available for the INDUST monitoring station. Upon examination of these data (DBKEY TA200), it appears these are a measure of water levels on the north side of S-310 in Lake Okeechobee. When the lock is open, these stage data are also a measure of stages in the Clewiston Canal to the south of this structure. However, when the S-310 lock is closed, Clewiston Canal stages are best estimated from the headwater stage data at S-169 (DBKEYs 12772 and 15591).

The TP concentrations in discharges to and from Lake Okeechobee at S-310 were estimated from sampling data retrieved from DBHYDRO for the INDUSCAN monitoring station. During the 12-year study period, data for 191 grab samples are available, which is an average collection interval of about once every 3.2 weeks. However, few of these samples were collected when flow was negative (from the Clewiston Canal to Lake Okeechobee) so the corresponding TP load estimates may not be very accurate.

2.2.7.2 S-169 Culverts

Structure S-169 is a series of three gated culverts that connect the Clewiston Canal with the C-21 Canal. These culverts are 84 inches in diameter, 60 feet long and have an invert elevation of approximately 6.0 feet. The gates for this culvert have three operating modes (Cooper, 1989):

- Auto: When stage in Lake Okeechobee are above 15.5 feet NGVD and the S-310 lock is closed, the gates at S-169 are adjusted automatically to maintain stages in the Clewiston Canal at about 15 feet (ranging from 14.7 to 15.2 feet). Any stormwater discharged to the Clewiston Canal when S-169 is on auto is released to the C-21 Canal to keep Clewiston Canal stages within the target range.
- Closed: The gates at S-169 are kept closed when lake stages are between 13.0 and 15.5 feet. Under this operating mode, S-310 is open and any stormwater discharge to the Clewiston Canal would flow directly to Lake Okeechobee.
- Open: When lake stages are below 13.0 feet or during hurricane alerts, the gates at S-169 are left open to allow water to pass freely between the Clewiston and C-21 canals.

The historic flow data for S-169 was retrieved from DBHYDRO using two database keys: DBKEY 12774 through 3/10/1993 and 15590 starting on 3/11/1993. Headwater and tailwater stage data are also available at this structure (headwater stages at DBKEYs 12772 and 15591 and tailwater stages at DBKEYs 12773 and 15592). Sampling data for TP were also retrieved from DBHYDRO for station S169. During the study period, 173 grab samples were collected and analyzed at this structure. None of the available 173 grab samples were collected on days with negative flow (from C-21 to Clewiston Canal) so none are considered valid for estimating TP loads in this flow direction.

2.2.7.3 S-4 Pump Station

S-4 is a pump station in the alignment of the Hoover Dike (L-D1) at the northern end of the C-20 Canal, which pumps stormwater from the S-4 Basin to Lake Okeechobee. The primary purpose of this feasibility study is to investigate ways to reduce these lake discharges, or to reduce the phosphorus contained therein. This station contains three pumps, each rated at 935 cfs, for a total station capacity of 2,805 cfs. Under normal operations, pumping is started at this station whenever stages in the C-20 Canal exceed 14.0 feet NGVD and stopped when canal stages fall to 11.0 feet. During hurricane alerts, stages in the C-20 Canal are drawn down to and held at approximately 10.0 feet (Cooper, 1989).

The historic flow data for S-4 was retrieved from DBHYDRO using DBKEY P1013. Headwater and tailwater stage data are also available from DBHYDRO at this location (DBKEYs TA221 and TA223, respectively). Sampling data for TP were also retrieved from DBHYDRO for station S4. The TP sampling data for this station includes both flow-weighted composite samples and grab samples. In estimating daily TP concentrations in the discharge from this station, the composite samples were given precedence over the grab sample data.

2.2.7.4 S-235 Culverts

Structure S-235 is a pair of gated culverts that connect the western end of the L-D1 Borrow Canal with the C-43 Canal (also known as the Caloosahatchee River). These culverts are located less than 400 feet downstream of the S-77 spillway, which controls discharge between Lake Okeechobee and the C-43 Canal. The S-235 structure consists of two 72-inch diameter by 70-foot long reinforced concrete pipe (RCP) culverts with invert elevations of 4.1 feet NGVD. The gates for these culverts are normally left open, and only closed when stages in Lake Okeechobee fall below 13.0 feet or during hurricane alerts (Cooper, 1989).

The majority of the flow through this structure (approximately 85 percent) is westward from the S-4 Basin to the C-43 Canal. Virtually all of the westward discharge at S-235 flows down the C-43 Canal toward Lake Hicpochee and the Gulf of Mexico; very little of this discharge flows back upstream into Lake Okeechobee. Therefore, S-4 Basin discharges at S-235 are not considered to be a target for diversion in this project.

The historic flow data for S-235 was retrieved from DBHYDRO using DBKEY P0984. Headwater and tailwater stage data for this same structure are also available in DBHYDRO (DBKEYs TA216 and TA218, respectively). Although there is a water quality testing station at this structure (S235), it appears these samples are not tested for phosphorus because there is no TP concentration data available for this station in DBHYDRO. Therefore, no TP load estimates were possible at this structure.

2.2.7.5 L-D1 Culverts

The L-D1 culverts are a series of gated culverts through the Herbert Hoover Dike that are distributed along the canals adjacent to Lake Okeechobee between S-235 and S-169. These culverts are primarily used to remove excess water from these canals when canal stages are higher than stages in Lake Okeechobee, although such conditions are rare. These culverts are described below (Cooper, 1989):

- Culvert #1: This culvert is located about 1.5 miles east of S-235 near DICD Pump Station No. 1 (EPD-15). This culvert, which is fitted with a single flap gate, is used to remove excess water from the L-D1 Canal when Lake Okeechobee stages are low.
- Culvert #1A: This culvert, which has vertical screw gates, is located directly opposite of DICD Pump Station No. 2 (EPD-14). This structure is rarely, if ever, operated.
- Culvert #2: Culvert #2 has five flap gates and a slide gate and is located about 500 feet west of S-169 on the C-21 Canal. These culverts can be used to remove excess water when lake stages are low but are primarily used for water supply. When the S-310 lock is closed, Lake Okeechobee releases into the S-4 Basin are made through these culverts. The Corps of Engineers is currently evaluating a project to relocate the S-169 structure farther west so that Culvert #2 would be located on the headwater (east) side of this structure.

There is no operational information available on these L-D1 culverts, thus they have not been included in the hydrologic and hydraulic modeling for this project.

2.2.8 S-4 Basin Discharge Summary

A discharge summary for the S-4 Basin structures discussed above is included in Table 2-1. This table lists average annual flow volumes in acre-feet (ac-ft), TP loads in kilograms (kg), and TP concentrations in part per billion (ppb). This table includes the SFCD original flows which are used during the existing conditions modeling effort.

Table 2-1: Recorded Discharge Summary for S-4 Basin Structures[a]

Station No.	Description	Discharge Destination	Avg. Ann. WY 1993-2004		
			Flow (ac-ft)	TP Load (kg)	TP Conc. (ppb)
EPD-06	U.S. Sugar Southline Pump Sta.	Clewiston C.	21,556	5,403	203
EPD-07[b]	SFCD P-5-W Pump Sta.	Clewiston C.	11,428	2,007	142
EPD-08	E. Hendry Co. Pump Sta. No. 1	Clewiston C.	1,415	553	317
EPD-09	Clewiston Pump Sta. No. 3	Clewiston C.	1,006	283	228
EPD-10	Clewiston Pump Sta. No. 2	C-21 Canal	2,336	643	223
EPD-11	Sugarland Main Pump Sta.	C-21 Canal	31,778	11,192	286
EPD-12	Clewiston C-21 Culvert	C-21 Canal	0	0	---
EPD-13	DICD Pump Sta. No. 4	L-D1 Canal	6,441	1,186	149
EPD-14	DICD Pump Sta. No. 2	L-D1 Canal	24,317	6,132	204
EPD-15	DICD Pump Sta. No. 1	L-D1 Canal	717	508	575
Total for EPD-Monitored Structures			100,993	27,906	224
S-4	Pump Sta. S-4	L. Okeechobee	27,110	5,988	179
S-169	S-169 Culverts	C-21 Canal	39,401	4,235	87
		Clewiston C.	4,454	---	---
S-235	S-235 Culverts	C-43 Canal	65,650	---	---
		L-D1 Canal	10,048	1,644	133
S-310	S-310 Lock	L. Okeechobee	22,099	4,788	176
		Clewiston C.	42,162	4,755	91

a. EPD monitoring data collected by RayRob Environmental. Data for other structures collected by SFWMD.

b. Data recorded at EPD-07 does not reflect impact of current SFCD diversion project, which began operation in 2006. Estimated post-diversion data for average annual flow, TP load and TP concentration are 493 ac-ft, 152 kg and 250 ppb, respectively.

2.2.9 Water Balance for Principal S-4 Basin Canals

The principal canals in the S-4 Basin are defined as those that convey the majority of the water into and out of the basin. These principal canals include the Clewiston (Industrial), C-20, C-21 and LD-1 canals. A daily water balance for these principal conveyance canals was developed using the available monitoring data. Ideally, these data should balance closely each day. That is, any recorded net discharges to these canals should be offset by net withdrawals of equal magnitude. This flow balancing rarely occurs in practice because there are unmonitored water deliveries and withdrawals from these canals. The largest of these unmonitored flows are the irrigation and other water supply withdrawals from these canals, which can be of significant magnitude at times each year. There are other inaccuracies in this methodology also such as the following:

- Seepage: Direct seepage gains or losses in these canals from the adjacent drainage districts or Lake Okeechobee would respectively contribute positively or negatively to the apparent water imbalances.

- Precipitation and Evaporation: Any direct precipitation on or evaporation from the water surface in these canals will respectively increase or decrease stages or discharge rates.
- Changes in Canal Storage: Canal stages and storage will increase or decrease, at least locally, whenever water is respectively discharged to or withdrawn from these canals. These changes in canal stage and storage are not monitored at most locations and would also contribute to an apparent water imbalance.
- Measurement Errors: Some error can be expected in all flow measurements. If these errors do not balance one another, they would contribute to an apparent flow imbalance.

Table 2-2 summarizes the inflows to, outflows from, and flow imbalances in these principal basin canals on an annual basis. The water imbalance term is calculated by subtracting canal inflows from outflows, so a negative imbalance indicates a net canal inflow. Review of this table shows there is a net canal inflow in most years. This result is expected because of irrigation withdrawals that are not monitored. Positive flow imbalances (for example, in WY 1996) generally occur in wet years when there would be less canal withdrawals for irrigation and more seepage from adjacent fields and Lake Okeechobee.

Table 2-2: Water Balance Summary for Principal S-4 Basin Canals[a]

Water Year	Canal Inflow by Source			Canal Outflow by Destination			Water Imbalance
	Internal EPD Stations	Lake Okeechobee @ S-310	C-43 Basin @ S-235	Lake Okeechobee @ S-310	Lake Okeechobee @ S-4	C-43 Basin @ S-235	
1993	114,181	19,259	12,306	16,991	25,645	107,985	4,876
1994	87,518	25,576	21,480	34,243	10,094	42,964	-47,272
1995	114,753	24,298	18,916	12,352	54,158	63,971	-27,486
1996	92,073	29,460	8,139	19,484	71,805	69,418	31,035
1997	77,569	57,135	585	35,119	12,074	74,921	-13,174
1998	90,632	31,624	8,691	30,328	34,304	66,063	-252
1999	90,092	86,595	4,282	6,036	30,985	63,871	-80,078
2000	178,089	27,624	3,160	27,252	44,070	70,274	-67,279
2001	94,566	57,782	3,891	14,250	649	28,571	-112,770
2002	61,814	72,922	13,758	18,438	15,212	65,568	-49,276
2003	110,404	44,280	14,419	36,152	6,870	68,499	-57,583
2004	100,220	29,386	10,954	14,548	19,456	65,689	-40,867
Average	100,993	42,162	10,048	22,099	27,110	65,650	-38,344

a. All data reported in acre-feet.

2.2.10 Destinations for S-4 Basin Drainage

The principal goal of the proposed diversion project is to reduce deliveries of S-4 Basin drainage to Lake Okeechobee. This section describes the results of analyses that attempt to quantify these historic basin discharges to Lake Okeechobee, and also to the C-43 Basin.

The water imbalances described in the previous section complicates this estimation process. On some days not all of the measured discharge from the EPD-monitored stations appears to make it to one of the three basin outlets: S-310 lock, S-4 pump station, or S-235 culverts. Conversely, there are days when the total discharge at these three outlets exceeds the total discharge from the ten EPD monitoring stations. To overcome these apparent discrepancies, the basin discharge at each outlet was assigned each day to the available supply sources on a prorated basis. This methodology is best illustrated by a simple hypothetical example for the western basin canals (C-21, C-20 and LD-1 canals):

- Assumed supply sources to western basin canals:
    - Total discharge for EPD-10 – EPD-15: 200 cfs
    - Westward discharge at S-169 assigned to Lake Okeechobee inflow at S-310: 25 cfs
    - Westward discharge at S-169 assigned to EPD-06 – EPD-09: 75 cfs
    - Total supply: 300 cfs

- Assumed discharge from western basin canals:
    - Discharge to Lake Okeechobee at S-4 Pump Station: 250 cfs
    - Discharge to C-43 Canal at S-235: 100 cfs
    - Total discharge: 350 cfs

- Calculated flow allocations:
    - EPD discharge to Lake Okeechobee at S-4 Pump Station: 250 * (200 + 75) / 300 = 229.2 cfs
    - EPD discharge to C-43 Canal at S-235: 100 * (200 + 75) / 300 = 91.7 cfs
    - S-4 Pump Station discharge assigned to lake inflow at S-310 = 250 * 25 / 300 = 20.8 cfs
    - S-235 discharge to C-43 Canal assigned to lake inflow at S-310: 100 * 25 / 300 = 8.3 cfs

Continuing with this same example, the corresponding TP loads in basin discharge, which originated from EPD drainage only, were estimated using the TP concentration data collected at the EPD monitoring stations.

- Assumed TP load in EPD discharge:
  - EPD-10 – EPD-15: 80.1 kg
  - EPD-06 – EPD-09: 40.4 kg
  - Total EPD TP load: 120.5 kg

- Calculated TP load allocations from EPD sources only:
  - EPD TP load to Lake Okeechobee at S-4 Pump Station: 250 / 350 * (40.4 + 80.1) = 86.1 kg
  - EPD TP load to C-43 Canal at S-235: 100 / 350 * (40.4 + 80.1) = 34.4 kg

Applying this methodology to the entire 12-year historic record yields the flow and TP load allocations that are summarized in Tables 2-3 and 2-4. These tables show that the total allocated discharge from EPD monitoring stations averages about 92,212 acre-feet and 26,517 kg annually. These volumes and loads are almost equally split between deliveries to Lake Okeechobee and to the C-43 Canal.

Table 2-3: S-4 Basin Discharge to Lake Okeechobee from EPD Sources Only

Water Year	At S-310 Lock		At S-4 Pump Station		Total	
	Volume (ac-ft)	TP Load (kg)	Volume (ac-ft)	TP Load (kg)	Volume (ac-ft)	TP Load (kg)
1993	15,613	4,802	25,466	10,278	41,078	15,080
1994	33,625	7,652	9,884	2,433	43,509	10,085
1995	12,242	3,020	51,907	10,381	64,148	13,401
1996	15,807	1,511	68,190	14,064	83,997	15,575
1997	33,698	6,057	12,057	3,187	45,755	9,245
1998	24,156	6,325	32,917	11,903	57,073	18,228
1999	5,248	2,117	29,317	8,796	34,565	10,912
2000	27,158	7,335	43,486	17,031	70,644	24,366
2001	13,255	6,146	441	24	13,695	6,170
2002	18,124	7,056	9,721	3,314	27,845	10,371
2003	35,850	9,511	6,542	2,863	42,392	12,374
2004	14,195	3,020	19,276	6,390	33,472	9,410
Avg. Annual	20,748	5,379	25,767	7,555	46,514	12,935
FWM TP Conc.[a]	---	210	---	238	---	225

a. Flow-weighted mean total phosphorus concentration for 12-year period.

Table 2-4: S-4 Basin Discharge to C-43 Canal from EPD Sources Only

Water Year	Volume (ac-ft)	TP Load (kg)
1993	74,730	19,402
1994	32,879	6,570
1995	56,135	11,134
1996	46,979	7,066
1997	42,942	8,717
1998	47,541	7,957
1999	37,507	11,925
2000	60,654	26,990
2001	14,361	19,400
2002	18,415	6,232
2003	55,891	16,268
2004	60,338	21,321
Avg. Annual	45,698	13,582
Avg. TP Conc.	---	241

Review of Tables 2-3 and 2-4 shows that discharge from EPD sources on average is split almost equally between discharges to Lake Okeechobee and the C-43 Canal — average annual discharge volumes are respectively 46,514 and 45,698 acre-feet/year while average annual TP loads are respectively 12,935 and 13,582 kg/year.

The flow and TP load allocations from EPD sources that are described above will be impacted by the SFCD diversion project that is now operational. Estimating the impact of the SFCD diversion project on these allocations is more speculative, both because there are no measurements of the continuing discharges from the SFCD's P-5-W pump station (EPD-07) and because there are frequent imbalances in the available discharge data. Given these limitations, estimates of the post-SFCD diversion discharges to Lake Okeechobee and the C-43 Canal from EPD sources only are presented in Table 2-5.

The SFCD diversion project is estimated to reduce total EPD discharges from the S-4 Basin by an average of more the 11,000 acre-feet and 4,000 kg per year. Volumetrically, approximately three quarters of these reductions would occur in Lake Okeechobee deliveries, but TP load reductions would be more evenly split between Lake Okeechobee and the C-43 Canal.

Table 2-5: S-4 Basin Discharges from EPD Sources after SFCD Diversion

Water Year	To Lake Okeechobee		To C-43 Canal		Total	
	Volume (ac-ft)	TP Load (kg)	Volume (ac-ft)	TP Load (kg)	Volume (ac-ft)	TP Load (kg)
1993	32,206	10,796	67,768	19,574	99,975	30,370
1994	29,005	6,281	32,773	5,508	61,778	11,789
1995	50,108	12,221	43,823	8,189	93,931	20,410
1996	64,351	19,827	39,047	8,151	103,398	27,977
1997	31,617	7,858	40,369	8,914	71,986	16,772
1998	51,020	18,354	45,223	9,770	96,243	28,124
1999	31,783	8,592	37,053	11,163	68,836	19,754
2000	64,848	18,465	59,218	19,824	124,067	38,289
2001	13,464	2,784	14,361	4,045	27,825	6,829
2002	24,961	9,403	18,342	5,792	43,303	15,195
2003	35,274	7,684	55,513	17,096	90,788	24,780
2004	28,417	8,645	57,085	19,969	85,502	28,613
Avg. Annual	38,088	10,909	42,548	11,500	80,636	22,409
Avg. TP Conc.[a]	---	232	---	219	---	225

a. Flow-weighted mean total phosphorus concentration for entire 12-year period.

2.3 C-139 BASIN

The C-139 drainage basin is classified as one of the Western Basins, those basins that drain into the Everglades Protection Area but are west of the EAA. This basin in located south of the S-4 Basin and covers approximately 243 square miles (Mock, Roos & Associates, 1993) (Figure 1-1). The C-139 Basin is bounded on the north and east by the L-1, L-2 and L-3 borrow canals and on the south by the Deer Fence Canal. A map showing portions of the C-139, S-8 and other drainage basins in the southern project area is included as Figure 2-3. The C-139 Basin structures, excluding those associated with STAs 5 and 6, are described below.

The principal control structures and other flow monitoring stations in this basin are discussed in the following sections.

Figure 2-3: Southern Project Area Map

2.3.1 G-135 Culvert

Structure G-135 is a gated culvert that connects the upper reaches of the L-1 Borrow Canal with Canal 6 in the Flaghole Drainage District. SFWMD operating criteria indicate that this culvert is normally closed and opened only as a bypass during high-flow events; however, it has been observed that this culvert is often left open at other times (ADA Engineering, 2006). Although negative flows have been recorded at this structure (that is, flow into the C-139 Basin), these negative flows are relatively insignificant. Most of the flow that passes through this structure enters the Flaghole Drainage District and C-43 Basin. These discharges from the C-139 Basin at G-135 are assumed to continue into the future and are not a target for redirection elsewhere (SFWMD, 2007).

The historic discharge, headwater stage and tailwater stage data for G-135 were retrieved from DBHYDRO using DBKEYs TA285, TA250 and TA251, respectively. There are no water quality data available for this structure in DBHYDRO.

2.3.2 G-96 Culverts

Structure G-96 is a double culvert in the L-1 Borrow Canal, located approximately three miles east of G-135. This structure is equipped with risers for installation of stop logs, but is normally left open (ADA, 2006). The historic flow, headwater stage and tailwater stage data for G-96 were retrieved from DBHYDRO using DBKEYs TA284, TA252 and TA253, respectively. There are no phosphorus concentration data available for this structure in DBHYDRO.

2.3.3 G-136 Culverts

Structure G-136 is a series of culverts that connect the L-1 and L-1 East (L-1E) borrow canals. This structure, G-150 and the L-1E Borrow Canal were constructed in the early 1980s to help alleviate flooding problems in the upper C-139 Basin. The G-136 structure consists of three 84-inch diameter by 80-foot long CMP culverts with invert elevations of 8.0 feet NGVD. The flow in these culverts is controlled by stop logs in risers, with the stop log crest normally set at elevation 13.0 feet. These stop logs are raised to 14.0 feet during dry conditions to help retain water in the L-1 Borrow Canal.

During periods of higher runoff, G-150 may be closed to divert all of the drainage from the upper C-139 Basin — that portion of the basin above G-150 — eastward through G-136 into the L-1E Canal and S-3/S-8 Basin.

The historic flow data for G-136 were retrieved from DBHYDRO using DBKEY 15195. Headwater and tailwater stage data are also available at this structure (DBKEYs TA235 and TS237, respectively). The associated water quality monitoring station (G136) has data for both flow-weighted composite and grab samples. The flow and TP discharge data for G-136 were provided by SFWMD staff (Burns & McDonnell, 2005a).

2.3.4 G-150 Culverts

Structure G-150 is a series of gated culverts located in the L-2 Borrow Canal, immediately south of its intersection with the L-1 Borrow Canal. The L-2W Canal intersects the L-2 Canal immediately below (south of) G-150. The flow through these culverts can be regulated by slide gates located on the south ends of these culverts.

During periods of higher runoff in the C-139 Basin, high stages in the L-2 Borrow Canal serve to restrict southward flow out of the upper basin through G-150, which can cause localized flooding in the upper basin. The critical flood stage in the L-1 Borrow Canal near G-136 is elevation 14 feet NGVD. When G-136 headwater elevations exceed this elevation, the gates at G-150 are closed to prevent backflow into the upper basin and all drainage from the upper basin is routed eastward through G-136 and the L-1E Borrow Canal to the Miami Canal.

