SOUTH FLORIDA WATER MANAGEMENT DISTRICT

Documentation of the South Florida Water Management Model *Version 5.5*

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sfwmd.gov

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PREFACE

This document provides information relevant to the South Florida Water Management Model (SFWMM), version 5.5. The South Florida Water Management Model is the most detailed, physically-based simulation model that combines the hydrology and management aspects of a greater portion of the South Florida Water Management District (SFWMD or District). The model is regional in spatial extent (covering most of South Florida) and it encompasses an area of substantial heterogeneity in both natural and managed hydrology. The most distinguishing characteristic of the model is that it has a 2-mile x 2-mile fixed-resolution grid system. Consequently, it is often referred to as the 2x2.

The SFWMM is a coupled surface water-groundwater model which incorporates overland flow, canal routing, unsaturated zone accounting and two-dimensional single layer aquifer flow. The model is site-specific because it was exclusively developed for the South Florida region. In addition to simulating the natural hydrology in South Florida, the model also simulates the management processes that satisfy policy-based rules (both existing and proposed) to meet flood control, water supply and environmental needs. The model domain encompasses complex natural and managed hydrologic systems and the system components are highly interdependent. Local changes within the regional system can have far-reaching impacts on the hydrology at other locations within the system. Evaluation of the impacts of proposed changes to the regional system is a complex and challenging task -- one that requires a thorough understanding and knowledge of the entire modeling domain.

The model runs on a daily continuous simulation mode, instead of event-based, for 36 years (1965-2000 period-of-record). The model has performed well in various applications using Sun Workstations™ running under the UNIX™ operating system. Current model applications require about 2 hours of run time on a nicely configured Sun Workstation™.

The main text of this document is divided into five chapters: General Introduction, Physical and Hydrologic Components, Policy and System Management Components, Calibration, and Sensitivity Analysis. The main text will be printed, while many of the appendices will only be available on CD. There are 18 accompanying appendices, 5 of which are printed in a separate volume. The remaining 13 appendices are available on CD. The appendices are organized into three sections: Model Application Information; Model Development Information; and pertinent Technical Memoranda (please refer to Table of Contents). The entire publication is available on the CD.

The model overview is presented in the first chapter where a general description of the SFWMM is given. A short introduction to the model is followed by a history of its evolution from the 1970s to the present. Although the model is referred to as a hydrologic simulation model, it goes beyond simulating the components of the hydrologic cycle. In fact, the majority of the model code deals with the complex operational and management aspects of the existing and proposed hydraulic infrastructure in the modeled area.

Hydrologic processes such as rainfall, evapotranspiration, overland flow, subsurface flow and canal routing are discussed in Chapter 2. Chapter 3 deals with the operational aspects of the extensive system of canals, structures, and operations that form the Central and Southern Florida Project (C&SF Project). The material is presented by geographical area. The different operating policies that apply in each area, together with their corresponding model implementations, are explained.

Calibration topics, covered in Chapters 4 through 5, include calibration and sensitivity analysis. Model calibration is used to reinforce the model's predictive capability by showing how well the simulated stage and discharge values match historical data. Results from a sensitivity analysis, expressed as correlation of model input parameter and model output, can be used as a guide during model calibration and as a tool for establishing priorities in future data collection activities. The appendices provide extensive detail on a variety of subjects presented in the main text. Generally, only a reader wanting detailed information will reference the appendices.

The intent and purpose of this document is to provide information about the SFWMM, its processes, capabilities and shortcomings. The reader should be aware that the discussions in the following chapters pertain to version 5.5 of the SFWMM and the information could be superseded in the future. This document supersedes the SFWMD publication entitled "A Primer to the South Florida Water Management Model (Version 3.5)" which was released in April, 1999. Additional information and updates to this documentation and the model may come in several forms: technical notes, memoranda, presentations or reports which may be provided on the SFWMM website.

Finally, this document is not intended to be a user's or programmer's manual. However, it does provide detailed information about the input files. It should be used as a reference guide to the structure and algorithms of the model, the sources and nature of the input files, and the basic capabilities of the model. This documentation was prepared with a broad audience base in mind. Proficiency with the use of the model itself cannot be gained merely by reading this document.

ACKNOWLEDGEMENTS

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ABBREVIATIONS

ac-ft	acre-feet
ac-ft/mo	acre-feet per month
ac-ft/yr	acre-feet per year
af	acre-feet
af/d	acre-feet per day
af/m	acre-feet per month
ADE	Alternating Direction Explicit
AFSIRS	Agricultural Field -Scale Irrigation Requirements Simulation Model
ASCII	American Standard Code for Information Interchange
ASR	Aquifer Storage and Recovery or Aquifer Storage and Retrieval
BCNP	Big Cypress National Preserve
BCR	Big Cypress Reservation
BMP	Best Management Practice
C&SF Project	Central and Southern Florida Flood Control Project
CERP	Comprehensive Everglades Restoration Plan
cfs or ft^3/s	cubic feet per second
Corps	U.S. Army Corps of Engineers (same as USACE, ACOE or COE)
CPU	Central Processing Unit
CSOP	Combined Structural and Operational Plan
CWMP	Caloosahatchee Water Management Plan
District	South Florida Water Management District (same as SFWMD)
DSS	Hydrologic Engineering Center's Data Storage System (same as HECDSS)
EAA	Everglades Agricultural Area
ECP	Everglades Construction Project
EIS	Environmental Impact Statement
ENP	Everglades National Park
ENR Project	Everglades Nutrient Removal Project
EPA	Everglades Protection Area
ET	evapotranspiration
FAO	Food and Agriculture Organization
FASS	Florida Agricultural Statistics Service
FGFWFC	Florida Game and Freshwater Fish Commission
FLUCCS	Florida Land Use and Cover Classification System
FPL	Florida Power and Light
ft	feet
GCC	GNU Complier Collection
GIS	Geographical Information System
GOF	Goodness-of-fit
GPS	Global Positioning System
HECDSS	Hydrologic Engineering Center's Data Storage System (same as DSS)
HW	Headwater
in.	inches
in/yr	inches per year

IOP	Interim Operational Plan
ISOP	Interim Structural and Operational Plan
lb	pounds
LEC	Lower East Coast
LECSA	Lower East Coast Service Area
LECRWSP	Lower East Coast Regional Water Supply Plan
LIDAR	Light Detection and Ranging
LNWR	Loxahatchee National Wildlife Refuge
LOK	Lake Okeechobee
LOSA	Lake Okeechobee Service Area
LWDD	Lake Worth Drainage District
MGD	million gallons per day
mgm	million gallons per minute
mi	miles
NAD83(90)	North American Datum 1983(1990)
NAVD88	North American Vertical Datum 1988
NEPA	National Environmental Policy Act
NESRS	Northeast Shark River Slough
NGVD	National Geodetic Vertical Datum
NOAA	National Oceanic and Atmospheric Administration
NOS	National Ocean Service
NPBSA	North Palm Beach Service Area
NSM	Natural System Model
PA	Position Analysis
PDE	Partial Differential Equation
PWS	Public Water Supply
QA/QC	Quality Assurance/Quality Control
RDO	Rain Driven Operations
Restudy	C&SF Project Comprehensive Review Study
RFP	Rainfall Plan
SCS	Soil Conservation Service
SCCS	Source Code Control System
SDCS	South Dade Conveyance System
sec	seconds
SRS	Shark River Slough
SFRSM	South Florida Regional Simulation Model
SFWMD	South Florida Water Management District (same as District)
SFWMM	South Florida Water Management Model
SSM	Supply-Side Management
STA	Stormwater Treatment Area
sq mi	square miles
SWIM Plan	Surface Water Improvement and Management Plan
TIN	Triangular Irregular Network
TW	Tailwater
USACE	U.S. Army Corps of Engineers (same as Corps, ACOE or COE)

USFWS	U.S. Fish and Wildlife Service
USGS	U.S. Geological Survey
UTM	Universal Transverse Mercator
WATBAL	Water Balance Model for AFSIRS
WCA	Water Conservation Area
WMA	Water Management Area
WSE	Water Supply and Environment

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1 GENERAL INTRODUCTION

1.1 THE CENTRAL AND SOUTHERN FLORIDA REGION

The Central and Southern Florida (C&SF) region generally refers to the watershed that starts in the Kissimmee River Basin (near Orlando, Florida) and flows southward through Lake Okeechobee to Florida Bay with waterways to the lower east and lower west coasts of Florida. The 17,930 square miles of the C&SF region are contained in the boundary of the South Florida Water Management District (Figure 1.1.1).

One of the most distinguishing characteristics of the region is the relatively flat terrain. From just south of Lake Okeechobee to Florida Bay, about 110 miles, the land surface elevation only drops about 16 feet. The average depth of Lake Okeechobee is less than 10 feet, the maximum depth is about 18 feet and it covers 730 square miles.

Prior to anthropomorphic influences, water flowed freely from the Kissimmee River Basin southward into Lake Okeechobee (Figure 1.1.2). As the rainy season progressed, from May to October, water began overflowing the southern rim of the Lake and provided an expansive sheet flow into the grassy wetlands of the Everglades. In most years, the overflow added to the rainy season runoff to create an extended period of flow into Shark River Slough and through, what is now, Everglades National Park (ENP). Shark River Slough rarely dried out.

Flows westward from Lake Okeechobee into the Caloosahatchee River Basin occurred only during the wettest years until a canal connected the Lake to the river in the late 1800s. Flows eastward to the St. Lucie Estuary did not occur until a canal connection was made in the early 1900s. Prior to development of the lower east coast of Florida, water from the Everglades flowed through the east coast ridge through narrow paths referred to as the transverse glades. Water along the western side of the Everglades flowed through several sloughs to the west coast of Florida into the Ten Thousand Islands area. Ultimately, the fresh water system fed into the Atlantic Ocean, Biscayne Bay, Florida Bay and the Gulf of Mexico.

With progressive development, the Kissimmee River Basin became a managed series of lakes and rivers. For flood control and navigation, the Caloosahatchee River was dredged and the St. Lucie Canal was created. A flood protection levee was built around most of Lake Okeechobee and Lake outflows became regulated for multiple purposes. In 1947, Everglades National Park (ENP) was established. In the 1950s, the Everglades south of the Lake were compartmentalized into several large areas; namely, the Everglades Agricultural Area (EAA) and five Water Conservation Areas (WCAs). The EAA is mostly comprised of sugar cane fields that are managed for flood control and water supply needs. The WCAs can be characterized as large, shallow reservoirs managed for several purposes. As the lower east coast of Florida developed, flood control and water supply became increasingly important operations.

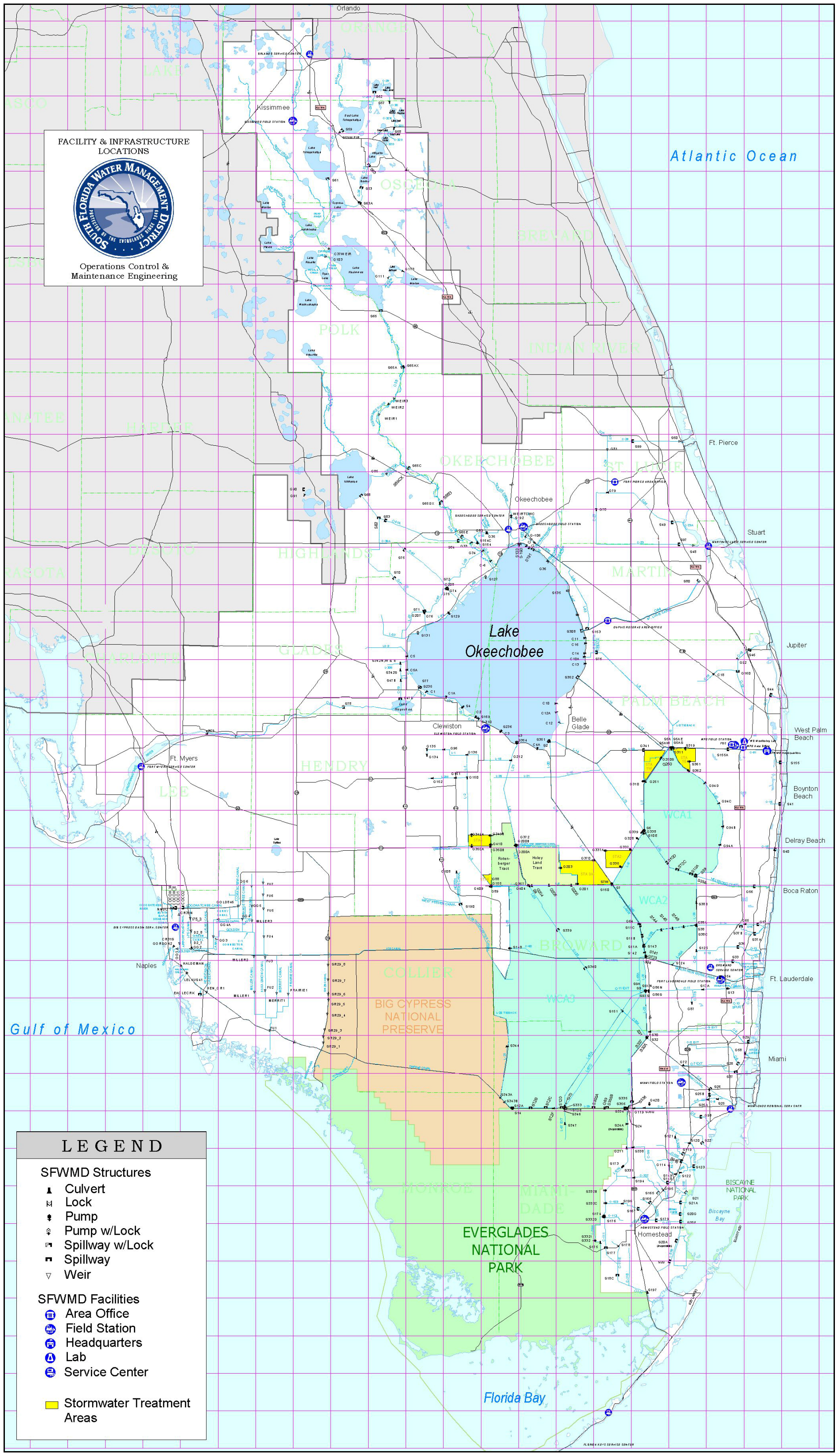

Figure 1.1.1 The Central and Southern Florida Region

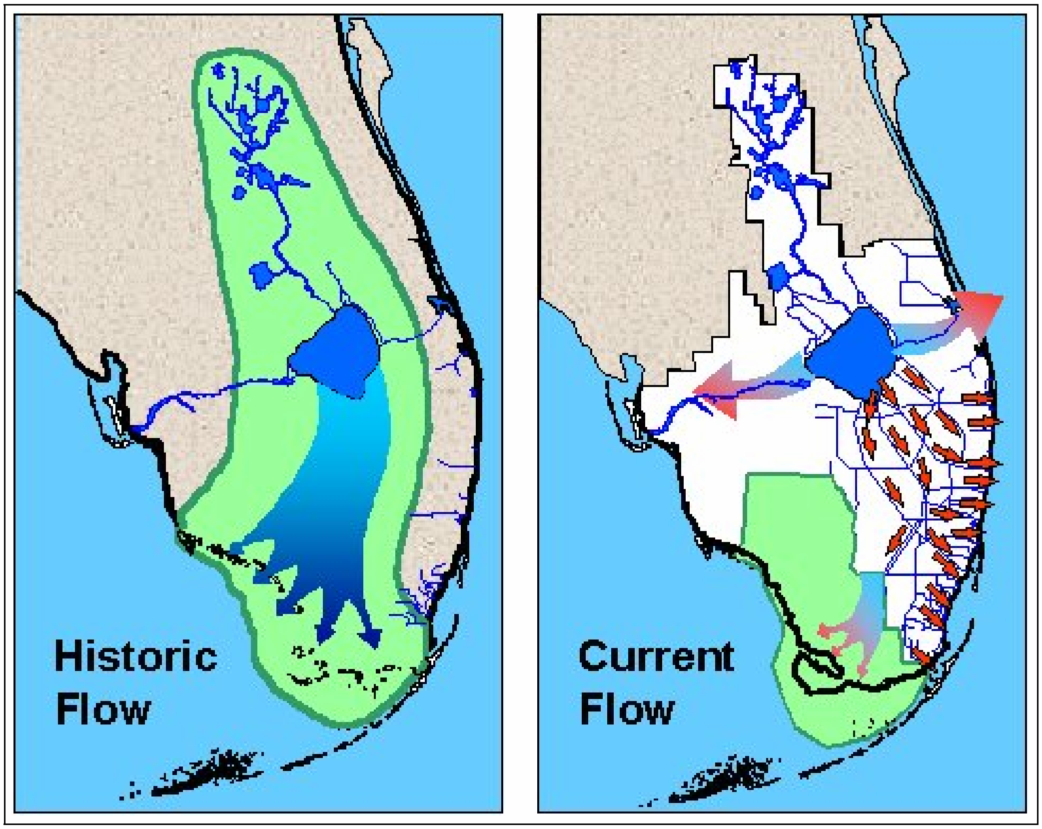

Figure 1.1.2 The Central and Southern Florida Regional Flow Characterization

Since the 1960s, water management has become more intensive and, often, controversial. With the dramatic changes in landscape and water management, many wetland species have been severely impacted. During wet periods, flood operations were designed to move water efficiently to the coast – resulting in salinity conditions too low to support stable estuarine environments. Droughts were exacerbated by the loss of fresh water during wetter times. A dry out in Shark River Slough (SRS) became an annual event. By the 1980s, there were over 1,100 miles of canals and levees and hundreds of water control structures. With the increase in water-related needs of the system, it was clear that a hydrologic numeric model was needed as a tool to evaluate and develop better water resource management options.

The South Florida Water Management Model is a regional-scale tool for addressing water management issues specific to South Florida, an area with diverse landscapes and a highly complex managed system. The objectives of the model are:

1. To simulate and evaluate the regional performance of potential changes to infrastructure or operations with respect to flood control, water supply, and environmental targets;
2. to gain insight on the performance of future scenarios by comparing alternatives or scenarios against each other, or against established baselines;
3. as a tool for operational planning, to simulate a range of probable outcomes based on the current state of the system and a range of climatologic inputs;
4. to provide a basis for further regional and sub-regional evaluations;
5. to provide quantitative hydrologic information for water resources planning strategies developed by the South Florida Water Management District;

6. to represent all the important hydrologic and related physical processes in South Florida;
7. to represent the important infrastructure, policies, and operation of the managed system;
8. to apply the most advanced modeling techniques and methodologies appropriate for each distinct area within South Florida; and
9. to apply all processes, techniques, and methodologies, in a manner consistent with the spatial and temporal scale of the model, which are justified by available data.

1.2 HISTORICAL BACKGROUND

“There are no other Everglades in the world. They are, they have always been, one of the unique regions of the earth; remote, never wholly known.” -- Marjory Stoneman Douglas

There are no other numerical models of the Everglades that can account for the suite of hydrologic processes and water management options that are unique to South Florida. The South Florida Water Management Model (SFWMM) is the primary tool used to evaluate the interaction of water supply and demand with hydrologic conditions in Palm Beach, Broward and Miami-Dade Counties and portions of seven other counties in South Florida. Initial work on the model started in the 1970s. The model was completed by the South Florida Water Management District (SFWMD or District) under contract (DACW17-81-C-0035) for the U.S. Army Corps of Engineers (USACE or Corps). Technical Publication 84-3 (TP84-3) "South Florida Water Management Model Documentation Report" was released in February 1984.

Driven by the need to evaluate additional complex water management options and longer periods of record, the SFWMM has evolved through several major revisions. In the early 1980s, the model ran a 14-year period of record from 1965 to 1984. By the late 1980s, the model was used to evaluate potential impacts of several major projects. In the early 1990s, the model was simulating a 19-year period of record with expanded capabilities. At that time, there was a special interest in simulating the natural system, specifically the remnant Everglades. By removing the water conveyance infrastructure and operational policies from the SFWMM, a new model was created that made simulation of the “natural conditions” possible. The new model was called the Natural System Model (NSM) and was completed in 1991. NSM is used to infer how the system might have behaved before anthropomorphic changes to the environment (SFWMD, 1998). Because the NSM uses the same hydrometeorological record as the SFWMM, comparison between “natural conditions” and managed systems can be made more reliably.

Throughout the early to mid-1990s, the SFWMM continued to expand in capability and application. However, not all improvements were made solely to the model code; some improvements were made to develop a SFWMM modeling system. Geographical Information System (GIS) products provided additional spatially-oriented features and the development of visualization tools enhanced the ability to review output. In 1997, a draft of “Documentation for the South Florida Water Management Model” was produced and a peer review was initiated and completed a year later.

In 1997, the SFWMM v3.5 modeling system was ready for the most ambitious application up to that time – the development and evaluation of the Central & Southern Florida (C&SF) Restudy. The C&SF Restudy was a holistic review of the C&SF region with the focus of improvement on restoration of the natural areas while respecting the other water-related needs of the region. By that time, the period of record spanned 31 years starting in 1965. All major operational rules for the system were simulated within the model. Not only were over 900 performance indicators and measures being produced, but they were available on the web for the public to evaluate and provide comments. The SFWMM was used to make sensitivity runs to better define operational and design guidelines for numerous components. Multi-agency teams developed the input criteria and evaluated modeling results. The model output and post-processed information allowed non-technical stake-holders, hydrologists, engineers, biologists, and ecologists to converse in common language. A restoration plan was developed and subsequently approved, in concept, by Congress in December of 2000.

Today, there are approximately 1,800 miles of canals and levees, 25 major pumping stations and about 200 larger and 2,000 smaller water control structures. The model has been dramatically improved and continues to play a crucial role in the evaluation of water resource management in South Florida. This document provides pertinent information about the science and capabilities of the model as they exist in the SFWMM, version 5.5. Figure 1.2.1 provides an evolution of the development and application of the SFWMM to date.

This documentation describes general model characteristics, hydrologic processes, management options, and simulation methods. The SFWMM is a useful tool to evaluate regional water budgets for establishing water reservations and to evaluate alternatives for managing the water resources of the C&SF region. It provides valuable information, such as boundary conditions, that can be used in local or smaller-scale hydrologic/hydraulic models in South Florida.

The most unique feature of the SFWMM is the ability to simulate operational scenarios, management options, and define regional water budgets. There are other surface/groundwater models that could be applied to the hydrology of the Everglades (especially at a sub-regional scale), but there are no other models that have the suite of management options and operational flexibility of the SFWMM for large-scale, system-wide interactions. Examples of the flexibility and operational features of the model will be discussed primarily in Chapter 3.