The historic flow data for G-150 were retrieved from DBHYDRO using DBKEY 15520. The corresponding headwater and tailwater stage data for G-150 are available for DBKEYs 15521 and 15522, respectively. There has been no regular water quality sampling at structure G-150 during the study period.

2.3.5 L-3 Borrow Canal Structures

All drainage from the lower C-139 Basin, plus any discharge from the upper basin through G-150, is conveyed southward via the L-2 and L-3 borrow canals. Prior to construction of STA-5 in 1999, this drainage continued south toward the common corner between Broward, Hendry and Palm Beach counties. This location is commonly referred to as Confusion Corner because of the many canals that intersect there. At this location, there were three control structures on the L-3 Borrow Canal:

- G-88 — Controls discharge into the L-4 Borrow Canal
- G-89 — Controls discharge into the L-28 Borrow Canal
- G-155 — Controls discharge into the L-3 Canal Extension and Water Conservation Area 3A (WCA-3A).

With the construction of STA-5, most of the drainage from the C-139 Basin — the remaining portion that is not diverted eastward through G-136 — is now diverted into this stormwater treatment area. Structure G-406 was constructed across the L-3 Borrow Canal, immediately south of its confluence with the Deer Fence Canal, to help force this drainage into STA-5. G-406 consists of two 10-foot by 9-foot RCB culverts with slide gates. These culverts are approximately 50 feet long with an invert elevation of 5 feet NGVD.

The gates at G-406 are normally closed. These gates begin to open when headwater stages reach 16.0 feet NGVD in an attempt to maintain the headwater stage at 16 feet. If the bypass rate required to maintain stages at 16 feet exceeds 400 cfs, then headwater stages are allowed to rise to 17.0 feet and bypass to 500 cfs. At a headwater stage of 17.0 feet, the gate opening will be progressively increased until fully open if necessary to maintain headwater stages at 17.0 feet. A 200-foot section of the G-406 embankment has also been lowered and armored to serve as an emergency overflow weir.

The historic flow data for G-406 were retrieved from DBHYDRO using DBKEY JU789. No headwater data are available for G-406; however, there are tailwater stage data for G-406 at DBKEY JJ155. In the years prior to G-406 operation, (5/1/1992 – 6/26/2000), the stage at G-406 was estimated using a modified G-155 headwater relationship. Regression analysis of data for the overlapping period of record (1999–2004) yielded an r^2 of 0.96. The stage data for G-155 head water is DBKEY TA267. Composite and grab sample data for TP concentrations are also available at structure G-406.

Prior to the construction of G-406, the discharge at this location was estimated using other available data. From the start of the study period until the end of calendar year 1995 (5/1/1992 – 12/31/1995), the historic discharge data for G-88, G-89 and G-155 were used to estimate lower C-139 Basin drainage. These discharge data are available from DBHYDRO for DBKEYs P1052, P1053 and P1039, respectively. Starting in 1996, SFWMD began monitoring flow in the L-3 Canal at a station known as L3DF (DBKEY 16243). The flow data at this station were used from 1/1/1996 – 6/25/2000, when G-406 discharge data became available. TP concentrations in these flows were estimated from sampling data for station L3.

2.3.6 C-139 Basin Discharge Summary

A discharge summary for the C-139 Basin structures discussed above is included in Table 2-6. The L-3 station shown in this table contains estimates of discharge in the L-3 Borrow Canal below the Deer Fence

Canal. Prior to the completion of G-406, these flows were estimated initially from the sum of the discharge at G-88, G-89 and G-155, and later from the data for station L3DF.

Table 2-6: Recorded Discharge Summary for C-139 Basin Structures[a]

Station No.	Description	Discharge Destination	Avg. Ann. WY 1993-2004		
			Flow (ac-ft)	TP Load (kg)	TP Conc. (ppb)
G-135	Culvert between C-139 and Flaghole Drainage Dist.	N. to Flaghole S. to C-139	4,152 264	--- ---	--- ---
G-96	Culverts in L-1 Canal	East West	8,102 931	--- ---	--- ---
G-136	Culverts between C-139 & S-3/S-8 Basins	E. into S-3/S-8	17,479	3,522	163
G-150	Culverts between L-1 & L-2 Canals	N. to L-1 S. to L-2	16,727 20,494	--- ---	--- ---
L-3[b]	L-3 Canal below Deer Fence Canal	South	166,723	43,874	213

a. Data collected by SFWMD and retrieved from DBHYDRO.
b. Composite record of data available at G-88, G-89, G-155, L3DF and G-406.

2.4 C-139 ANNEX BASIN

The basin known as the C-139 Annex is the U.S. Sugar Corporation's Southern Division Ranch, Unit No. 1. This agricultural area, which is primarily dedicated to citrus production, is located in southeastern Hendry County. Presently, this area drains to the L-28 Borrow Canal, which conveys this drainage to WCA-3A via pump station S-140. A new pump station, with a capacity of about 450 cfs, is planned that will discharge this drainage eastward into the L-3 Borrow Canal for subsequent treatment in STA-6.

At present, there is only one outlet from this basin to the L-28 Borrow Canal. The discharge at this point, known as USSO, is monitored at an ultrasonic velocity meter (UVM) station in this outlet canal. The discharge records for this monitoring station begin in December 1995, so these data do not cover the entire 12-year study period. For the period December 1995 through April 2004, the average annual discharge at station USSO is as follows:

- Average annual volume: 37,650 acre-feet/year
- Average annual TP load: 4,586 kg/year
- Average TP concentration: 99 ppb

2.5 STORMWATER TREATMENT AREAS 5 AND 6

As already stated, it is intended that all of the drainage from the C-139 and C-139 Annex basins be passed through a STA for treatment prior to being discharged to WCA-3A. STAs 5 and 6 have been constructed to satisfy this goal. These STAs are considered the most likely destination for S-4 Basin diversions also. These STAs are located in the western S-3/S-8 Basin but are discussed herein separately from the rest of that drainage basin, which is described below in Section 2.6.

The original development of STA-5 was completed in 1999 with construction of the first two flowways. These two flowways have an effective treatment area of approximately 4,110 acres. A third flowway, which provides an additional treatment area of 1,985 acres, was constructed in 2006. This new flowway is flow capable but has not yet been placed in service because of the recent drought conditions. A further expansion of STA-5 is planned for the near future. The remaining area between STA-5 and STA-6, which is known as Compartment C, will be converted into additional treatment area by 2011.

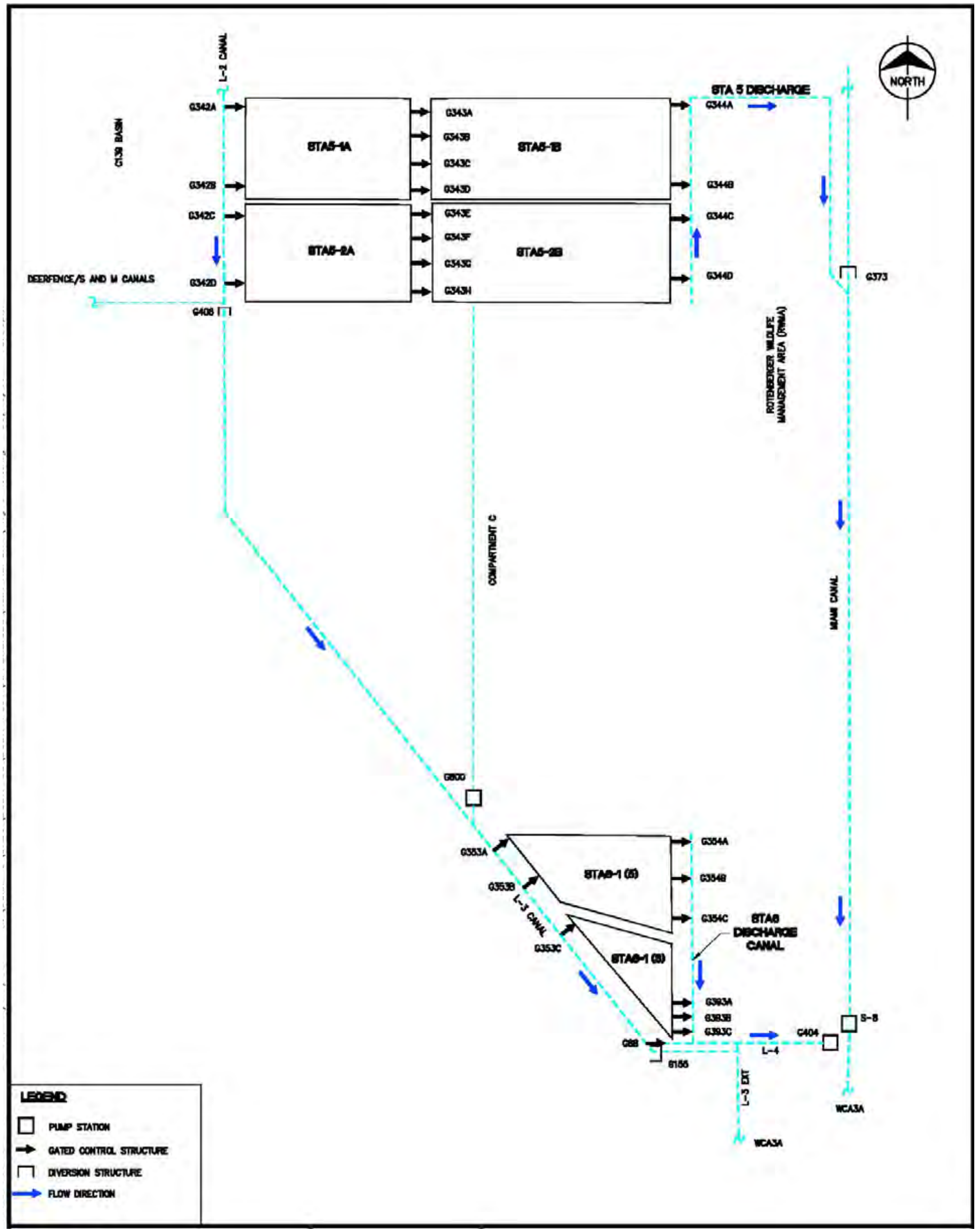
STA-6 is located at the southern end of the L-3 Borrow Canal near Confusion Corner, in the extreme southwestern corner of the S-3/S-8 Basin. The original STA-6 construction was completed in 1996 and provided 897 acres of effective treatment area. A 1,387-acre expansion of STA-6, known as Section 2, was recently completed and is now in its start-up phase. To date, STA-6 has been used to treat drainage from Compartment C, but will be retasked to treat drainage from the C-139 Annex in the immediate future.

Brief descriptions of the current and planned configurations of STAs 5 and 6 are included below. More thorough descriptions of these facilities can be found in URS Corporation (2007).

2.5.1 Existing Phase

The original construction of STAs 5 and 6 has been termed the Existing Phase. A schematic showing the configuration of STAs 5 and 6 and their associated control structures for this phase is included in Figure 2-4. Under this phase, STA-5 included two flowways (STA5-1 and STA5-2), which are divided into two treatment cells each (STA5-1A, STA5-1B, STA5-2A and STA5-2B). The flow through these cells is controlled by the following structures:

Figure 2-4: Existing Phase Schematic for STAs 5 and 6



Adapted from URS Corporation (2007), Figure 6-1.1 (not to scale).

- Inflow control structures G-342A — G-342D: There are four inflow structures, two for each flowway, that control the inflow to STA-5. These gated culverts are constructed through the levee between the L-3 Borrow Canal and STA-5. Each of these culverts consists of a 10-foot by 6-foot reinforced concrete box (RCB) culvert approximately 53 feet long with an invert elevation of 7.3 feet NGVD. Inflow from the L-3 Canal flows by gravity into these two STA-5 flowways. The historic flow through these inflow structures is available from DBHYDRO for DBKEYs J6406, J6398, J6407 and J6405.
- Intermediate control structures G-343A — G-343H: Flow between the A and B cells of STA-5 is controlled by eight control structures. These gated RCB culverts are 10 feet wide by 6 feet high by 60 feet long, with invert elevations of 5 feet NGVD.
- Outflow control structures G-344A — G-344D: The outflow from the STA5-1 and STA5-2 flowways passes into the STA-5 discharge canal through four gated culverts. These structures are 10-foot by 10-foot RCB culverts that are approximately 53 feet long. The inverts of these culverts are at elevation 0.0 feet NGVD.

The STA-5 discharge canal runs north along the eastern boundary of this STA and then turns east, following the northern boundary of the Rotenberger Wildlife Management Area until it meets the Miami Canal. Initially, all discharge from STA-5 entered the Miami Canal at this location.

The Existing Phase of STA-6 is also referred to as Section 1 (STA6-1). Section 1 consists of two cells, Cells 3 and 5 that operate in parallel. Under this phase, the inflow to STA-6 came primarily from Compartment C via the G-600 pump station. The original STA-6 control structures are listed below:

- Inflow control structures G-353A — G-353C: At STA-6, an inflow canal has been constructed parallel to the L-3 Borrow Canal. There are three inflow structures that control the flow between this inflow canal and Cells 3 and 5. G-353A and G-353B control inflow to Cell 5 and G-353C controls inflow to Cell 3. The inflow to these two cells flows by gravity from the adjacent inflow canal.
- Cell 5 outflow control structures G-354A — G-354C: The discharge from Cell 5 is controlled by the three G-354 control structures. The discharge through these structures flows into the STA-6 discharge canal where it travels south into the L-4 Borrow Canal for subsequent delivery to WCA-3A.
- Cell 3 outflow control structures G-393A — G-393C: Near the southern corner of Cell 3 there are three control structures that control the discharge from this treatment cell into the discharge canal.

2.5.2 Interim Phase

The configuration known as the Interim Phase at STAs 5 and 6 includes the initial expansion of these facilities that was completed in 2006. Figure 2-5 is a schematic showing the treatment cells and control structures included in the Interim Phase configuration. For STA-5, this includes construction of the third flowway (STA5-3), with two treatment cells in series, and related control structures:

- Inflow control structures G-342E and G-342F
- Intermediate control structures G-343I and G-343J
- Outflow control structures G-344E and G-344F

The new inflow control structures for the third flowway are located south of the G-406 diversion structure, so the gates in this structure will remain open during interim operations. The function provided by G-406 has been replaced by a new G-407 structure that is located farther downstream near Confusion Corner (labeled G407A on the schematic).

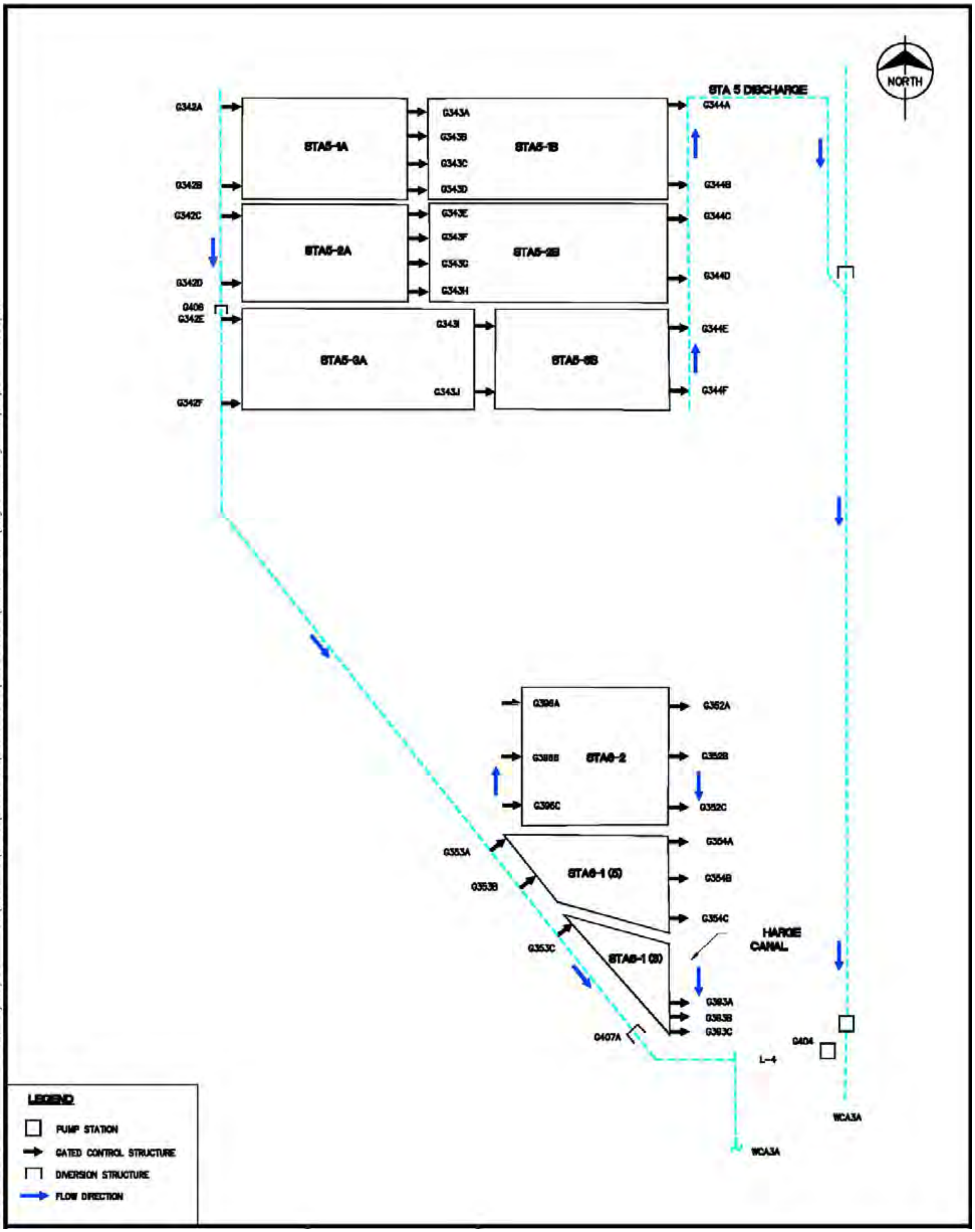
The STA-5 discharge canal was extended to the south to intercept the discharge from STA5-3, but the flow in this discharge canal continues to move north and then east toward the Miami Canal. By 2005, the new STA-5 outlet canal was constructed parallel to the Miami Canal from its confluence with the STA-5 discharge canal to just south of structure G-373. This new canal intercepts the treated discharge from STA-5 and keeps it from becoming mixed with untreated drainage in the Miami Canal that is subsequently diverted through STA-3/4.

For STA-6, the Interim Phase included construction of Section 2 (STA6-2) and six new control structures:

- Inflow control structures G-396A — G-396C
- Outflow control structures G-352A — G-352C

Under interim phase operations, STA-6 will be tasked with treating drainage from the remaining parts of Compartment C and bypasses from STA-5 until the new U.S. Sugar Corporation pump station from the C-139 Annex becomes operational.

Figure 2-5: Interim Phase Schematic for STAs 5 and 6



Adapted from URS Corporation (2007), Figure 6-1.2 (not to scale).

2.5.3 Compartment C Buildout Phase

The final phase of construction at STAs 5 and 6 will see the remainder of Compartment C developed into treatment area. Although the final design of these facilities is not complete, their planning is in a relatively advanced state. The proposed configuration for the Buildout Phase is shown in the schematic in Figure 2-6. For STA-5, the Compartment C Buildout will include construction of two additional flowways (STA5-4 and STA5-5) with two treatment cells each. The associated new control structures are listed below:

- Inflow control structures: G-342G — G-342M (structures G-342G and G-342H will be new inflow structures for STA5-3, replacing the current function of G-342E and G-342F)
- Intermediate control structures: G-343K — G-343O
- Outflow control structures: G-344G — G-344K

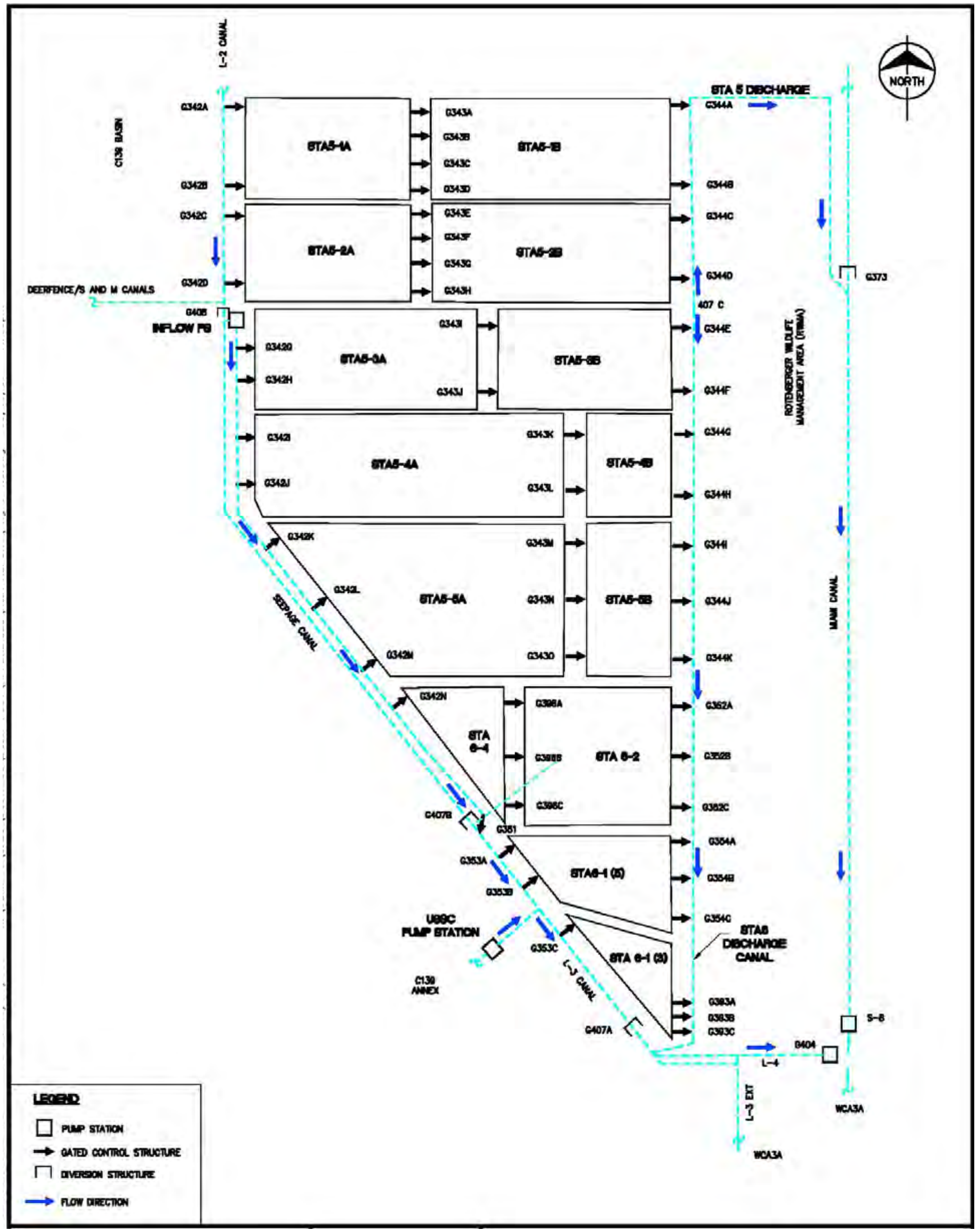
The remaining treatment area included in the buildout phase will be STA6-4. This small, triangular area will serve as a lead-in cell for STA6-2, and require construction of a single new inflow structure, G-342N.

The inflow to STA5-1 and STA5-2 will continue to flow by gravity from the L-3 Borrow Canal, but a new inflow pump station and inflow canal will be constructed to feed the remaining treatment areas in STAs 5 and 6. The addition of the C-139 Annex pump station will also require a new divide structure in the L-3 Borrow Canal, G-408 (labeled as G-407B on the schematic).

On the discharge side, the STA-5 discharge canal will be extended southward to meet the existing STA-6 discharge canal. A new divide structure will be built in this combined discharge canal between G-344D and G-344E. This structure will force outflow from the first two flowways at STA-5 to flow north and direct all other discharge to the south into the L-4 Borrow Canal.

The preceding description is only a brief summary of the planned changes with the Compartment C Buildout. Readers are referred to URS Corporation (2007) for a more comprehensive description of these facilities. As proposed, STAs 5 and 6 will effectively be merged into a single treatment facility after the Compartment C Buildout is complete. In this final configuration, this combined treatment area will be referred to as STA-5/6 in subsequent sections.

Figure 2-6: Compartment C Buildout Phase Schematic for STAs 5 and 6



Adapted from URS Corporation (2007), Figure 6-1.3 (not to scale).

2.6 S-3/S-8 BASIN

The S-3 and S-8 drainage basins are both drained by the Miami Canal so they are normally considered together. These two drainage basins are located respectively east and southeast of the S-4 Basin. The S-3/S-8 Basin, specifically the Miami Canal, serves as a sink for discharges from the L-1E borrow canal and STA-5 discharge canal, so Miami Canal stages are of primarily interest in this study. In the future, this basin could also become a destination for S-4 Basin diversions if additional treatment areas are developed in the S-3/S-8 Basin. These two structures are S-3/S-354 and S-8. The S-3/S-8 Basin is not part of the study area for the S-4 Basin Feasibility Study except to provide information about stages at G-212 and as a comparison for STA-5 discharges.

2.6.1 Structure G-212

Structure G-212 is located at the junction between the L-1E and Miami canals, but there are no monitoring data available for this structure. Therefore, headwater elevation data for the S-3 pump station was used to estimate tailwater stages in the Miami Canal below G-212. The headwater stage data for S-3 Pump Station was retrieved using DBKEY 06633.

2.6.2 STA-5 Discharge and Outlet Canal Stages

Prior to construction of the STA-5 Outlet Canal, the STA-5 Discharge Canal emptied directly into the Miami Canal. Stages in the Miami Canal at this location are available from the nearby "Miami.15" stage gage (DBKEY 03722). There is also no stage monitoring station located at the confluence of the STA-5 Outflow and Miami canals. Therefore, Miami Canal stages below this outflow canal were estimated using headwater elevation data for the G-200A pump station. This pump station is located on the Miami Canal at the northwest corner of the Holey Land wildlife management area, and is the closest structure on the Miami Canal that is also south of control structure G-373. The headwater stage data for G-200A was retrieved using DBKEY 13152.

2.7 METEOROLOGICAL DATA

Meteorological data on historic rainfall and evaporation were also collected for the S-4 and C-139 drainage basins. One station was selected from those available in DBHYDRO to characterize rainfall for the S-4 Basin:

- SFWMD Clewiston Field Station: Station CLEW.FS_R (DBKEY 16696)

Two stations were selected from those available in DBHYDRO to characterize rainfall for the C-139 Basin, and the resultant record was developed from the average of the data at these two stations:

- Paige Ranch- L-2 Borrow Canal, ¼-mile south of G-150: Station PAIGE_R (DBKEY IV151)
- Alico South Canal - a mile west of L-2 Borrow Canal: Station ALICO_R (DBKEY 16224)

The precipitation data for these three stations are summarized in Table 2-7.

Table 2-7: Rainfall Data Summary for S-4 and C-139 Basins

Water Year	Annual Rainfall Depth (inches)			
	S-4 Basin (Clewiston FS)	C-139 Basin		
		Paige Ranch	Alico S. Canal	Average
1993	46.7	57.5	46.3	51.9
1994	45.9	41.8	44.3	43.0
1995	48.6	58.8	60.8	59.8
1996	54.5	58.4	60.9	59.7
1997	45.9	57.6	56.4	57.0
1998	50.1	55.5	53.7	54.6
1999	46.6	44.3	47.6	46.0
2000	60.3	50.4	58.4	54.4
2001	27.1	36.8	38.5	37.7
2002	49.9	47.9	37.9	42.9
2003	46.6	47.8	43.7	45.7
2004	47.9	48.3	33.9	41.1
Avg. Annual	47.5	50.4	48.5	49.5
Min. Annual	27.1	36.8	33.9	37.7
Max. Annual	60.3	58.8	60.9	59.8

Evaporation data were also collected for the study vicinity. Unfortunately, there are no meteorological stations within the S-4 or C-139 basins with continuous evaporation data for the entire study period. The closest such station is located at Pump Station S-5A or at the WRA-1W/ENR (Everglades Nutrient Removal) areas, which are approximately 34 miles east of the study area.

Like rainfall, a composite was developed for the S-4 Basin and for the C-139 basin. Lake evaporation estimates were calculated from the pan evaporation data using a pan coefficient of 76 percent.

The following stations were used to estimate evaporation data for the S-4 Basin:

- Belle Glades Experiment Station: Station BELLE_GL_E (DBKEY 06357)
- Clewiston: Station CLEW_E (DBKEY 06365)
- Caloosahatchee Hurricane Gate: Station HGS1_E (DBKEY 06364)
- Pump Station S-5A: Station S5A_E (DBKEY PN932)

The pan evaporation data for these four stations are summarized in Table 2-8.

Table 2-8: Summary of S-4 Basin Evaporation Data

Water Year	Annual Pan Evaporation (inches)					Lake Evaporation (inches)
	Belle Glade	Clewiston	HGS1	S-5A	Composite	
1993	71.4	44.5	69.9	51.5	45.7	34.7
1994	56.9	59.3	70.6	56.0	60.0	45.6
1995	67.7	68.6	65.7	52.1	69.4	52.7
1996	72.7	37.9	64.8	49.6	70.7	53.8
1997	70.0	54.4	75.5	47.2	83.0	63.1
1998	67.8	42.5	69.1	49.3	71.8	54.6
1999	37.2	18.1	48.6	51.3	65.6	49.9
2000	---	---	---	49.3	61.6	46.8
2001	---	---	---	50.3	59.0	44.9
2002	---	---	---	52.9	62.5	47.5
2003	---	---	---	56.2	57.2	43.5
2004	---	---	---	54.2	60.3	45.8
Avg. Ann.	63.4	46.5	66.3	51.6	63.9	48.6
Min. Ann.	37.2	18.1	48.6	47.2	45.7	34.7
Max. Ann.	72.7	68.6	75.5	56.2	83.0	63.1

Evaporation estimate for the C-139 Basin were developed using the average of the S-4 Basin evaporation estimates show in Table 2-4 and two additional stations farther south:

- S-140 Spillway on Levee L-28: Station S140 SPW_E (DBKEY 06329)
- S-7 Control Structures on North New River: Station S7_E (DBKEY 06384)

These data are summarized in Table 2-9.

Table 2-9: Summary of C-139 Basin Evaporation Data

Water Year	Annual Pan Evaporation (inches)					Lake Evaporation (inches)
	S-140	S-7	South Stations	S-4 Basin	Composite	
1993	70.1	60.8	70.2	45.7	61.0	46.4
1994	70.7	58.9	68.1	60.0	72.2	54.9
1995	71.5	58.4	71.0	69.4	73.6	56.0
1996	72.1	63.3	74.4	70.7	78.8	59.9
1997	71.9	60.0	75.3	83.0	86.5	65.8
1998	69.0	61.3	71.1	71.8	76.5	58.2
1999	75.5	65.8	79.8	65.6	79.6	60.5
2000	75.5	50.6	71.3	61.6	72.6	55.2
2001	76.7	66.9	77.4	59.0	76.4	58.0
2002	66.4	66.3	72.1	62.5	76.3	58.0
2003	69.0	62.0	72.9	57.2	71.6	54.4
2004	73.4	63.2	72.0	60.3	71.2	54.1
Avg. Ann.	71.8	61.5	73.0	63.9	74.7	56.8
Min. Ann.	66.4	50.6	68.1	45.7	61.0	46.4
Max. Ann.	76.7	66.9	79.8	83.0	86.5	65.8

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Section 3.0

Baseline Conditions Modeling

3.0 BASELINE CONDITIONS MODELING

This section of the report documents the hydraulic and water quality models developed to characterize baseline conditions in the S-4 and adjacent drainage basins prior to implementation of the proposed diversion project. These baseline conditions are used as a point of comparison to the projected conditions described in Section 5.0.

3.1 HYDRAULIC MODELING

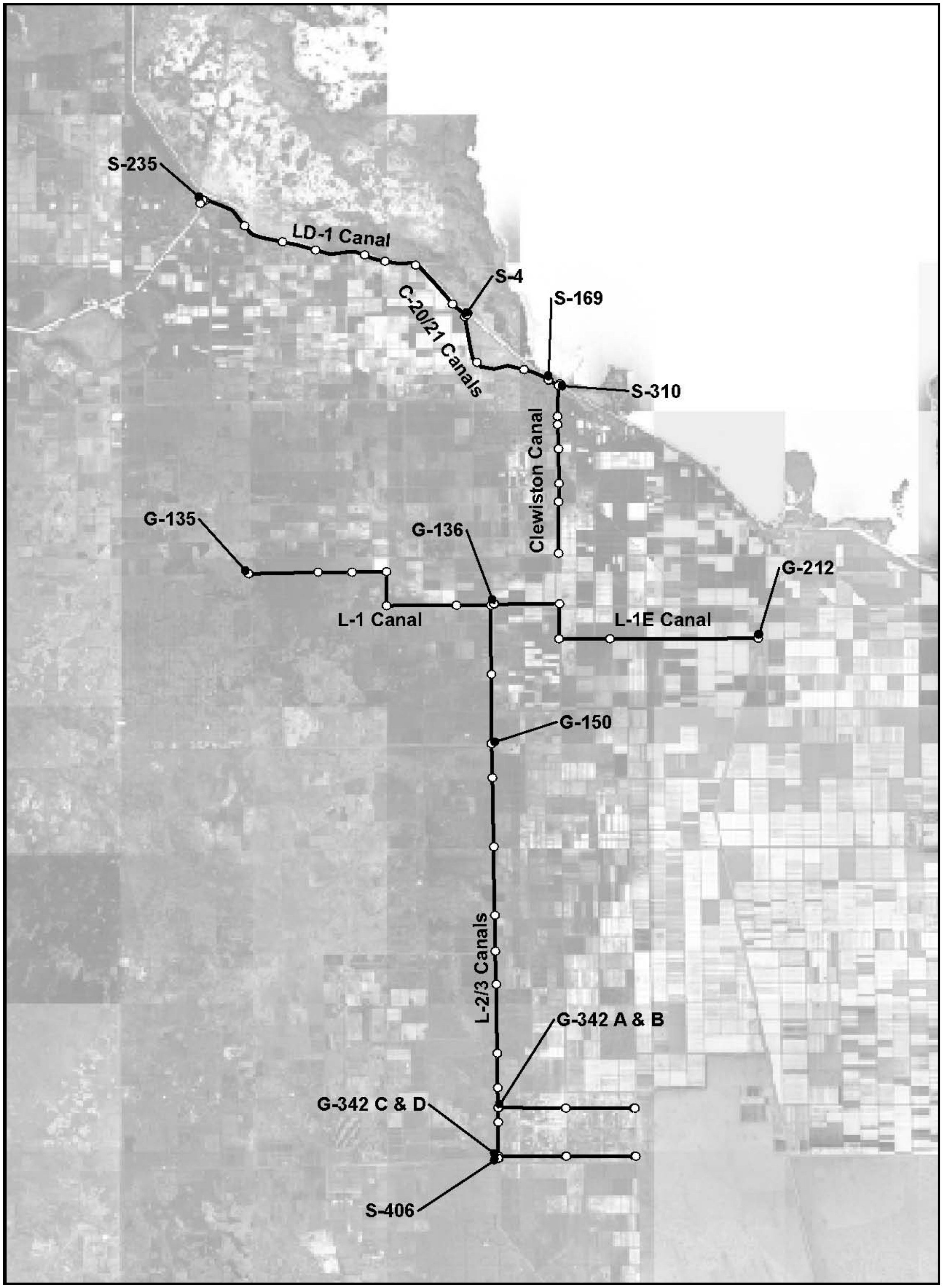
A hydraulic model was developed for the S-4 Basin and portions of the C-139 and S-3/S-8 basins. This model was built using a version of the U.S. Environmental Protection Agency's Storm Water Management Model (SWMM) that was developed by XP Software (XPSWMM). XPSWMM is a dynamic rainfall-runoff simulation model used for single event or long-term (continuous) simulation of runoff quantity and quality from urban and non-urban areas. For this project, only the hydraulic simulation capabilities of XPSWMM were used to route runoff quantities through the primary drainage canals within the project area. The hydrologic routines for runoff generation and the water quality modeling capabilities of XPSWMM were not used.

XPSWMM uses nodes (connection points such as structures and canal intersections) and links (conduits such as channels and pipes) to represent the hydraulic system. It routes flow through this system by solving the St. Venant equations for each model time step. For this project, a five-minute time step was used. XPSWMM is able to route flow through the drainage network, modeling the effects of backwater, flow reversal, surcharging, looped connections, and pressure flow. With the optional real time control (RTC) module, XPSWMM can also be programmed to operate pump stations, gates, and weirs using complex logic based on velocity, flow or water level sensors.

3.1.1 Canal Network

As stated above, only the primary drainage canals within the affected basins were explicitly modeled in XPSWMM. These canals are shown on the XPSWMM schematic included as Figure 3-1 and are listed below:

Figure 3-1: XPSWMM Schematic for Existing Conditions



- S-4 Basin
  - Clewiston (Industrial) Canal from U.S. Sugar Corporation's (USSC) Southline Pump Station (EPD-06) north to Lake Okeechobee at S-310 lock
  - C-21 and C-20 canals, including S-169 control structure, from confluence with Clewiston Canal on east to S-4 Pump Station
  - LD-1 perimeter canal between C-20 Canal on east and C-43 Canal at S-235 control structure on west
- C-139 Basin
  - L-1 Borrow Canal from downstream (east) of G-135 to G-150
  - L-2/L-3 borrow canals from G-150 to south of G-406
- S-3/S-8 Basin
  - L-1E Borrow Canal from G-136 to Miami Canal at G-212 structure

None of the other tributary canals within these basins were modeled explicitly, although a few of these tributaries within the Sugarland Drainage District and U.S. Sugar Townsite area were added later in the XPSWMM models developed for the project alternatives. The inflow from or outflow to these tributaries were modeled as point sources or sinks at the locations where these tributaries join the primary basin canals.

Also, flow through the existing and proposed STA-5 and STA-6 flowways was not modeled explicitly in the hydraulic model. Only the flow through the corresponding inflow structures (G-342A – G-342D and proposed new STA-5/6 inflow pump station) were included in the XPSWMM model. The reason for this simplification is discussed in Section 3.1.4.2 below.

Within the S-4 Basin, representative canal cross sections for the above canals were obtained from recent surveying by Johnson-Prewitt and Associates (JPA). JPA surveyed cross sections in the Clewiston, C-20, C-21 and LD-1 canals approximately every mile. Cross sections for selected internal canals and laterals within the Sugarland Drainage District were also surveyed for later use in the alternatives models.

For C-139 and S-3/S-8 basin canals, cross section data were obtained from as-built drawings provided by SFWMD. These data were supplemented by additional canal survey information for the L-1, L-2 and L-3 canals collected by A.D.A Engineering (2007).

3.1.2 Control Structures

The canal network used in the baseline XPSWMM model includes several inline and terminal control structures that are included in the XPSWMM model. These structures, which are all described in Section 2.0, are listed below by basin:

- S-4 Basin
  - S-310 lock at junction of Clewiston Canal and Lake Okeechobee
  - S-169 culverts in C-21 Canal
  - S-4 Pump Station between C-20 Canal and Lake Okeechobee
  - S-235 culverts between LD-1 and C-43 canals
- C-139 Basin
  - G-96 culverts in L-1 Borrow Canal
  - G-136 culverts between L-1 and L-1E borrow canals
  - G-150 culverts in L-2 Borrow Canal immediately south of intersection with L-1 Borrow Canal
  - Inflow structures from L-3 Borrow Canal into STA5-1 and STA5-2 flowways (G-342A – G-342D)
  - Proposed new inflow pump station between L-3 Borrow Canal and proposed inflow canal for STA5-3, STA5-4, STA5-5, STA6-1 and STA6-2 flowways (included in Baseline Conditions model but not Existing Conditions model)
  - G-406 culverts in L-3 Borrow Canal
- S-3/S-8 Basin
  - G-212 culverts between L-1E and Miami canals

Additional control structures, both existing and proposed, were later added to the XPSWMM models for the alternatives (Section 5.0). Where applicable the published operating rules for these structures are also described in Section 2.0. However, some of these default rules were adjusted as part of the model calibration process. Where applicable these modified operating rules are described below.

3.1.3 Historic Runoff Data

The simulation period for the hydraulic model was selected to match that of the EPD-collected monitoring data for the S-4 Basin. The 12-year period of record for these data includes water years 1993 to 2004 (5/1/1992–4/30/2004). The data for the ten EPD monitoring stations and for several other SFWMD-monitored structures within the project area were used to estimate the runoff data used in the hydraulic model. All of these data, whether compiled by the EPD or SFWMD, are in the form of average daily flow rates. The methods used to develop the inflow/outflow hydrographs used in the hydraulic model varied by basin, as discussed below.

3.1.3.1 S-4 Basin

With limited exception, most of the historic discharge to the primary canals within the S-4 Basin comes from the pump stations that were included in the EPD's monitoring program. The discharge record at each of these EPD monitoring stations was used as an inflow hydrograph to the respective canal segments in the S-4 Basin. Unfortunately, there are no corresponding records for irrigation withdrawals from these canals so these withdrawals were accounted for using a water balance term described below.

Most of the flow out of and into the S-4 Basin for the model simulation period was recorded at one of the three basin outlet structures: S-310 lock, S-4 pump station and S-235 culverts. Structure S-169 serves as a divide structure that separates the eastern (Clewiston Canal) portion of the basin from the western portion (C-20, C-21 and LD-1 canals). As discussed in Section 2.2.9 (Table 2-2), flows at these basin boundary structures do not always balance with recorded discharges at the EPD monitoring stations. This is partially caused by irrigation withdrawals, which are not measured. These imbalances proved to be problematic in developing a hydraulic model that can replicate existing conditions. To alleviate some of the problem, the flow records at S-310 and S-235 were adjusted to ignore basin inflow (that is, the flow at these structures was assumed to be zero on days with a net inflow). Within the east and west subbasins, a new series of water balance terms were calculated and introduced as an additional inflow/outflow hydrograph to the corresponding canal segments.

3.1.3.2 C-139 Basin

Within the C-139 Basin, the available flow monitoring stations were discussed in Section 2.3. Unlike the S-4 Basin, there are no direct measurements of drainage into the primary C-139 Basin canals (L-1, L-2 and L-3 borrow canals). The apparent runoff into these canals was estimated strictly based on daily water balance terms calculated from the flow data at basin boundary structures. Similar to the S-4 Basin, the C-139 Basin was divided into two subbasins: a northern subbasin tributary to the L-1 Borrow Canal above G-150 and a southern subbasin south of G-150 that is tributary to the L-2 and L-3 borrow canals. Separate water balance terms were calculated for each subbasin.

These subbasin water balance terms can be negative on days with significant irrigation withdrawals. Again, this proved to be problematic in the hydraulic model because it assumes that STA-5 and its related control structures are present throughout the 12-year simulation period. During the early years of the simulation period — before these STA-5 structures were actually constructed — they interfered with historical flow patterns, making it impossible at times to satisfy irrigation demands (that is, negative runoff). To resolve these modeling problems, negative water balance terms were zeroed.

Within each subbasin, the apparent runoff each day was distributed to specific canal segments based on estimates of contributing drainage area. These flow distributions are summarized in Table 3-1.

Table 3-1: Distribution of C-139 Basin Runoff

C-139 Subbasin	Inflow Hydrograph Location	Contributing Drainage Area[a] (acres)	Percent of Subbasin Runoff[b]
North of G-150	Above G-135	3,607	19
	Between G-135 & G-96	3,891	20
	Between G-96 & G-136	6,313	33
	Between G-136 & G-150	5,303	28
	North Subbasin Total	19,114	100
South of G-150	At L-2W Canal confluence	26,481	18
	At Midway Canal confluence	11,601	8
	At Alico South Canal confluence	40,416	28
	At G-342A	8,587	6
	At S&M Canal confluence	19,175	13
	At Deer Fence Canal confluence	36,970	26
	South Subbasin Total	143,230	100

a. Estimated from drainage area data presented in A.D.A. Engineering, 2007.
b. Portion of total subbasin runoff that accrues at specified point. Totals may not add to 100 percent because of rounding.

3.1.4 Model Boundary Conditions

Historic water elevation data (stages) at the locations listed below were used as boundary conditions in the hydraulic model:

- Lake Okeechobee near S-310 lock
- S-4 Pump Station tailwater in Lake Okeechobee
- S-235 tailwater in C-43 Canal
- Miami Canal at G-212
- G-406 tailwater in L-3 Borrow Canal
- G-342A&B tailwater in STA-5 cell 1A

- G-342C&D tailwater in STA-5 cell 2A

Historic mean daily stage data for the entire 12-year simulation period are available from DBHYDRO for many of these locations. The exceptions are for those structures that did not exist at the start of this period: G-406 and the four G-342 structures. These latter structures were developed in conjunction with STA-5, which began operation in 1999.