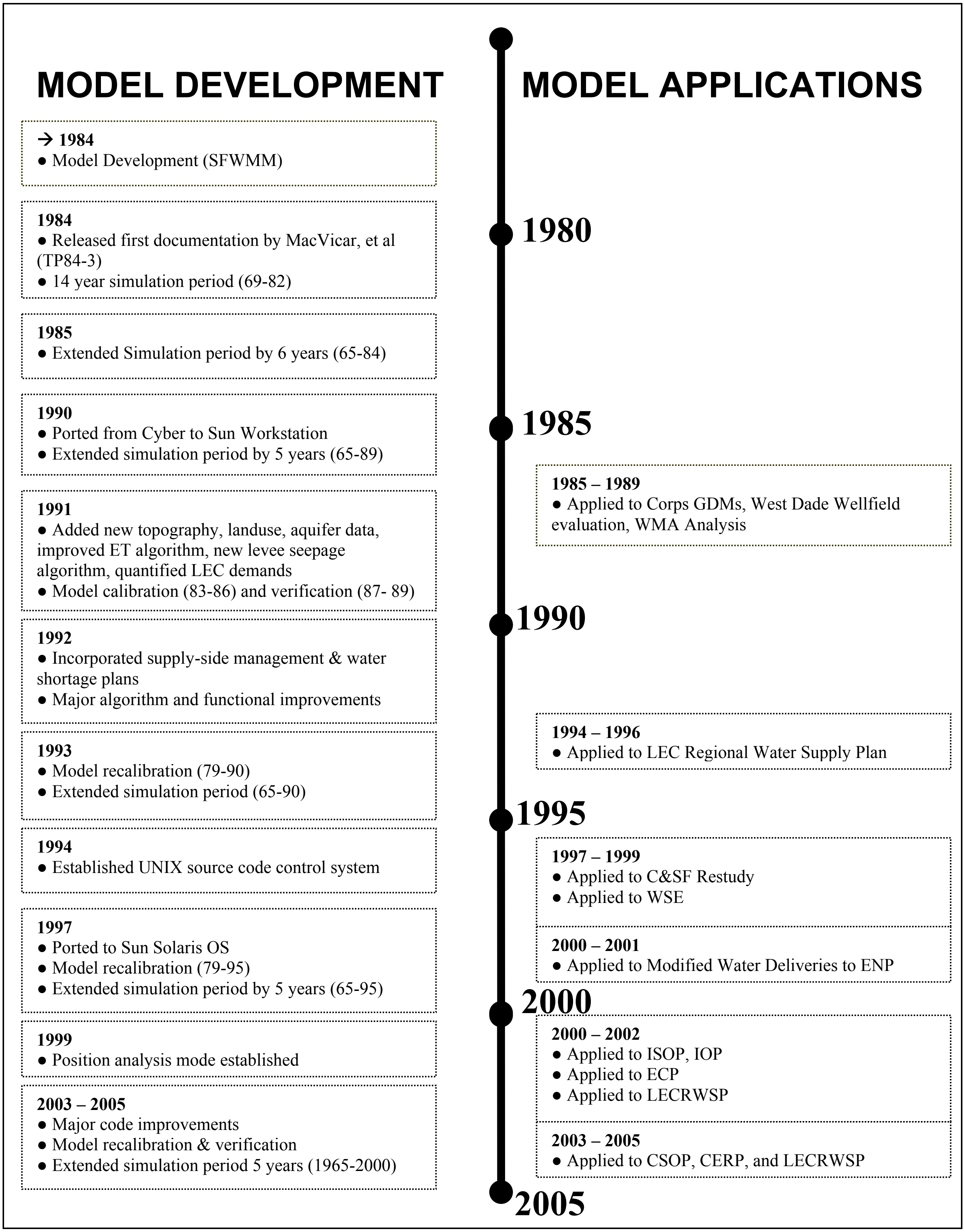

Figure 1.2.1 Evolution of the South Florida Water Management Model

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1.3 MODEL CHARACTERISTICS

The South Florida Water Management Model (SFWMM) is a regional-scale hydrologic model that simulates physical processes in the natural (coupled surface water and groundwater) and man-made (canals, structures and reservoirs) systems in South Florida. It includes management and operational rules established, mostly by the U.S. Army Corps of Engineers (USACE or Corps), for operating the Central and Southern Florida Project (C&SF Project) for flood control and other purposes. As a planning tool, the model can be used to predict the response of the hydrologic system to proposed changes in hydraulic infrastructure and/or operating rules. The design of the model takes into consideration the distinct hydrologic and geologic features of subtropical South Florida which includes: 1) the strong interaction between canals and the highly permeable surficial aquifer, especially in the eastern portion of the region; and 2) the effects of rainfall, evapotranspiration, overland flow and groundwater movement within the Water Conservation Areas (WCAs) and Everglades National Park (ENP). To jointly simulate these complex processes, a distributed parameter/cell-based network is used. The SFWMM integrates hydrologic processes with the hydraulic infrastructure and associated policy-based rules and guidelines related to water management in South Florida.

Figure 1.3.1 shows the model boundary relative to South Florida. The model is conceptualized at varying levels of detail (as described below) for three different major geographic areas: (1) for Lake Okeechobee, (2) for the combined extent of the Everglades Agricultural Area (EAA), the Everglades Protection Area (EPA) and the Lower East Coast (LEC) and (3) for non-EAA Lake Okeechobee Service Area (LOSA) basins. The necessity to break the model into these areas is primarily due to issues of data availability which, in turn, require different computation methods. The SFWMM employs both lumped and distributed modeling techniques in its approach to model these areas.

Lake Okeechobee is modeled as a lumped system, or regarded as a single point in space without dimensions where simulated water levels and/or flow rates are spatially averaged. The amount, timing and distribution of structure flows in and out of Lake Okeechobee are dictated by management rules related to flood control, water supply, and environmental restoration. One might note that some of these rules (e.g. regulation schedules and supply-side management) are actually in operation but a few more are incorporated in the model to address proposed operating policies, specifically those related to the Everglades environmental restoration.

The gridded portion of the model domain describes the extent of the finite difference solution to the governing overland and groundwater flow equations and is defined just south of Lake Okeechobee. The network is comprised of 2-mile square grid cells that cover the large coastal urban areas of Palm Beach, Broward and Miami-Dade Counties; the EAA; the WCAs and ENP. The total coverage of the model is 1,746 grid cells. The model assumes homogeneity in physical as well as hydrologic characteristics within each grid cell. With this assumption, a grid cell may also be referred to as a nodal point or simply, a node. In addition to water levels at grid cells, and surface and groundwater flow between cells, the model also calculates discharges for the major hydraulic structures within the model grid.

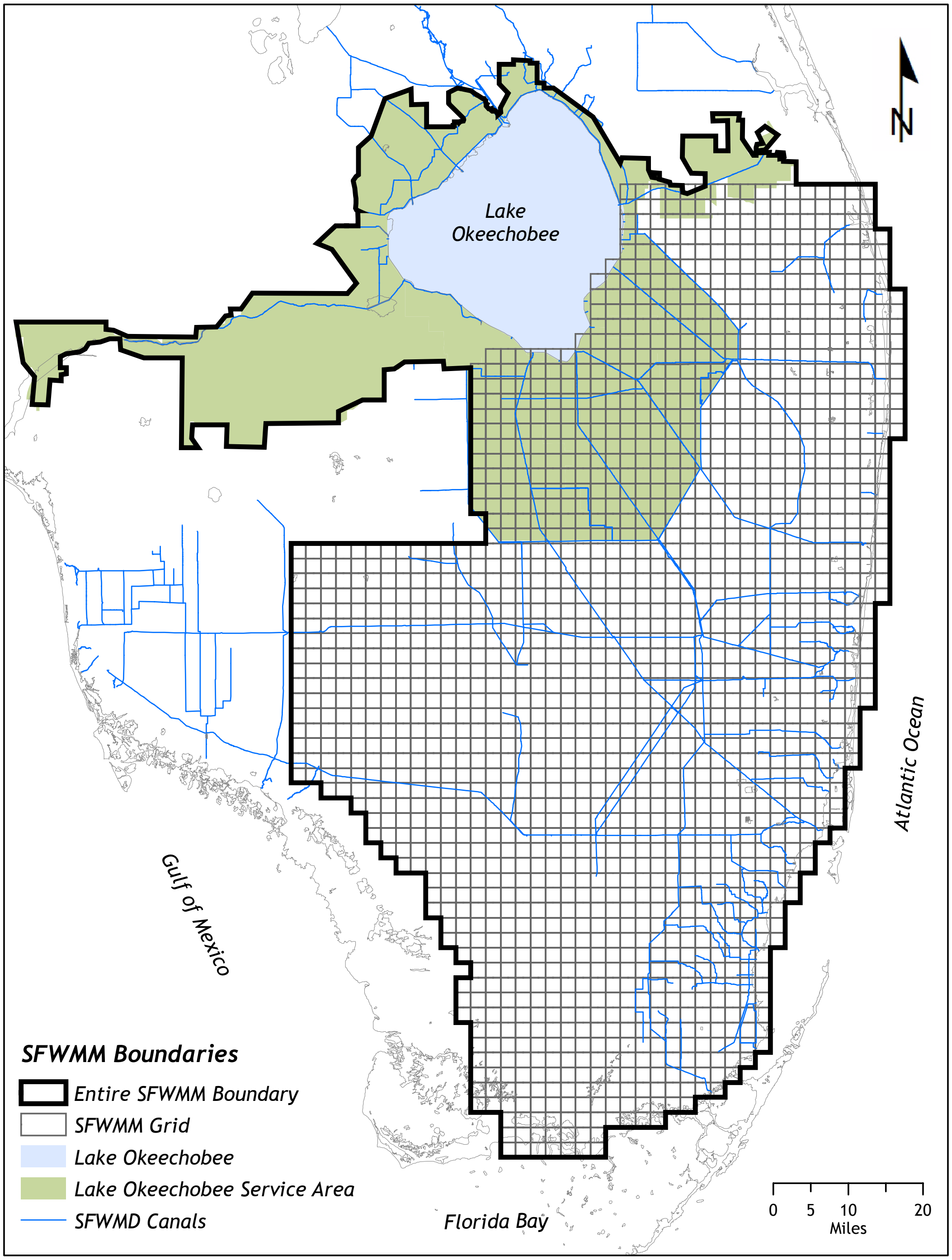

Figure 1.3.1 South Florida Water Management Model Boundaries

Finally, a simple flow balance procedure is used for the rest of the LOSA excluding the EAA. In these basins, pre-processed user-input demand and runoff characteristics are combined with appropriate system operational rules to calculate flow distributions. Hydrologic characteristics such as rainfall and evapotranspiration amounts and basin internal flux terms are accounted for in the pre-processing tools. As a result of the way in which these basins are conceptualized, water levels, overland flow and groundwater flow are not simulated in this portion of the LOSA. They are assumed to be consistent with the time series of demand and runoff quantities which are otherwise calculated in the gridded portion of the model domain.

A fixed time step of one day is used in the model; however, for overland flow, time slicing is used as discussed in Section 2.3. The selection of this time step is consistent with the minimum time increment for which hydrologic data such as rainfall, evaporation and structure discharge are generally available. Rainfall and potential evapotranspiration (PET) are the primary driving processes. Therefore, the longest total simulation time for the model is a function of the available historical (or an estimate of historical) rainfall and PET data. The SFWMM version 5.5 can be run for as short as one month and for as long as 36 years from January 1, 1965 through December 31, 2000. The hydrologic processes are generally modeled sequentially within one time step. A continuous unconfined groundwater system is assumed to underlie the gridded portion of the model domain. To simplify programming and reduce computational time, no iteration is performed between surface water and groundwater routines within a time step. Calculations for more transient phenomena, such as channel flow routing, are performed before less transient phenomena, such as groundwater flow, within a time step. The bulk of the computer code, on the other hand, is comprised of the operational rules that drive the human management of the entire system. The close relationship between the natural hydrology and hydraulic infrastructure in South Florida makes the SFWMM unique.

Data required to describe the physical features of the modeling domain such as land elevation and land use types are readily available from the District's Geographical Information System (GIS) database. Many physical parameters such as seepage rate factors, overland flow roughness coefficients and aquifer transmissivity were estimated within reasonable ranges. A calibration of the model was recently performed to ascertain the values of these parameters. In general, the purpose of this effort was to verify and/or improve the predictive capability of the model by: (1) incorporating the best available data; (2) introducing new/improved algorithms into the model; and (3) adjusting calibration parameters to obtain a close agreement between model output and historical flow and/or stage data. Included in this report is a representative sample of calibration results in different areas within the system (refer to Chapter 4).

Sun™ FORTRAN was the programming language used in coding the SFWMM and the Sun™ GCC compiler is used to create the executable code. In version 5.5, the source code has a total of about 77,000 lines of code grouped into more than 95 subroutines and 150 functions. The subroutines are liberally documented and each subroutine has a short description of the purpose of the subroutine. The model can be run on a Sun Sparcstation™ under the SunOS™ 8.0 (or later) operating system. Flat: text or American Standard Code for Information Interchange (ASCII) format; binary: Grid_io (Van Zee, 1993); and Hydrologic Engineering Center's Data Storage System (HECDSS) (U.S. Army Corps of Engineers, 1994) formats are used on both input and output. Total execution time varies according to central processing unit (CPU) speed, network traffic and the scenario being simulated. As of this writing, the execution time is about 1.5 hours on a Sunblade 2000™ workstation; however, execution times can exceed 2 hours depending upon the management options selected.

The general hydrologic processes simulated by the gridded portion of the SFWMM are depicted in Figure 1.3.2. The loss of water to the atmosphere by evapotranspiration is considered by the model to occur from above and below the land surface. This distinction makes it possible to produce a water budget for the entire layer of the soil column as well as the saturated and unsaturated zones that comprise the subsurface region of the model. Overland flow can be partitioned into a cell-to-cell transport of surface water (sheetflow) and movement of surface water directly into a receiving canal (drainage). Other processes such as seepage across levees, and leakance/seepage into and out of canals fall under the general category of groundwater flow and are discussed in more detail in the following sections. Finally, canal flow describes the passage of water from one water body, typically a canal reach, across a hydraulic structure into another water body such as a downstream canal reach, reservoir or detention facility.

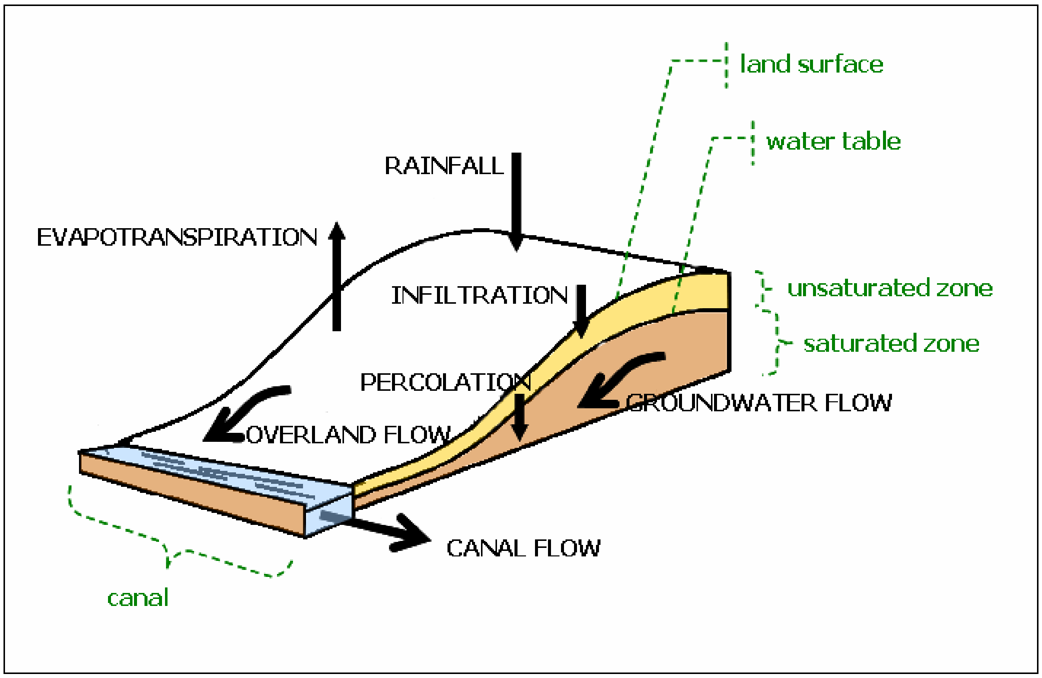

Figure 1.3.2 General Hydrologic Processes in the South Florida Water Management Model

The degree of complexity increases as one superimposes the hydraulic structures and corresponding operational rules on the system. Thus, Figure 1.3.2 only shows the natural hydrology simulated in the model. The operational and management component is more complex and the discussion of the corresponding processes will be made with respect to the areas where they apply. Figure 1.3.3 is a simplified flowchart of the overall organization of the model. An expanded "call tree" flowchart is given in Appendix H. A written presentation of the model process description is provided in Appendix J. A short description of the purpose of each subroutine used in the model is given in Appendix I. The SFWMM v5.5 source code is shown in Appendix K.

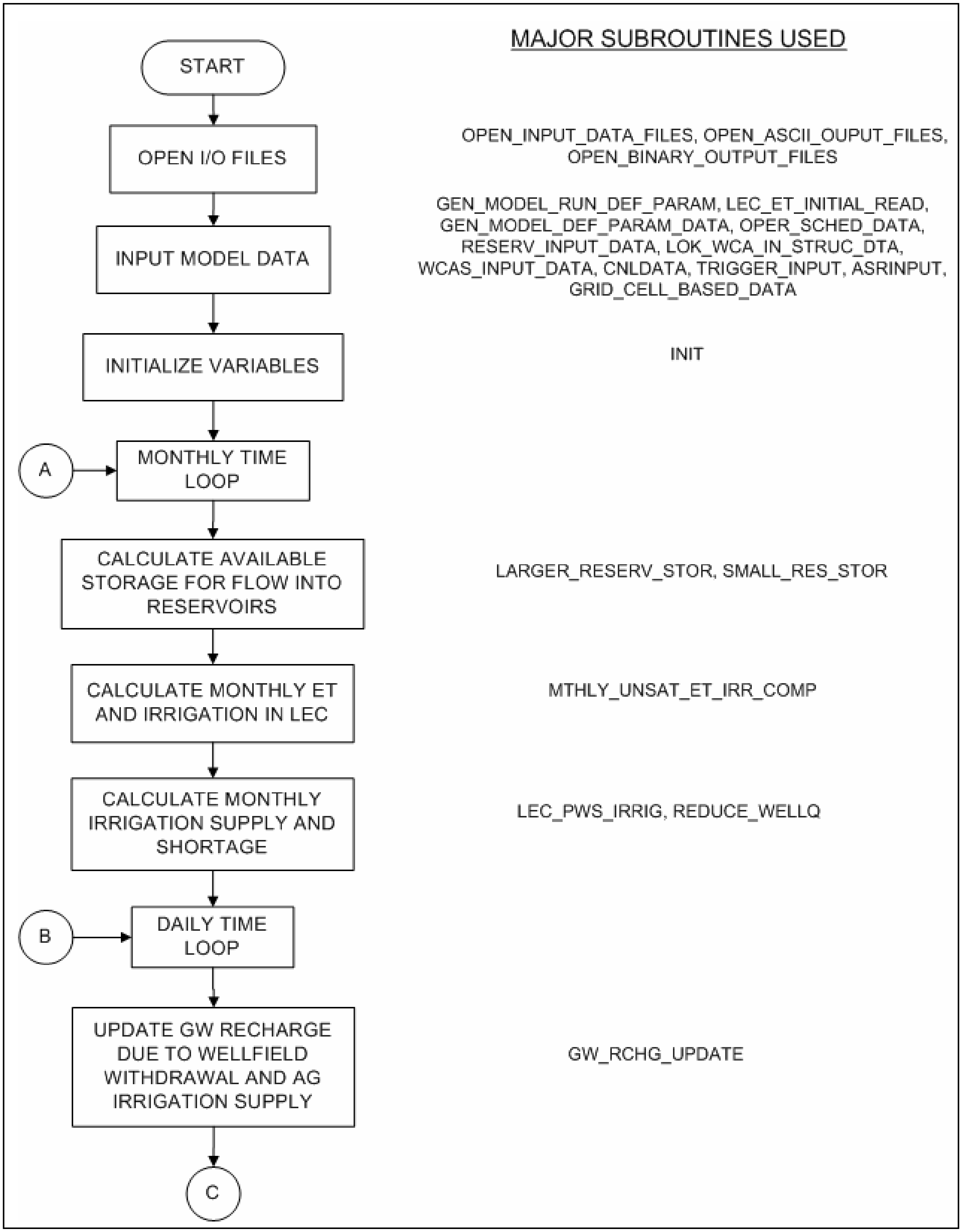

Figure 1.3.3 Simplified Flowchart for the South Florida Water Management Model v5.5

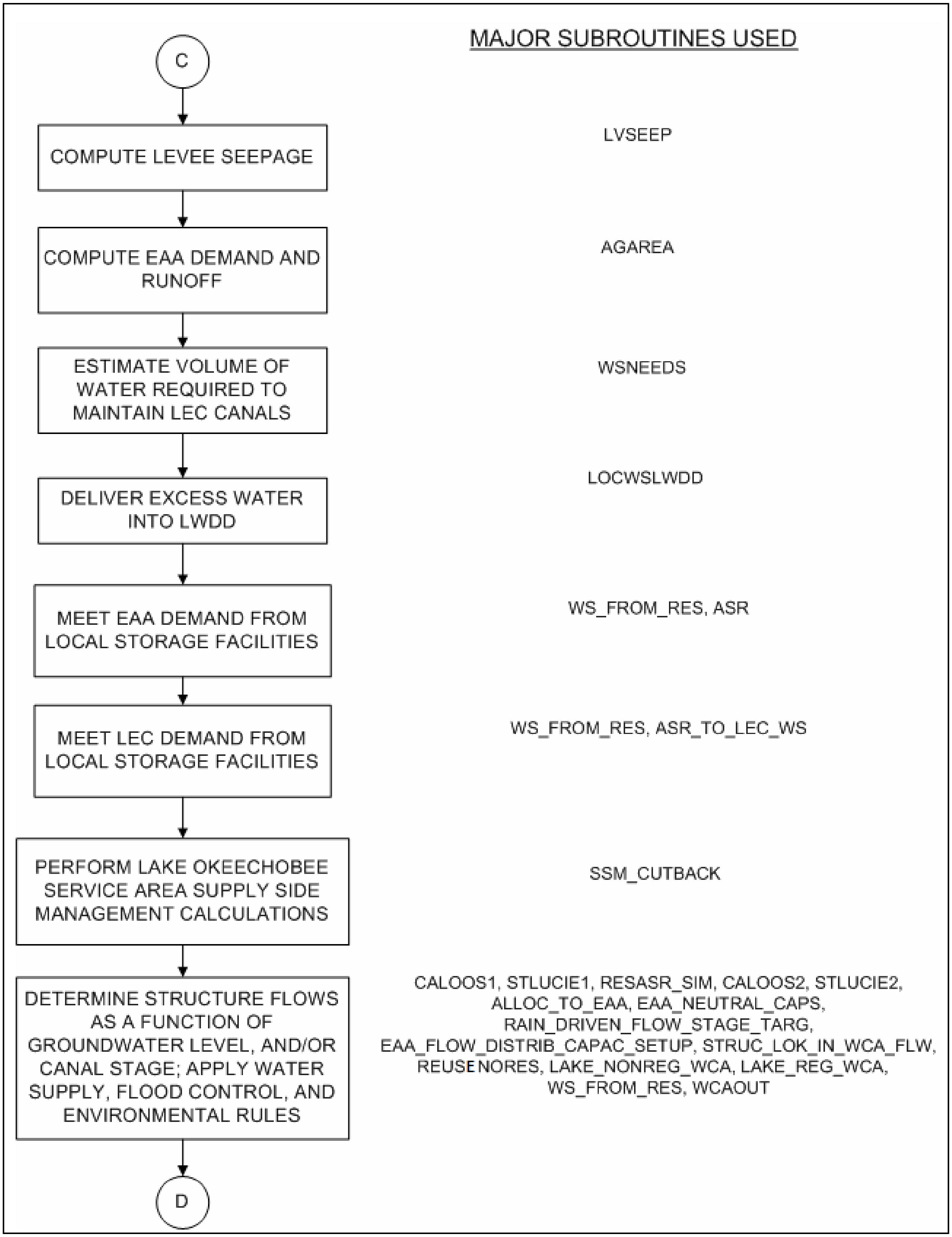

Figure 1.3.3 (*cont.*) Simplified Flowchart for the South Florida Water Management Model v5.5

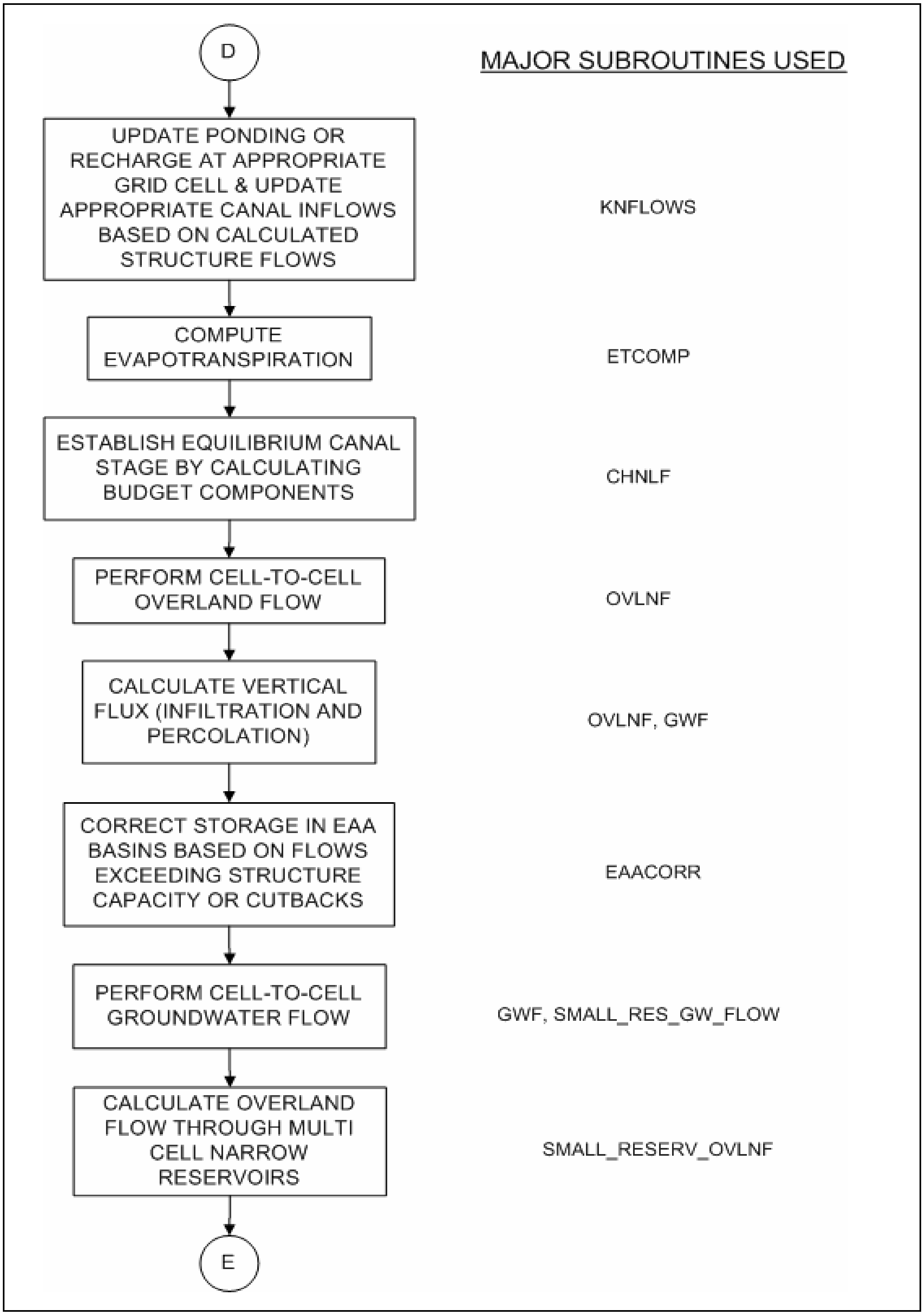

Figure 1.3.3 (*cont.*) Simplified Flowchart for the South Florida Water Management Model v5.5