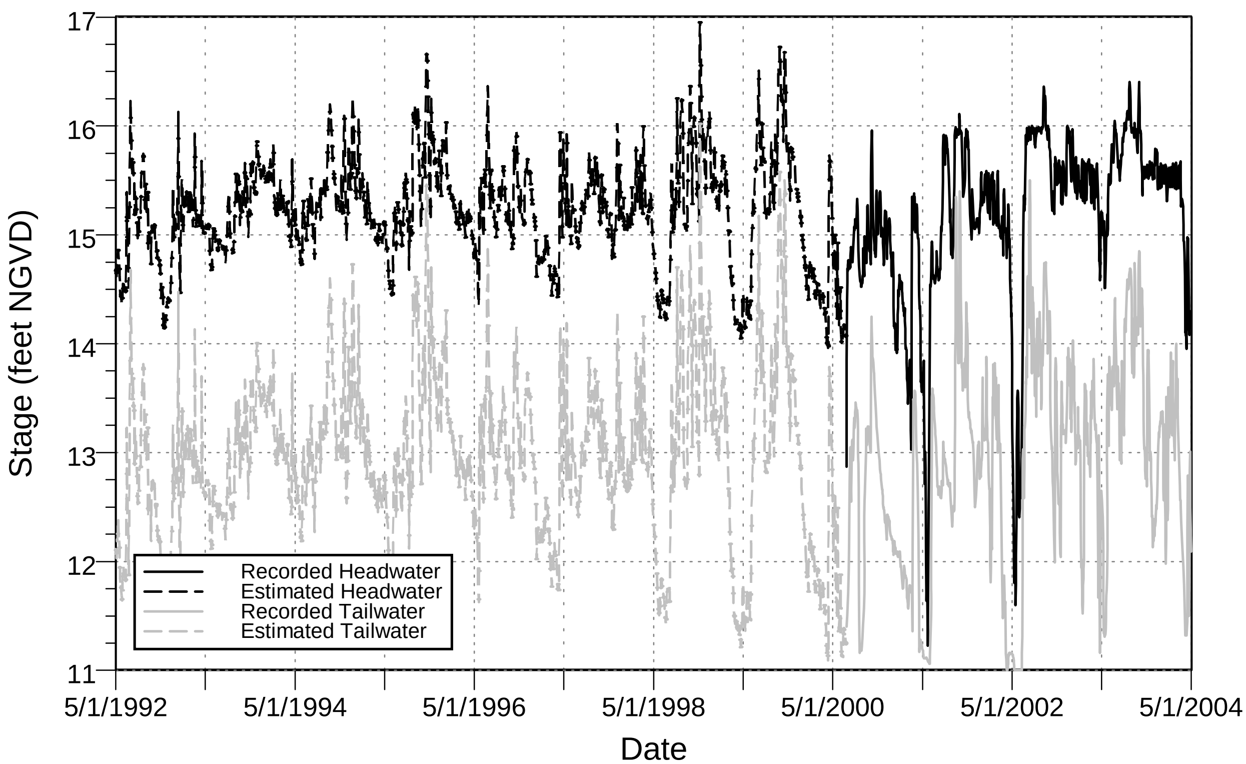
3.1.4.1 G-406 Tailwater

In DBHYDRO, the record for tailwater stage data at G-406 begins on June 26, 2000. For the balance of the model simulation period, these G-406 tailwater data were synthesized using other available stage data for the L-3 Borrow Canal. The only downstream structure that has stage information directly comparable to G-406 tailwater stages and data for the entire 12-year period is G-155. This weir (spillway) structure is located at Confusion Corner where the L-3 and L-3 Extension canals meet. The regression analyses that compared G-406 tailwater and G-155 headwater stages for their overlapping periods of record have a coefficient of determination (r^2) of 0.96. Thus, G-406 tailwater stages for the period prior to June 26, 2000, were initially synthesized using G-155 headwater stages.

Subsequent comparison of the simulated versus measured G-406 tailwater stages (pre-STA vs. post-STA period) showed them to be generally higher in the early pre-STA years. This discrepancy is explained by the reduced flow in the lower L-3 Borrow Canal between G-406 and G-155 after STA-5 and G-406 became operational. With less bypass flow in the L-3 Borrow Canal, headwater stages at G-155 would be lower and water surface gradients between G-155 and G-406 also reduced (that is, flatter).

Therefore to more accurately estimate what the G-406 tailwater would be, a comparison between measured G-406 tailwater and headwater stages was required. Unfortunately, no G-406 headwater data exists so headwater stages at G-342D were used as a surrogate. G-342D is located less than 300 feet upstream of G-406. In order to simulate the G-406 headwater and tailwater in the pre-STA period, a visual comparison of each variable's characteristics during the post-STA period was performed. From this comparison, a relationship between G-406 headwater and tailwater stages was developed and used to adjust the simulated G-406 tailwater stage estimates that were initially developed based on G-155 data. These G-406 tailwater stage estimates are not perfect but because this structure is normally closed, small errors in these estimates should not significantly impact the modeling results. Figure 3-2 is a graph of the recorded and estimated headwater and tailwater stages at G-406. This graph shows that the estimated data compares favorably with the actual recorded data. Only the G-406 tailwater stage data were used as boundary conditions in the XPSWMM model.

Figure 3-2: G-406 Headwater and Tailwater Stage Data (feet NGVD)



3.1.4.2 G-342 Tailwater

Initial attempts to simulate the flow through the initial two flowways at STA-5 were not very successful. That is, this flow could not be predicted readily based on the difference between inflow stages in the L-3 Borrow Canal and outflow stages in the STA-5 discharge canal. Therefore, analyses were conducted to find an alternate method for modeling STA-5 inflow. These analyses found that tailwater stages at the G-344 outlet structures correlated well with stages in the Miami Canal, but upstream, the correlation yielded an r^2 of 0.01. Several different types of analyses were performed to determine how STA-5 can be modeled with respect to the Miami Canal; the drop in stages as flow progresses from west to east was found to be uneven, with little relationship to the fluctuations in the Miami Canal. The initial results from the XPSWMM modeling effort yielded little success in mimicking STA-5, thus the XPSWMM model boundary condition was moved upstream to the G-342 tailwater stages.

Extensive analyses of the G-342 tailwater stages (of both flowways averaged) showed that two separate relationships existed. The first relationship showed that when the STA-5 flow (total for G-342A-D) is high (generally greater than 150 cfs), the correlation between the G-342 tailwater stages and the G-406 headwater stages yielded an r^2 of 0.95. The second relationship showed that when the total STA-5 flow is generally less than 150 cfs, the correlation between the G-342 tailwater stages and the Miami Canal (Miami.15 as described in Section 2) yielded an r^2 of 0.43. Using the equations developed showing both relationships, it was found that the correlation between the simulated and actual G-342 tailwater stages yielded an r^2 of 0.86. Thus, G-342 tailwater stages were synthesized for the pre-STA period (5/1/1992–6/25/2000) based on the Miami Canal stages and the simulated G-406 headwater values developed earlier. The last step required was determining the average tailwater stage for each flowway, which was based on the average G-342 stages through comparison of the actual data (6/26/2000–4/30/2004).

3.1.5 Existing Conditions Models

The initial effort to develop a hydraulic model focused on simulating historic or existing conditions within the affected drainage basins. There is currently no hydraulic connection between the S-4 and C-139 drainage basins so separate XPSWMM hydraulic models were developed for each of these basins.

3.1.5.1 S-4 Basin Model

As stated above, only the principal canals and control structures within the S-4 Basin were represented in the hydraulic model. Calibrating this model to match historic conditions was complicated by the following limitations:

- Historic discharge and stage data are mean daily values: This limitation was probably the most significant because the available historic discharge and stage data do not necessarily represent peak conditions each day. This problem makes it difficult to accurately simulate basin performance because peak discharges and canal stages are dampened by the daily averaging process.

- Irrigation and other canal withdrawals are not monitored: As stated above, there are no records of irrigation withdrawals so these withdrawals can only be inferred by examining daily canal water balances. To combat this problem, the historic basin runoff data were adjusted to eliminate any inflow to the S-4 Basin, which theoretically excludes irrigation from the hydraulic simulation.

- Control structure operation does not rigorously follow published rules: The published operating rules for principal basin control structures (S-310, S-169, S-4 and S-235) are discussed in Section 2.2 but these rules are not rigorously followed. A primary example of this discrepancy is in the operation of the S-310 lock structure. The operating rule states that the lock is closed whenever Lake Okeechobee stages exceed 15.5 feet NGVD; however, review of recorded stage data for the lake and Clewiston Canal show this lock is more typically closed at lake stages near 16.0 feet. Also, the discharge data for this same structure show basin inflow from the lake when the lock is obviously closed.

Because of the first and last modeling limitations discussed above, the published structure operating rules were used only as guidelines in the hydraulic model. These rules were then adjusted as necessary to calibrate the model to existing conditions. The model was considered to be calibrated when the modeled basin discharge at S-310, S-4 and S-235 closely matched historic values on a long-term basis. Table 3-2 presents a comparison of historic and modeled discharge data for these three structures. These same data are shown graphically in Figure 3-3.

Table 3-2: Comparison of S-4 Basin Discharge for Existing Conditions[a]

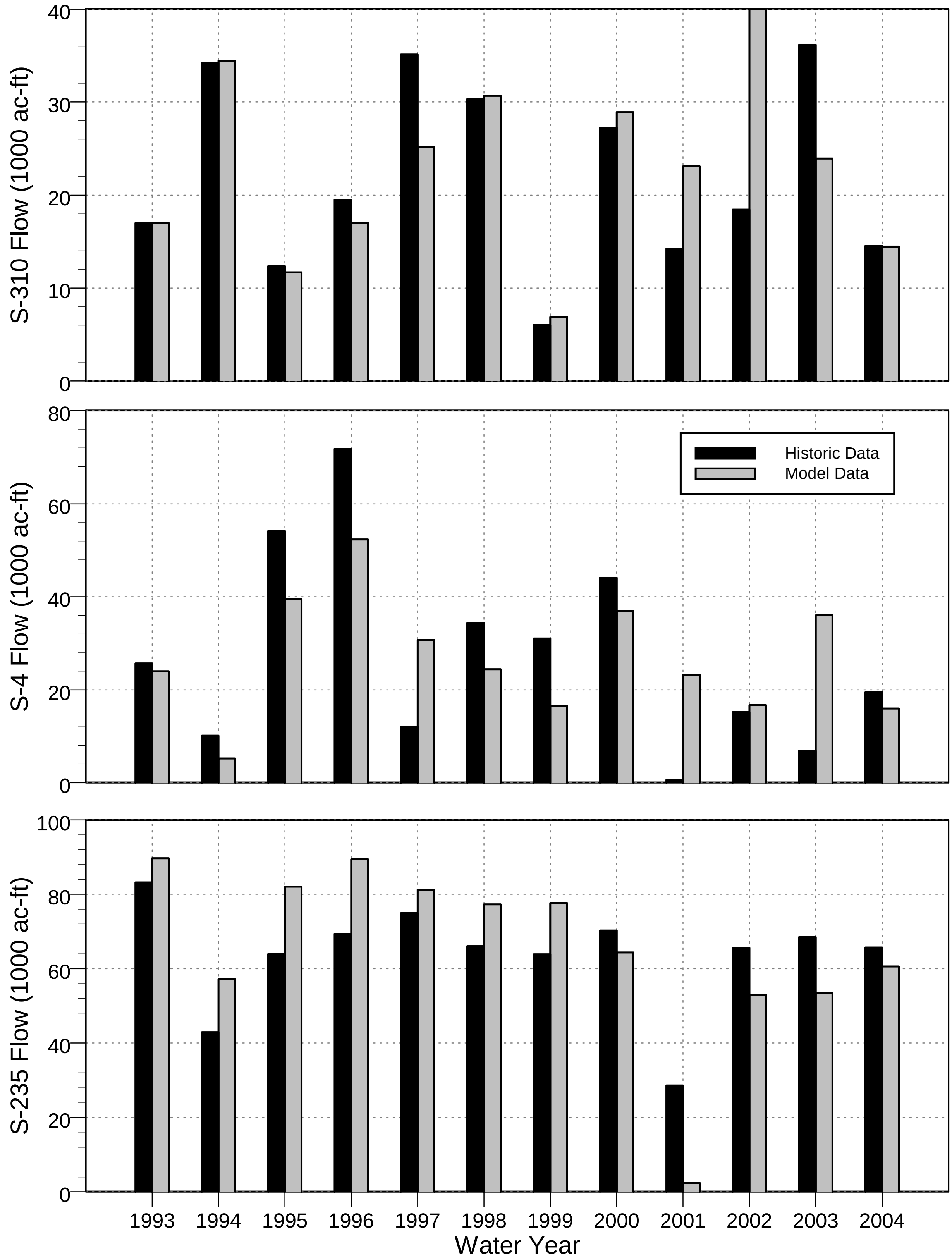
Water Year	L Okeechobee @ S-310[b]		L Okeechobee @ S-4		C-43 Canal @ S-235[b]	
	Historic	Modeled	Historic	Modeled	Historic	Modeled
1993	16,991	17,017	25,645	24,009	83,176	89,677
1994	34,243	34,427	10,094	5,208	42,964	57,147
1995	12,352	11,677	54,158	39,478	63,971	82,034
1996	19,484	17,010	71,805	52,302	69,418	89,386
1997	35,119	25,174	12,074	30,743	74,921	81,222
1998	30,328	30,685	34,304	24,375	66,034	77,329
1999	6,036	6,885	30,985	16,555	63,871	77,665
2000	27,252	28,923	44,070	36,915	70,274	64,383
2001	14.250	23,094	649	23,213	28,571	2,376
2002	18,438	40,873	15,212	16,687	65,568	52,961
2003	36,152	23,918	6,870	35,975	68,499	53,535
2004	14,548	14,467	19,456	15,960	65,689	60,578
Total	265,193	274,151	325,320	321,421	762,956	788,291

a. All discharge volumes in acre-feet per year.

b. Basin inflow at these structures ignored for historic data. Modeled data are net discharges at these structures.

Review of Table 3-2 or Figure 3-3 shows that historic and modeled discharges at the three S-4 Basin discharge structures match closely in many years; however, there are a few significant exceptions. The water years with the largest mismatches tended to be those when Lake Okeechobee stages were low for some or all of the year. The worst such water year was 2001, a drought year, when Lake Okeechobee stages ranged from about 13 feet to less than 10 feet NGVD. Although the model was less successful at replicating conditions during droughts, potential diversions are also near zero at these times.

Figure 3-3: Annual S-4 Basin Discharge Comparison for Existing Conditions

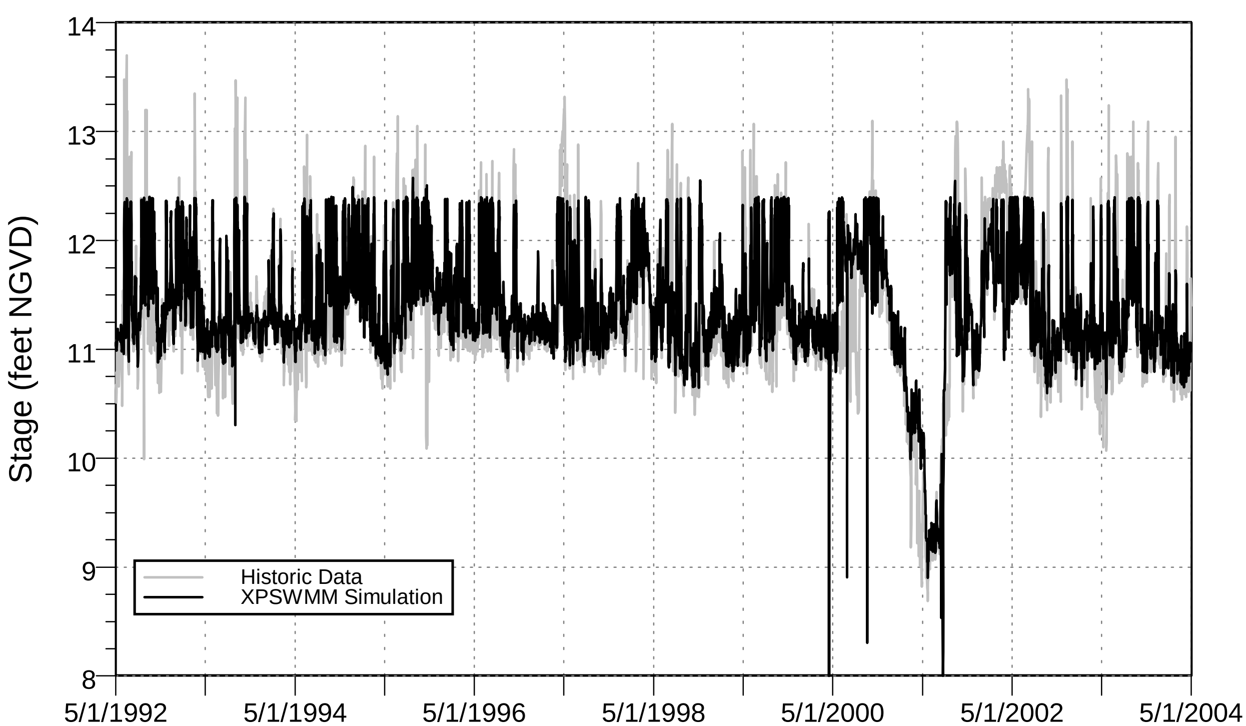


Although there are some water years with significant differences between historic and modeled flows, the XPSWMM model was calibrated based on total discharges for the entire 12-year simulation period. As shown in Table 3-2, the total simulated discharge from the XPSWMM model at the three S-4 Basin discharge structures closely matches the corresponding historic values. On a percentage basis, the total modeled discharge volumes for the 12-year simulation period compare as follows to historic values:

- Discharge to Lake Okeechobee
    - At S-310 lock: +3.4 percent
    - At S-4 pump station: -1.2 percent
    - Total (S-310 + S-4): +0.9 percent
- Discharge to C-43 Canal at S-235: +3.3 percent

Discharge volumes were the primary metric used to calibrate the XPSWMM model but stage data were also compared. Figure 3-4 is a plot that compares the recorded historic headwater stage data at the S-4 Pump Station with simulated stages. Other than a few anomalies during drought conditions when simulated stages were very low for brief periods, these stages appear to be similar. Peak simulated stages are lower than historic stages because of the adjustments made to the station operating rules (see below).

Figure 3-4: S-4 Pump Station Headwater Stages for Existing Conditions



To achieve the close agreement between these total historic and modeled discharges, the operating rules for the some of the basin's control structure were adjusted from the published rules as follows:

- S-310 Lock: As stated in Section 2.0, the S-310 lock should be closed whenever Lake Okeechobee stages exceed 15.5 feet NGVD. Review of the historic data for the 12-year simulation period shows this lock was more typically closed at lake stages between 15.75 and 16.0 feet. In the XPSWMM model, a Lake Okeechobee elevation of 15.8 feet was adopted for closure of S-310.

- S-4 Pump Station: The operating criteria for the S-4 pump station states that the pumps should be started whenever its headwater stages exceed 14.0 feet NGVD and stopped after headwater stages fall below 11.0 feet NGVD. In the XPSWMM model, the first S-4 pump was started at headwater elevation 12.6 feet and stopped at elevation 12.2 feet. The other two pumps were started at elevation 14.2 and 14.4 feet and both stopped at an elevation of 13.8 feet.

The operating rules used in the XPSWMM model for the S-169 and S-235 culverts closely approximated the published operating rules.

3.1.5.2 C-139 Basin Model

A similar Existing Conditions model was developed for the C-139 Basin. Many of the same modeling limitations described above, such as use of mean daily flow data and no records of irrigation withdrawals, apply to this basin as well.

Some of the key features in this basin, namely STA-5 and G-406, were constructed in 1999 and 2000 after the start of the model simulation period. The flow monitoring data for STA-5 (G-342A-D) and G-406 begin respectively on October 1, 1999, and June 26, 2000, which implies that they began operation on these dates. These new structures fundamentally altered the hydraulics of the lower C-139 Basin so the "historic" flow and stage data for the pre-STA era in the lower C-139 Basin were adjusted to account for these impacts. Readers should remember when comparing historic versus modeled data for these structures before water year 2001, that these historic data are themselves estimates.

For the C-139 Basin, model calibration consisted primarily of adjusting the operating rules for G-406, specifically the relationship between allowable bypass discharge and headwater stage. The resulting annual flow volumes at the key model discharge points — G-136, STA-5 (G-342A-D) and G-406 — are summarized in Table 3-3. These same data are shown graphically in Figure 3-5.

Table 3-3: Comparison of C-139 Basin Discharge for Existing Conditions[a]

Water Year	L-1E Canal @ G-136		STA-5 @ G-342A-D		L-3 Canal @ G-406	
	Historic	Modeled	Historic[b]	Modeled	Historic[b]	Modeled
1993	11,671	12,193	101,326	124,438	32,973	17,252
1994	20,113	37,066	112,612	115,051	6,948	227
1995	35,986	44,968	229,006	298,984	36,578	27,923
1996	20,790	50,924	165,255	205,830	99,007	78,884
1997	13,091	28,569	142,914	140,629	14,076	8,628
1998	20,776	55,673	152,080	129,802	4,366	718
1999	13,734	71,150	110,612	75,727	16,456	31,052
2000	24,859	69,924	147,426	123,493	33,579	59,007
2001	3,294	4,163	52,038	57,190	2,645	875
2002	17,062	21,370	158,673	157,935	32,149	26,100
2003	15,154	26,988	170,177	164,914	36,450	47,338
2004	13,221	19,474	152,984	140,602	35,655	51,024
Total	209,753	442,461	1,695,103	1,734,564	350,881	349,026

a. All discharge volumes in acre-feet per year.