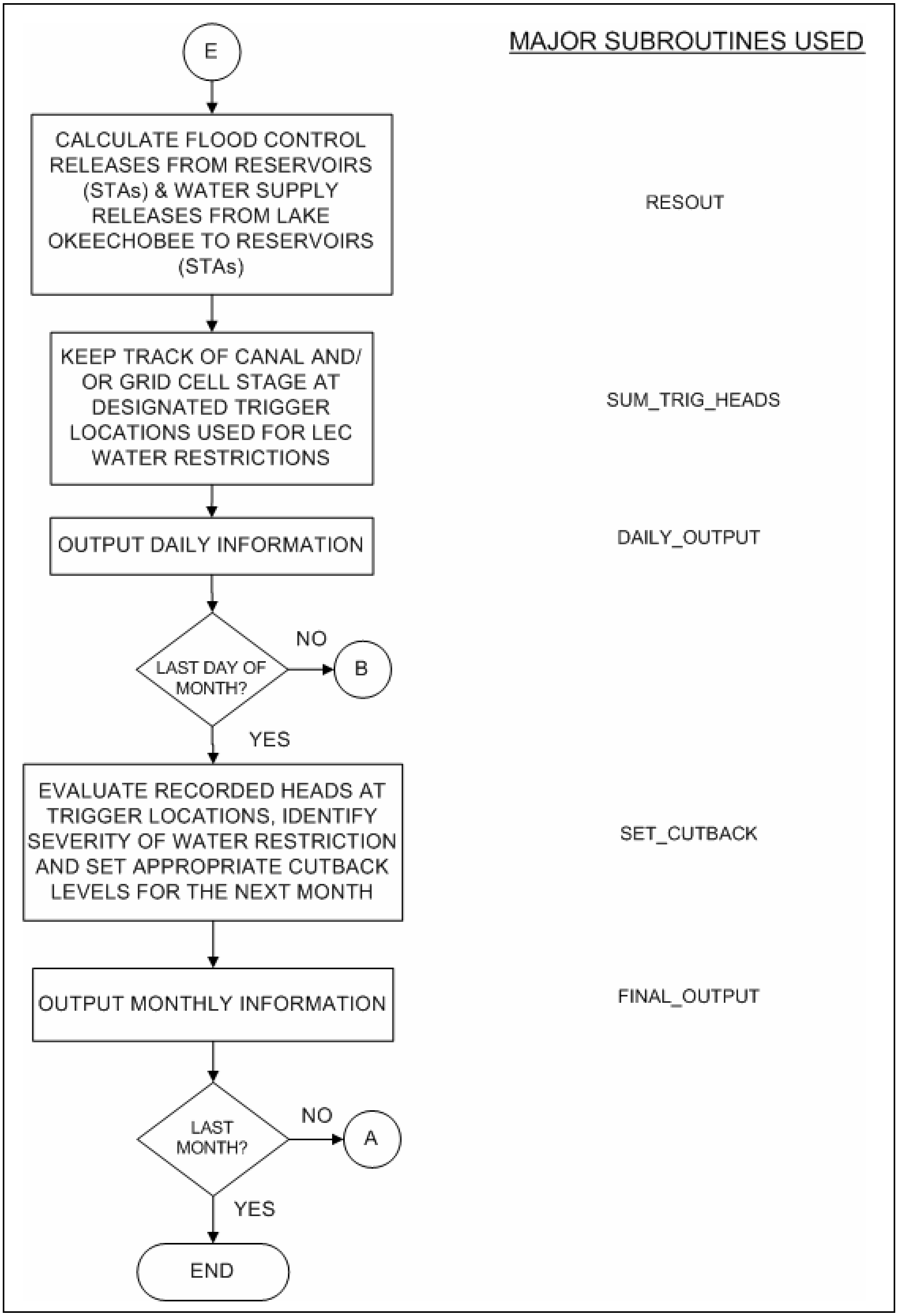

Figure 1.3.3 (*cont.*) Simplified Flowchart for the South Florida Water Management Model v5.5

Figure 1.3.4 illustrates how the SFWMM relates to a variety of support utilities (pre- and post-processors). Due to the enormous amounts of input required and output generated by the model, an entire suite of utility programs has been developed. A substantial increase in efficiency in evaluating modeling scenarios is realized by using these programs. The interaction between data and computer programs shows that the model should not be considered only as a single computer program but as an entire modeling system or package.

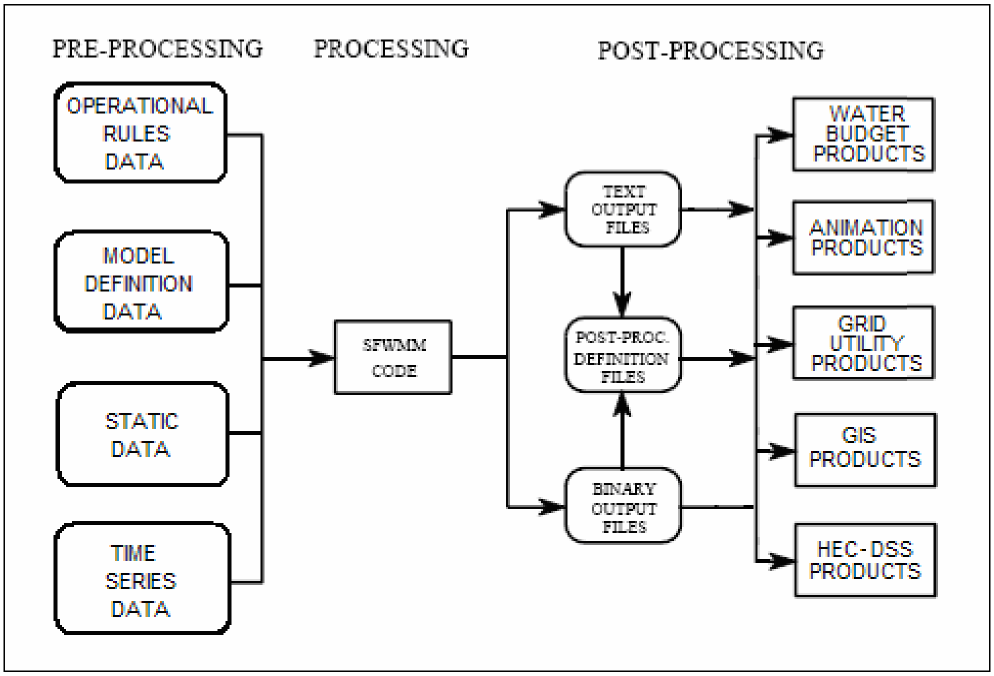

Figure 1.3.4 The South Florida Water Management Modeling System

One of the most effective ways of summarizing model output is by way of water budgets. A water budget is an accounting of all components of the hydrologic cycle within a bounded region. In the SFWMM, a water budget provides a quantitative breakdown of these components across the boundaries of areas in South Florida idealized as series of horizontal and vertical segments separating 2-mile by 2-mile grid cells. Figure 1.3.5 shows the major geographical areas included in the SFWMM. With the exception of some of the smaller LOSA basins, and small inflow basins north of Lake Okeechobee, water budget summaries are produced by the water budget program. Knowledge of water budgets for different subregions within the model enables one to make relative comparisons of the quantity and distribution of water within the entire modeling domain. Within the EAA (shown in Figure 1.3.5), water budgets for several subregions are generated which include Stormwater Treatment Areas (STAs), Wildlife Management Areas (WMAs), and storage reservoirs (when modeled). A discussion of input and output files, and performance measures is provided in Appendix A. Also included is a discussion of post-processing programs. Appendix B presents the UNIX "manpages" that describe the input files in detail.

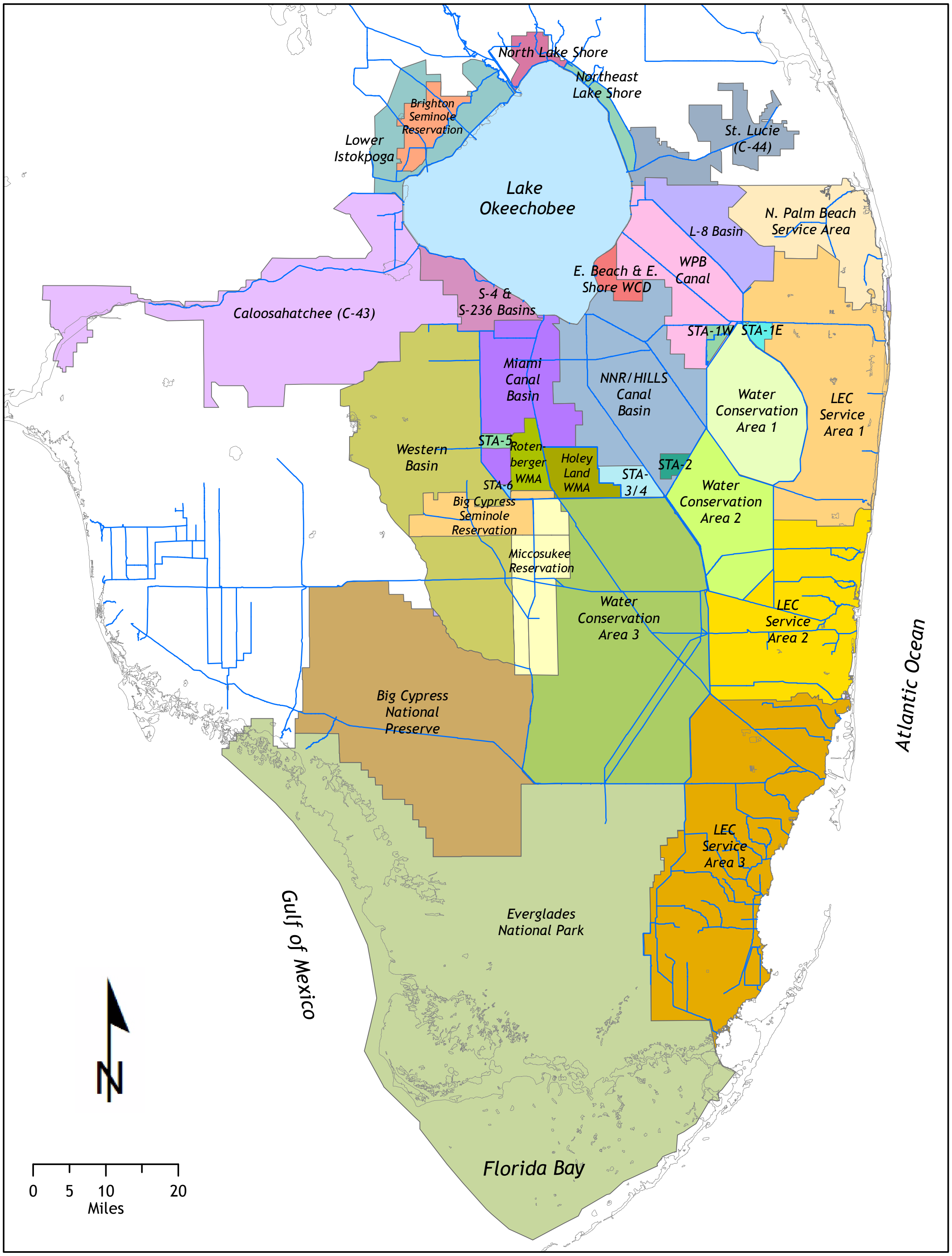

Figure 1.3.5 Major Geographical Areas within the South Florida Water Management Model

2 PHYSICAL AND HYDROLOGIC COMPONENTS

2.1 TOPOGRAPHY AND LAND USE

The primary non-calibrated static terms used in the South Florida Water Management Model (SFWMM) are topography and land use. The data input to the model for these components is considered to be constant throughout the period of simulation. While much of the topography data used in the model was developed under previous versions, several new data sets were available for SFWMM v5.5. The topography update process and results are provided in this section. The land use section describes the types of land covers available for simulation in the model and includes aerial photographs which show the relative differences between some of the more unique land cover types.

2.1.1 Topography

Topography data sources as utilized in the SFWMM are illustrated in Figure 2.1.1.1. The newer data sets used in updating SFWMM v5.5 covered the Everglades Agricultural Area (EAA) and the natural areas to the south of the EAA. Other sources of data came from a variety of sources and represented the best available data from past years where metadata were not generally recorded. The sources of the older topography data will be discussed at the end of this section.

After considering existing documentation, spatial location, and quality of several new topography datasets, five datasets were selected for incorporation into this update. Additionally, it was decided to uniformly lower the elevation of the EAA based on a uniform subsidence rate. The EAA has several factors which cause rapid subsidence, most importantly aerobic microbiological decomposition (oxidation). Measured rates of subsidence (Shih et al., 1997) were used to determine a rate of subsidence in the EAA for the last decade. The Holey Land and Rotenberger Wildlife Management Areas were excluded from this subsidence adjustment. The new datasets are listed below.

1. *High-Accuracy Elevation Data collection from the United States Geological Survey (USGS).* This data consists of elevation values on a regular grid of 400 meters, throughout the Everglades National Park (ENP) and portions of southern Miami-Dade County. Data in the western limits of the ENP have not been collected or finalized. The data was collected in the North American Datum 1983 (1990) [NAD83(90)] horizontal datum and the North American Vertical Datum 1988 (NAVD88) vertical datum. The stated vertical accuracy is 0.5 feet.
2. *LIDAR (Light Detection and Ranging) elevation data collected for Water Conservation Area (WCA) 3A, north of Interstate 75 (I-75).* This data was contracted by the USGS to EarthData International, Inc. The raw data was re-sampled to 5-meter pixels and processed by the contractor, using proprietary algorithms, to represent bare-surface elevation. The stated vertical accuracy is 15 centimeters, or approximately 0.5 ft.
3. *The Rotenberger Wildlife Management Area Survey, 1999.* This survey was conducted by Lindahl, Browning, Ferrari, and Hellstrom, Inc. Using Global Positioning Survey (GPS) technology and airboats, six east-west cross-sections were traversed, with elevations collected at approximately 0.25 mile spacing. The reported vertical accuracy of this data is 0.2 feet.

4. *The Stormwater Treatment Areas (STAs) 1990s.* These elevations were compiled by the Everglades Construction Project (ECP) and are based upon the best available data. The only data available are mean elevations for the STA cells.
5. *The 8.5 Square-Mile Area Survey, 1986.* This area was surveyed by Aero-metric Corporation under contract to the United States Army Corps of Engineers (USACE or Corps), from January to April 1986. Elevations were collected on a 300-foot grid using conventional methods. The purpose of the survey was to produce cross-sections for hydrologic modeling. The vertical accuracy is reported to be 0.1 meter, or about 0.33 ft.

Other sources of data that were not used fell into two categories: not within the model domain or not appropriate to natural surface elevation modeling. The first category is clear; examples of the second category are as follows:

- *The LIDAR data collected by the USACE along and to the east of the levee separating the urban area of South Florida from the Everglades.* This data covered a relatively small area in comparison to the voluminous amount of data it contained. Also, it was not collected with regional-scale hydrology in mind, which seeks to represent the elevation of the natural terrain as opposed to man-made features such as roads and levees. Consequently, this data was not incorporated into the current elevation update.
- *The Truck Survey and the Airboat Survey conducted as part of the USGS High-Accuracy Elevation Data Collection.* These surveys were conducted differently from the more comprehensive Helicopter Survey (which represents the bulk of the collection). The documentation on these sets is sparse, and they were conducted in the urban portion of Miami-Dade County. An analysis of the data shows that the Truck Survey in particular did not target natural ground elevation specifically. For these reasons, the datasets were excluded.

The High-Accuracy Elevation Data (2001) collection was created using GPS technology in conjunction with numerous vehicles, including helicopter, truck, and airboat platforms. The portions of this dataset east of the eastern protective levee were excluded. The eastern area was collected primarily by airboat and truck platform, while the helicopter technique was used almost exclusively west of the eastern protective levee (Figure 2.1.1.2). Examination of the data showed that the data east of the eastern protective levee was inconsistent with other data sources and would not be used. The data west of the eastern protective levee was determined to be of good quality because it was consistent with existing knowledge and used a logical and defensible method of collection. The processing of this dataset involved the following steps:

1. Converting the vertical datum from NAVD88 to National Geodetic Vertical Datum 1929 (NGVD29) using the VERTCON 2.0 program provided by the National Geodetic Survey.
2. Projecting to Florida State-Plane East feet using Arc/Info.
3. Masking out the roads and canals using the SFWMD major canals coverage buffered 50 feet, and the ETAK major roads (1994) buffered by 50 feet. The ETAK roads (produced by Etak Inc., a leading publisher of digital street map databases) were chosen because of the higher locational accuracy of the linework. The SFWMD has newer road coverages, which are considered better in attribution.
4. Projecting the horizontal data from Universal Transverse Mercator (UTM) to Geographic (Lat-Long) using the Arc/Info 'project' command (VERTCON 2.0 requires Lat-Long coordinates).

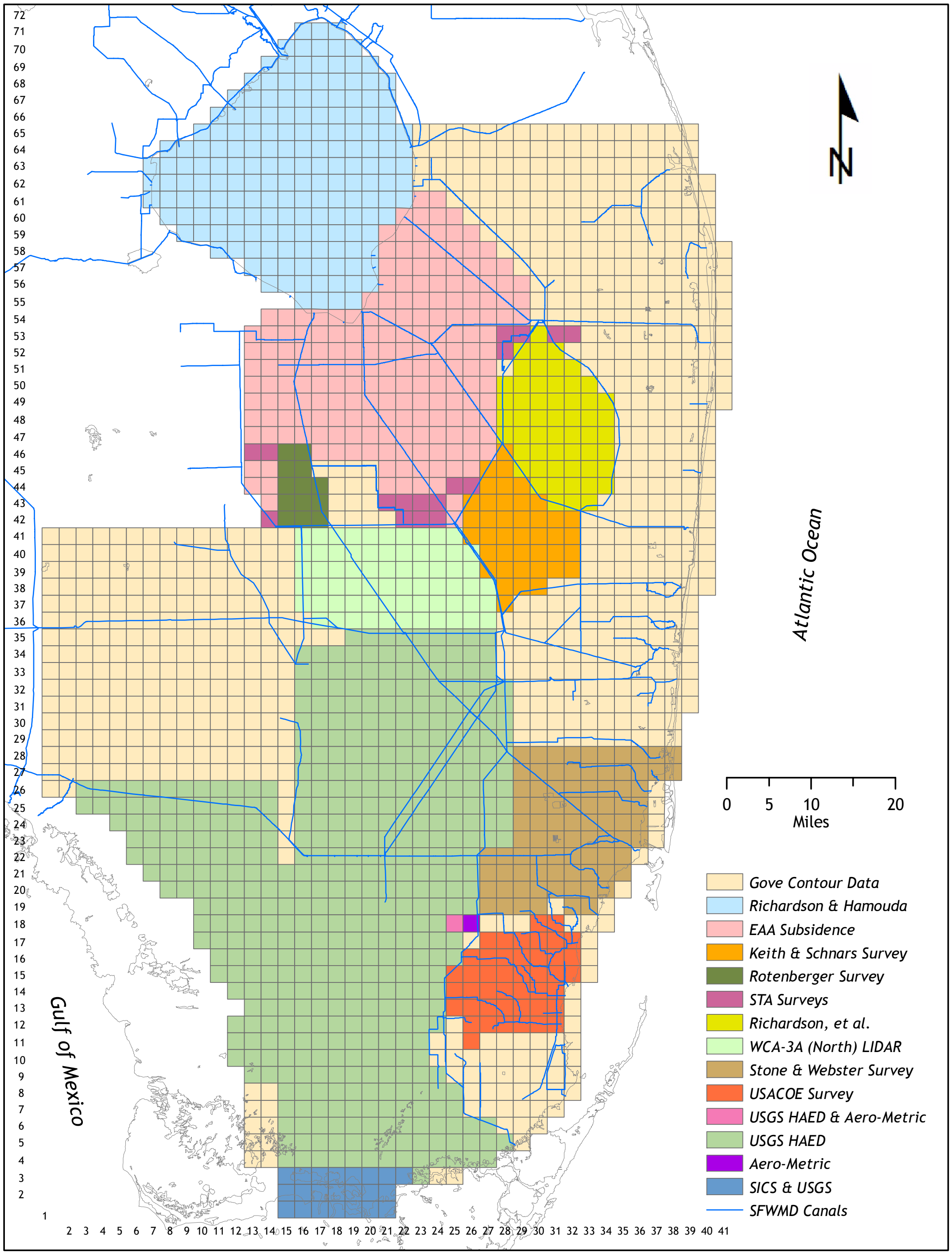

Figure 2.1.1.1 Sources of Topography for South Florida Water Management Model v5.5

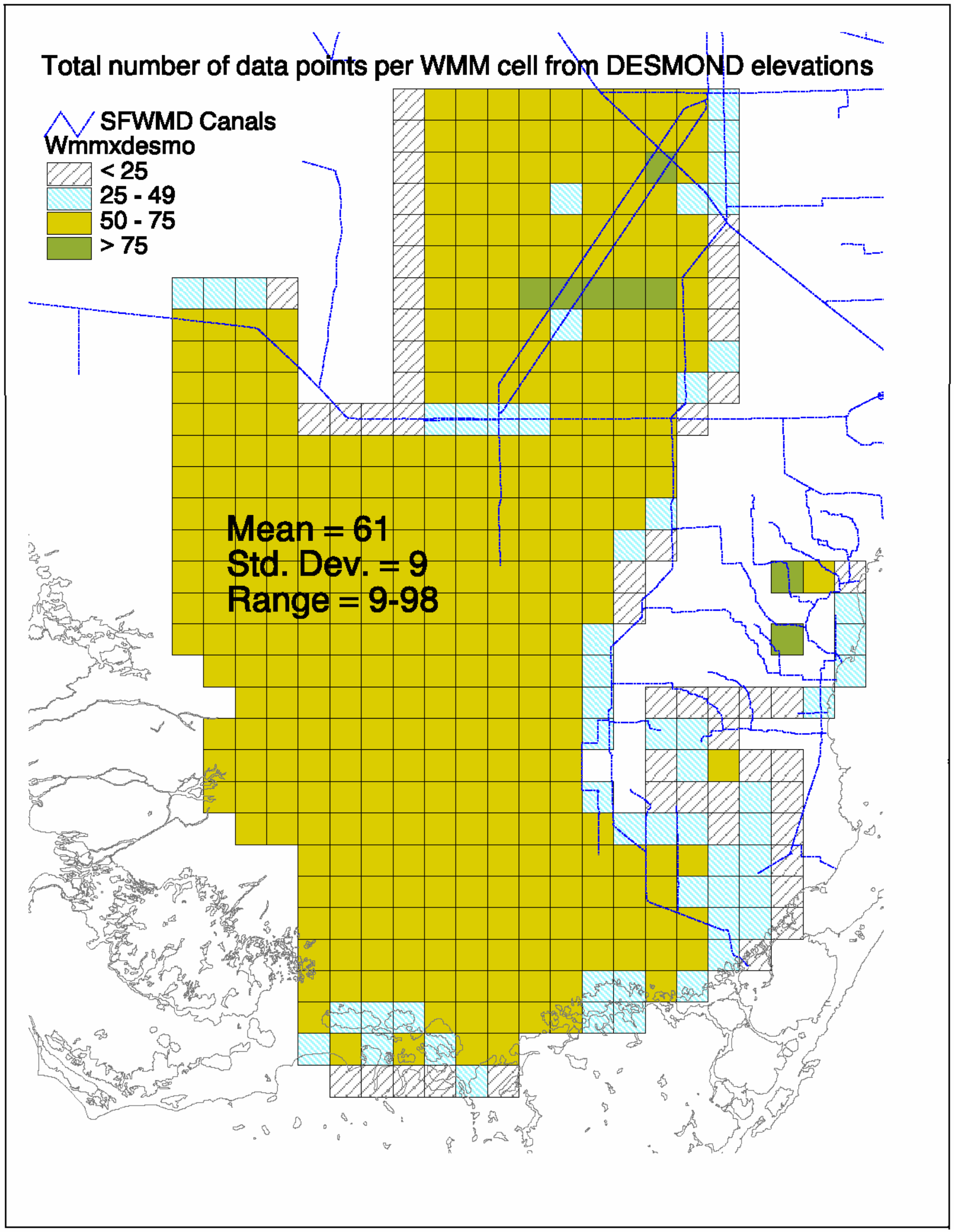

Figure 2.1.1.2 Total Number of Data Points per South Florida Water Management Model Cell for the High-Accuracy Data Collection

5. Aggregating the remaining data to the SFWMM cells by averaging the points that fell within each cell. The process produced an average of 61 points per cell, ranging from 9 to 98 points with a standard deviation of 9. The SFWMM cells containing relatively few points were located on the fringe of the model and were excluded from the final values provided.
6. Calculating and removing outlier data points per SFWMM cell based on a value being approximately 2 standard deviations from the mean value for the cell. These values represent man-made features or localized features not representative of natural ground elevation.
7. Updating 356 cells in the SFWMM.

The WCA-3A LIDAR data was masked to exclude areas outside of the natural internal portion of WCA-3 north of I-75. An analysis of the data showed some abnormal variance in the data moving north-south, but it was determined that for regional-scale modeling, this variance would be aggregated out of the data. In the majority of SFWMM cells, over 400,000 points of LIDAR elevation data were aggregated to one value (Figure 2.1.1.3). The processing of this dataset involved the following steps:

1. Masking out the roads and canals using the SFWMD major canals coverage buffered by 50 feet, and the ETAK major roads buffered by 100 feet, except for I-75 which was buffered by 150 feet. The final mask eliminated all data outside of the internal buffer distance, although some data points had been collected outside of the conservation area.
2. Aggregating the data to 100-meter pixels from the original 5-meter pixels that were received.
3. Projecting the data from UTM to Geographic (Lat-Long) projection.
4. Converting the vertical datum from NAVD88 to NGVD29 using the VERTCON 2.0 program released by the National Geodetic Survey.
5. Projecting the horizontal data from Geographic to Florida State-Plane East feet using the Arc/Info 'project' command.
6. Converting the elevation from meters to feet.
7. Aggregating the remaining data per SFWMM cell by averaging the values that fell within each cell. The process produced an average of 730 points per cell, ranging from 23 to 1,055 points with a standard deviation of 371. Some SFWMM cells along the fringe of the dataset were excluded from the final values provided.
8. Calculating and removing outlier data points per SFWMM cell based on a value being approximately 2 standard deviations from the mean value for the cell. These values are man-made features or localized features not representative of natural ground elevation. For the WCA-3A LIDAR, a manual approach was taken to retain "patches" of outlier points that could represent a large-scale natural feature. Only points which were randomly spaced were removed.
9. Updating 68 cells in the SFWMM.

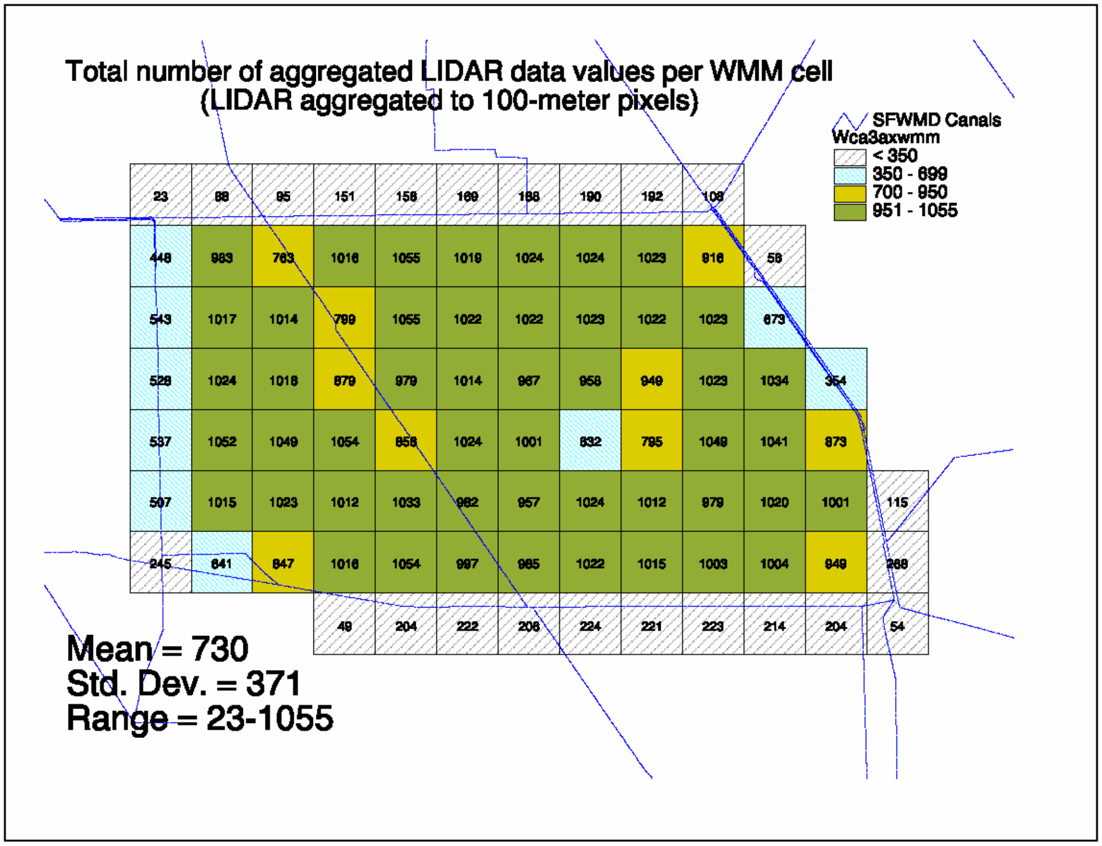

Figure 2.1.1.3 Total Number of Data Points per South Florida Water Management Model Cell for the United States Geological Survey LIDAR data

The EAA was determined to be subsiding at a long-term average rate of between 1 and 1.2 inches per year (Ingebritsen et al., 1999). These rates of subsidence are consistent with Stephens and Johnson (1951), Shih et al. (1979), and Stephens et al. (1984). In the previous revision of elevation data for the SFWMM, a rate of 0.1 foot per year was applied to the 1960 USACE 1-foot contour map data for 28 years (1960-1988) to achieve what became the 1990 updated SFWMM topography (Gove, 1993). According to Shih et al. (1997) subsidence since 1978 has occurred at an average rate of 0.57 inches per year. Measured rates ranged from 0.31 to 0.77 inches per year. In spite of the limited area from which subsidence measurements were taken, and the lack of a clear pattern of subsidence, the average rate of 0.57 inches per year was applied to all EAA cells (123 SFWMM cells) for ten years (1990-2000) to arrive at a current elevation value (Figure 2.1.1.4). Note that the Holey Land and Rotenberger Wildlife Management Areas were excluded from this update. Both of these areas are managed differently from the rest of the EAA and each other (Smith, 2001). Both areas were surveyed with conventional methods in 1992 by the Florida Game and Freshwater Fish Commission (FGFWFC) and updated in the SFWMM.

For the Rotenberger Wildlife Management Area Survey (1999), corresponding SFWMM cells were updated based on the surveyed data and a manually devised weighting mechanism (Brion, 2001). Thirteen SFWMM cells were updated.

The STA elevations were drawn from design dots and/or construction plans. The current information available consists of mean elevations for the cells of each STA. The Save Our Rivers and STA levee coverages were used to create a coverage representing the STAs. The mean elevations were then applied to the appropriate STA cells. A weighted average elevation per SFWMM cell was created using the elevations from the STA cells and SFWMM v3.7 elevations for portions of SFWMM cells not covered by an STA. Seventeen cells in the SFWMM were updated.

For the 8.5 Square Mile Area Survey (USACE, 1999), remaining elevations were averaged for one SFWMM cell, Row Column (18, 26). Elevation points collected along the L-31 Levee were manually removed. These values were approximately 6 feet higher than the rest of the data.

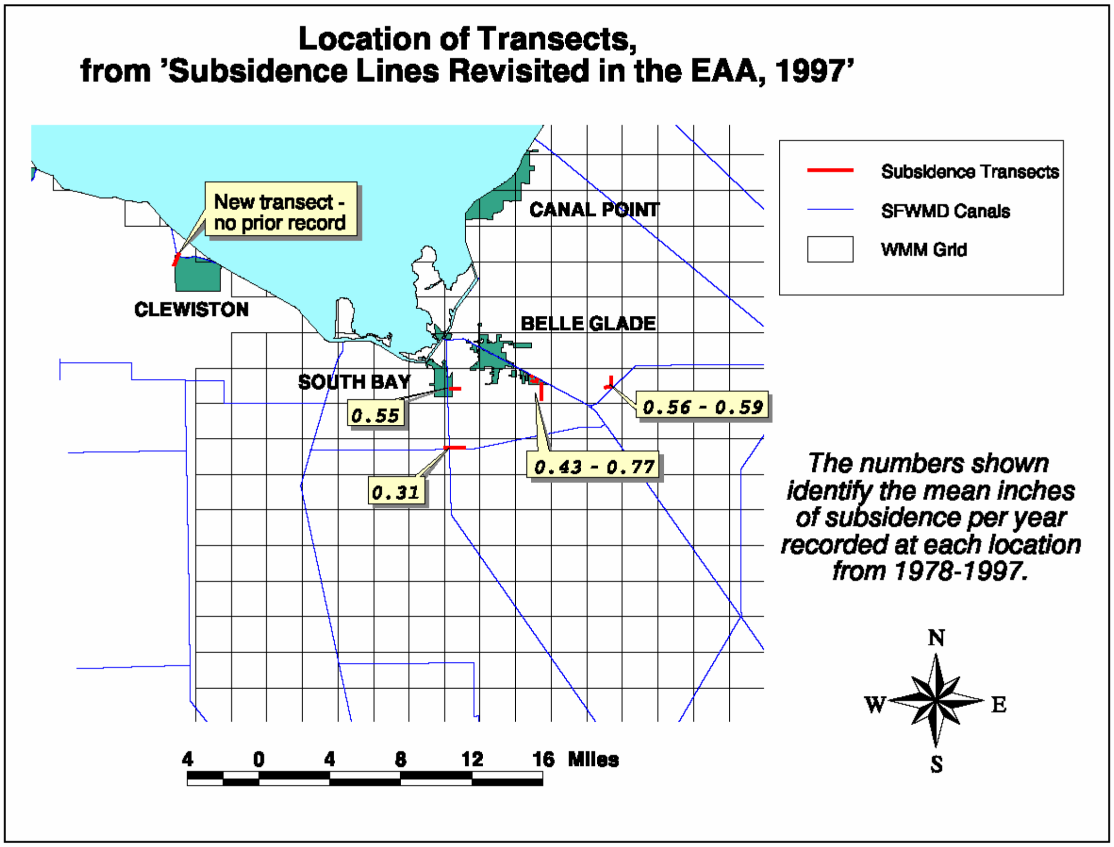

Figure 2.1.1.4 Location of Transects of Measured Subsidence in the Everglades Agricultural Area (Shih et. al., 1997)

Older Data Sources

There were six data sets used to construct the topography in areas not replaced by new information. These data sets are considered to be legacy information and are not addressed in detail in this documentation. The sources for the data are:

1. For the Big Cypress National Preserve and parts of Broward, Palm Beach and Martin Counties, a memorandum for Charles Gove, dated November 11, 1993 was used. The source included both one-foot and five-foot contour data.

2. For WCA-2A, a data survey by Keith and Schnars, dated April 2, 1993, and titled GPS Topography Survey of WCA-2A.
3. For Lake Okeechobee, the source was a report from J. R. Richardson and E. Hamouda titled “BIS Modeling of Hydroperiod, Vegetation, and Soil Nutrient Relationships in the Lake Okeechobee Marsh Ecosystem,” Arch. Hydrobiol., Advances in Limnology, 45, 95-115, 1995.
4. For WCA-1, the source was a report from Richardson, et al., “An Evaluation of Refuge Habitats and Relationships to Water Quality, Quantity, and Hydroperiod.” 1990.
5. For north Miami-Dade County, the data was based on an undated survey from Stone and Webster.
6. For south Miami-Dade County, the data was based on an undated survey from USACE.

Figure 2.1.1.5 displays the final elevations for the SFWMM. More information on the update process for the new topography is provided in Appendix M.

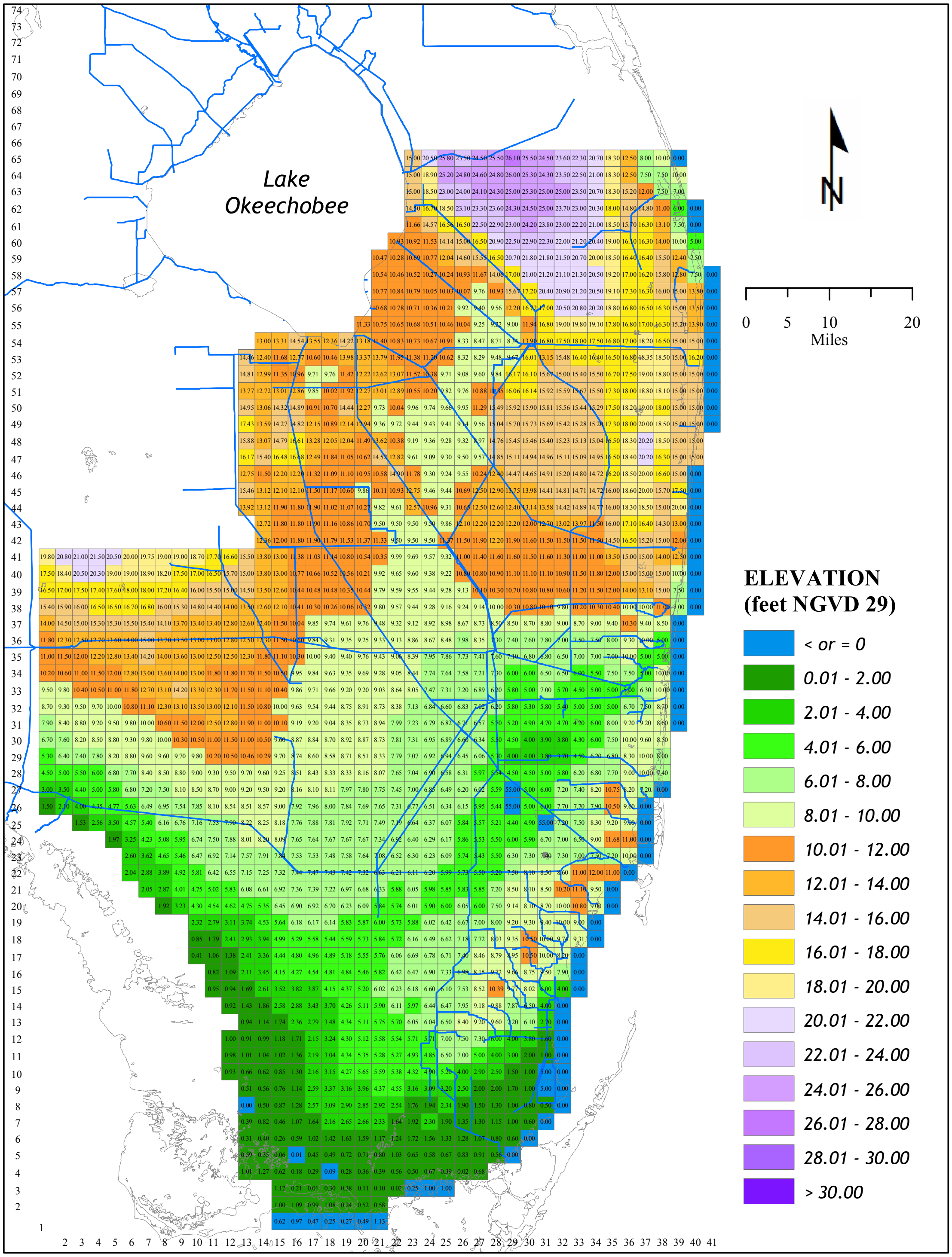

Figure 2.1.1.5 South Florida Water Management Model v5.5 Grid Cell Elevation Values

2.1.2 Land Use

This section describes the South Florida Water Management Model (SFWMM) land use or vegetation developed to represent the years 1988, 2000 and 2050 for each 2-mile by 2-mile model grid cell. The 1988 land use map is required for calibration purposes, while the 2000 and 2050 maps help to illustrate the changes between current and future representation of the South Florida system in the model. The final maps are shown in Figures 2.1.2.1, 2.1.2.2 and 2.1.2.3 for 1988, 2000, and 2050. An effort was made to use the most recent or most accurate data. Since no detailed, uniform map of vegetation exists for the entire SFWMM area several data sources were used to create a composite high resolution Geographic Information System (GIS) dataset to represent the year 2000. The data sources for the vegetative classes and the locations of the datasets are shown in Figure 2.1.2.4 (Rutchey and Vilchek, 1994), (Richardson, et al. 1990), (Welch, et al. 1995), (IFAS, 2001). The land use cover classification was expanded for SFWMM v5.5 (a "crosswalk" of the old classification to the updated classification is provided in Appendix T). Helicopter flights were used to visually check the natural areas, and photographs are included to illustrate the new classification scheme. This section also describes the sources of data and a description of each land use class is provided with emphasis on its hydrological differences. Values for overland flow resistance coefficients and evapotranspiration (ET) parameters from the calibrated version of the SFWMM v5.5 are provided in Table 2.4.2.1 within Section 2.4.2.

Sources and Classification Method

2000 Land Use
The Florida Land Use and Cover Classification System (FLUCCS) is the primary source for land use/land cover input to the SFWMM. Since FLUCCS does not include detailed vegetation information, the best available alternative data sources were used for vegetation classification within the Water Conservation Areas (WCAs) and Everglades National Park (ENP) (Figure 2.1.2.4). A composite GIS coverage of these sources was developed and intersected with the SFWMM grid in order to produce a majority land use type for each cell. Checks were performed including a visual check against 2000 satellite imagery to evaluate each grid cell's former and new land use class. In areas where the majority land use type from the land use data did not match the satellite image, the satellite image took precedence. A draft SFWMM 2000 land use map was verified by aerial survey resulting in adjustments to several classifications in the natural areas and parts of the Everglades Agricultural Area.

1988 Land Use
The SFWMM 2000 land use map was used as a base for revision of the 1988 land use map. It was assumed that natural areas in 2000 were also natural areas with the same land use type as in 1988. Urban and agricultural cells in the earlier version of the 1988 land use map and the SFWMM v5.5 2000 land use map were cross checked. Cells designated as agricultural in the original 1988 map, and as urban in the 2000 map, reverted to agricultural in the revised 1988 map. A check of urban cells was also performed.