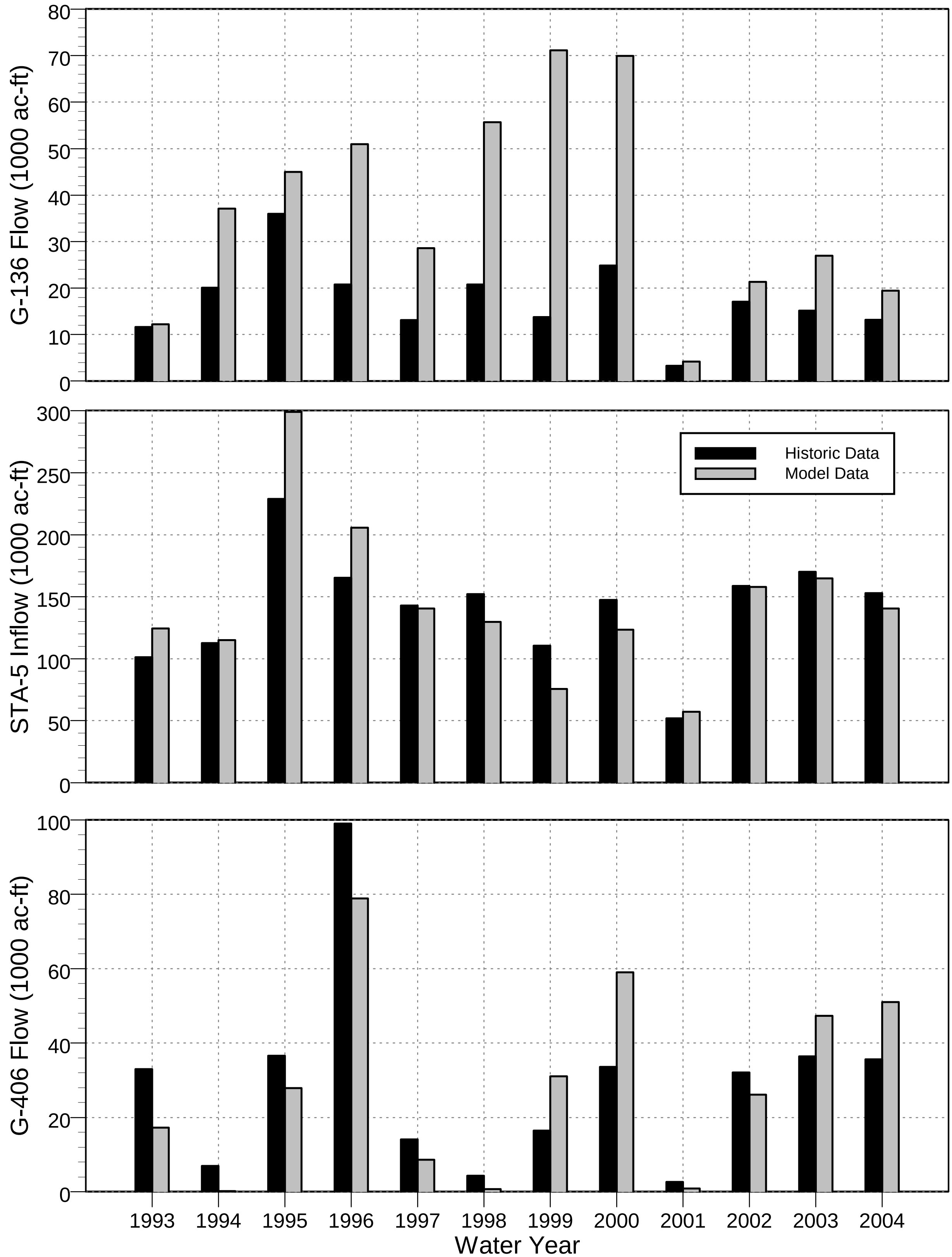
b. Historic inflow to STA-5 and flow at G406 are based on estimates prior to 10/1/1999 and 6/26/2000, respectively.

Review of Table 3-3 or Figure 3-5 shows that modeled discharge at G-136 is significantly higher than the estimated historic values, especially during the early years of the simulation period. These higher flows occur because the hydraulic model simulates the presence of the G-406 structure in these years before it was actually constructed. With G-406 closed much of the time, stages in the L-2 Borrow Canal would have been typically higher than under actual historic conditions. These higher stages south of G-150 restrict southward flow through this structure so more drainage from the upper C-139 Basin must be discharged at G-136. In total, the modeled G-136 discharge is more than double historic values (211 percent).

In Figure 3-4, inflow to STA-5 is shown to be a close match between historic and modeled values in most years. Overall, the modeled inflow to STA-5 is about 2.3 percent higher than historic.

The total simulated STA bypass through G-406 is within a half percent of historic totals. However, from year to year the annual totals are often quite different (Figure 3-5). Most of the calibration effort for the C-139 hydraulic model was expended adjusting G-406 bypass volumes. These flows were adjusted by incrementally changing the G-406 operating criteria. The existing operating rules for this structure are

Figure 3-5: C-139 Basin Discharge Comparison for Existing Conditions



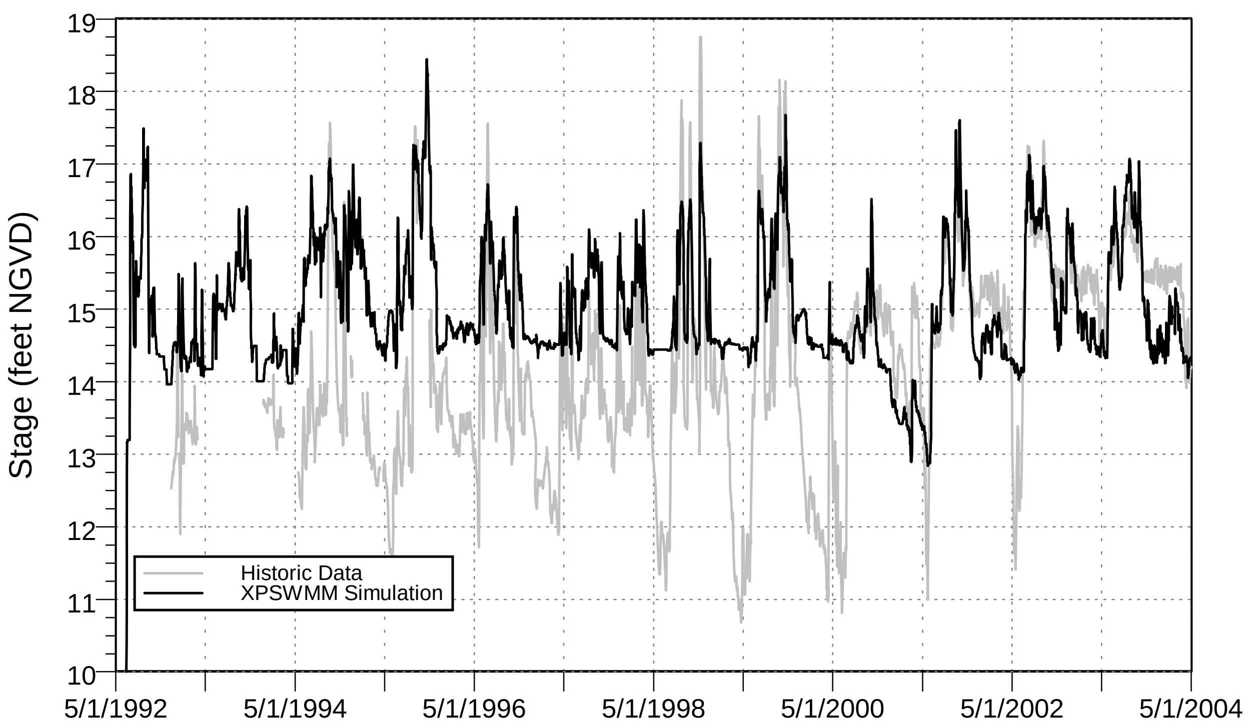
described in Section 2.3.5. In the XPSWMM model, G-406 bypass flows were simulated using the following gate opening schedule:

- G-406 headwater elevation ≤ 15.9 feet NGVD – Gates closed
- G-406 headwater elevation = 16.0 feet NGVD – Gates open 16 percent
- G-406 headwater elevation = 16.2 feet NGVD – Gates open 80 percent
- G-406 headwater elevation ≥ 16.5 feet NGVD – Gates open 100 percent

G-406 is a double box culvert structure so it has two gates. In XPSWMM both gates are assumed to be open the same amount. For headwater elevations between those listed above the gate opening is assumed to vary linearly.

Discharge volumes were the primary metric used to calibrate the XPSWMM model but stage data were also compared. Figure 3-6 is a plot that compares the recorded historic headwater stage data at structure G-150 with simulated stages from the hydraulic model. The headwater side of G-150 is assumed to be on

Figure 3-6: G-150 Headwater Stages for Existing Conditions



the south in the L-2 Borrow Canal. This graph shows there is relatively close agreement between the historic and modeled stages when these stages are above approximately 15 feet NGVD. Simulated stages rarely fell below about 14 feet but recorded stages were as low as about 11 feet. Most of the low G-150 headwater stage readings occurred in the early years of the simulation period before STA-5 and G-406 were constructed. In the XPSWMM model, G-406 was assumed to exist for the entire period so this structure would tend to prevent low headwater stages at G-150.

3.1.6 Baseline Conditions

Baseline Conditions are defined as those that will exist in the future at the time S-4 Basin diversions may begin. Baseline conditions within the S-4 Basin itself are identical to the existing conditions that are described above. Within the C-139 Basin, the only change from existing to baseline conditions is the planned expansion of STA-5 within Compartment C. In the XPSWMM model, STAs 5 and 6 are not modeled explicitly so the only addition to the hydraulic model necessary to represent baseline conditions is the planned new STA-5 inflow pump station. This pump station will be located immediately north of G-406 and discharge water into a new inflow canal that will supply all of the treatment area south of G-406 (flowways STA5-3, STA5-4, STA5-5, STA6-1 and STA6-2).

The principal impact of the change from existing to baseline conditions is to increase total STA-5/6 inflow while reducing the volume of required bypass through G-406 (or its successor structures G-407 and G-408). The resulting annual STA-5/6 inflow and bypass volumes are compared in Table 3-4. These same data are shown graphically in Figure 3-7.

In the baseline model it was assumed to be desirable that the inflow to each flowway would be balanced after completion of the Compartment C buildout. With recognition of the addition STA-5/6 inflow that will come from the C-139 Annex, achieving a balanced inflow requires that approximately 35 percent of C-139 Basin runoff be routed through the STA5-1 and STA5-2 flowways (G-342A-D) and the remainder (about 65 percent) through the new inflow pump station to the southern flowways. These inflows were balanced by adjusting the operating criteria for the new inflow pump station. These revised operating criteria are discussed further below.

Review of Table 3-4 shows that inflow to the first two flowways at STA-5 will decrease by an average of about 80,000 acre-feet per year after the Compartment C buildout. Deliveries to the southern flowways from the new pump station are predicted to average about 109,000 acre-feet annually. The differences between this average inflow decrease to the upper flowways and increase to the lower flowways (about

Table 3-4: STA-5/6 Inflow and Bypass for Existing and Baseline Conditions[a,b]

Water Year	STA5-1 & STA5-2 @ G-342A-D		Balance of STA-5/6 @ Inflow Pump Station		STA-5/6 Bypass @ G-406	
	Existing	Baseline	Existing	Baseline	Existing	Baseline
1993	124,438	76,058	0	65,003	17,252	0
1994	115,051	77,985	0	36,624	227	0
1995	298,984	136,048	0	192,234	27,923	0
1996	205,830	104,328	0	180,065	78,884	37
1997	140,629	66,004	0	83,966	8,628	0
1998	129,802	71,592	0	56,631	718	0
1999	75,727	25,985	0	81,746	31,052	0
2000	123,493	41,939	0	137,617	59,007	0
2001	57,190	46,250	0	12,185	875	0
2002	157,935	63,027	0	120,564	26,100	0
2003	164,914	30,563	0	181,244	47,338	0
2004	140,602	31,124	0	163,353	51,024	0
Total	1,734,564	770,873	0	1,311,232	349,026	37

a. All discharge volumes in acre-feet per year.
b. STA-5/6 inflow from C-139 Basin only. STA-5/6 inflow from C-139 Annex is not included in these data.

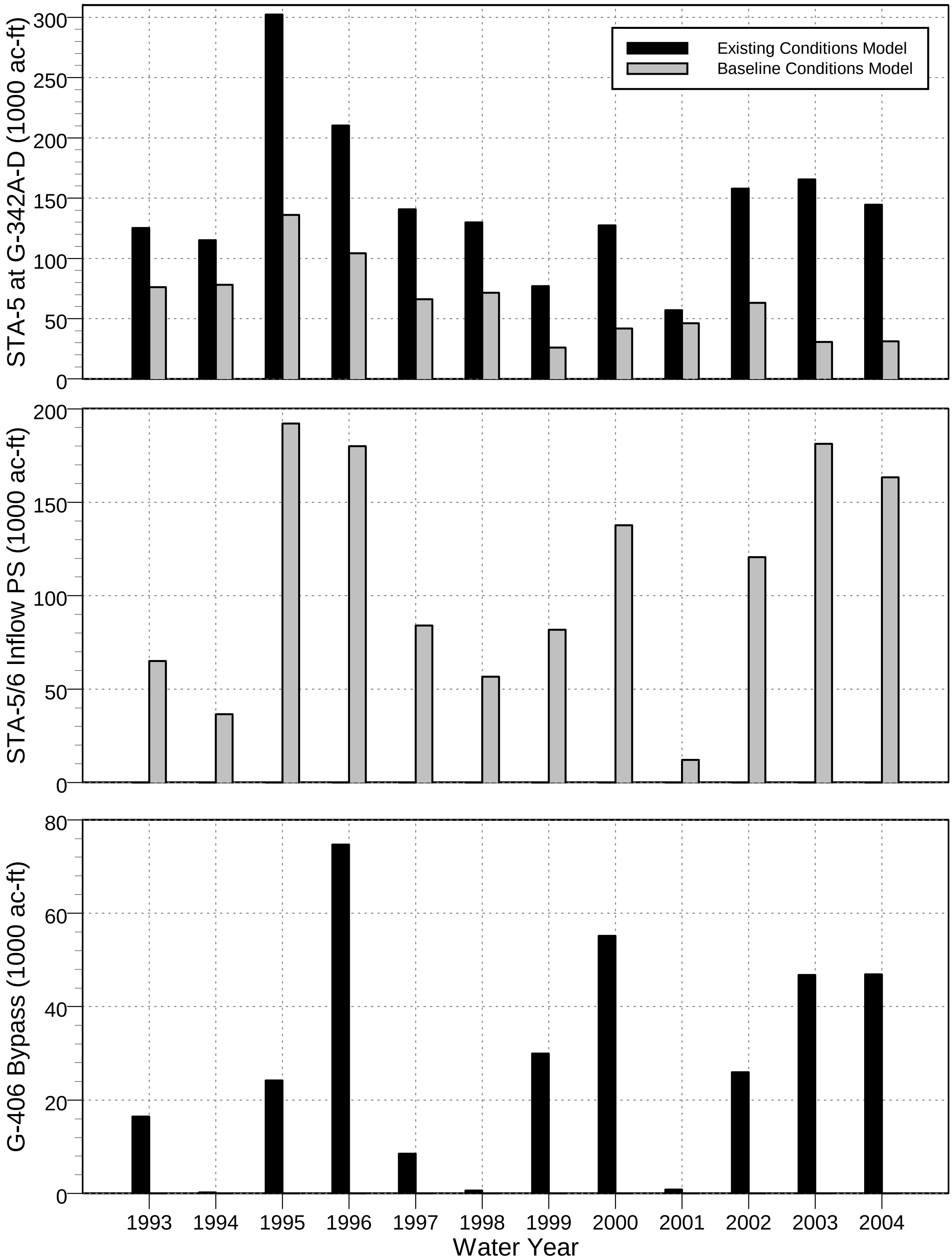
29,000 acre-feet) are made up by reduced bypass at G-406, which goes from an average of about 29,000 acre-feet per year to essentially zero. The modeled inflow balance between the upper and lower flowways at STA-5/6 is approximately 37 percent to 63 percent, respectively, over the entire study period.

The operating criteria for the planned STA-5/6 inflow pump station were adjusted to achieve the stated flow balance. URS Corporation (2007) states the new pump station will have two 100-cfs electric pumps and four 470-cfs diesel pumps, with one of the diesel pumps used as a redundant spare. The headwater stages used to start and stop each of these pumps are listed in Table 3-5.

Table 3-5: Assumed Operating Criteria for STA-5/6 Inflow Pump Station

Pump No.	Pump Capacity (cfs)	Headwater Elevation (feet NGVD)	
		Start	Stop
1	100	15.3	15.0
2	100	15.4	15.0
3	470	15.5	15.0
4	470	15.7	15.0
5	470	16.0	15.0

Figure 3-7: Comparison of Annual STA-5/6 Inflow and Bypass



3.2 WATER QUALITY MODELING

Diversions from the S-4 Basin will have to be treated before these diversions can be discharged into the Everglades Protection Area. Once STA-5 has been fully expanded through the build out of Compartment C, it and STA-6 will become the most likely facilities to treat this water. As currently proposed, STAs 5 and 6 will be transformed into a single, combined treatment area with the Compartment C buildout; therefore, this expanded treatment area is referred to as STA-5/6 below.

If additional drainage volumes from the S-4 Basin are diverted to STA-5/6 for treatment, it will be important to estimate the impact of these diversions on the performance this STA. Therefore, an analysis of STA-5/6 performance under Baseline Conditions was prepared for comparison to similar analyses for proposed conditions that were developed later. This analysis utilized the Dynamic Model for Stormwater Treatment Areas, Version 2 (DMSTA), which was developed by Walker and Kadlec (2008). The most recent version of this model is dated January 2, 2008.

3.2.1 DMSTA Configuration

The STA-5/6 configuration used in the DMSTA model was based on Alternative 3 from the Compartment C Basis of Design Report (BODR) (URS, 2007). The BODR identifies Alternative 3 as the recommended alternative for the Compartment C build out. This proposed layout was simplified slightly in the DMSTA model because the proposed STA-5/6 complex will include seven flowways of two treatment cells each, for a total of 14 cells. The DMSTA program only allows 12 treatment cells to be modeled in a single case. If each cell within the STA-5/6 complex were individually modeled, two separate model runs would be needed for each analysis. To reduce the number of cells used in the DMSTA model, flowways STA5-4 and STA5-5 were combined into a single flowway by merging cells STA5-4a and STA5-5a together and cells STA5-4b and STA5-5b together. This simplified STA-5/6 system with 12 total treatment cells is easier to model in DMSTA. Combining these cells for water quality modeling purposes should not alter the results significantly.

3.2.2 Baseline DMSTA Dataset

A baseline inflow dataset, which includes daily inflow rates and TP concentrations for water years 1995–2004, was used in the DMSTA model. This dataset is a combination of two separate datasets, one for STA-5 and other for STA-6, that were developed for the EAA Regional Feasibility Study (RFS) (Burns & McDonnell, 2005). Inflow to STA-5 was assumed to come from C-139 Basin drainage and inflow to STA-6 from C-139 Annex drainage. The combined baseline dataset is two years shorter than the dataset used in the XPSWMM hydraulic model because there is little C-139 Basin water quality data available in the earlier years of the study period.

Runoff quantity and quality data for the C-139 Annex were collected at station USSO (DBKEY TA425). The record at this station starts on December 2, 1995, which is approximately 19 months after the start of the STA-5 inflow dataset from the C-139 Basin (May 1, 1994). Data for the missing period at USSO (5/1/1994–12/1/1995) were estimated so the two datasets would both cover the same 10-year period. These data were estimated based on two characteristics: average data by date and a weighting factor based on flows in C-139 basin. The available daily data at station USSO (12/2/1995–4/30/2004) were averaged for each day of the year and used to fill in the missing data period for the C-139 Annex. These generated data were then adjusted using a weighting factor developed from comparison of the corresponding data for the C-139 Basin. Examination of the C-139 Basin data showed that flows during the first two years of the study period (WY 1993 and 1994) were relatively higher than the 10-year average. Therefore, the estimated average daily flows used for the C-139 Annex were adjusted upward using this same ratio.

Table 3-6 is an annual summary of the C-139 Basin and C-139 Annex runoff and the combined baseline inflow dataset for STA-5/6. The rainfall and evapotranspiration data used in the DMSTA model for this ten-year period are described in Section 2.7.

Table 3-6: Baseline Inflow to STA-5/6 for DMSTA Modeling

Water Year	C-139 Basin		C-139 Annex[a]		Combined STA-5/6 Inflow		
	Volume (ac-ft)	TP Load (kg)	Volume (ac-ft)	TP Load (kg)	Volume (ac-ft)	TP Load (kg)	TP Conc[b] (ppb)
1995	262,327	53,812	58,460	7,270	320,787	61,082	154
1996	261,192	55,522	51,273	7,044	312,465	62,566	162
1997	151,440	38,438	40,196	5,108	191,635	43,546	184
1998	149,152	27,193	46,081	4,022	195,233	31,215	130
1999	122,038	28,242	24,270	3,131	146,307	31,373	174
2000	176,867	39,980	46,365	6,417	223,232	46,397	168
2001	52,964	14,979	26,831	4,565	79,795	19,544	199
2002	182,608	55,375	37,722	3,846	220,330	59,221	218
2003	209,265	63,935	43,921	5,262	253,186	69,197	222
2004	190,713	58,088	46,858	5,732	237,571	63,820	218
Total	1,758,566	435,564	421,976	52,397	2,180,542	487,962	181
Avg. Ann.	175,857	43,556	42,198	5,240	218,054	48,796	181

a. C-139 Annex data for 5/1/1994–12/1/1995 are estimates.
b. Flow-weighted mean TP concentrations.

A detailed listing of DMSTA input and output variables for Baseline Conditions is included in Appendix A. Some of the key DMSTA results are summarized in Table 3-7. As shown, the projected FWM TP concentration in STA-5/6 outflow is 12.8 ppb.

Table 3-7: Baseline DMSTA Modeling Summary for STA-5/6

Parameter	Units	Baseline Case[a]
Average Annual Inflow (WY 1995-2004)		
Volume	1000 ac-ft	218.0
TP Load	1000 kg	48.8
FWM TP Concentration[b]	ppb	181
Average Annual Outflow (WY 1995-2004)		
Volume	1000 ac-ft	208.4
TP Load[c]	1000 kg	3.3
FWM TP Concentration[b]		
Median	ppb	12.8
10-90% Conf. Limits[d]	ppb	10.1-17.0
Geometric Mean TP Conc		
Median	ppb	8.9
10-90% Conf. Limits4	ppb	6.5-12.8

a. Baseline case assumes treatment of runoff from C-139 Basin and C-139 Annex.
b. Flow-weighted mean (FWM) TP concentration.
c Outflow TP load based on median FWM TP concentration.
d. Mean TP concentrations for 10-90% confidence limits based on simulations using 10th and 90th percentile settling rate (K) estimates. Base (median) concentration estimates are based on simulations with median K values.

* * * * *

Section 4.0

DIVERSION AND TREATMENT ALTERNATIVES

4.0 DIVERSION AND TREATMENT ALTERNATIVES

This report section describes the various alternatives that have been developed to reduce the discharge of phosphorus to Lake Okeechobee from the S-4 Basin. This goal can be accomplished by (1) diverting basin drainage away from Lake Okeechobee or (2) by treating this drainage to remove phosphorus before it is discharged to Lake Okeechobee. Even under the first diversion option, it is likely these diversions will have to be first treated before they can be released to other sensitive water bodies. Therefore, all plans conceived for this project will involve both diversion and treatment components, which have been classified respectively as diversion and treatment alternatives. The diversion and treatment alternatives identified for evaluation in this feasibility study have been developed through consultation with EPD and SFWMD representatives. These alternatives are described in the balance of this section.

Many of these alternatives will require construction of new control structures and canals, or modifications to existing structures and canals. No specific design capacity recommendations for these new or modified structures and canals are included in this chapter. These capacity requirements were established later through the hydraulic modeling of these alternatives, which is described in Section 5.0.

4.1 DIVERSION ALTERNATIVES

The diversion alternatives are defined as potential systems that would allow for diversion of S-4 Basin drainage away from Lake Okeechobee. This diverted drainage would have to be delivered into one of the SFWMD's principal drainage canals. There are three such canals that pass adjacent to the borders of the S-4 Basin. These canals include the L-1 and L-1E borrow canals to the south, and C-43 Canal to the west. In the prior Diversion Study (Burns & McDonnell, 2006), the C-43 Canal was discounted as a feasible destination for additional S-4 Basin diversions because none of the planned projects in the C-43 Basin (for example, the C-43 West Reservoir Project) are sized to accommodate additional S-4 Basin diversions. This conclusion leaves the L-1 and L-1E borrow canals as the most logical delivery points for S-4 Basin diversions.