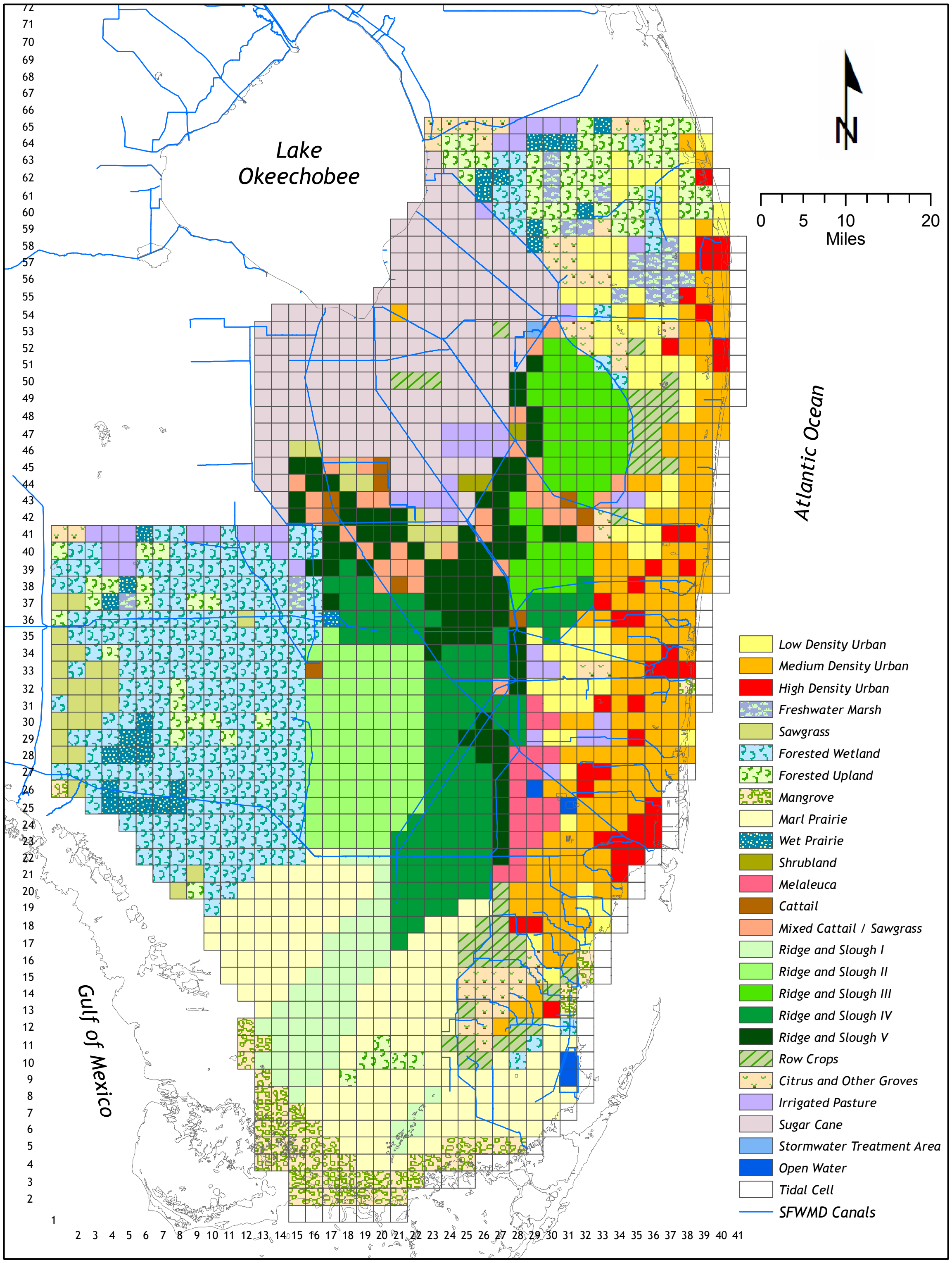

Figure 2.1.2.1 1988 Land Use Map

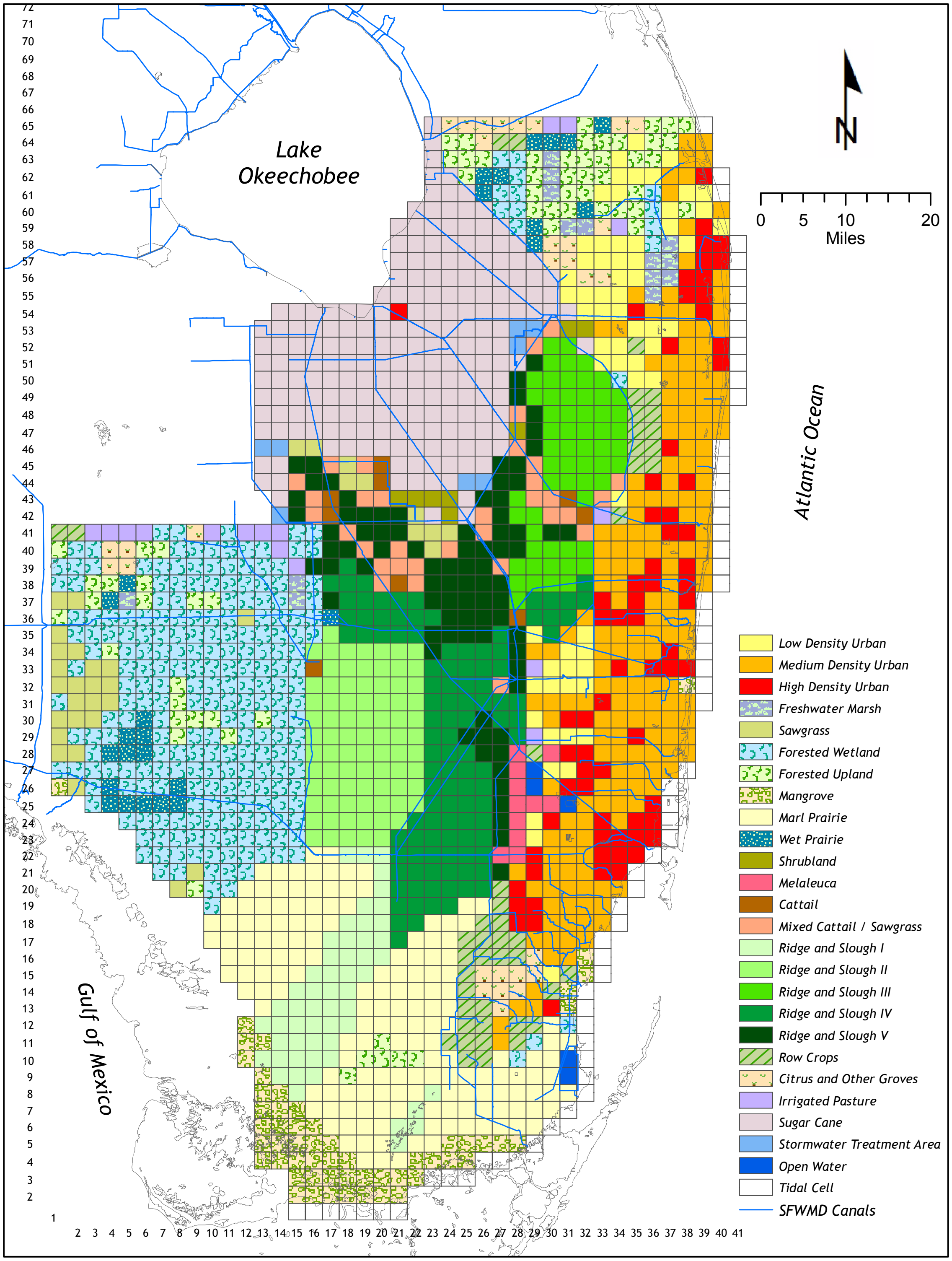

Figure 2.1.2.2 2000 Land Use Map

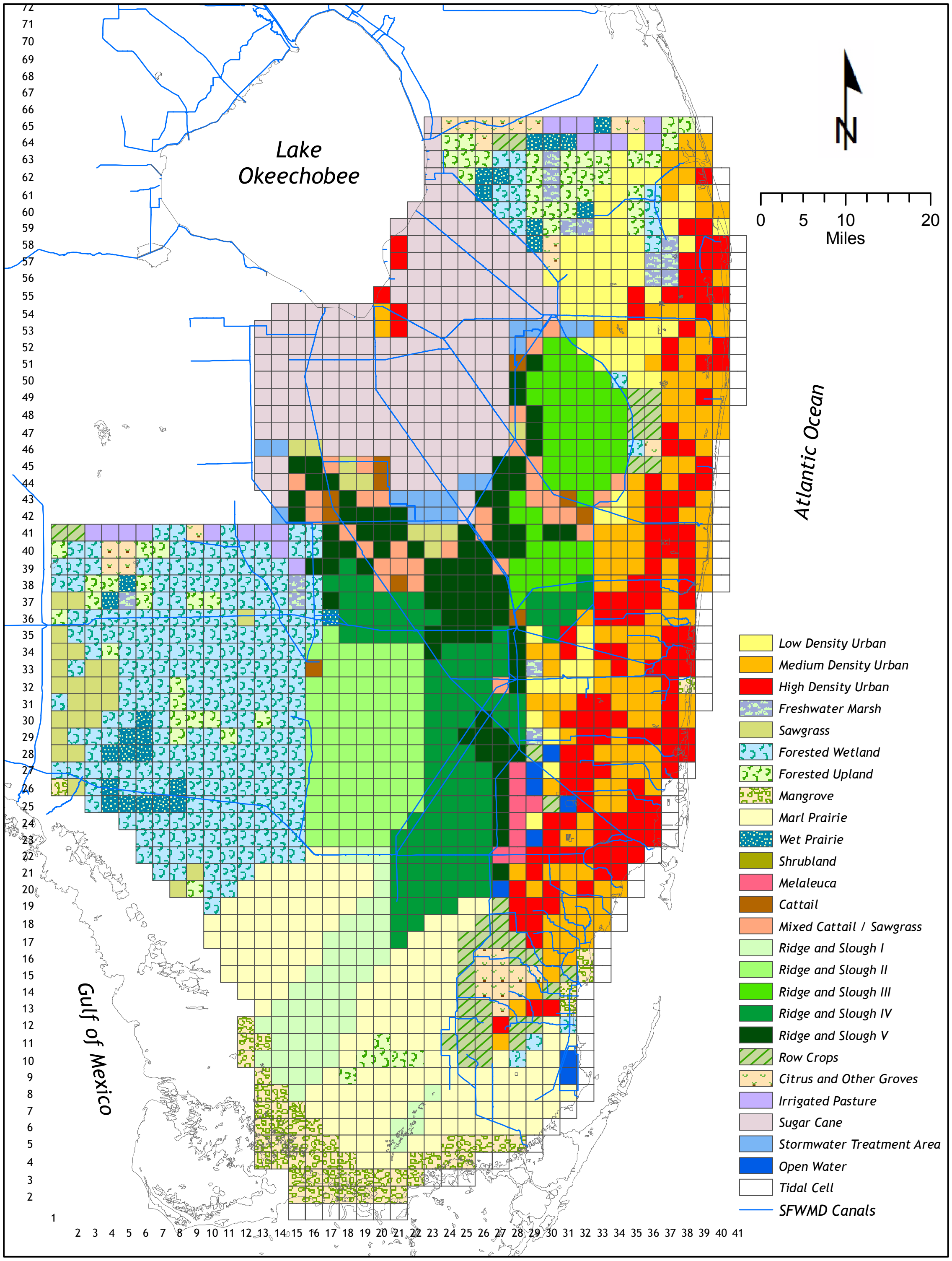

Figure 2.1.2.3 2050 Land Use Map

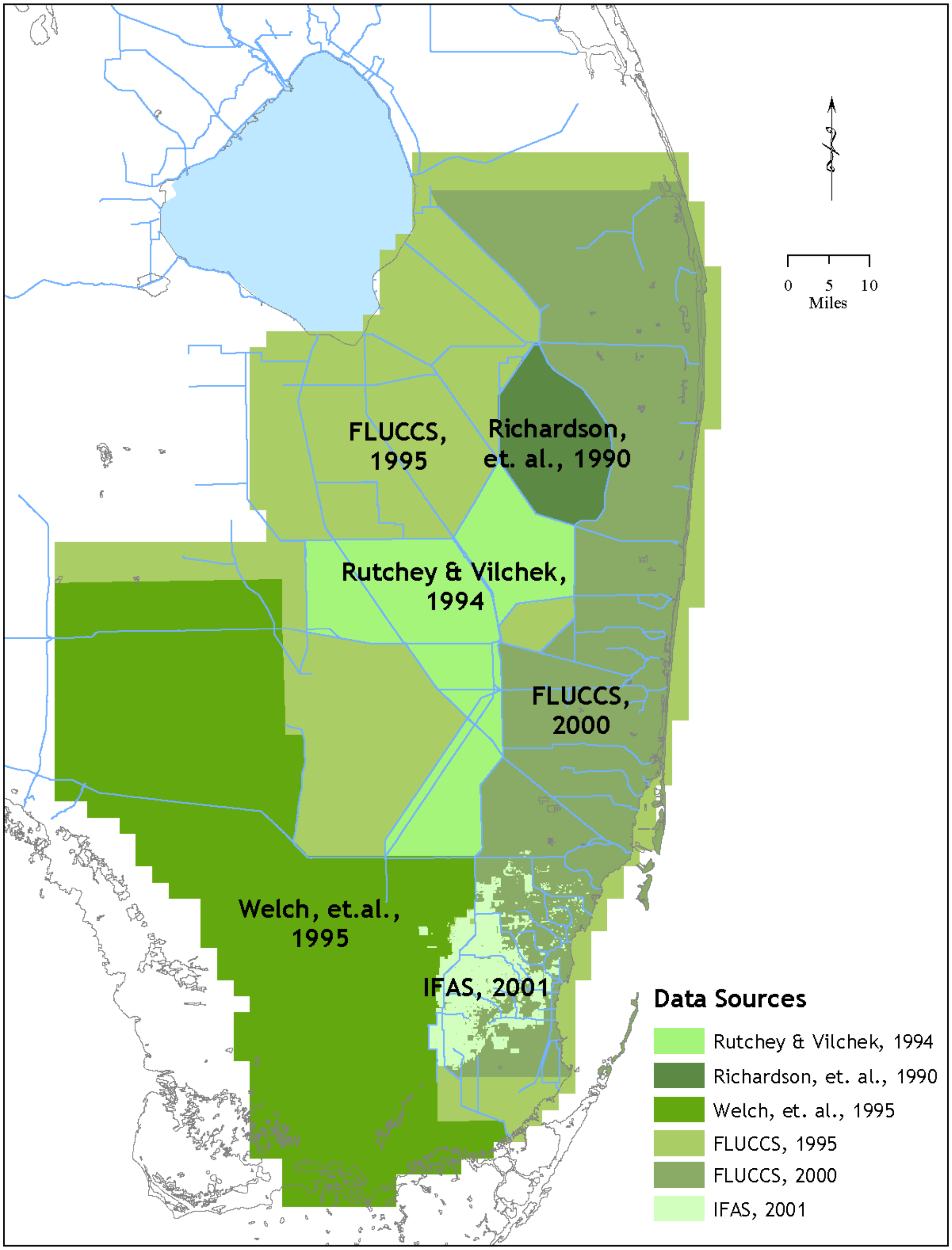

Figure 2.1.2.4 Data Source for Vegetative Classes

2050 Land Use
The 2000 land use coverage was used as a starting point for the 2050 land use projections. All polygons with the potential to be developed were extracted from the 2000 land use coverage. These polygons were then updated with Comprehensive Plan projections from Palm Beach, Broward and Miami-Dade Counties. The 2050 land use coverage was then intersected with the SFWMM grid and the majority land use was assigned to each grid cell. The natural areas were assumed to be the same as 2000 except in areas of urban development. Land use updates for Martin County were included which changed the projections for six of the SFWMM cells (R64C32, R64C33, R64C34, R64C36, R65C32, and R65C36 were all changed from Forested Upland to Irrigated Pasture).

Land Use/Landscape Description

This section will outline all of the land use classifications available within the gridded extent of the SFWMM. The model can only accept one land use classification per grid cell and the associated assignment is assumed to apply over the entire spatial extent of the cell. This assumption is reasonable in a majority of the model domain where changes in landscape occur gradually. In the Lower East Coast Service Areas (LECSAs), where land use can change more rapidly from urban to agricultural to natural classifications, additional consideration is made for land use variability at a scale smaller than a 2-mile by 2-mile grid cell. This is accomplished as part of a pre-processed sub-module within the SFWMM, known as the ET-Recharge model. The details related to this feature are explained in Section 2.3.5. The SFWMM classifies land use as one of the following choices:

High Density Urban
Model grid cells with greater than 50 percent impervious cover. Areas comprised of industrial sites, shopping centers with large paved areas, and high density residential areas are designated as high density urban.

Medium Density Urban
Model grid cells with 25 to 50 percent impervious cover. Medium density residential areas or mixtures of low density and high density within the same grid cell are classified as medium density urban.

Low Density Urban
Model grid cells with less than 25 percent impervious cover. This category includes golf courses, small holdings and low density residential areas; it may also contain agricultural or natural areas within urban land uses.

Ridge & Slough
The most extensive landscape in the remnant Everglades, Ridge & Slough, can be characterized as a mosaic of sawgrass ridges interspersed with open water sloughs and dotted with tree islands. Ridges vary from consisting only of sawgrass, to ridges with shrub cover or tree islands. Slough conditions range from open water to dense aquatic vegetation cover (e.g. water lilies). Periphyton communities are established to varying degrees in some areas. Due to shortened hydroperiods, sawgrass and other macrophyte encroachment into sloughs has resulted in an increased resistance

to flow. The Ridge & Slough landscape is highly directional in places (Central WCA-3A), and has non-directional characteristics in other places (WCA-1). Because of the uniqueness of Ridge & Slough landscape habitat, some aerial photographs are included.

Due to water management practices, the current Ridge & Slough landscape is a modified form of the pre-drainage Everglades landscape. It is reduced in spatial extent as well as modified in terms of vegetation community composition. For the purpose of SFWMM land cover classification, current vegetation occurring within the boundary of Ridge & Slough landscape as defined in the Natural System Model (NSM), was classified as (modified) Ridge & Slough, and divided into five categories representing different resistances to flow.

Ridge & Slough I consists of linear directional sawgrass ridges interspersed with predominantly open water sloughs. This subclass of Ridge & Slough has lower resistance to flow than other Ridge & Slough subclasses because it has more open water with fewer water lilies, little to no invasion of the sloughs with sawgrass and other species and little periphyton. The Ridge & Slough I landscape is found in Shark River Slough (SRS) and Taylor Slough in ENP.

Ridge & Slough II is comprised of directional sawgrass with open water sloughs that have been slightly filled in with sparse sawgrass and other species, increasing resistance to flow. Periphyton growth on submerged stems of the emergent vegetation in the sloughs increases flow resistance. The Ridge & Slough II landscape is found in WCA-3A south of Alligator Alley and west of the Miami Canal (Figure 2.1.2.5).

Ridge & Slough III is predominantly non-directional consisting of circular and irregular shaped sawgrass ridges interspersed with open water sloughs. Shrubs and trees are present on many of the ridges. In places, water lilies are present in the sloughs. Ridge & Slough III landscape is found in WCA-1 and WCA-2A (Figure 2.1.2.6). Resistance to flow is expected to be higher than Ridge and Slough II due to lack of directionality.

Ridge & Slough IV consists of non-directional to slightly directional sawgrass ridges with little evidence of shrubs or tree islands. Sloughs often have water lilies or periphyton in them. Areas of Ridge & Slough IV landscape include WCA-2B, parts of WCA-3A north of Alligator Alley and southeast of the Miami Canal / Alligator Alley intersection, WCA-3B and Northeast SRS (Figure 2.1.2.7).

Ridge & Slough V consists of Ridge & Slough vegetation that has been considerably modified by in-filling of sloughs with sawgrass and other wet prairie species. Resistance to flow is higher than the other Ridge & Slough subclasses and slightly less than that of the sawgrass landscape. Areas of Ridge & Slough V landscape include parts of northwest and northeast WCA-3A, parts of the Rotenberger and Holey Land Wildlife Management Areas, northern WCA-3B and the Pennsuco Wetlands (Figure 2.1.2.8).

Figure 2.1.2.5 Examples of Ridge & Slough II Landscape

Figure 2.1.2.6 Examples of Ridge & Slough III Landscape

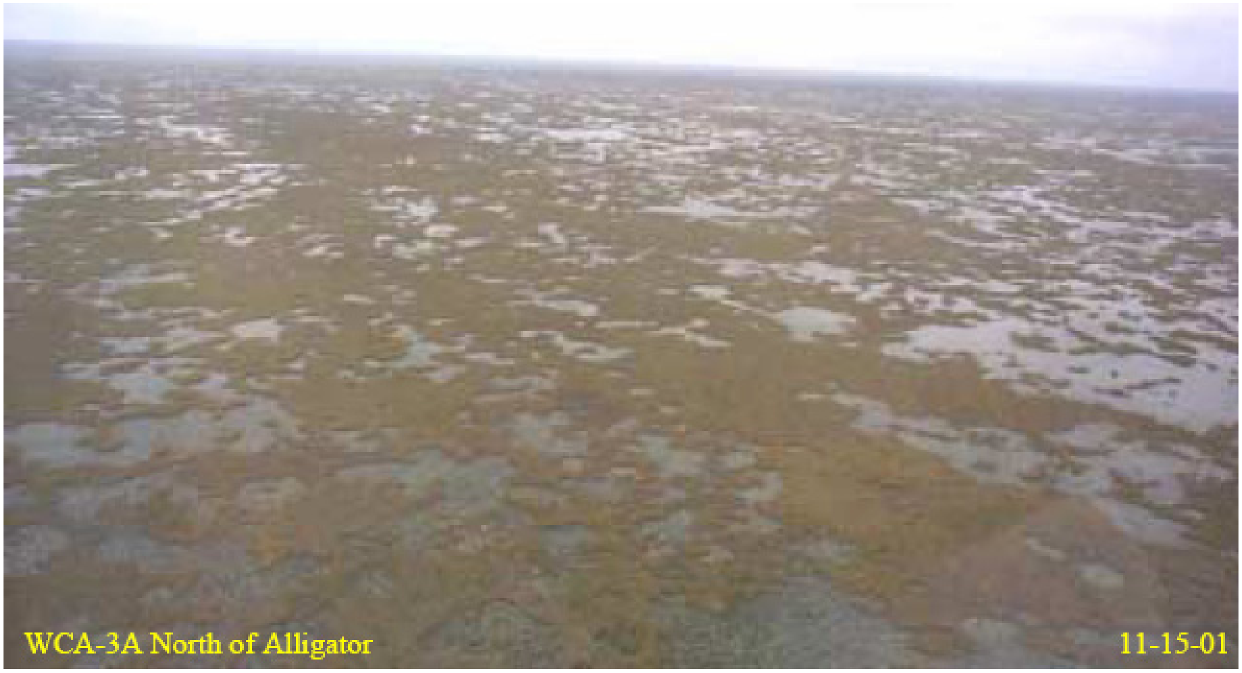

Figure 2.1.2.7 Examples of Ridge & Slough IV Landscape

Figure 2.1.2.8 Examples of Ridge & Slough V Landscape

Freshwater Marshes

Freshwater marshes are inundated areas outside of the Ridge & Slough boundary. Marshes are dominated by emergent and floating vegetation. Freshwater marshes occur in deeper depressions than prairies and have longer hydroperiods.

Wet Prairie

Wet prairie landscape is found in shallow depressions among flatwoods, in pastures, and at the edges of cypress domes and marshes. In this classification, wet prairie is a grassy landscape mixed with open water. The dominant vegetation of wet prairies include wiregrass, spike rush, muhly grass, beak rush, cordgrass, maidencane, and St. John's wort.

Marl Prairies
Marl Prairies are comprised of relatively (compared to Ridge & Slough landscapes) sparse, low stature sawgrass on marl soils. Open water sloughs with no prominent directional pattern occur in marl prairies. The Marl Prairie landscape was defined by intersecting model grid cells with predominantly sawgrass vegetation and marl soils. The resulting Marl Prairies correspond closely with those identified in several studies (Davis 1943, Davis et al, 1994, McVoy and Park 1997) as having a distinct boundary with the Ridge & Slough landscape. Resistance to flow in the Marl Prairies is lower than in the Ridge & Slough landscapes because of the relatively sparse sawgrass.

Sawgrass
Sawgrass classification is applied to areas outside of the Ridge & Slough boundary that are dominated by contiguous areas of medium to dense sawgrass. In some places there are breaks in the sawgrass due to open water where periphyton and bladderwort may be found.

Cattail
Cattail (Typha spp.) is a marsh species that thrives under high-nutrient conditions. It occurs naturally in disturbed areas or around gator holes and can be found downstream of the EAA in areas where nutrient enrichment has occurred.

Mixed Cattail and Sawgrass
Mixed cattail and sawgrass is a mixture of cattail patches and sawgrass (Figure 2.1.2.9) and is used to represent SFWMM grid cells that contain greater than 20 percent cattail and greater than 20 percent sawgrass. It is found in areas where cattails have invaded sawgrass, such as parts of northern WCA-3A and along parts of the edges of WCA-2A and the Loxahatchee National Wildlife Reserve (LNWR).

Figure 2.1.2.9 Example of Mixed Cattail and Sawgrass Landscape

Stormwater Treatment Areas
Stormwater Treatment Areas (STAs) include large, constructed, treatment wetlands designed to serve as biological filters to reduce the phosphorous concentration in agricultural runoff entering the Everglades Protection Area (EPA). Vegetation varies by STA, and consists mainly of cattail, mixed marsh and submerged aquatic vegetation communities.

Forested Wetlands
Forested wetlands include cypress swamps, hardwood and wetter species forming a mosaic of pine flatwoods and depressed wetlands.

Forested Uplands
Forested uplands are pinelands on higher sands or areas of former mosaic of pine flatwoods and depressed wetlands that have been dehydrated by artificial drainage.

Mangrove Forests
Mangrove forests are coastal landscapes containing red, white or black mangrove that may extend inland such as in the southern and southwestern Everglades. Mangroves are permanently to regularly flooded by tidal waters.

Melaleuca
Melaleuca is an exotic species (Melaleuca quinquenervia) forming monotypic stands that dominate the landscape. Melaleuca exists in both upland habitats, and lower areas which have experienced prolonged inundation.

Shrubland
Shrubland includes areas where trees are not present but shrubs are the dominant vegetation. Shrubs may include: Brazilian pepper, wax myrtle and saw palmetto. Shrubland is an upland community which rarely experiences inundation.

Open Water Bodies
Open water bodies such as lakes, canals or deep excavated reservoirs are included in the open water category.

2.2 RAINFALL

In all South Florida Water Management Model (SFWMM) runs, rainfall is assumed to have the same temporal and spatial distribution as that which occurred historically over the period of simulation. Since rainfall is the main driving force in the hydrology of South Florida, it serves as a good control variable for evaluating alternative ways of managing the system as a whole. For the distributed mesh portion of the model, a daily time series of rainfall depths for each grid cell is used. For Lake Okeechobee and other lumped hydrologic systems, a single daily time series of rainfall depths is input and assumed to apply over the spatial extent of the basin. The general procedure for the development of the rainfall data set in the SFWMM can be described as follows: data collection and associated quality assurance/quality control (QA/QC) or screening of rainfall station data; and transformation of rainfall point data into grid-based data.

2.2.1 Quality Assurance/Quality Control of Rainfall Data

Rainfall data was collected with the goal of generating a 2-mile x 2-mile "super grid" covering nearly the entire South Florida Water Management District (SFWMD or District) for the 1914 to 2000 period of record. The spatial extent of the super grid was determined to be larger than that of the computational grid for the SFWMM in order to allow for determination of rainfall in the Natural System Model (NSM) as well as to provide rainfall information for the lumped portions of the SFWMM. The primary reason for creating a rainfall data file with a greater period of record than required by the modeling period of simulation (1965 to 2000) was to support identification of monthly and annual data trends.

Because of data availability issues, the rainfall data for the period from 1914 to 1998 were processed separately from the period of 1999 to 2000; however, the exact same procedure was used for both time periods. For the period from 1914 to 1998, there were 860 rainfall stations covering 11 counties (Broward, Highlands, Martin, Palm Beach, Collier, Glades, Monroe, Miami-Dade, Hendry, St. Lucie and Okeechobee). For the period 1999-2000, rainfall data at 964 stations covering the same counties were available. Figure 2.2.1.1 identifies the location of rainfall stations used in the creation of the SFWMM data set.

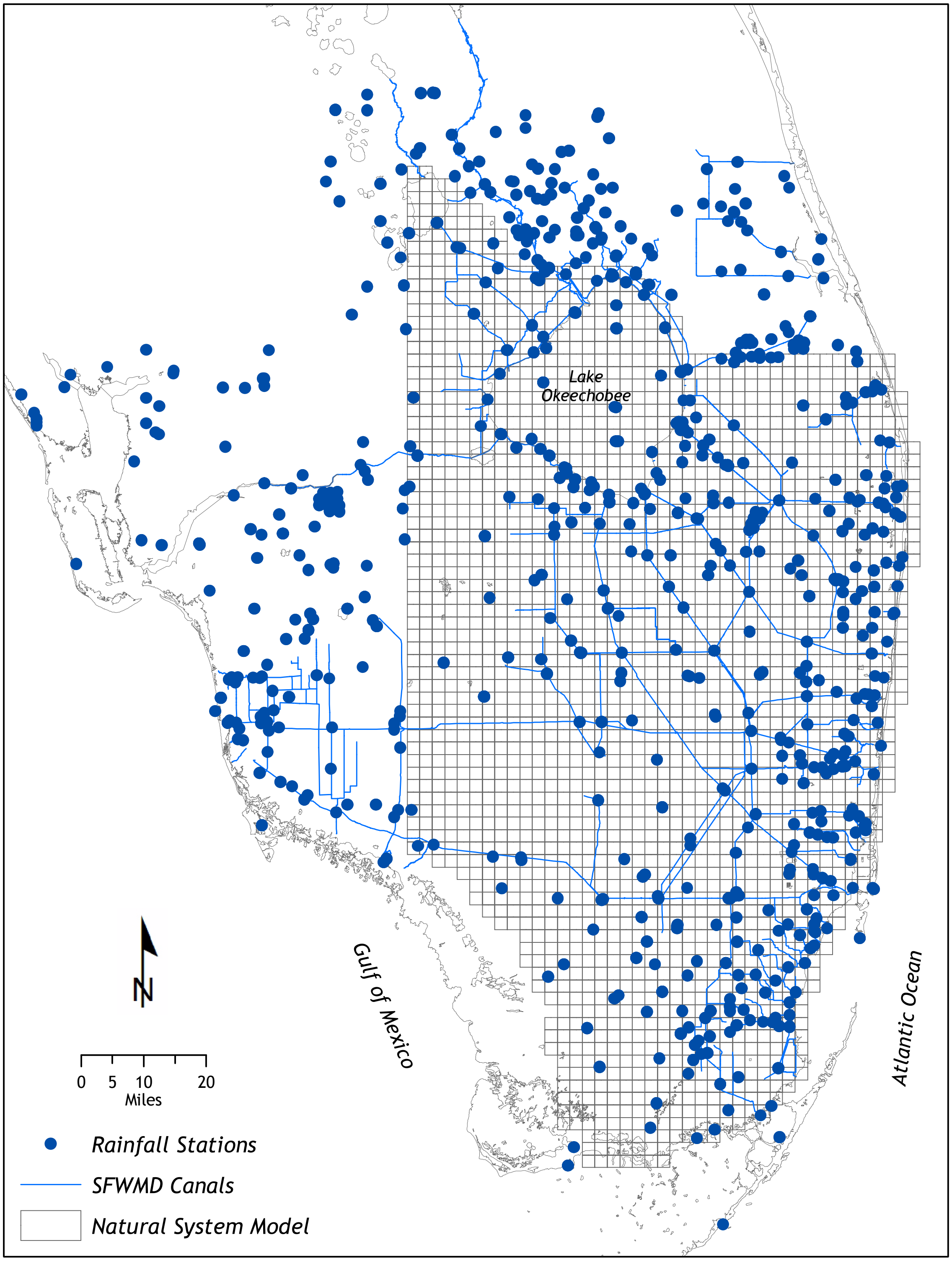

Figure 2.2.1.1 Location of Rainfall Stations

QA/QC of rainfall station data sets was carried out in five phases, with a number of methodical steps to complete each phase. The five phases were as follows:

I. Review and classification of daily data having extreme values.
II. Testing and elimination of some extreme daily values.
III. Screening of data with zero monthly rainfall.
IV. Screening of rainfall data having extreme low annual values and high monthly values.
V. Data screening through visualization.