During high rainfall events, it will not be possible to divert all (if any) of the required S-4 Basin drainage so the basin's existing drainage system must remain intact and operational even after implementation of the proposed diversion system. With this absolute requirement in mind, three diversion alternatives have been identified that would enable S-4 Basin drainage to be diverted into the L-1 and/or L-1E borrow canals. These alternatives are described below.

4.1.1 Diversion Alternative 1

Diversion Alternative 1 uses a southerly extension of the Clewiston (Industrial) Canal to divert S-4 Basin drainage into the L-1E Borrow Canal. From this point, these diversions could be directed east into the Miami Canal, or approximately two miles west into the L-1 Borrow Canal.

4.1.1.1 Overview

For this diversion alternative, all S-4 Basin drainage targeted for diversion must first be delivered to the Clewiston Canal. Four of the basin's drainage areas already have pump stations that discharge directly to this canal:

- U.S. Sugar Corporation's Townsite Area (USSC Townsite) at Southline Pump Station (EPD-06)
- South Florida Conservancy District (SFCD) at P-5-W Pump Station (EPD-07). Note, because of a prior SFCD diversion project the discharge at this pump station should be much, much less than historic levels.
- East Hendry County Drainage District (EHCDD) at Pump Station No. 1 (EPD-08)
- Clewiston Drainage District (CDD) at Pump Station No. 3 (EPD-09)

In order to maximize diversions from CDD, discharge at its Pump Station No. 2 (EPD-10) would have to be minimized by preferentially operating EPD-09. An alternate plan to treat rather than divert CDD drainage is discussed below in Section 4.2.2.3.

The biggest operational change would occur for the Sugarland Drainage District (SDD), which currently discharges all of its drainage northward into the C-21 Canal. Although this district abuts the Clewiston Canal over a distance of about two miles, there is no existing outlet into this adjacent canal. A new eastern outlet into the Clewiston Canal would be required to divert the drainage from this district.

Implementation of this diversion alternative could potentially divert all drainage from the S-4 Basin except that from the Disston Island Conservancy District (DICD). This district's drainage is omitted because less than a third of this district's drainage is currently discharged to Lake Okeechobee; the balance is delivered to the C-43 Canal. Also, this district is the most remote hydraulically from the Clewiston Canal, making the capture of this drainage relatively difficult. The most feasible scenario to accomplish this diversion would likely require pumping this district's drainage southward into the Sugarland Drainage District and using SDD canals to convey this drainage to the Clewiston Canal. This option was not considered feasible because it would disrupt operations within the SDD.

Figure 4-1 is a schematic that shows the anticipated new control structures that would be required to implement this diversion alternative. In this and other similar schematics, all new structures, or structures that must be modified or relocated, are shown in red. For easier reference, these new or modified structures have been named after a nearby existing structure. For example, new structures near the G-136 culverts are named G-136A, G-136B, etc. A general description of the new structures and other required improvements within the S-4 Basin is included below. The new structures shown in this schematic on the L-1 and L-1E borrow canals, which are outside of the S-4 Basin, are discussed in connection with Treatment Alternative A (Section 4.2.1).

4.1.1.2 Clewiston Canal Improvements

Under this diversion alternative, the Clewiston Canal would be used to convey all basin diversions into the L-1E Borrow Canal. The southern terminus of the Clewiston Canal is presently located at USSC's Southline Pump Station (EPD-06). This pump station would have to be relocated and this canal extended southward, approximately 1.5 miles, to meet Levee 1 East at the extreme southeast corner of the S-4 Basin. There is an existing USSC canal (Lateral 0-S) along the alignment of this canal extension that could potentially be enlarged and used for this purpose.

As mentioned above, the Southline Pump Station would have to be relocated (replaced) under this alternative. The replacement pump station (designated EPD-06A in the schematic) would likely be constructed across the Townsite Main Canal, immediately southwest of its current location, and be oriented to discharge eastward into the extended Clewiston Canal. A new north-south canal would also be required to connect the Hageman and Main canals within the Townsite area. This new canal would replace the conveyance function currently provided by Lateral 0-S. Once these new facilities are operational, the existing Southline Pump Station could be demolished.

At the southern end of the Clewiston Canal extension, a new gated culvert structure (G-136B) would be required to control flow between the Clewiston and L-1E canals. This structure would be opened when diverting basin drainage and closed when it is necessary to direct basin drainage back north toward Lake Okeechobee. There is a rail line that passes through this corner of the S-4 Basin. This new control structure would also need to carry this rail line across the new Clewiston Canal extension.

The last improvement contemplated in the Clewiston Canal would be a new gated culvert structure (S-310A) that would prevent any diverted drainage from flowing northward into Lake Okeechobee. Ideally, this control structure would be located south of the U.S. Highway 27 bridges over this canal but

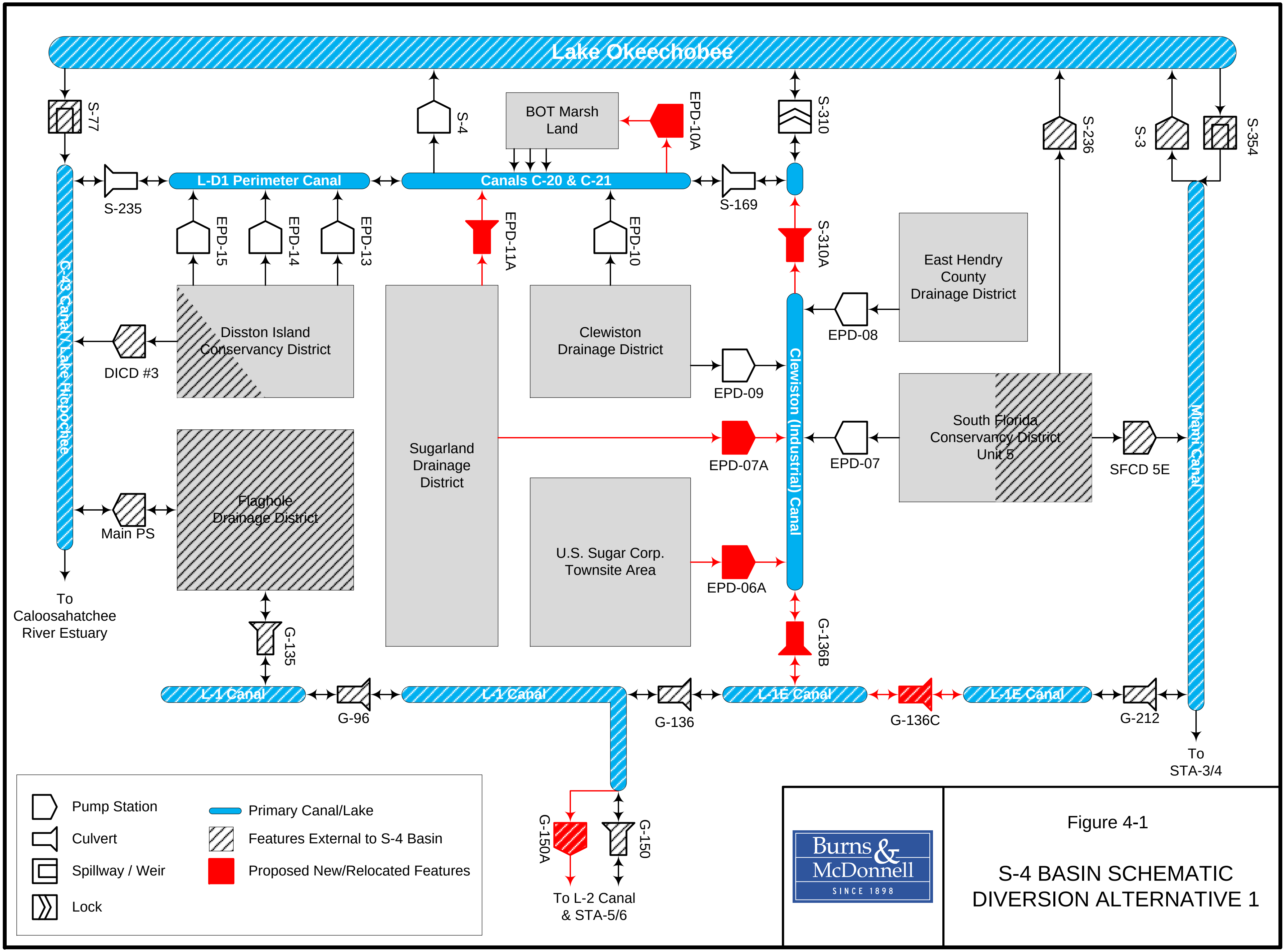


Figure 4-1

S-4 BASIN SCHEMATIC DIVERSION ALTERNATIVE 1 north of the pump station from EHCDD (EPD-08). This control structure would normally remain closed and be opened only during high rainfall events when bypass to Lake Okeechobee may still be necessary or to supply water from Lake Okeechobee to the southern Clewiston Canal for irrigation. With construction of this new control structure, no changes in the operation of the S-310 lock would be required; therefore, little if any recreational impacts for boaters should occur within Clewiston.

After initially proposing that the new S-310A structure be constructed near U.S. Highway 27, comments were received from EPD representatives that indicate future residential and recreational development of the southern Clewiston Canal is being considered. This new control structure would impede any associated boating activity along the Clewiston Canal. This structure could be relocated farther south, but doing so may reduce the amount of basin drainage that could potentially be diverted.

4.1.1.3 Sugarland Drainage District Improvements

Diverting drainage from the Sugarland Drainage District into the Clewiston Canal will require a number of internal improvements. The existing Sugarland Main pump station will require some modification. The drainage pumps in this station are no longer in service; water levels within the district are controlled via a gated culvert that passes through this pump station. The gate on this culvert is manually operated at present. In the future, this culvert must be fitted with a remotely-operated gate or replaced entirely. This modified or replacement gate structure has been designated as EPD-11A on the schematic.

There is also an irrigation pump in this pump station that supplies water to the entire Sugarland Drainage District. If the existing pump station is decommissioned or demolished, the function of the existing irrigation pump in this structure must be preserved or replaced.

There is a small portion of SDD, approximately 320 acres, that drains to the Nine Mile Canal. This area is generally north of U.S. Highway 27 and immediately west of County Road 720. The Nine Mile Canal discharges to the C-21 Canal downstream (north) of the Sugarland Main pump station. This diversion scheme would not capture the relatively small quantities of drainage from the Nine Mile Canal.

Diversions from this district into the Clewiston Canal would occur through a new pump station (or possibly a new gated culvert) that would be located at the southeast corner of the district (EPD-07A). This location is almost directly across the Clewiston Canal from the SFCD’s P-5-W Pump Station (EPD-07).

The existing east-west canals along the southern SDD boundary are relatively small laterals. One or more of these laterals will have to be enlarged to convey the required flows to this new pump station. These canal enlargements will be somewhat problematic because of existing facilities in this area. The USSC has a series of treatment and retention ponds located south of its sugar mill complex along the eastern SDD border. Enlarging one of the existing laterals to feed a new EPD-07A pump station would require reducing the size of one of these ponds, which could impair its intended purpose.

There are also two rail lines that pass through the area immediately west of the proposed EPD-07A pump station. One of these lines runs north-south along the west bank of the Clewiston Canal. The other rail line is a spur that branches off of the north-south line and runs east-west along the southern SDD border. The existing culverts under these rail lines would have to be replaced with larger box culverts to accommodate an enlarged supply canal for the new diversion pump station. Any canal improvements would also impact miscellaneous road crossing culverts and structures that control runoff between fields and adjacent laterals. These smaller structures have not yet been inventoried.

4.1.2 Diversion Alternative 2

A schematic for Diversion Alternative 2 is included in Figure 4-2. For this diversion alternative, S-4 Basin drainage would be diverted into both the L-1 and L-1E borrow canals. From these delivery points, these diversions could be directed eastward into the Miami Canal or southward into the C-139 Basin. This diversion alternative is very similar to the first one, differing only in the manner that SDD drainage would be diverted.

4.1.2.1 Overview

Under this diversion alternative, a Clewiston Canal extension to intersect the L-1E Borrow Canal would still be required. Diverted drainage from CDD, EHCDD, SFCD and USSC Townsite would still travel southward through this canal extension into the L-1E Canal. All of the Clewiston Canal improvements described above in Section 4.1.1.2 would still be necessary. The only exception is that the hydraulic capacity of the Clewiston Canal extension itself and structure G-136B may be reduced slightly from those required under Diversion Alternative 1.

Drainage from the Sugarland Drainage District would be diverted directly into the L-1 Borrow Canal at the south end of this district through a new pump station referred to as G-96A. The related internal SDD improvements are described in Section 4.1.2.2 below.

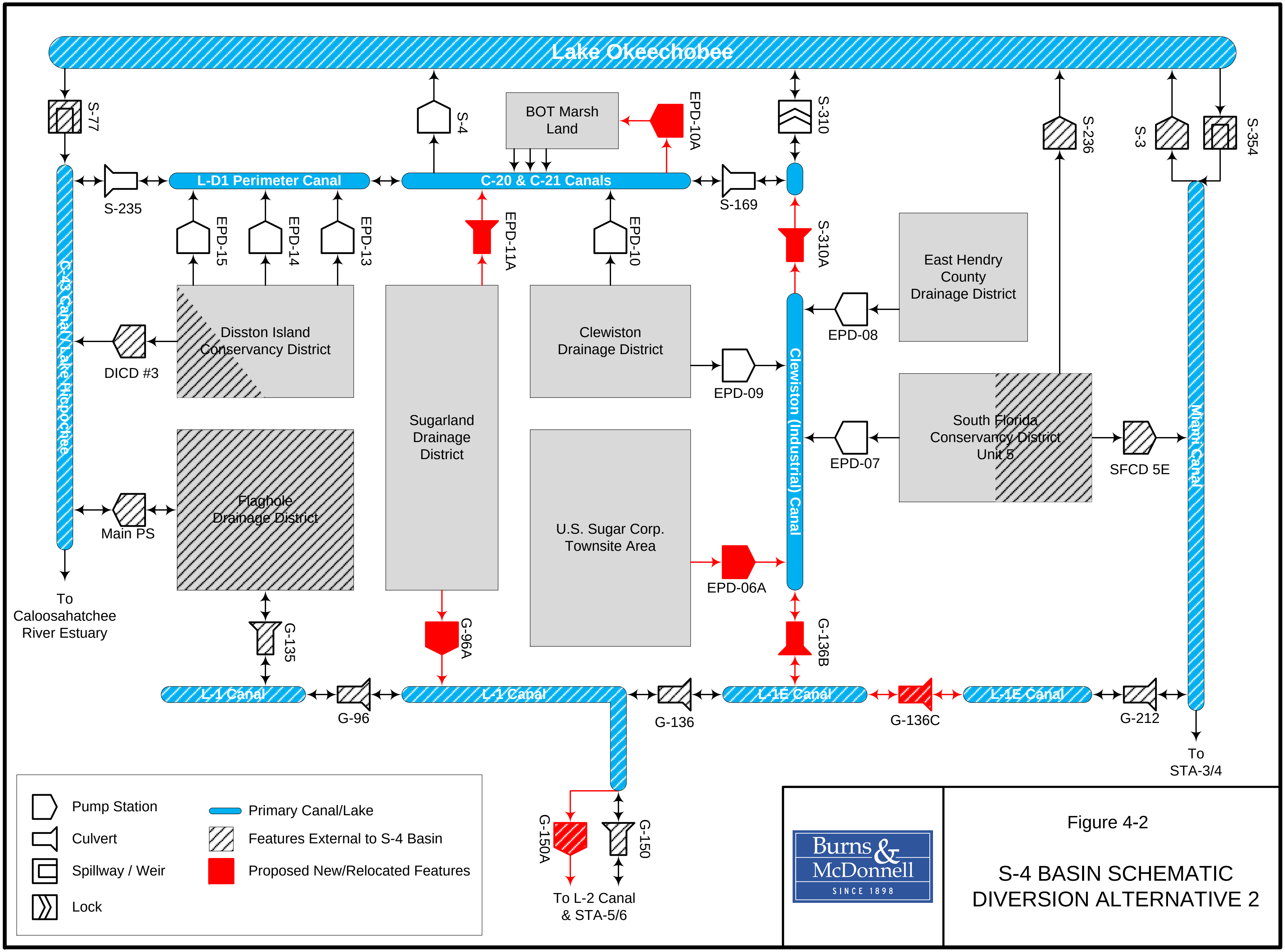


Figure 4-2

S-4 BASIN SCHEMATIC DIVERSION ALTERNATIVE 2

As with all of the diversion alternatives, this alternative would make no attempt to divert DICD drainage or the relatively small amount of SDD drainage that is carried by the Nine Mile Canal. The majority of this drainage flows into the C-43 Canal rather than Lake Okeechobee. An alternate plan to treat, rather than divert, CDD drainage (Section 4.2.2.3) could be implemented with this alternative also.

4.1.2.2 Sugarland Drainage District Improvements

Diverting drainage from the Sugarland Drainage District into the L-1 Borrow Canal will require a number of internal improvements. The existing gated culvert in the Sugarland Main pump station (EPD-11A) will have to be automated or replaced as described in Section 4.1.1.3.

Diversions from this district into the L-1 Borrow Canal would occur through a new pump station or gated culvert (G-96A) that would be located at the extreme southern end of the district where the district boundary and L-1 Borrow Canal meet at a common corner. There is a small drainage lateral (L-15) that connects this remote corner of the district with Canal 4. Canal 4 is one of the main internal SDD canals. It runs north-south from the L-15 lateral up to the southern edge of U.S. Highway 27 and then turns east toward the Sugarland Main pump station. Lateral L-15 and the southern reaches of Canal 4 would have to be enlarged to convey this district's drainage to the new discharge pump station or culvert.

Approximately two miles north of its southern terminus, Canal 4 crosses under a rail line and adjacent road through a 48-inch culvert. There is also a drainage pump at this location. This rail/road crossing culvert will have to be replaced with a larger structure. Other miscellaneous road culverts and field drainage structures that may be impacted by these canal enlargements will also need to be replaced.

4.1.3 Diversion Alternative 3

The third and final diversion alternative is significantly different from the first two alternatives. The proposed control structures included in this alternative are shown schematically in Figure 4-3. For this alternative, all S-4 Basin diversions would be delivered into the L-1 Borrow Canal. These diversions could then stay in this canal and the C-139 Basin, or be directed eastward into the L-1E Borrow Canal and S-3/S-8 Basin.

4.1.3.1 Overview

With Diversion Alternative 3, the SDD and USSC Townsite areas would both be diverted directly into the L-1 Borrow Canal. There would be no other diversions under this plan, although the alternate treatment plan discussed in Section 4.2.2.3 would be required under Diversion Alternative 3 to treat CDD drainage.

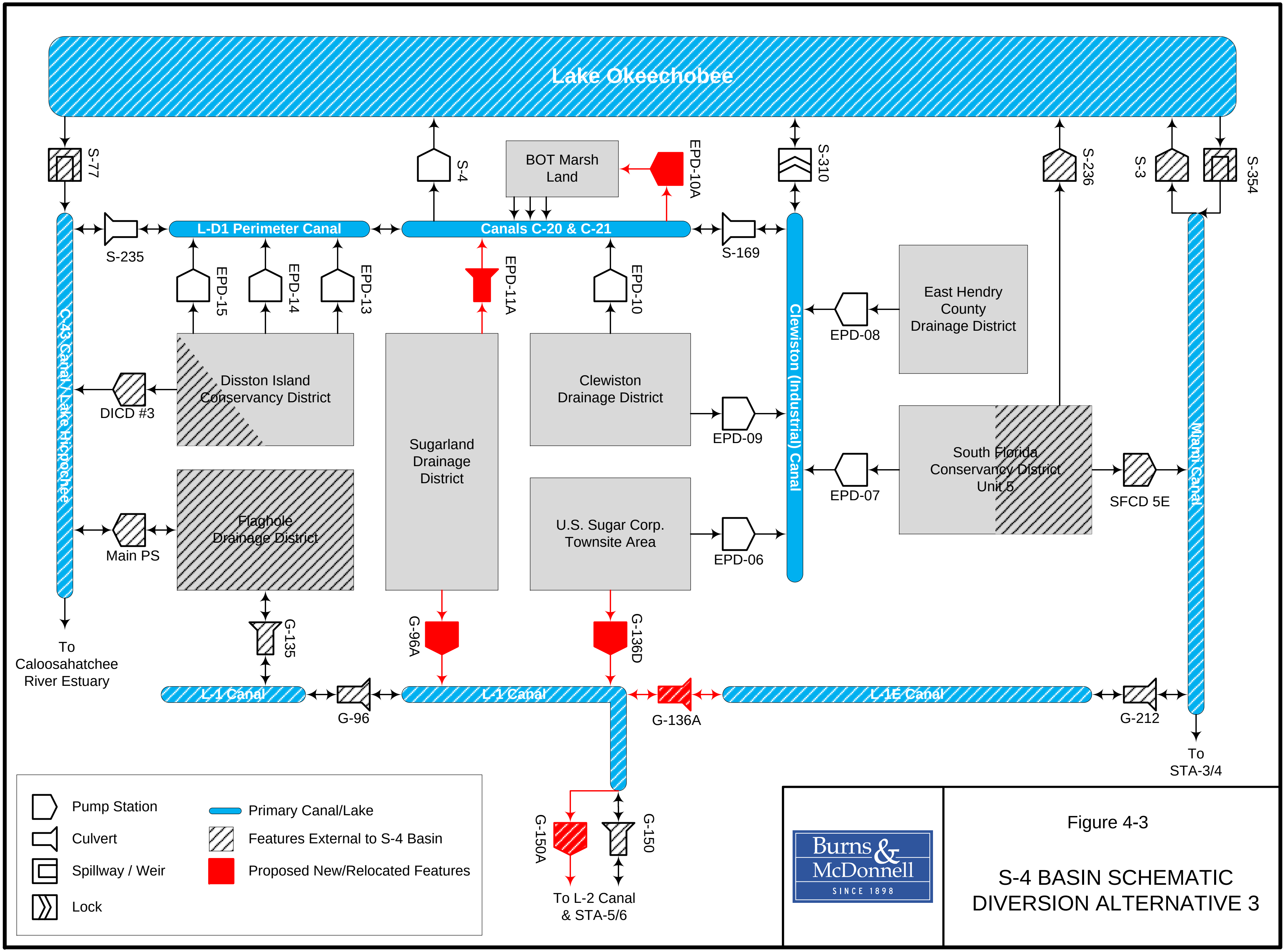


Figure 4-3

S-4 BASIN SCHEMATIC
DIVERSION ALTERNATIVE 3

The first two diversion alternatives both require extensive improvements within the Clewiston Canal, which may impede future development along this canal. In particular there was concern about the proposed S-310A gate structure (Figures 4-1 and 4-2) that would block boater access to the southern Clewiston Canal. This third diversion alternative was proposed because it specifically avoids any modifications within the Clewiston Canal, including construction of S-310A. The disadvantage of this plan is that it precludes diversion of EHCDD drainage discharged at EPD-08 and any remaining SFCD drainage discharged at EPD-07. Although relatively small in magnitude, the majority of the drainage from these two districts would make its way into Lake Okeechobee.

4.1.3.2 Sugarland Drainage District Improvements

Diverting drainage from the Sugarland Drainage District into the L-1 Borrow Canal would be accomplished by the same internal modifications described for Diversion Alternative 2 (Section 4.1.2.2).

4.1.3.3 U.S. Sugar Townsite Improvements

Within the USSC Townsite drainage area, Diversion Alternative 3 would require markedly different improvements from the prior two alternatives. For these prior diversion alternatives, all drainage from this area would be discharged from the replacement EPD-06A pump station into the Clewiston Canal. Depending on current conditions, this drainage could either travel north as it does today or be diverted southward. With Diversion Alternative 3, diverted versus non-diverted drainage would be discharged via separate pump stations. Because no changes to the Clewiston Canal are contemplated under this alternative, the existing Southline Pump Station would not be relocated and all non-diverted drainage would continue to be discharged from this facility.

All drainage from this area that is to be diverted would be discharged from a new pump station (designated G-136D in Figure 4-3). This new pump station would be located through Levee 1 immediately west of G-136. Two existing laterals (Laterals 4 and 4-N) would be enlarged to convey Townsite drainage to this new pump station. These laterals intersect the two primary east-west canals within this drainage area, the Main and Hageman canals.

4.2 TREATMENT ALTERNATIVES

The three diversion alternatives described above address options for diverting S-4 Basin drainage into the L-1 and/or L-1E borrow canals. The majority of these S-4 Basin diversions would eventually make their way into the Everglades Protection Area, so they must be treated to remove excess phosphorus first. The treatment alternatives described in this section discuss options for treating these S-4 Basin diversions after they are delivered into the L-1 or L-1E borrow canals, and also options for treating S-4 Basin drainage in lieu of diversion.

4.2.1 Treatment Alternative A

Treatment Alternative A would deliver S-4 Basin diversions to STA-5/6 for treatment. The planned buildout of STA-5 within Compartment C would be required to make these diversions feasible. From their delivery point(s) in the L-1 and/or L-1E borrow canals, these diversions would be conveyed to the L-2 Borrow Canal and hence into the L-3 Borrow Canal and STA-5/6. The new or modified control structures and other improvements required for Treatment Alternative A are described below.

4.2.1.1 L-1 Borrow Canal Improvements

For delivery of diversions to STA-5/6, some enlargement of the L-1 Borrow Canal may be necessary. For Diversion Alternative 1, these enlargements would be confined to the reach between G-136 and G-150. For the other two diversion alternatives, these canal improvements could extend as far upstream (west) as the proposed new G-96A pump station. The need for and size of any canal enlargements was investigated with the hydraulic system model. The results of these analyses are presented in Section 5.0.

A new pump station will be required at G-150 where the L-1 and L-2 borrow canals meet (designated G-150A in Figures 4-1, 4-2 and 4-3).[1] This pump station will be oriented to lift water from the L-1 Borrow Canal into the L-2 Borrow Canal. This pump station is necessary because canal stages in the L-2 Borrow Canal are typically higher than stages to the north, in the L-1 Borrow Canal, since construction of STA-5 and G-406. As a result, southward flows through G-150 have become quite rare, as evidenced by recent flow monitoring data. Future plans for STA-5/6 show that stages in the L-2 and L-3 canals could be even higher than recent historic levels, ranging up to 15 feet NGVD or more. Stages in the L-1 Borrow Canal need to be maintained at 14 feet NGVD or less to prevent flooding of adjacent pasture lands. Without construction of this pump station, the frequency with which S-4 Basin drainage could be diverted south to STA-5/6 would be severely limited.

There are times however, when gravity flow in a northerly direction may still be desirable at G-150. Therefore, either the existing G-150 structure should be preserved or a similar gated culvert incorporated into the new pump station structure.

[1] In actuality, G-150 is located in the L-2 Borrow Canal. The intersection of the L-1 and L-2 borrow canals is approximately 100 feet north of G-150.

4.2.1.2 L-1E Borrow Canal Improvements

In the L-1E Borrow Canal, a new gated culvert structure (G-136C, Figures 4-1 and 4-2) will be required downstream (east/south) of the confluence of this canal and the new Clewiston Canal extension. This structure, which is applicable only for Diversion Alternatives 1 and 2, is necessary to force any S-4 Basin diversions delivered into the L-1E Borrow Canal to flow back through G-136 into the L-1 Borrow Canal. The stop logs in G-136 would have to be removed to allow free discharge through these culverts in either direction.

For Diversion Alternative 3, a similar gated culvert structure (G-136A in Figure 4-3) will be required in the L-1E Borrow Canal; however, this new structure could replace the existing G-136.

It is not anticipated that any conveyance improvements in the L-1E Borrow Canal will be required.

4.2.2 Treatment Alternative B

For Treatment Alternative B, S-4 Basin diversions would be delivered to the S-3/S-8 Basin for treatment. Whether discharged initially into the L-1 Borrow Canal or directly into the L-1E Borrow Canal, diverted drainage would flow eastward in the L-1E Borrow Canal to the Miami Canal.

At present, the only treatment area accessible from the Miami Canal is STA-3/4. During high rainfall events, drainage from the northern C-139 Basin (north of G-150) is currently discharged through G-136 into the S-3/S-8 Basin, where they would eventually make their way to STA-3/4 for treatment. These discharges are expected to continue into the future.

With consideration of the existing C-139 Basin drainage that passes through G-136, SFWMD staff has stated that the existing treatment capacity of STA-3/4 is fully allocated. Therefore, delivery of additional diversions from the S-4 Basin into the S-3/S-8 Basin could overload this treatment area. In order for Treatment Alternative B to be considered feasible, additional wetland treatment areas will probably have to be constructed within the S-3/S-8 Basin, possibly in conjunction with construction of the EAA Reservoir Project, Phase 2 (Compartment A-2). At the current time, there has been little advance planning for development of Compartment A-2 because of uncertainty surrounding the operating characteristics of the A-1 Reservoir (now under construction). Until these issues are resolved, no further consideration can be given to treatment of diverted drainage from the S-4 Basin within the S-3/S-8 Basin.

4.2.2.1 L-1 Borrow Canal Improvements

For delivery of S-4 Basin diversions to the Miami Canal, some enlargement of the L-1 Borrow Canal may still be necessary under Diversion Alternatives 2 and 3. These enlargements would be confined to the reach between the proposed G-96A pump station (see Figures 4-2 and 4-3) and G-136. These potential canal enlargements would not be required for Diversion Alternative 1.

No other improvements, such as the new G-150A pump station, would be required in the L-1 Borrow Canal for Treatment Alternative B.

4.2.2.2 L-1E Borrow Canal Improvements

For Treatment Alternative B, no new structures would be required in the L-1E Borrow Canal for Diversion Alternative 1. For the other two diversion alternatives, it may be necessary to replace the existing G-136 structure with gated culverts that allow for remote operation (see G-136A in Figure 4-3).

It is not anticipated that any conveyance improvements in the L-1E Borrow Canal will be required to implement Treatment Alternative B.

4.2.3 Clewiston Drainage District Treatment Alternative

An alternate treatment plan for CDD drainage is now in the planning stages. Located immediately north of the City is a 728-acre parcel of public marsh land that is bounded by the Herbert Hoover Dike, C-20 Canal and C-21 Canal. This public land is under the control of the Board of Trustees of the Internal Improvement Trust Fund (BOT). Within this area is a small parcel (approximately 24 acres) that is owned by the U.S. Sugar Corporation. This BOT land is now being considered for a water storage project intended to reduce Lake Okeechobee discharges and help restore the hydrology of this area (Powell Kugler; MacVicar, Federico & Lamb; 2007).

Under the current project plan, water would be pumped onto this BOT land from the C-21 Canal and stored on site at depths up to two feet maximum. Future plans would redirect all of the drainage from CDD into this area, which would then function as a small stormwater treatment area. The inflow pump station for this STA would be located in the eastern portion of the BOT land, directly north of CDD Pump Station No. 2 (EPD-10). This new inflow pump station has been designated EPD-10A on the basin schematics. After passing through this STA, CDD drainage would be discharged back into the C-20 Canal through three existing culverts. After treatment, it is anticipated that this CDD drainage would be of sufficient quality that it could be discharged directly to Lake Okeechobee.

If this alternate CDD treatment project should come to fruition, CDD drainage could be eliminated from any plans to divert S-4 Basin drainage southward (Diversion Alternatives 1 and 2). This alternative CDD treatment plan would likely be required to make Diversion Alternative 3 feasible.

4.2.4 Intrabasin Treatment Alternative

One alternative to diversion of S-4 Basin drainage is to pre-treat all basin drainage before it is discharged to Lake Okeechobee. This option was investigated in the previous Diversion Study (Burns & McDonnell, 2006) and discounted, but a summary of these results are repeated here for convenience.

In the previous analysis, the gross size of a STA required to treat CDD and SDD drainage to at TP concentration of 40 ppb was estimated at 3,360 acres based on preliminary DMSTA modeling. The 40-ppb TP criteria is an estimate of the limits that may be imposed in the future on discharges to Lake Okeechobee. The added requirement to treat drainage from the EHCDD, SFCD and USSC Townsite areas would increase the hydraulic loading to a potential intrabasin STA by about 65 percent and the TP loading by approximately 50 percent. While no additional DMSTA modeling was performed for this intrabasin treatment alternative, given these statistics the gross area of a STA required to pre-treat all of this drainage would be at least 5,000 acres. A 5,000-acre STA located within the eastern portion of the S-4 Basin would consume about 20 percent of the total land area. The economic impact of converting this much arable land from private to public ownership is considered unattractive for the following reasons:

- Anticipated high capital cost relative to that for diversion of basin drainage, particularly with respect to the cost of land acquisition;
- Impact on local government tax bases;
- Reduced impact on total phosphorus load discharged to Lake Okeechobee, as compared to all other alternatives considered;
- Uncertain effect of future regulatory actions on allowable discharges to Lake Okeechobee.

For the above reasons, the intrabasin treatment alternative was not considered to be a viable option in the current Feasibility Study.

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Section 5.0

ALTERNATIVES ANALYSIS

5.0 ALTERNATIVES ANALYSIS

This report section is not yet available.

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Section 6.0

OPINION OF PROBABLE COSTS

6.0 OPINION OF PROBABLE COSTS

This report section contains opinions of the probable capital and operating costs for development of each project alternative. The identified improvements that will be necessary for development of these alternatives are described in prior sections and below. These designs should be considered conceptual only and will likely be revised upon completion of more detailed analysis and design.

6.1 DISCLAIMER

The opinions of probable cost presented in this section are based on the experience and qualifications of Burns & McDonnell's professional engineering staff and its familiarity with the construction industry. Burns & McDonnell has no control over the cost of labor, materials, equipment or services furnished by others or over contractors' methods for determining prices, or over competitive bidding or market conditions; therefore, Burns & McDonnell cannot and does not guarantee that proposals, bids or actual total construction costs will not vary from the opinions of probable cost presented below.

6.2 UNIT COSTS

The unit costs used to develop the opinions of cost for each alternative are described below. These unit costs are segregated into those initial costs for capital improvements and right-of-way acquisition, and those annual costs for operations and maintenance. Most of these unit costs come from completed SFWMD projects or from proposed SFWMD projects where detailed engineering and opinions of cost have been completed. The unit costs developed from these SFWMD sources were supplemented as necessary with unit costs available from standard construction industry cost guides. The next sections present these unit costs and document their sources.

6.2.1 Unit Costs for Capital Improvements

The unit costs used in the opinion of capital costs were pulled together from various sources including those listed below:

- Means 2008 Heavy Construction Cost Data
- STA-6 Section 2 Opinion of Capital Costs (2005)
- Supplemental Technology Standard of Comparison Guidelines for Cost Estimates (STSOC 2001)
- Big Cypress Basin 1 Construction Costs (2006)
- STA-3/4 Construction Costs (2001)

Review of these sources revealed a wide range in the reported opinions of cost and actual construction costs for comparable capital improvements, so these costs were compared to one another in many cases to determine the unit costs that are considered the most applicable at the present time. The challenge in identifying reasonable unit costs was complicated by the following factors:

1. The damage caused by the very active hurricane seasons of 2004 and 2005 pulled the construction industry's resources away from its normal business activity to attend to rebuilding efforts.
2. The housing industry in South Florida has experienced a significant recession because of prior overbuilding in a speculative market.

The effect of these two factors — one that tends to increase demand and prices and the other that has the opposite effect — is difficult to show quantitatively. For some projects, the effect of the dual hurricane seasons was a price inflation of about 50 percent, but with the recent housing industry problems, prices have decreased. However, if active hurricane seasons are experienced again in the future, these prices will likely increase again.

The following are the assumptions used in developing the unit capital costs:

- Culverts – All culverts were assumed to be constructed with corrugated aluminum pipe. Small culverts were assumed to be 48 inches in diameter by 50 feet long while large culverts were 84 inches in diameter by 60 feet long. Culvert unit costs include an allowance for installation of risers and flashboards when necessary for stage control.

- Box Culverts – Box culverts would be constructed of reinforced concrete with small box culverts assumed to be 10 feet wide by 6 feet high. Large box culverts were assumed to be 12 feet by 10 feet. Box culverts can be simple culverts for road crossings, etc. or gated with electric gate operators and telemetry allowing remote operation. Separate units cost were developed for simple and gated box culverts in the two size ranges specified. Double box culverts are assumed to be twice the cost of single box culverts.

- Pumping Stations – Pumping stations are sized for the maximum required diversion capacity. The existing drainage systems within the S-4 and C-139 basins will be retained, so pumping stations used only for diversion service will not require the same level of redundancy (spare pumps, backup power, etc.) as drainage pump stations. SFWMD uses standardized pump sizes (25 and 100-cfs electric pumps and 470 and 940-cfs diesel pumps) when practicable in its stations. The proposed L-2 Borrow Canal pump station at G-150 is assumed to be configured with 100 and 470-cfs pumps. It is assumed that pump stations within the S-4 Basin will be configured similar to existing pump stations with electric pumps in increments of 50, 100, and 200 cfs. The pumping station prices were assumed to differ by size as shown in the STSOC:

- Very small pumping stations: 40 cfs or less (these small stations tend not to have all of the equipment as larger pump stations)
- Small pumping stations: 41 to 60 cfs
- Large pumping stations: 61 to 500 cfs
- Very large pumping stations: greater than 500 cfs (these large stations are often able to achieve some economies of scale)

- Canal excavation – Because of the presence of near surface rock, canal excavation will require blasting and rock excavation.

- Levee construction – A berm for canal maintenance will be included for levees taller than five feet.

- Clearing – Only light clearing consisting of grass and small shrubs is expected

- Electrical distribution – Pump stations must be served by a suitable three-phase electric distribution line. Distribution line construction costs will be included for each new pump station. The origin of these new distribution lines is assumed to be the existing Florida Power & Light substation that is located within the USSC Townsite area. For gated box culvert structures, which have much more modest electric power requirements than pump stations, can be served by single-phase power lines. Per discussions with EPD representatives, it is expected that suitable single-phase power lines exist in the immediate vicinity of each proposed gated box culvert; thus no costs for extension of power lines to these structures in included.

- Stilling wells – Each gated structure and pump station will likely require a pair of stilling wells with stage recording equipment to measure headwater and tailwater elevations. Because of the lack of existing stage information in the Clewiston (Industrial) Canal, additional stage monitoring stations are expected. Unit costs for stilling wells include costs of stage recording equipment, telemetry and electrical work.

- Pump station demolition – Demolition is assumed to include environmental factors such as lead abatement and asbestos handling. Some salvaging of equipment is expected.

The unit prices compiled for this study were updated to 2008 levels using available Engineering News Record (ENR) cost indices; the closest city to the project area with available indices is Atlanta, Georgia. The Atlanta ENR cost index shows considerable variability in the annual rate of increase; with the use of several different sources, the effect of these rates will, hopefully, be minimal.

Table 6-1 shows the unit prices that were used in this study and their principal source. All costs in this table have been updated to year 2008 using the ENR cost indices.

Table 6-1 Unit Costs and Reference Sources for Capital Improvements

Item	Unit Cost (2008$/unit)	Units	Original Data Year	Principal Data Source
Canal Excavation	5	CY	2005	STA-6 Section 2
Levee Construction	4	CY	2005	STA-6 Section 2
Clearing	400	acre	2001	STSOC
Seeding/Mulching	3,700	acre	2006	Big Cypress Basin 1 Construction Cost
Small Culverts	6,700	each	2008	2008 Means
Large Culverts	16,000	each	2008	2008 Means
Small Box Culverts	*	each		
Large Box Culverts	*	each		
Small Gated Box Culverts	194,300	each	2001	STA-3/4 Construction Cost
Large Gated Box Culverts	291,500	each	2001	STA-3/4 Construction Cost
Structure Electrical	55,700	each	2001	STA-3/4 Construction Cost
Elect. Power Dist.	103,600	mile	2001	STSOC
Stilling Wells	11,700	each	2001	STA-3/4 Construction Cost
Very Large Pump Sta.	9,700	cfs	2001	STSOC
Large Pump Station	12,800	cfs	2001	STSOC
Medium Pump Station	12,300	cfs	2001	STSOC
Small Pump Station	9,800	cfs	2001	STSOC
Pump Station Removal	100,000	each	2008	Engineering Judgment

* Unit cost still under development.

6.2.2 Unit Costs for Land Acquisition

The unit costs used in the opinions of cost for land and right-of-way acquisition were based on engineering judgment and not on extensive market research. A common unit cost of $10,000 per acre was adopted for use in this study. The following assumptions were used to estimate the quantity of land required:

- Levee – Based on required width or additional width, including width of adjacent berms and levees, times required length
- Control structures – 1 acre per control structure
- Pumping stations – 3 acres per pump station

6.2.3 Unit Costs for Annual Operation & Maintenance

The unit costs used in the opinions of operation and maintenance (O&M) costs were obtained from the Supplemental Technology Standard of Comparison Guidelines for Cost Estimates (STSOC 2001). This reference is the only known source with a compilation of O&M costs that is suitable for this type and location of study.

The following are the assumptions used in developing the unit O&M costs:

- Levees – O&M costs include periodic mowing and other basic levee maintenance activities

- Control structures – Costs include labor and materials for operation and periodic maintenance

- Lead operators – For major pump stations, lead operators are needed. In this study, any required new pump stations will be in close proximity to one another so only one lead operator is included.

- Engine operator/maintenance mechanic – Pump stations require engine operators and maintenance mechanics. Some of the proposed new pump stations in this study may consist entirely of electric motor driven pumps, significantly reducing maintenance requirements. Also, because these stations will be in close proximity to one another, sharing of maintenance personnel is assumed practicable. Therefore, only one engine operator/maintenance mechanic is included per each large pump station.

- Mechanical maintenance – Pump units require mechanical maintenance; pumps larger than 500 cfs require considerably more maintenance due to the pump size and sophistication.

- Building maintenance – Periodic maintenance of pumping station housing is required

- Energy consumption – The operational cost of pumping water, whether electrical power or diesel fuel, was estimated based on the volume of water pumped.

Table 6-2 shows the unit O&M costs used for this study. All costs in this table have been updated to year 2008 using the ENR costs indices. The exception to this is the increase in the price of fuel which has increased by approximately 250 percent since 2001.

Table 6-2: Annual Unit Costs for Operation & Maintenance

O&M Cost Item	Unit Cost (2008$/unit)	Units
Levee	2,000	mile
Control Structures	15,500	each
Lead Operator	77,700	each
Engine Operator/Maintenance Mechanic	64,800	each
Pump Mechanical Maintenance (> 500 cfs)	29,800	each unit
Pump Mechanical Maintenance (< 500 cfs)	13,000	each unit
Building Maintenance	15,500	each
Diesel Pump Energy Cost	1.25	acre-foot
Electric Pump Energy Cost	*	acre-foot

* Unit cost still under development.

6.3 METHODOLOGY

The unit costs listed above were used to develop opinions of probable capital and O&M cost for each project alternative. These upfront capital and annual O&M costs were then combined to yield an estimate of the present worth of total project expenditures over an assumed 50-year operating life. The methodology and common assumptions used to develop these estimates of present worth project expenditures are described below:

6.3.1 Capital Costs

The opinions of probable capital cost for each project alternative were estimated using the following methodology:

- Direct Construction Costs: The hydraulic modeling and other analyses described in Section 5.0 were used to estimate the new physical works required to implement each project alternative. These takeoff quantities were multiplied by the applicable unit costs and totaled to yield estimated direct construction costs.

- Indirect Capital Costs: Indirect capital costs include items such as the costs of planning, permitting, engineering, owner's overhead, land acquisition, and contingency. Except for land acquisition costs, these indirect costs were all estimated as a percentage of construction cost subtotals. These indirect cost rates were calculated as follows:

    - Planning, Engineering and Design (PE&D) – 10 percent of Direct Construction Cost
    - Program and Construction Management (P&CM) – 10 percent of Direct Construction Cost
    - Contingency – 30 percent of Direct Construction Costs plus PE&D and P&CM
    - Land Acquisition – Estimated land requirements times unit land cost ($10,000 per acre)

- Total Capital Costs: Total capital costs are the sum of direct construction costs plus indirect capital costs (PE&D, P&CM, contingency and land acquisition).

6.3.2 Operation and Maintenance Costs

The O&M cost opinions developed for each alternative are considered to be incremental. That is, only those additional O&M costs directly attributable to the proposed diversion project are included in these estimates. The unit costs shown in Table 6-2 are multiplied by the applicable quantities and then totaled to yield annual O&M cost subtotals. A 30 percent contingency was then added to these subtotals to give estimates of Total Incremental O&M Cost.

6.3.3 Present Worth of Total Project Expenditures

The present worth of total project expenditures was calculated using the following cash flow assumptions to achieve a start-up date of January 1, 2012:

- Land acquisition: 2009
- Planning, Engineering & Design: 2009
- Program and Construction Management: 50 percent in 2010 and 50 percent in 2011
- Construction: 50 percent in 2010 and 50 percent in 2011
- O&M: Each year of operation (2012–2061)

All costs, both capital and O&M, are escalated from the estimate date to the expenditure year at 3 percent per year. Present worth costs are discounted from the expenditure date to December 31, 2008 using a discount rate of 6.375 percent.

The following section presents example opinion of probable cost data for Alternative 1A. This section is intended to demonstrate how we plan to apply the unit cost data and estimating methodology described above to an example project alternative. The data shown below are subject to change as we finalize our analyses.