The first two phases were designed to identify and remove daily values that were highly questionable according to a prescribed classification scheme, while the third and fourth phases were designed to identify and remove data associated with stations that were not consistent with monthly and annual trends. The last phase provides final QA/QC through data visualization. Appendix P presents a memorandum describing, in detail, the phases and steps used. Short descriptions of the QA/QC phases are provided in the following sections. It is important to note that during these phases, screening criteria were developed from both the raw rainfall station data and from analysis of the gridded representation of the data. The methodology for the development of the gridded data will be discussed in Section 2.2.2.

Phase I: Identification and Classification of Extreme Daily Rainfall Values

In the first pass, daily rainfall values greater than 16 inches were flagged as questionable. Additionally, daily rainfall values less than 16 inches but higher than 5.5 inches in Miami-Dade, Broward and Palm Beach counties, and 5 inches in the other counties of the SFWMD area were flagged as questionable. The lower threshold values for questionable data represent approximately the 99.9 percentile in each respective county. For each day when at least one questionable data point was identified, values from the nearest six stations were extracted into a data set. For each of the resulting data sets, a classification scheme (having seven classes based on distance and value difference) was used to automatically accept or mark values for further review. After automatic acceptance of two of the classes, and marking the other five classes as questionable, the rainfall data set was recreated and reviewed using grid summaries and viewing programs.

Phase II: Examination of Extreme Daily Rainfall Data

During this phase, the values identified as questionable in Phase I, were further analyzed for either acceptance or rejection. Using the nearest six stations, a manual examination of the questionable values was conducted which included consideration for: distance, direction, difference in values, number of neighbors with high values, time of year, frequency of re-occurrence in the period of record and known tropical storm events.

Phase III: Examination of Daily Data Corresponding to Zero Monthly Rainfall

In this phase, efforts were made to identify and verify rainfall data for calendar months with zero rainfall. The objective was to reject or accept such data based on prescribed criteria. Part of this process was automated and part was performed manually. For each county, calendar months with zero rainfall data are extracted into a file and the average rainfall was calculated (excluding the site under investigation) and compared to the questionable site. A monthly value of zero during dry seasons was not considered unreasonable, however zero monthly rainfall values during the wet season where nearby stations averaged $\geq$ 5 inches, were considered highly suspect. Considerations for acceptance or rejection of data included: the nearby averages, historical monthly average tables which included surrounding areas, the repetition of zero values from other sites for the same month, seasonality, the number of consecutive zero values at a given site, and whether or not the nearby site average was below the long-term monthly average. A final evaluation was made for stations with zero rainfall for three or more consecutive months by examining the quality of the daily rainfall.

Phase IV: Examination of Annual Rainfall below 30 Inches and Monthly Rainfall above 20 Inches

Visual examination of the data set showed annual rainfall was below 30 inches in some areas. Similarly, the monthly rainfall was greater than 20 inches in some areas. The examination of such data was carried out in three steps: investigation of the corresponding data, comparison with rainfall local statistics, and a visual inspection of annual snapshots extracted from the revised rainfall data set.

The investigation of the corresponding data consisted of a visual review of the daily data for the records that did not meet the criteria. About 6 percent of cases that had annual rainfall below 30 inches, 22 years of daily data were found to be of poor quality (a combination of unrealistically low and missing values) and were consequently removed. Of the cases that had a monthly rainfall that was greater than 20 inches, only month of rainfall was rejected where high rainfall was reported in an area with an average rainfall of 0.65 inches; the rest of the cases were accepted.

For the cases that had annual rainfall below 30 inches and had a maximum of two months of missing data, the following statistics were generated: the average, the standard deviation, the annual rainfall excluding the missing months, and the annual rainfall after counting for the missing month {(using the following approximation: Adjusted value = [(value)(12) / [(12 – number of missing months)]}. If the number of stations used to compute the statistics was two or less, discretion (based on a visual evaluation) was used to either reject or accept the daily data set for the year. In cases where the number of stations used to compute the statistics is more than two, the daily data set for a given year was rejected if the associated adjusted value was as follows:

1. Below 20 inches; or
2. Less than 1/2 of the average rainfall (for the given county and given year based on all locations except the one of interest); or
3. Less than (AVG-2.5)(STD) where STD is the standard deviation of annual rainfall within that county and that year.

Of the 98 cases identified, 53 daily data sets were rejected.

Phase V: Final QA/QC through Data Visualization

During Phase V, a visual examination of daily, monthly, and annual snapshots of the rainfall data set was performed. Some areas of very low rainfall still existed. Associated stations were identified and a visual inspection of the daily values was performed. At some stations, daily data were of poor quality as indicated by an overwhelmingly large number of missing data for a given year. As a result of the visual evaluation, six records were rejected for at least one year, one record was rejected for two years, and three stations were dropped for the entire period of record.

2.2.2 Transformation to Grid-Based Data Set

Once the rainfall data QA/QC was completed, a Triangular Irregular Network (TIN) approximation method was performed to assign a representative rainfall depth for each day and grid cell. This was necessary because rainfall gauging stations do not normally coincide with the centroid of the grid cells and most grid cells do not contain rainfall gauging stations.

The normal TIN approximation involves using the centroid of the grid cell as a reference point for determining which three rainfall stations are used for estimating the daily rainfall value. If rainfall stations are fairly sparse, model grid cells are small, or rain events are spatially large, this would be a suitable application. However, in South Florida, the rainfall stations are not sparsely located, the model grid cells are large (4 square miles each), and heavy rainfall events can be localized. Therefore, a variation of the normal TIN approximation method was developed for this application.

The new method involved dividing each model grid cell into 100 sub-cells. Because each cell was equally divided horizontally and vertically by 10, the methodology is referred to as TIN-10. The sub-cells were over-laid by a triangular pattern of rainfall stations (with stations at each apex as shown in Figure 2.2.2.1). For the sub-cells contained within a single triangle, a daily rainfall value was calculated based on the rainfall stations at each apex. The calculated values were the weighted (based on distance from each station to each sub-cell centroid) average of the three nearest stations. Once the daily rainfall for each sub-cell was determined, the values were averaged to compute the grid cell daily rainfall value used by the model.

From Figure 2.2.2.1, the normal TIN approximation method would apply the rainfall at stations B, C, and D to the centroid of the grid cell even though only 38 percent of the sub-cells fell within the triangle. Consequently, the influence of two other rainfall stations would not be considered for the remaining 42 percent of sub-cells. For the TIN-10 method, the influences of the other two stations would be included in the approximation.

A comparison between the two methods revealed only small differences in annual averages with the TIN-10 method being slightly lower. The monthly average differences were generally less than 0.2 inches with the TIN-10 method having consistently lower maxima. The differences between the two methods were more evident during the wet season months. The TIN-10 method tends to decrease the dominance of any one station thus minimizing the effect of a localized rain event on a grid cell.

Average annual results of the generation of the rainfall data set by the process for data collection, QA/QC and transformation to grid are provided in Figure 2.2.2.2. The seasonal variability of the end product is shown in Figure 2.2.2.3.

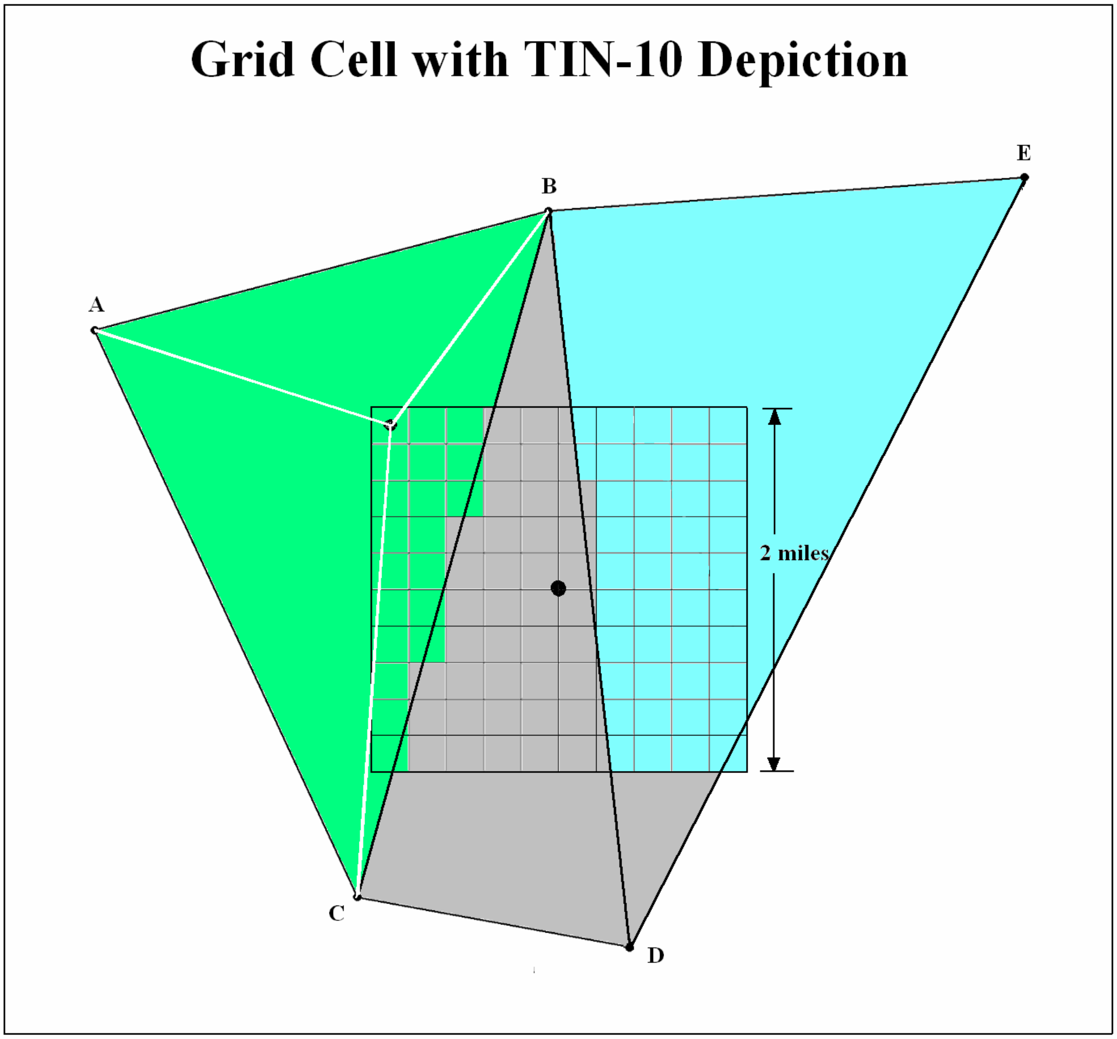

Figure 2.2.2.1 Example of TIN-10 Estimation for Model Grid Cell

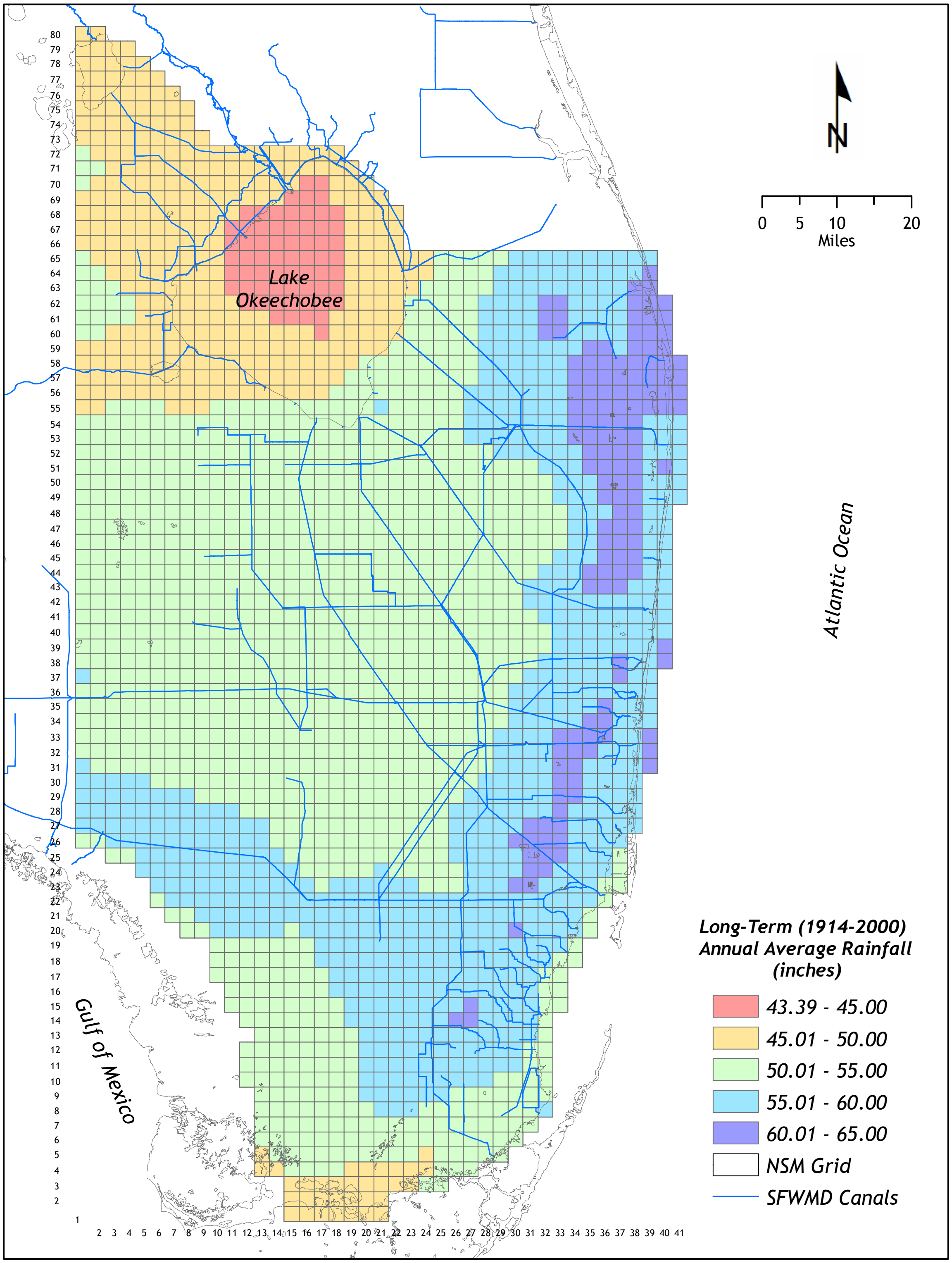

Figure 2.2.2.2 Grid Values of Annual Average Rainfall

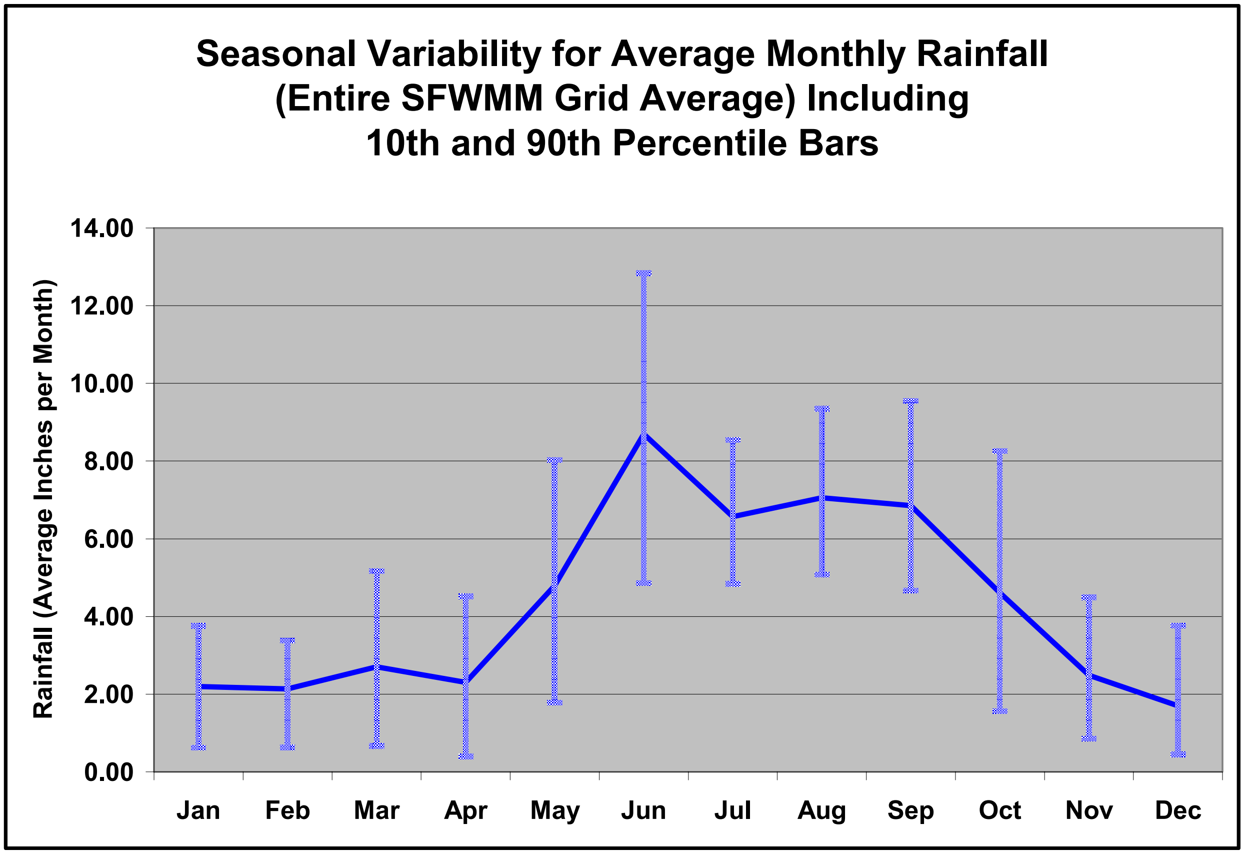

Figure 2.2.2.3 Monthly Mean with 10th and 90th Percentile Bars for Rainfall

2.3 EVAPOTRANSPIRATION

The calculation of evapotranspiration (ET) in the South Florida Water Management Model (SFWMM) is based on reference crop potential ET which is adjusted according to crop type, available soil moisture content, and location of the water table. Algorithms used to calculate actual evapotranspiration vary geographically because of different data availability, calibration approaches and varying physical and operational characteristics of different areas within the model domain. For Lake Okeechobee, the pan evaporation method is used to calculate open water and marsh zone ET. In the Everglades Agricultural Area (EAA), total ET is the sum of its components from the saturated, unsaturated and open water zones. In non-irrigated areas such as the Everglades, the unsaturated zone is not modeled and total ET is calculated as the sum of open water evaporation and saturated zone (water table) ET. Finally, in irrigated areas within the Lower East Coast (LEC), an application of Agricultural Field-Scale Irrigation Requirements Simulation Model (AFSIRS) was used to calculate ET and recharge while saturated and open water ET are calculated as described below.

2.3.1 Determination of Potential Evapotranspiration

In the SFWMM, predicted evapotranspiration is calculated by spatial interpolation of the reference or potential evapotranspiration between the sites, and by the application of landscape-specific crop coefficients that are a function of water depth. These landscape-specific crop coefficients are obtained by calibration as part of the SFWMM calibration/verification effort. Several potential methods for estimating potential or reference evapotranspiration for use in these regional long-term continuous simulation models were examined. The selected method for potential evapotranspiration estimation is presented here.

The SFWMD Simple Method (Abtew, 1996; Equation 2.3.1.1) was selected to provide estimates of long-term historical (1965-2000) *wet marsh potential ET* for long-term hydrological modeling

$$ET_p = \frac{K_1 R_s}{\lambda} \tag{2.3.1.1}$$

where:

ET_p = wet marsh potential evapotranspiration [mm dd^{-1}];
K_1 = coefficient (0.53 for mixed marsh, open water and shallow lakes);
R_s = solar radiation received at the land surface [MJ m^{-2} d^{-1}]; and
λ = latent heat of evaporation [MJ kg^{-1}].

It is important to keep in mind that due to the difference in roughness characteristics between marsh and reference grass surfaces, the crop coefficients developed with respect to a grass reference ET may need to be modified for use with wet marsh potential ET. Due to the scarcity of solar radiation and cloud cover data, the self-calibrating K_r method (Hargreaves and Samani, 1982; Allen, 1997; Equation 2.3.1.2) was chosen for estimating solar radiation (R_s) for potential ET estimation since it depends on a single parameter with low spatial variability.

$$R_s = \tau R_a = K_r \left(T_{max} - T_{min}\right)^{0.5} R_a \qquad (2.3.1.2)$$

where:

R_s = solar radiation received at land surface [MJ m^{-2} d^{-1}];
τ = atmospheric transmissivity;
K_r = empirical coefficient;
T_{max} = mean daily maximum temperature over the period of interest [˚C];
T_{min} = mean daily minimum temperature over the period of interest [˚C]; and
R_a = extraterrestrial solar radiation [MJ m^{-2} d^{-1}].

Extraterrestrial solar radiation (R_a) is calculated from latitude and time of year by integrating the instantaneous radiation intensity at the outer atmosphere from sunrise to sunset:

$$R_a = \frac{(24)(60)}{\pi} G_{sc} d_r \left(\omega_s \sin\varphi \sin\delta + \cos\varphi \cos\delta \sin\omega_s\right) \qquad (2.3.1.3)$$

where:

R_a = extraterrestrial solar radiation [MJ m^{-2} d^{-1}];
G_{sc} = solar constant = 0.8202 (Duffie and Beckman, 1991) [MJ m^{-2} min^{-1}];
d_r = relative distance from the sun to the Earth;
ω_s = sunset hour angle [rad];
φ = station latitude [rad]; and
δ = declination of the sun [rad].

The relative distance from the sun to the Earth (d_r), the declination of the sun (δ) and sunset hour angle (ω_s) are given by:

$$d_r = 1 + 0.033 \cos\left(\frac{2\pi J}{365}\right) \qquad (2.3.1.4)$$

$$\delta = 0.409 \sin\left(\frac{2\pi J}{365} - 1.39\right) \qquad (2.3.1.5)$$

$$\omega_s = \arccos\left(\tan\varphi \tan\delta\right) \qquad (2.3.1.6)$$

where:

J = Julian day of the year.

The K_r method was applied at 17 National Oceanic and Atmospheric Administration (NOAA) stations with long-term (1965-2000) daily temperature data to provide long-term estimates of R_s for hydrologic modeling. For Lake Okeechobee, the average estimated R_s at Canal Point, Moore Haven and Belle Glade data collection stations was used. The NOAA temperature data was thoroughly checked and patched to correct systematic errors, trends and missing values with the purpose of producing the best possible temperature dataset for R_s and ET estimation.

In order to guarantee reasonable estimates, the following two constraints were incorporated into the R_s estimation:

- A constant upper bound for the transmissivity is set to 0.75 across South Florida (i.e. clear-sky transmissivity defined as 75% of the extraterrestrial solar radiation; Smith, 1991).
- A lower bound for the transmissivity is set at 10% of the clear-sky transmissivity.