6.4 ALTERNATIVE 1A

Alternative 1A is a combination of Diversion Alternative 1 and Treatment Alternative A. For this alternative, all diverted drainage from the S-4 Basin will be first delivered into the Clewiston Canal and then conveyed south via an extension of this canal into the L-1E Canal. From this point, these diversions would flow west into the L-1 Borrow Canal and then southward via the L-2 and L-3 borrow canals for treatment in STA-5/6. The opinion of probable cost developed for this alternative is documented in the following sections.

6.4.1 Capital Costs

The following list is a summary of the anticipated physical works necessary for implementation of Alternative 1A. The structure designations in this list are from the schematic in Figure 4-1.

- Clewiston (Industrial) Canal
  - Demolish existing USSC Southline Pump Station — replacement pump station discussed below
  - Extend Clewiston Canal southward to meet L-1E Borrow Canal. Portions of the existing Clewiston Canal south of proposed new Sugarland Drainage District outlet may also require enlargement.
  - Replace existing road crossing culvert located immediately south of Hageman Canal confluence
  - Construct new gated culvert structure (G-136B) at intersection of extended Clewiston and L-1E canals
  - Construct new gated culvert structure (S-310A) in Clewiston Canal south of U.S. Highway 27

- Sugarland Drainage District (SDD)
  - Construct new pump station (EPD-07A) between SDD and Clewiston Canal
  - Enlarge existing SDD canals and laterals to convey drainage to new EPD-07A structure — will also require replacement of several road and rail crossing culverts within SDD
  - Replace or modify gated culvert structure in Sugarland Main Pump Station (EPD-11A) to allow for remote operation

- USSC Townsite Area
  - Construct replacement to Southline Pump Station (EPD-06A)
  - Make internal canal improvements required to route drainage to relocated pump station

- L-1E Borrow Canal
  - Construct new gated culvert structure (G-136C) in L-1E Canal
  - Enlarge L-1E Canal between G-136 and new G-136C (may not be necessary)
  - Remove flashboards in G-136 or entire structure

- L-1/L-2 borrow canals
  - Enlarge L-1 Canal between G-136 and G-150 (may not be necessary)
  - Construct new pump station (G-150A) between L-1 and L-2 canals

The assumed physical dimensions or capacities of the required physical works for Alternative 1A are presented in Table 6-3.

Table 6-3: Assumed Physical Dimensions for Alternative 1A

Item No.		Description	Quantity	Unit	Quantity	Unit	Total Quantity	Unit
1	Canal Excavation: Sugarland canals to Clewiston Canal							
	a	Canal Excavation	15,800	feet	300	SF	175,556	CY
	b	Levee Construction	15,800	feet	408	SF	238,756	CY
	c	Clearing and Site Preparation	15,800	feet	160	feet	58.0	acre
	d	Seeding & Mulching	15,800	feet	100	feet	36.3	acre
	e	Replace small field culverts	6	each	8	each	48	each
	f	Replace large field culverts	17	each	1	each	17	each
	g	Small Double Box Culvert	2	each	1	each	2	each
2	Canal Excavation: Clewiston Canal							
	a	Canal Excavation	7,900	feet	160	SF	46,815	CY
	b	Levee Construction	7,900	feet	136	SF	39,793	CY
	c	Clearing and Site Preparation	7,900	feet	100	feet	18.1	acre
	d	Seeding & Mulching	7,900	feet	60	feet	10.9	acre
	e	Replace small field culverts	2	each	1	each	2	each
	f	Replace large field culverts	3	each	1	each	3	each
3	Canal Excavation: L-1E Canal (end of Clewiston to G-136)							
	a	Canal Excavation	10,600	feet	0	SF	0	CY
	b	Levee Construction	10,600	feet	0	SF	0	CY
	c	Clearing and Site Preparation	10,600	feet	0	feet	0.0	acre

Item No.	Description		Quantity	Unit	Quantity	Unit	Total Quantity	Unit
	d	Seeding & Mulching	10,600	feet	0	feet	0.0	acre
	e	Replace small field culverts	7	each	0	each	0	each
	f	Replace large field culverts	3	each	0	each	0	each
4	Canal Excavation: L-1 Canal (G-136 to G-150)							
	a	Canal Excavation	10,600	feet	300	SF	117,778	CY
	b	Levee Construction	10,600	feet	255	SF	100,111	CY
	c	Clearing and Site Preparation	10,600	feet	120	feet	29.2	acre
	d	Seeding & Mulching	10,600	feet	100	feet	24.3	acre
	e	Replace small field culverts	2	each	8	each	16	each
	f	Replace large field culverts	8	each	1	each	8	each
5	Control Structure: EPD-11A							
	a	Small Double Box Culvert	1	each	1	each	1	each
	b	Structure Electrical	1	each	1	each	1	each
	c	Existing Structure Removal	1	each	1	each	1	each
6	Control Structure: S-310A							
	a	Large Double Box Culvert	1	each	1	each	1	each
	b	Structure Electrical	1	each	1	each	1	each
	c	Stilling Wells	1	each	2	each	2	each
7	Control Structure: G-136A							
	a	Large Double Box Culvert	1	each	1	each	1	each
	b	Structure Electrical	1	each	1	each	1	each
	c	Stilling Wells	1	each	2	each	2	each
8	Control Structure: G-136B							
	a	Large Double Box Culvert	1	each	1	each	1	each
	b	Structure Electrical	1	each	1	each	1	each
9	Pumping Station: New Sugarland PS EPD-07A							
	a1	Pump	1	each	200	cfs	200	cfs
	a2	Pump	3	each	100	cfs	300	cfs
		Pumping Station Total	1	each			500	cfs
10	Pumping Station: Relocate USSC Southline PS EPD-06A							
	a1	Pump	2	each	45,000	gpm	201	cfs
	a2	Pump	5	each	22,000	gpm	245	cfs
	a	Pumping Station Total	1	each			446	cfs
	b	Inflow Canal Excavation	1,000	feet	200	SF	7,407	CY
	c	Existing Structure Removal	1	each	1	Ea	1	each
11	Pumping Station: New PS G-150A							
	a1	Pump	2	each	470	cfs	940	cfs
	a2	Pump	2	each	100	cfs	200	cfs
	a	Pumping Station Total	1	each			1,140	cfs

Applying the required dimension information in Table 6-3 to the unit costs for capital improvements yields the opinion of probable capital cost summarized in Table 6-4.

6.4.2 Annual O&M Costs

Listed below is a summary of the anticipated incremental operation and maintenance requirements for Alternative 1A:

- Maintain new levees associated with creation or enlargement of Clewiston, L-1E, L-1 and SDD internal canals.
- Operate and maintain new control structures EPD-11A, S-310A, G-136B and G-136C
- Operate and maintain new pump stations, including hiring of new maintenance staff, at structures EPD-07A and G-150A. Pump Station EPD-06A is a direct replacement for the USSC Southline Pump Station so it is assumed there would be no incremental O&M cost for this new pump station.

The opinion of probable O&M cost for Alternative 1A is included in Table 6-5.

6.4.3 Present Worth of Total Project Expenditures

The total present worth costs of Alternative 1A is presented in Table 6-6. These present worth costs are discounted to March 31, 2008.

Table 6.4 Opinion of Probable Capital Cost, Alternative 1A

Item		Item No.	Description	Estimated Quantity	Unit	Estimated Unit Cost	Estimated Total Cost
Canal Excavation	Sugarland canals to Clewiston Canal	1a	Canal Excavation	175,556	CY	$5	$877,778
		1b	Levee Construction	238,756	CY	$4	$955,022
		1c	Clearing and Site Preparation	58	AC	$400	$23,214
		1d	Seeding & Mulching	36	AC	$3,700	$134,206
		1e	Replace small field culverts	48	Ea	$6,700	$321,600
		1f	Replace large field culverts	17	Ea	$16,000	$272,000
		1g	Small Double Box Culvert	2	Ea	$388,600	$777,200
	Clewiston Canal	2a	Canal Excavation	46,815	CY	$5	$234,074
		2b	Levee Construction	46,815	CY	$4	$187,259
		2c	Clearing and Site Preparation	18	AC	$400	$7,254
		2d	Seeding & Mulching	11	AC	$3,700	$40,262
		2e	Replace small field culverts	2	Ea	$6,700	$13,400
		2f	Replace large field culverts	3	Ea	$16,000	$48,000
	L-1E Canal (end of Clew. to G-136)	3a	Canal Excavation	0	CY	$5	$0
		3b	Levee Construction	0	CY	$4	$0
		3c	Clearing and Site Preparation	0	AC	$400	$0
		3d	Seeding & Mulching	0	AC	$3,700	$0
		3e	Replace small field culverts	0	Ea	$6,700	$0
		3f	Replace large field culverts	0	Ea	$16,000	$0
	L-1 Canal (G-136 to G-150)	4a	Canal Excavation	117,778	CY	$5	$588,889
		4b	Levee Construction	117,778	CY	$4	$471,111
		4c	Clearing and Site Preparation	29	AC	$400	$11,680
		4d	Seeding & Mulching	24	AC	$3,700	$90,037
		4e	Replace small field culverts	16	Ea	$6,700	$107,200
		4f	Replace large field culverts	8	Ea	$16,000	$128,000
Control Structure	EPD-11A	5a	Small Box Culvert	1	Ea.	$194,300	$194,300
		5b	Structure Electrical	1	Ea.	$55,700	$55,700
		5c	Removal of Existing Pump Station	1	Ea.	$100,000	$100,000
	S-310A	6a	Large Double Box Culvert	1	Ea.	$583,000	$583,000
		6b	Structure Electrical	1	Ea.	$55,700	$55,700
		6c	Stilling Wells	2	Ea.	$11,700	$23,400
	G-136A	7a	Large Double Box Culvert	1	Ea.	$583,000	$583,000
		7b	Structure Electrical	1	Ea.	$55,700	$55,700
		7c	Stilling Wells	2	Ea.	$11,700	$23,400
	G-136B	8a	Large Double Box Culvert	1	Ea.	$583,000	$583,000
		8b	Structure Electrical	1	Ea.	$55,700	$55,700
Pump Sta.	EPD-07A	9	Large Pump Station	500	cfs	$12,800	$6,400,000
	EPD-06A	10a	Large Pump Station	446	cfs	$12,800	$5,703,719
		10b	Inflow Canal Excavation	7,407	CY	$5	$37,037
		10c	Removal of Existing Pump Station	1	Ea.	$100,000	$100,000
	G-150A	11	Very Large Pump Station	1,140	cfs	$9,700	$11,058,000
	Subtotal, Estimated Construction Costs						**$30,899,800**
	Planning, Engineering & Design			10 %			$3,090,000
	Program & Construction Management			10 %			$3,090,000
	Total Estimated Cost, Without Contingency						**$37,079,800**
	Contingency			30 %			$11,123,900
	Land Acquisition						
			Canal Improvements	105	AC		
			Control Structures (1 ac ea)	6	AC		
			Pump Stations (3 ac ea)	12	AC		
			Total	123	AC	$10,000	$1,233,700
	TOTAL ESTIMATED CAPITAL COST						**$48,204,000**

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Table 6.5 Opinion of Probable Incremental O&M Cost, Alternative 1A

Item	Item No.	Description	Estimated Quantity	Unit	Estimated Unit Cost	Estimated Total Cost
Levee	1	SDD Levees	3.0	mile	$2,000	$5,985
	2	Clewiston Canal Levee	1.5	mile	$2,000	$2,992
	3	L-1E Canal Levee	0.0	mile	$2,000	$0
	4	L-1 Canal Levee	2.0	mile	$2,000	$4,015
C.S.*	5	Water Control Structures	6	each	$15,500	$93,000
Pump Station	6	Lead Operator	1	each	$77,700	$77,700
	7	Engine Operator/Mechanic	3	each	$64,800	$194,400
	8	Mechanical Maintenance	15	unit	$13,000	$195,000
	9	Building Maintenance	3	each	$15,500	$46,500
	10	Power for P.S. EPD-07a	25,400	ac-ft	$1.25	$31,750
	11	Power for P.S. EPD-06a	17,200	ac-ft	$1.25	$21,500
	12	Power for P.S. G-150A	68,900	ac-ft	$1.25	$86,125
	Subtotal, Estimated Incremental Operation & Maintenance Costs					**$759,000**
	Contingency		30 %			$227,700
	TOTAL INCREMENTAL O&M COST					**$987,000**

C.S.* - Control Structure
P.S.* - Pumping Station
Quantity estimated = 80% times avg ann acft

Table 6.6 Total Present Worth, Alternative 1A

Annual Discount Rate		6.375%		Date of Pricing Data		03/31/08
Present Cost as of		12/31/2008				
Annual Escalation Rate		3.000%		Convenience Rate		3.277%
		Capital Costs				Present
Year	Land Acq.	PED	P&CM	Const.	Total	Worth
2009	$1,517,295	$3,800,310			$5,317,606	$3,450,159
2010			$1,957,160	$25,835,966	$27,793,126	$21,705,965
2011			$2,015,875	$26,611,045	$28,626,919	$21,017,292
Total Capital Cost					$5,317,606	$46,173,416
Incremental Costs for Operation and Maintenance						**Present**
From	**To**			**Total O&M Cost**		**Worth**
2012	2061				$149,618,893	$18,581,170
Total Present Worth of Alternative						**$64,754,586**

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Section 7.0

CONCLUSIONS

7.0 CONCLUSIONS

This chapter will be provided in a later draft report.

* * * * *

Section 8.0

REFERENCES

8.0 REFERENCES

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Appendix A

DMSTA DATA

DMSTA2- Inputs & Out

Project: PROJECT_STA5C6 | Model Release: 01/02/08 | Current Date: 1/17/2008

Input Varia	Units	Value	Case Description:						
Design Cas	-	2010 Base	STA-5 Expanded to Include Full Build-out of Compartment C & STA-6; Cells 4 & 5 condensed to singular flowway - Cell 45						
Input Series Name		TS_Base	2010-2014; downstream cells considered as SAV_3 (PEW_3 for STA-6); Inflows limited to C-139 Basin & C-139 Annex runoff						
Starting Da	-	05/01/94	Historic Inflow Concentrations Reduced by 10% for ongoing BMP implementation in basin						
Ending Dat	-	04/30/04	Triangle Area on southern end of Comp C converted to use as Cell 6A and STA-6 Section 2 as Cell 6B.						
Starting Da	-	05/01/94	Cells 3 & 5 of STA-6 Section 1 converted to use as Cells 8 & 7, respectively.						
Integration	-	4	Simulation Type:	Uncertainty Analysis					
Number of	-	0	Output Variable	Mean	Lower CL	Upper CL	Diagnostics		
Output Ave	days	30	FWM Outflow C (ppb)	12.8	17.0	10.1	H20 Balance Error Mean & Max	0.0%	0.0%
Inflow Conc	-	1	GM Outflow C (ppb)	8.9	12.8	6.5	Mass Balance Error Mean & Max	0.0%	0.0%
Rainfall P C	ppb	10	Load Reduction %	93%	91%	95%	Iterations & Convergence	3	0.2%
Atmospheri	mg/m2-yr	20	Bypass Load (%)	0.0%			Warning/Error Messages	22	

Cell Number -->		1	2	3	4	5	6	7	8	9	10	11	12
Cell Label	-	1A	1B	2A	2B	3A	3B	45A	45B	6A	6B	7	8
Vegetation	-->	EMG_3	SAV_3	EMG_3	SAV_3	EMG_3	SAV_3	EMG_3	SAV_3	EMG_3	SAV_3	PEW_3	PEW_3
Inflow Frac	-	0.14		0.14		0.169		0.364		0.127		0.043	0.017
Downstrea	-	2		4		6		8		10			
Surface Are	km2	3.38	4.94	3.38	4.94	6.31	3.72	17.05	4.53	1.90	5.61	2.53	0.99
Mean Widtl	km	1.56	1.56	1.56	1.56	1.56	1.56	3.90	3.90	1.14	2.40	1.26	0.61
Number of	-	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0	3.0
Minimum D	cm												
Release 1 Series Name													
Release 2 Series Name													
Outflow Series Name													
Depth Series Name													
Outflow Co	cm	38	45	38	45	38	45	38	45	38	45	38	38
Outflow We	cm	10	10	10	10	10	10	10	10	10	10	10	10
Outflow Co	-	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5	3.5
Outflow Co	-	1.2	0.9	1.2	0.9	1.2	0.9	1.2	0.9	1.2	0.9	1.2	1.2
Bypass De	cm												
Maximum I	hm3/day												
Maximum C	hm3/day												
Inflow Seep	(cm/d) / cm												
Inflow Seep	cm												
Inflow Seep	ppb												
Outflow Se	(cm/d) / cm	0.0075	0.0075										
Outflow Se	cm	-46	-38										
Max Outflo	ppb	20	20										
Seepage R	-	1	2										
Seepage R	-	1	1										
Seepage D	-												
Initial Wate	ppb	30	30	30	30	30	30	30	30	30	30	30	30
Initial P Sto	mg/m2	500	500	500	500	500	500	500	500	500	500	500	500
Initial Wate	cm	45	45	45	45	45	45	45	45	45	45	45	45
C0 = Conc	ppb	3	3	3	3	3	3	3	3	3	3	3	3
C1 = Conc	ppb	22	22	22	22	22	22	22	22	22	22	22	22
C2 = Conc	ppb	300	300	300	300	300	300	300	300	300	300	300	300
K = Net Se	m/yr	16.8	52.5	16.8	52.5	16.8	52.5	16.8	52.5	16.8	52.5	34.9	34.9
Z1 = Satura	cm	40	40	40	40	40	40	40	40	40	40	40	40
Z2 = Lower	cm	100	100	100	100	100	100	100	100	100	100	100	100
Z3 = Upper	cm	200	200	200	200	200	200	200	200	200	200	200	200

Output Var	Units	1	2	3	4	5	6	7	8	9	10	11	12	Overall
Execution T	sec/yr	12.10	12.60	13.00	13.40	13.80	14.20	14.70	15.10	15.50	15.90	16.30	16.80	16.80
Run Date	-	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08	01/17/08
Starting Da	-	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94
Starting Da	-	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94	05/01/94
Ending Dat	-	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04	04/30/04
Output Dur	days	3653	3653	3653	3653	3653	3653	3653	3653	3653	3653	3653	3653	3653
Cell Label		1A	1B	2A	2B	3A	3B	45A	45B	6A	6B	7	8	Total
Downstream Cell Label		1B	Outflow	2B	Outflow	3B	Outflow	45B	Outflow	6B	Outflow	Outflow	Outflow	-
Network Si	-	none	none	none	none	none	none	none	none	none	none	none	none	none
Simulation	-	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta	Uncerta
Surface Are	km2	3.38	4.94	3.38	4.94	6.31	3.72	17.05	4.53	1.90	5.61	2.53	0.99	59.27
Mean Raini	cm/yr	126.66	126.66	126.66	126.66	126.66	126.66	126.66	126.66	126.66	126.66	126.66	126.66	126.7
Mean ET	cm/yr	147.31	146.23	147.31	146.17	147.31	146.36	147.01	146.34	147.31	146.48	146.47	146.47	146.8
Cell Inflow	hm3/yr	37.7	37.0	37.7	37.0	45.5	44.2	98.0	94.5	34.2	33.8	11.6	4.6	269.1
Cell Inflow	kg/yr	6834.9	2782.9	6834.9	3041.7	8250.7	2477.6	17770.8	4170.1	6200.3	3735.1	2099.3	830.0	48821
Cell Inflow	ppb	181	75	181	82	181	56	181	44	181	111	181	181	181.4
Treated Ou	hm3/yr	37.0	36.0	37.0	36.0	44.2	43.4	94.5	93.6	33.8	32.7	11.1	4.4	257.2
Treated Ou	kg/yr	2782.9	374.6	3041.7	384.5	2477.6	532.4	4170.1	1446.8	3735.1	337.3	159.5	64	3299
Treated FW	ppb	75	10	82	11	56	12	44	15	111	10	14	15	12.8
Upper Con	ppb	88.3	12.8	98.3	13.4	71.8	16.2	58.5	21.5	124.1	12.6	19.4	19.6	17.0
Lower Con	ppb	62.0	8.7	66.3	8.9	42.2	9.8	32.4	11.6	95.7	8.8	11.3	11.4	10.1
Total Outflo	hm3/yr	37.0	36.0	37.0	36.0	44.2	43.4	94.5	93.6	33.8	32.7	11.1	4.4	257.2
Total Outflo	kg/yr	2782.9	374.6	3041.7	384.5	2477.6	532.4	4170.1	1446.8	3735.1	337.3	159.5	64.0	3299.0
Total FWM	ppb	75.3	10.4	82.2	10.7	56.1	12.3	44.1	15.5	110.5	10.3	14.4	14.6	12.8
Bypass Loa	kg/yr	0	0	0	0	0	0	0	0	0	0	0	0	0.0
Bypass Loa	%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0%	0.0
Maximum I	hm3/d	0.49	0.50	0.49	0.50	0.60	0.60	1.29	1.30	0.45	0.45	0.15	0.06	3.53
Maximum C	hm3/d	0.50	0.50	0.50	0.50	0.60	0.60	1.30	1.31	0.45	0.46	0.16	0.06	3.59
Surface Loa	kg/yr	4227	2562	3793	2657	5773	1945	13601	2723	2465	3398	1940	766	45522
Load Trapp	kg/yr	3533	2540	3904	2818	5979	2067	14157	2871	2527	3581	2022	798	46798
Overall Loa	%	59%	87%	55%	87%	70%	79%	77%	65%	40%	91%	92%	92%	93%
Lower Con	%	0.5	0.9	0.5	0.9	0.6	0.8	0.7	0.6	0.3	0.9	0.9	0.9	91%
Upper Con	%	0.7	0.9	0.6	0.9	0.8	0.8	0.8	0.6	0.5	0.9	0.9	0.9	95%
Daily Geon	ppb	69.4	6.4	75.7	6.2	52.9	7.5	42.2	10.5	100.5	5.9	10.6	10.8	#N/A
Outflow Ge	ppb	68.3	7.0	74.6	6.9	50.9	8.3	40.0	11.2	99.4	6.6	10.9	11.1	8.9
Upper Con	ppb	81.1	9.2	90.4	9.3	66.6	11.9	54.3	16.8	112.7	8.6	15.9	16.2	12.8
Lower Con	ppb	55.3	5.5	59.0	5.4	37.2	6.1	28.3	7.7	84.7	5.3	7.9	8.0	6.5
Frequency	%	100%	18%	100%	19%	100%	35%	100%	61%	100%	17%	57%	64%	43%
Frequency	%	100%	0%	100%	0%	100%	2%	100%	4%	100%	0%	2%	3%	10%
Frequency	%	94%	0%	98%	0%	58%	0%	7%	0%	100%	0%	0%	0%	2%
Freq Outflo	%	100%	44%	100%	47%	100%	64%	100%	86%	100%	43%	82%	85%	69%
95th Perce	ppb	87.76	14.54	96.97	15.15	65.41	16.52	51.23	19.27	134.67	14.76	19.49	19.63	17