For each NOAA station, the K_r was selected so that the long-term average annual wet marsh potential ET estimated by the Simple method (Equation 2.3.1.1) matched an expected north to south gradient (Visher and Hughes, 1969). Figure 2.3.1.1 shows that the selected K_r values do not vary significantly from station to station with generally lower values occurring in the interior (e.g. minimum value of 0.154 at Devil's Garden) and higher values near the coast (e.g. maximum of 0.210 at Miami International Airport). In general, the selected K_r values agree with Hargreaves' (1994) recommendation of using K_r=0.16 for interior regions and K_r=0.19 for coastal regions. Annual time series and summary statistics of wet marsh potential evapotranspiration estimated at 17 NOAA stations and Lake Okeechobee are presented in Table 2.3.1.1.

The TIN method was selected for spatially-interpolating the wet marsh potential ET across a 2-mile by 2-mile super grid covering most of South Florida (Figure 2.3.1.2). Unlike rainfall stations, there is a scarcity of stations where wet marsh potential ET was estimated. Furthermore, ET is likely to be less localized than rainfall. Therefore, it was appropriate to apply the TIN methodology which resulted in a smoother spatial variation of potential ET.

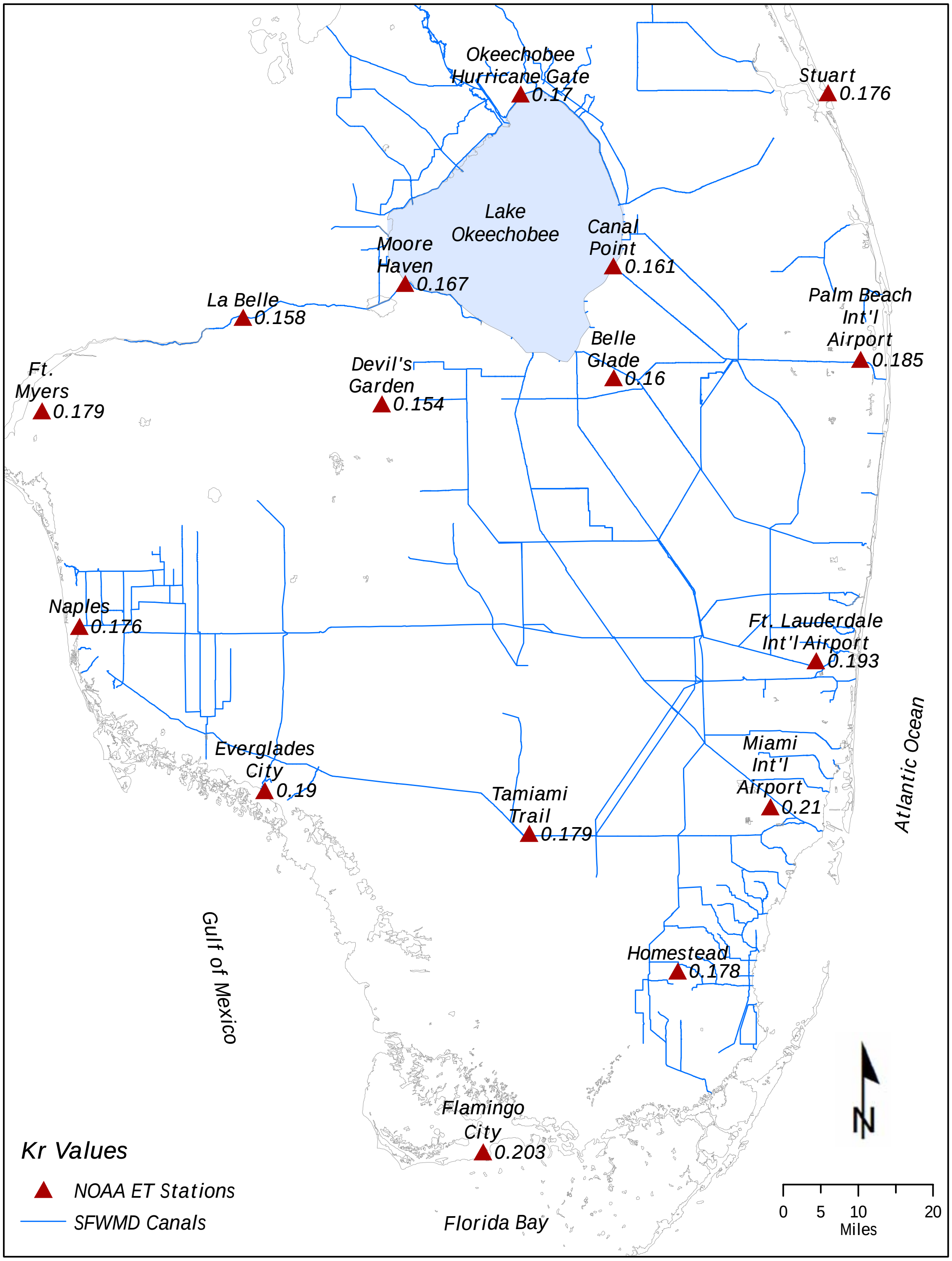

Figure 2.3.1.1 Selected K_r Values for 17 National Oceanic and Atmospheric Administration Stations with Long-Term Daily Temperature Data

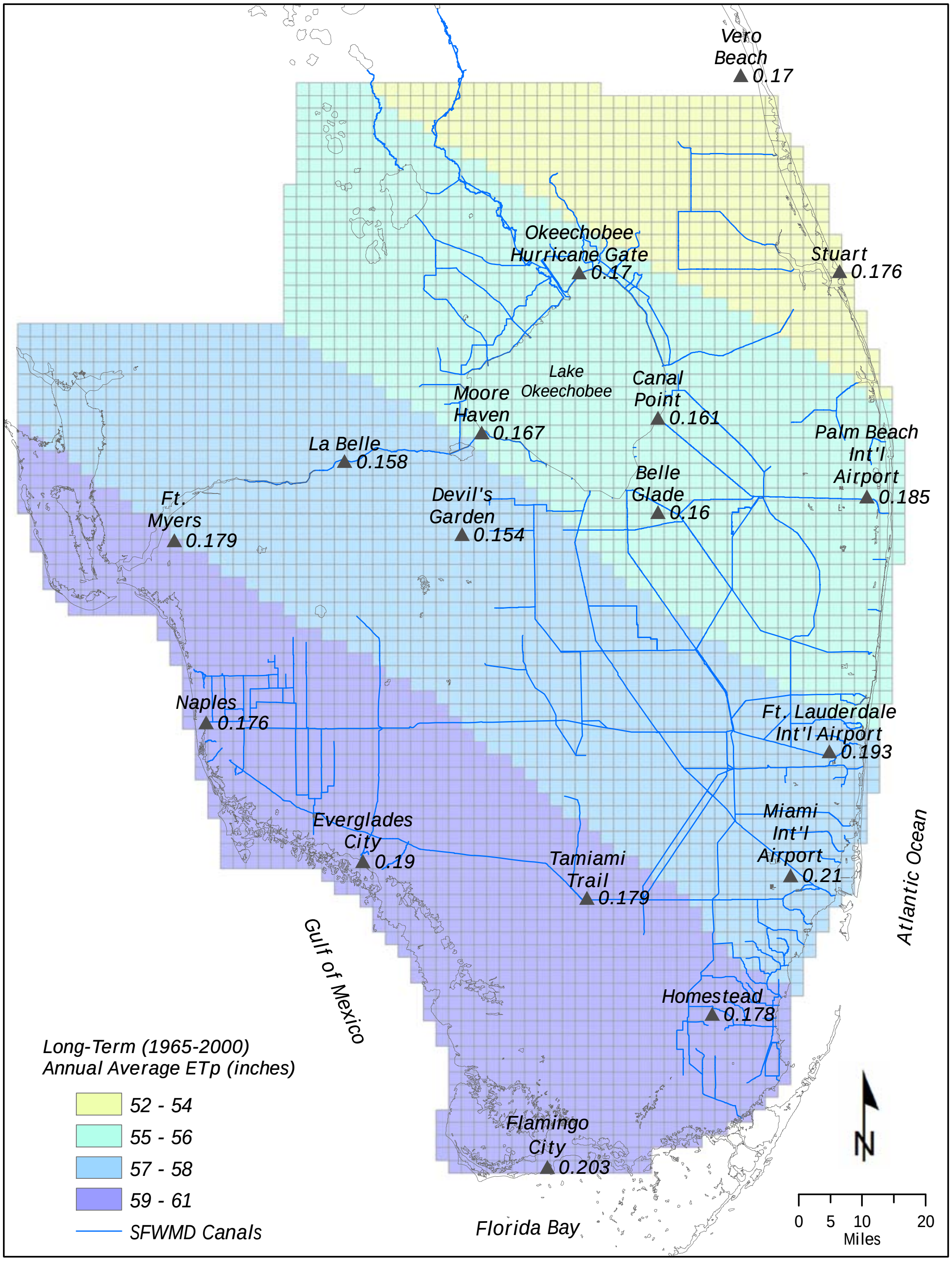

Figure 2.3.1.2 Estimated Annual Average Wet Marsh Potential Evapotranspiration for a 2-mile x 2-mile Super-Grid which Includes the South Florida Water Management Model and Natural System Model Grids

Table 2.3.1.1 Annual Time Series and Summary Statistics of Wet Marsh Potential Evapotranspiration Estimated at 17 National Oceanic and Atmospheric Administration Stations Plus Lake Okeechobee

Year	LOK	La Belle	Devils Garden	Ft Myers	Naples	Ever-glades City	Flamin-go	Home-stead	Tami-ami Trail	Miami	Ft Lauder-dale	WPBIA	Canal Point	Belle Glade	Moore Haven	Okee-chobee Hurr. Gate	Stuart	Vero Beach
Kr	N/A	0.158	0.154	0.179	0.176	0.190	0.203	0.178	0.179	0.210	0.193	0.185	0.161	0.160	0.167	0.170	0.176	0.170
1965	55.27	56.57	54.88	57.96	59.53	62.05	59.58	61.53	60.80	57.74	58.76	55.87	55.16	56.25	54.39	56.14	52.66	52.69
1966	52.74	54.92	53.90	56.94	57.94	60.51	56.77	58.36	56.16	56.85	57.67	53.80	53.13	53.83	51.25	54.77	51.78	52.01
1967	56.58	58.40	55.75	56.46	59.36	60.73	58.52	60.24	63.63	54.76	57.70	57.05	56.86	56.49	56.38	53.21	54.85	54.47
1968	54.57	57.37	54.53	57.70	58.36	60.22	57.89	59.07	59.78	55.59	58.36	56.27	54.31	54.79	54.61	53.97	54.87	53.17
1969	53.03	56.72	53.54	53.86	58.11	60.46	58.24	57.29	56.65	58.07	57.37	53.63	53.77	53.04	52.30	50.78	52.73	50.73
1970	54.75	58.85	55.27	55.86	60.22	58.52	58.93	59.37	53.54	56.73	57.92	54.79	55.18	53.65	55.41	51.84	55.93	52.94
1971	56.90	61.77	58.13	57.34	61.43	60.25	61.70	61.54	61.22	58.62	60.41	57.96	56.89	55.59	58.21	56.08	53.13	53.75
1972	55.60	59.76	56.42	59.32	60.88	58.41	59.34	58.77	58.83	55.98	58.19	54.90	54.34	54.04	58.41	55.23	52.56	52.61
1973	55.71	57.06	56.50	59.23	61.91	60.27	60.01	58.02	59.57	54.62	57.74	54.32	55.03	54.40	57.70	55.47	52.47	51.50
1974	55.95	58.07	57.64	59.90	62.95	60.58	57.60	59.85	60.10	57.24	57.72	53.94	55.66	54.77	57.43	56.04	55.65	53.09
1975	56.29	58.97	56.33	59.61	62.70	58.42	60.72	59.97	59.04	57.45	56.73	54.92	56.09	55.04	57.75	55.73	55.75	54.59
1976	55.38	57.73	57.58	59.14	62.31	60.21	62.08	58.09	56.12	56.63	55.27	54.38	54.92	53.90	57.32	53.63	56.06	54.08
1977	55.66	58.69	56.96	57.89	61.44	59.61	62.38	58.36	57.40	56.14	55.13	55.03	54.66	54.54	57.77	52.47	53.77	53.75
1978	53.65	58.38	53.99	57.57	59.82	59.58	61.30	57.30	55.98	54.80	56.13	55.06	53.85	52.65	54.45	52.95	53.31	53.72
1979	53.84	56.35	54.59	57.93	60.48	57.97	60.21	57.48	58.29	52.95	56.48	54.57	54.01	52.68	54.83	52.07	50.32	52.24
1980	55.30	57.67	55.35	58.56	60.36	58.80	61.83	59.01	59.75	55.86	57.58	57.78	55.20	53.85	56.84	54.41	54.37	54.06
1981	57.27	59.41	59.09	60.05	63.16	60.43	63.72	59.75	62.67	59.88	59.25	57.32	55.96	54.93	60.92	57.32	55.32	55.58
1982	54.03	55.33	52.69	56.76	60.70	57.69	60.75	58.33	60.47	56.36	56.56	50.83	53.31	53.18	55.59	55.91	51.90	50.85
1983	54.50	54.48	53.74	54.26	59.79	57.51	60.58	58.16	57.95	59.52	58.15	52.08	53.43	55.99	54.09	55.69	51.86	52.57
1984	54.58	55.53	54.30	56.73	58.12	60.35	61.41	62.29	56.93	59.23	55.56	52.67	54.25	55.10	54.40	55.00	54.78	50.41
1985	56.16	56.87	59.21	58.30	57.75	60.30	62.75	57.98	61.93	61.09	57.03	54.34	54.33	56.71	57.44	54.11	54.18	51.41
1986	55.96	56.85	55.05	59.85	58.34	61.27	63.42	57.82	57.20	60.40	55.53	54.59	54.87	56.89	56.11	55.00	53.76	54.64
1987	55.65	55.08	55.81	58.74	56.96	60.21	62.85	56.81	56.57	59.10	54.64	53.79	54.00	56.82	56.12	55.25	53.09	53.13
1988	55.65	56.33	59.00	60.61	58.36	63.59	58.07	55.40	57.99	58.80	55.10	53.90	54.60	56.51	55.83	55.00	52.60	52.85
1989	57.94	57.56	59.25	61.41	58.70	56.99	57.89	58.52	64.46	60.38	56.12	55.87	57.08	57.80	58.93	57.62	54.25	54.85
1990	57.10	56.37	57.11	60.83	58.71	56.90	61.55	58.10	63.73	58.41	54.95	54.20	56.64	57.36	57.30	51.22	51.26	52.09

Table 2.3.1.1 *(cont)* Annual Time Series and Summary Statistics of Wet Marsh Potential Evapotranspiration Estimated at 17 NOAA Stations plus Lake Okeechobee

Year	LOK	La Belle	Devils Garden	Ft Myers	Naples	Ever-glades City	Flamin-go	Home-stead	Tami-ami Trail	Miami	Ft Lauder-dale	WPBIA	Canal Point	Belle Glade	Moore Haven	Okee-chobee Hurr. Gate	Stuart	Vero Beach
Kr	N/A	0.158	0.154	0.179	0.176	0.190	0.203	0.178	0.179	0.210	0.193	0.185	0.161	0.160	0.167	0.170	0.176	0.170
1991	55.58	55.61	57.80	58.12	56.90	59.62	61.47	57.95	59.45	57.54	52.72	53.19	54.81	56.37	55.55	50.10	52.16	51.55
1992	55.38	54.66	57.45	58.23	57.35	57.69	61.20	59.44	59.79	58.21	54.26	54.71	54.61	56.23	55.30	52.79	52.86	53.44
1993	55.87	54.35	57.63	57.82	57.95	60.45	61.48	58.35	54.22	57.55	54.17	53.73	54.58	57.41	55.63	55.49	52.47	53.34
1994	53.90	56.24	58.28	57.11	55.85	59.39	60.74	59.24	56.36	55.41	51.19	54.97	53.30	55.05	53.35	52.68	51.93	51.59
1995	53.80	54.83	61.34	55.46	55.62	58.75	61.79	56.86	54.22	56.58	57.04	57.06	54.32	54.61	52.48	52.53	54.62	51.53
1996	55.72	54.60	61.28	57.27	58.11	62.45	62.67	56.75	58.31	57.51	54.99	53.58	55.46	56.03	55.66	53.70	54.03	51.88
1997	55.32	55.18	58.50	59.45	56.89	59.47	61.30	56.20	57.63	56.56	54.01	52.51	54.94	55.21	55.82	55.58	55.61	49.72
1998	54.67	53.60	58.50	56.51	56.33	56.20	63.82	55.19	56.44	56.20	54.42	53.33	54.86	55.10	54.05	54.62	51.79	51.06
1999	55.71	56.08	58.02	57.63	56.67	57.31	64.79	57.93	56.16	58.08	55.70	54.21	55.67	56.22	55.23	53.94	52.79	52.62
2000	58.19	55.22	56.82	58.85	57.49	58.12	58.63	57.32	56.67	57.53	55.02	53.94	58.24	58.99	57.32	54.81	52.52	53.09
Ann Ave	55.39	56.71	56.73	58.04	59.10	59.48	60.78	58.41	58.50	57.34	56.27	54.59	54.95	55.33	55.89	54.25	53.44	52.71
Stdev	1.25	1.81	2.14	1.71	2.07	1.63	1.98	1.57	2.70	1.81	1.91	1.57	1.16	1.51	1.99	1.78	1.48	1.36
Max	58.19	61.77	61.34	61.41	63.16	63.59	64.79	62.29	64.46	61.09	60.41	57.96	58.24	58.99	60.92	57.62	56.06	55.58
Min	52.74	53.60	52.69	53.86	55.62	56.20	56.77	55.19	53.54	52.95	51.19	50.83	53.13	52.65	51.25	50.10	50.32	49.72
Max-Min	5.45	8.16	8.64	7.56	7.53	7.39	8.02	7.10	10.92	8.14	9.22	7.12	5.12	6.34	9.66	7.52	5.74	5.86

2.3.2 Lake Okeechobee Evapotranspiration

Although Lake Okeechobee (LOK) ET is predominantly open water ET, spatial variation is accounted for by conceptualizing the Lake as made up of three distinct zones (Figure 2.3.2.1): an open water zone, a marsh (wetted or inundated littoral) zone, and a no-water (dry littoral) zone. The surface areas of these zones vary with Lake stage. Lake Okeechobee ET computation was originally based on the pan evaporation method (Shih, 1980), expanded to take into consideration the no-water zone (Ahn and Ostrovsky, 1992), and improved to account for reference crop ET calculations based on first the Penman-Monteith method (Trimble, 1996) and later the SFWMD Simple Method (Irrizary, 2003). The following equation is used in the model on a daily basis:

$$ET_{LOK,\,t} = ET_{ref,\,t}\,[A_{w,\,t} + k\,(A_{m,\,t} + A_{n,\,t})] \qquad (2.3.2.1)$$

where:

$ET_{LOK,\,t}$ = total LOK evapotranspiration [ac-ft];
k = evapotranspiration coefficient taken as 1.2 (Shih, 1980);
$A_{w,\,t}$ = LOK open water surface area [acre];
$A_{m,\,t}$ = LOK marsh surface area [acre];
$A_{n,\,t}$ = LOK no-water surface area [acre]; and
$ET_{ref,\,t}$ = wet marsh reference crop evapotranspiration [ft].

No-water zone ET is assumed to be limited by the total Lake monthly rainfall. Therefore, the total monthly dry littoral zone ET from the Lake cannot exceed the total monthly Lake rainfall. Daily dry littoral zone ET has a maximum value equal to the product of the total monthly Lake rainfall and the ratio of the daily pan evaporation to the total monthly pan evaporation.

The marsh zone exists where the bottom elevation of the Lake is above 11.5 ft National Geodetic Vertical Datum (NGVD) (Shih, 1980). The following conditional equations conceptualized in Figure 2.3.2.1 are used to calculate open water, marsh and no-water areas, respectively:

$$\begin{aligned} A_{w,t} &= fn(H_t) && \text{if } H_t \leq 11.5 \text{ ft NGVD} \\ &= A_{w,max} && \text{otherwise} \end{aligned} \qquad (2.3.2.2)$$

$$\begin{aligned} A_{m,t} &= 0 && \text{if } H_t \leq 11.5 \text{ ft NGVD} \\ &= fn(H_t) - A_{w,max} && \text{otherwise} \end{aligned} \qquad (2.3.2.3)$$

$$A_{n,t} = A_{LOK} - (A_{w,t} + A_{m,t}) \qquad (2.3.2.4)$$

where:

H_t = stage in Lake Okeechobee at time t [ft NGVD];
$A_{w,max}$ = Lake Okeechobee open-water surface area at 11.5 ft NGVD or higher [acres];
A_{LOK} = Lake Okeechobee surface area at 20 ft NGVD or higher (466,000 acres);
= defines upper limit of area enclosed by the peripheral levee around the Lake; and
$fn(H_t)$ = stage-area relationship for Lake Okeechobee, defined for stage less than or equal to 11.5 ft.

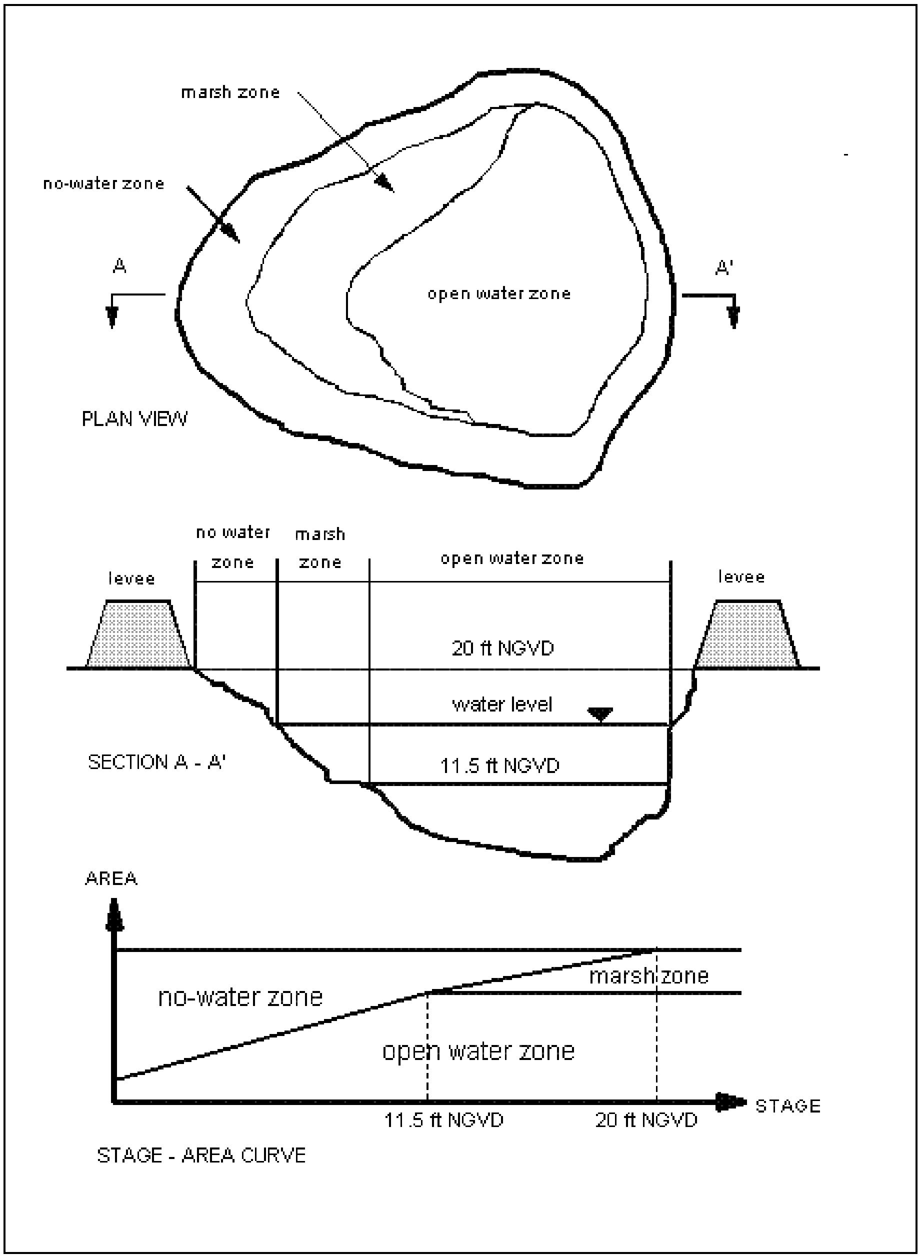

Figure 2.3.2.1 Conceptual Representation of the Different Lake Okeechobee Evapotranspiration Zones as Implemented in the South Florida Water Management Model

2.3.3 Everglades Agricultural Area

The calculation of ET in the EAA is strongly influenced by the operating rules governing the management of the EAA. The details of this topic will be discussed in Section 3.2. The remainder of the model domain, non-LOK and non-EAA, can be partitioned into non-irrigated and irrigated areas. The latter includes an unsaturated zone ET accounting procedure while the former makes simplifying assumptions for the unsaturated zone.

2.3.4 Non-irrigated Areas

Vegetation in the non-irrigated areas of the Lower East Coast (LEC) receives its water from rainfall and moisture from the unsaturated zone (or water table if the unsaturated zone dries up). For non-irrigated areas in the Water Conservation Areas (WCAs), Everglades National Park (ENP) and portions of Big Cypress National Preserve (BCNP), the following assumptions are made: (1) moisture content between land surface and water table does not change; (2) ET comes only from the saturated zone (ETS) and/or ponding (ETP); and (3) infiltration equals percolation.

The generalized form of the ET function in the model is:

$$ET = (KFACT)\,(ETR) \tag{2.3.4.1}$$

where:

ET = actual evapotranspiration;

KFACT = adjustment factor that takes into account vegetation/crop type and location of the water table relative to land surface as defined in Table 2.3.4.1 and Figure 2.3.4.1;

ETR = wet marsh potential ET (from Section 2.3.1).

Table 2.3.4.1 Variation of KFACT as a Function of Water Table Location

Zone	**Depth from Land Surface to Water Level** DWT: water table condition (below ground) PND: ponding condition (above ground)	Adjustment Factor, KFACT
I	DWT ≥ DDRZ	0.0
II	DSRZ < DWT < DDRZ	[(DDRZ - DWT) / (DDRZ - DSRZ)] (KVEG)
III	0 ≤ DWT ≤ DSRZ	KVEG
IV	0 < PND ≤ OWPOND	KVEG + (KMAX - KVEG) (PND / OWPOND)
V	PND > OWPOND	KMAX

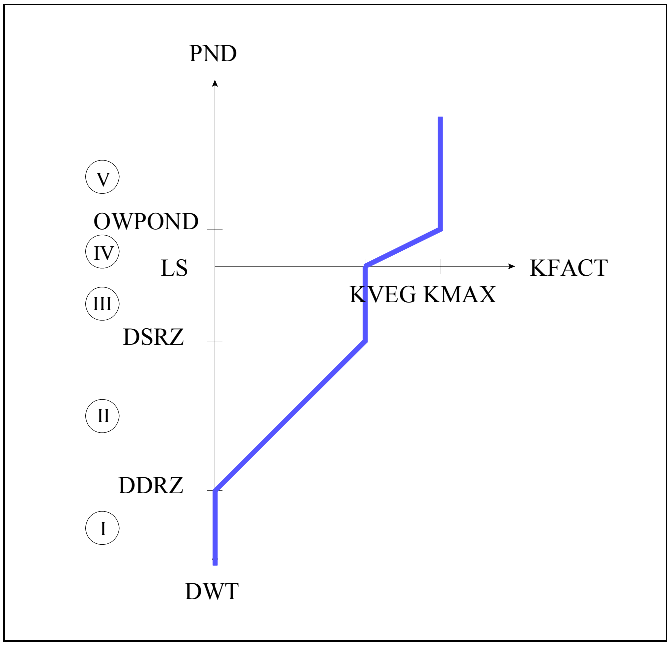

Figure 2.3.4.1 KFACT as a Function of Water Table Location

The definitions of the variables used in Figure 2.3.4.1 and Table 2.3.4.1 are as follows: *OWPOND* = minimum ponding depth above which ET for open-water is assumed, e.g., plants are fully submerged such that transpiration no longer contributes to ET; *LS* = land surface; *DSRZ* = depth from land surface to the bottom of the shallow root zone; *DDRZ* = depth from land surface to the bottom of the deep root zone; *PND* = depth from land to top of ponding; *DWT* = depth from land surface to water table; *KVEG* = calibrated vegetation/crop coefficient which is interpolated based on mid-month values assigned for each land use; *KMAX* = coefficient applied to ET for open water condition.

Tables 2.3.4.2 and 2.3.4.3 show the ET parameters associated with the modeled land cover/land use types as defined in Section 2.1. Note that:

1. Land uses 7 through 9, and 10 pertain to the three EAA agricultural types and Stormwater Treatment Area (STA) wetland classification, respectively.
2. Land uses 3-6, 12, 13 and 16 (McVoy and Park, 1997) can also be found in the Natural System Model (NSM) land use classification scheme.
3. The final or calibrated KVEG values for the three land uses in the EAA (land uses 7, 8 and 9 as shown in Table 2.3.4.3.) are the products of two parameters: (a) field-scale calibrated KVEG; and (b) calibration/adjustment factor KCALIB which are used to convert theoretical KVEG from field-scale to regional-scale. KCALIB values were determined during the calibration of the EAA (refer to Section 4.1).

For accounting purposes, if the water level goes above land surface (LS), the evapotranspiration is referred to as open-water ET (ETP). ETP is limited by the available ponding for the current day, i.e., previous day ponding plus current day rainfall. The portion of ET calculated from Equation (2.3.4.1) in excess of available ponding for the day is assumed to come from the saturated zone (ETS).

Table 2.3.4.2 Static Evapotranspiration Parameters used in the South Florida Water Management Model

	Land Use/Description	KMAX	OWPOND (ft)	DSRZ (ft)	DDRZ (ft)
1	Urban/low density	1.0	1.0	1.5	4.0
2	Agriculture/citrus	1.0	3.0	2.0	4.0
3	Wetland/freshwater marsh	1.0	4.0	0.0	1.2
4	Wetland/sawgrass plains	1.0	7.0	0.0	4.5
5	Wetland/wet prairie	1.0	3.5	0.0	2.0
6	Rangeland/shrubland (scrub and shrub)	1.0	3.5	0.0	7.0
7	Agriculture/row (or truck) crops	1.0	1.0	1.5	3.0
8	Agriculture/sugar cane	1.0	1.0	1.5	3.8
9	Agriculture/irrigated pasture	1.0	1.0	1.0	2.0
10	Wetland/stormwater treatment area and above-ground reservoir	1.0	4.0	0.5	5.0
11	Urban/high density	1.0	1.0	1.0	1.5
12	Forest/forested wetlands	1.0	10.0	0.0	9.0
13	Forest/mangroves	1.0	7.0	0.0	0.7
14	Forest/melaleuca	1.0	10.0	1.5	7.0
15	Wetland/cattail	1.0	6.0	0.0	3.0
16	Forest/forested uplands	1.0	10.0	4.8	11.0
17	Wetland/Ridge & Slough I	1.0	4.5	0.0	2.8
18	Wetland/marl prairie	1.0	3.0	0.0	6.5
19	Wetland/mixed cattail / sawgrass	1.0	7.0	0.0	4.0
20	Water/open water (deep excavated reservoirs)	1.0	0.0	0.0	0.0
21	Wetland/Ridge & Slough II	1.0	6.5	0.0	3.0
22	Wetland/Ridge & Slough III	1.0	3.0	0.0	1.5
23	Wetland/Ridge & Slough IV	1.0	6.8	0.0	3.0
24	Wetland/Ridge & Slough V	1.0	6.9	0.0	4.0
25	Urban/medium density urban	1.0	1.0	1.0	2.5

Notes: OWPOND is the minimum ponding depth above which ET for open-water is assumed.
DSRZ is the depth from the land surface to the bottom of the shallow root zone.
DDRZ is the depth from the land surface to the bottom of the deep root zone.

Table 2.3.4.3 Calibrated Vegetation/Crop Coefficient (KVEG) as a Function of Land Use and Month as Implemented in the South Florida Water Management Model

Land Use	Jan.	Feb.	Mar.	Apr.	May	Jun.	Jul.	Aug.	Sep.	Oct.	Nov.	Dec.
1	0.546	0.512	0.534	0.542	0.552	0.572	0.638	0.706	0.705	0.676	0.604	0.562
2	0.701	0.693	0.610	0.542	0.661	0.710	0.744	0.810	0.822	0.772	0.723	0.700
3	0.780	0.750	0.800	0.830	0.850	0.900	0.940	0.970	0.970	0.902	0.840	0.800
4	0.830	0.800	0.840	0.870	0.890	0.900	0.910	0.960	0.960	0.880	0.860	0.840
5	0.780	0.750	0.790	0.800	0.810	0.830	0.850	0.880	0.880	0.835	0.810	0.790
6	0.820	0.790	0.830	0.840	0.850	0.860	0.870	0.880	0.880	0.850	0.835	0.820
7[a]	0.640	0.690	0.870	0.950	0.860	0.660	0.610	0.660	0.710	0.870	0.930	0.880
8[a]	0.800	0.600	0.550	0.800	0.950	1.000	1.050	1.050	1.050	1.000	0.950	0.900
9[a]	0.650	0.700	0.750	0.950	0.950	0.980	0.980	0.980	0.940	0.800	0.870	0.650
10	0.830	0.782	0.810	0.835	0.848	0.860	0.880	0.920	0.920	0.872	0.844	0.830
11	0.413	0.381	0.392	0.401	0.412	0.422	0.435	0.455	0.480	0.483	0.442	0.415
12	0.700	0.670	0.710	0.720	0.740	0.750	0.770	0.780	0.780	0.760	0.730	0.710
13	0.710	0.700	0.730	0.750	0.790	0.830	0.890	0.950	0.950	0.870	0.790	0.730
14	0.770	0.740	0.790	0.820	0.850	0.880	0.900	0.930	0.930	0.850	0.810	0.780
15	0.805	0.780	0.810	0.820	0.832	0.848	0.862	0.890	0.890	0.840	0.815	0.807
16	0.730	0.700	0.740	0.760	0.800	0.870	0.940	0.980	0.980	0.950	0.870	0.750
17	0.760	0.740	0.770	0.790	0.810	0.850	0.920	0.980	0.980	0.910	0.810	0.770
18	0.780	0.750	0.790	0.820	0.850	0.890	0.960	0.995	0.995	0.950	0.860	0.800
19	0.815	0.790	0.825	0.835	0.850	0.870	0.882	0.930	0.930	0.860	0.835	0.820
20	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000	1.000
21	0.610	0.600	0.630	0.650	0.670	0.690	0.720	0.740	0.740	0.720	0.675	0.620
22	0.780	0.750	0.790	0.800	0.820	0.850	0.870	0.880	0.880	0.870	0.840	0.820
23	0.770	0.750	0.780	0.800	0.830	0.860	0.910	0.960	0.960	0.910	0.840	0.780
24	0.830	0.800	0.840	0.870	0.890	0.900	0.910	0.960	0.960	0.880	0.860	0.840
25	0.500	0.450	0.475	0.490	0.510	0.530	0.560	0.600	0.600	0.570	0.540	0.510

[a]For the EAA, these values are multiplied by an additional calibration coefficient KCALIB (refer to Section 4.1).

2.3.5 Irrigated Areas in the Lower East Coast

For irrigated areas, primarily LEC Service Area grid cells, the unsaturated zone is treated as a separate control volume where infiltration, percolation, evapotranspiration and changes in soil moisture are accounted for. The reasons for the unsaturated zone accounting are: (1) the desire to implement the Water Shortage Plan in the LEC (SFWMD, 1991) which entails cutbacks in irrigation amounts and frequencies; (2) the need to quantify LEC irrigation applied to the unsaturated zone; and, consequently, (3) the need to more accurately assess changes in irrigation requirements associated with changes in land use.

In irrigated areas in the LEC, a two-step approach is taken to calculate total ET from each irrigated grid cell. In the first step, unsaturated zone moisture accounting is performed for the irrigated portion of a model grid cell. If ΔS represents the change in soil moisture content, then the water balance equation for the unsaturated zone at the end of each time step is:

$$\Delta S = NIRRSUPTOT - ETU + INFILT - PERC \qquad (2.3.5.1)$$

where:

NIRRSUPTOT = total net irrigation application depth [in.)];
ETU = evapotranspiration from the unsaturated zone [in.];
INFILT = total flux across land surface due to ponding/rainfall [in.]; and
PERC = total flux across the water table used as recharge to the saturated zone [in.].

NIRRSUPTOT and ETU represent the preprocessed (input to the model) total net irrigation supply and unsaturated zone evapotranspiration. They are calculated from the ET-Recharge model (Restrepo and Giddings, 1994) which is an extension of the Agricultural Field-Scale Irrigation Requirements Simulation (AFSIRS) program (Smajstrla, 1990) outlined in Appendix Q. Computational requirements, central processing unit (CPU) time and data storage for field-scale unsaturated zone moisture accounting are very prohibitive such that pre-processing ETU data was opted for the SFWMM.

Infiltration depth, INFILT, is the minimum of ponding depth, infiltration rate, and available unsaturated zone storage. Therefore, the time-dependent moisture content in the unsaturated zone (S_t) can be expressed as:

$$S_t = S_{t-1} + INFILT + NIRRSUPTOT - ETU \qquad (2.3.5.2)$$

If S_t is less than the water-holding capacity of the unsaturated zone (SWSCAP), then percolation, PERC, is zero. Otherwise, PERC becomes the soil-moisture content in excess of SWSCAP and the final moisture-content for time step t equals SWSCAP.

In the second step, the saturated zone evapotranspiration (ETS) is calculated using ET from the generalized ET function, Equation (2.3.4.1):

$$ETS = ET - ETU \qquad (2.3.5.3)$$

Due to differences in scale and assumptions used between the ET-Recharge model and SFWMM, there are instances when the unsaturated zone moisture accounting cannot be carried out due to the lack (or absence) of moisture in the unsaturated zone. In such cases, the unaccounted for ETU is taken directly out of the saturated zone, thus lowering the water table by a corresponding amount.

ET-Recharge Model

In the LEC service areas (Figure 1.3.5), irrigation supply and unsaturated zone ET are pre-processed, i.e., pre-calculated quantities input to SFWMM, and used in the unsaturated zone moisture accounting. These quantities, among others, were output from the ET-Recharge model (Giddings and Restrepo, 1995). This model was originally used to provide a more accurate method for estimating the recharge component for the District's countywide groundwater models. The model was later enhanced to handle any user-specified model grid, e.g., SFWMM grid system.

The necessary input to the model can be classified into the following two categories:

1. A text file description of basic element areas (BEAs): area; levels 1, 2, and 3 land use codes; soil code equivalent to AFSIRS SOIL.DAT file; cell (row and column numbers) location within the SFWMM grid system; vertical hydraulic conductivity of the soil; active/inactive designation for cell; flag indicating if the BEA is located east of the saltwater interface; and
2. A reference table for each BEA in (1) relating the District's level 3 land use classification (Florida Department of Transportation, 1985) to the following: runoff coefficients; crop type; growing season; percent pervious area; switch indicating if a BEA is irrigated or not; and water use type classification.

To perform a crop root zone water balance on a daily basis, the following approach is taken.

First, BEAs are defined for the LEC. By definition, a BEA is a polygon having a unique combination of attributes such as land use, soil type, percent irrigated, non-irrigated and impervious area, vertical hydraulic conductivity, and SFWMM cell location. As mentioned in Section 2.1.2, use of BEAs allows the SFWMM to capture land use variability at a scale smaller than the 2-mile by 2-mile discretization of the overall model grid. The size of a SFWMM grid cell is the upper limit on the size of a BEA.

Once defined, if a BEA falls within a pervious area, AFSIRS is called to perform crop root zone water balance on a daily basis. AFSIRS calculates irrigation requirements and crop evapotranspiration rates as a function of crop type, soil type, irrigation system, growing season, and climatic conditions. It assumes that crop requirements are met from the unsaturated zone through rainfall or supplemental irrigation. An irrigation management option within AFSIRS was selected such that the exact amount and timing of the irrigation is to be used to restore the root zone to field capacity (i.e., maximum yield and thus, maximum or potential ET is always maintained).

Some of the most important assumptions in AFSIRS as applied to the irrigated areas of the LEC are as follows:

1. The calculated drainage does not distinguish between runoff and percolation.
2. The crop root zone is entirely in the unsaturated zone.
3. Lateral flow is neglected in the unsaturated zone.
4. Crop requirements are met from the unsaturated zone through rainfall or supplemental irrigation.
5. Crop-water requirements are calculated based on maximum yield.
6. AFSIRS does not compute yield but calculates the quantity and frequency of irrigation necessary to avoid crop stress.
7. The calculated net irrigation requirement does not include leaching, freeze protection or crop cooling requirements.

Daily rainfall and wet marsh potential ET (ETR) are defined as inputs to AFSIRS. Since rainfall and ETR amounts are defined for each SFWMM grid cell, rainfall (RF) and ETR for a basic element area is taken as the value assigned to the SFWMM cell where the BEA is located. AFSIRS calculates the potential evapotranspiration for crop c (ETc) using the formula:

$$ETc = (kc)\,(ETR) \qquad (2.3.5.4)$$

where: kc is the crop coefficient that varies with crop type and crop growth stage.

The rate at which water is returned from the soil to the atmosphere by ET is controlled by two factors: atmospheric demand and soil-water availability (Jensen, et al., 1990). At the end of each time step, the AFSIRS water balance equation for the crop root zone is:

$$\Delta STO = RAIN + NIRR - DRAIN0 - RUNOFF - ET \qquad (2.3.5.5)$$

where:

ΔSTO = change in root zone soil water storage [in.];
RAIN = rainfall [in.];
NIRR = net irrigation requirement or irrigation supply [in.];
DRAIN0 = drainage [in.];
RUNOFF = surface runoff [in.]; and
ET = evapotranspiration [in.].

In the ET-Recharge model, the runoff and drainage terms are combined to form the variable DRAIN, i.e., RUNOFF + DRAIN0. All BEAs within a SFWMM grid cell can be combined and Equation (2.3.5.5) can be rearranged, and written in terms of NIRR (an input to the SFWMM):

$$NIRR = \Delta STO - RAIN + DRAIN + ET \qquad (2.3.5.6)$$

Drainage is calculated as the difference between rainfall and available soil water storage (storage beyond field capacity) at the time rain occurs. By implementing an extended form of the Soil Conservation Service (SCS) runoff estimation method (McCuen, 1982), the DRAIN term can be partitioned back into total direct runoff and the original drainage term DRAIN0 in Equation 2.3.5.5 (Giddings and Restrepo, 1995). This approach involves the use of the Curve Number

(CN) for major storm events. AFSIRS assumes that the crop root zone is entirely within the unsaturated zone (ET = ETU). The maximum unsaturated zone ET, ETUmax, can vary depending on whether a BEA is impervious or pervious.

For impervious areas, the ET-Recharge model assumes negligible ETUmax. For SFWMM grid cells with non-irrigated pervious areas, ETUmax = (ETc) (% of pervious area). For SFWMM grid cells with irrigated pervious areas, ETUmax = (ETc - supplemental requirement) (% of pervious area), i.e., ETUmax is limited by the amount of available soil moisture in the unsaturated zone. Supplemental irrigation requirements can be met from the water table.

The ET-Recharge model aggregates output from BEAs to SFWMM grid values. A list of output information generated on a daily basis from the model pertinent to the SFWMM is as follows (in inches per day):

1. composite crop PET per LEC model grid cell (ET_{p_cell});
2. unsaturated zone ET per LEC model grid cell (ETU_{cell});
3. unsaturated zone ET for irrigated portion of each LEC model grid cell ($ETIU_{cell}$); and
4. irrigation deliveries per water use type (landscape, golf course, agricultural overhead, agricultural low volume, and agricultural other) for each LEC model grid cell.

A FORTRAN program is used to aggregate the irrigated (pervious) acreages for the BEAs into a composite acreage per LEC grid cell for each water use type. These acreages appear in the model as the independent terms LSC (landscape), GLF (golf course), AOH (agricultural overhead), ALV (agricultural low volume), and AOT (agricultural other).

Irrigation deliveries calculated from the ET-Recharge model are treated as target irrigation demands in the SFWMM. These irrigation demands can be met from the water table, wastewater reuse and public water supply (PWS), and are the basis for implementing the LEC trigger and cutback modules (refer to Table 3.5.4.1).

Irrigation Demands met by Alternate Sources

In the LEC, some areas may be irrigated by local municipal water (PWS pumpage) or wastewater reuse and do not rely on the surficial aquifer (water table). In order to account for the reduced impact of these acreages on the SFWMM grid cell water budget, a method was developed to reduce the irrigation demands that are met from the water table. This method identifies three parameters which represent the reduction in demands. These parameters are: FLI (fraction of landscape irrigation from PWS), FLR (fraction of landscape irrigation from wastewater reuse) and FGI (fraction of golf course irrigation from wastewater reuse). Values of FLI, FLR and FGI are defined as fractions of the total landscape (FLI and FLR) or golf course (FGI) irrigation and are subtracted from the total irrigation demands as calculated by the ET-Recharge model prior to influencing the SFWMM grid cell. Figures 2.3.5.1, 2.3.5.2 and 2.3.5.3 show the values used in the SFWMM for the FLI, FLR and FGI terms, respectively, for the 2000 condition. During drought conditions, the acreages whose irrigation demands are met by alternate sources will remain unaffected by water shortages and only the net water table demands (after reduction) will be cut back. For a more detailed description of this method, see Appendix S.

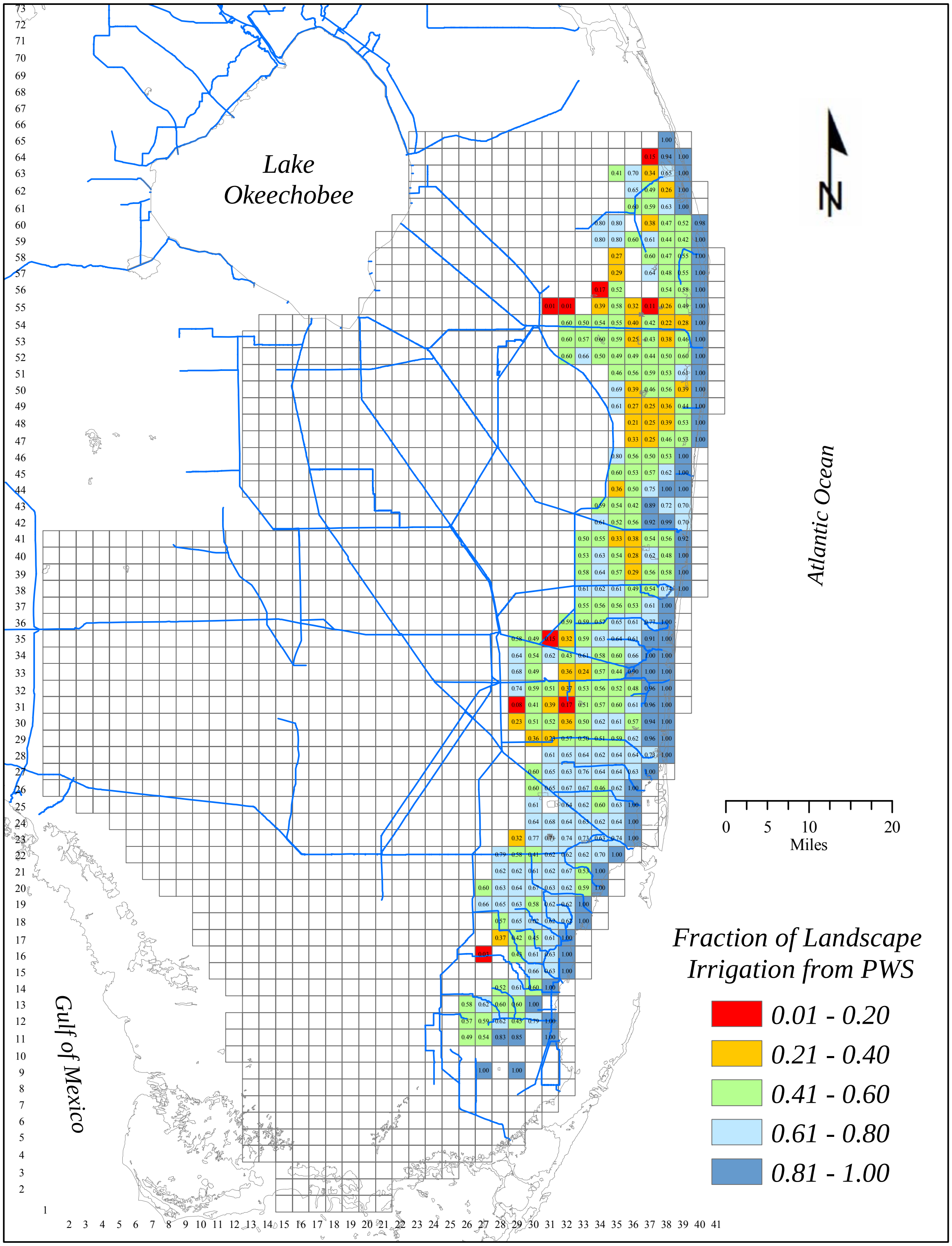

Figure 2.3.5.1 2000 Fraction of Landscape Irrigation from Public Water Supply Map

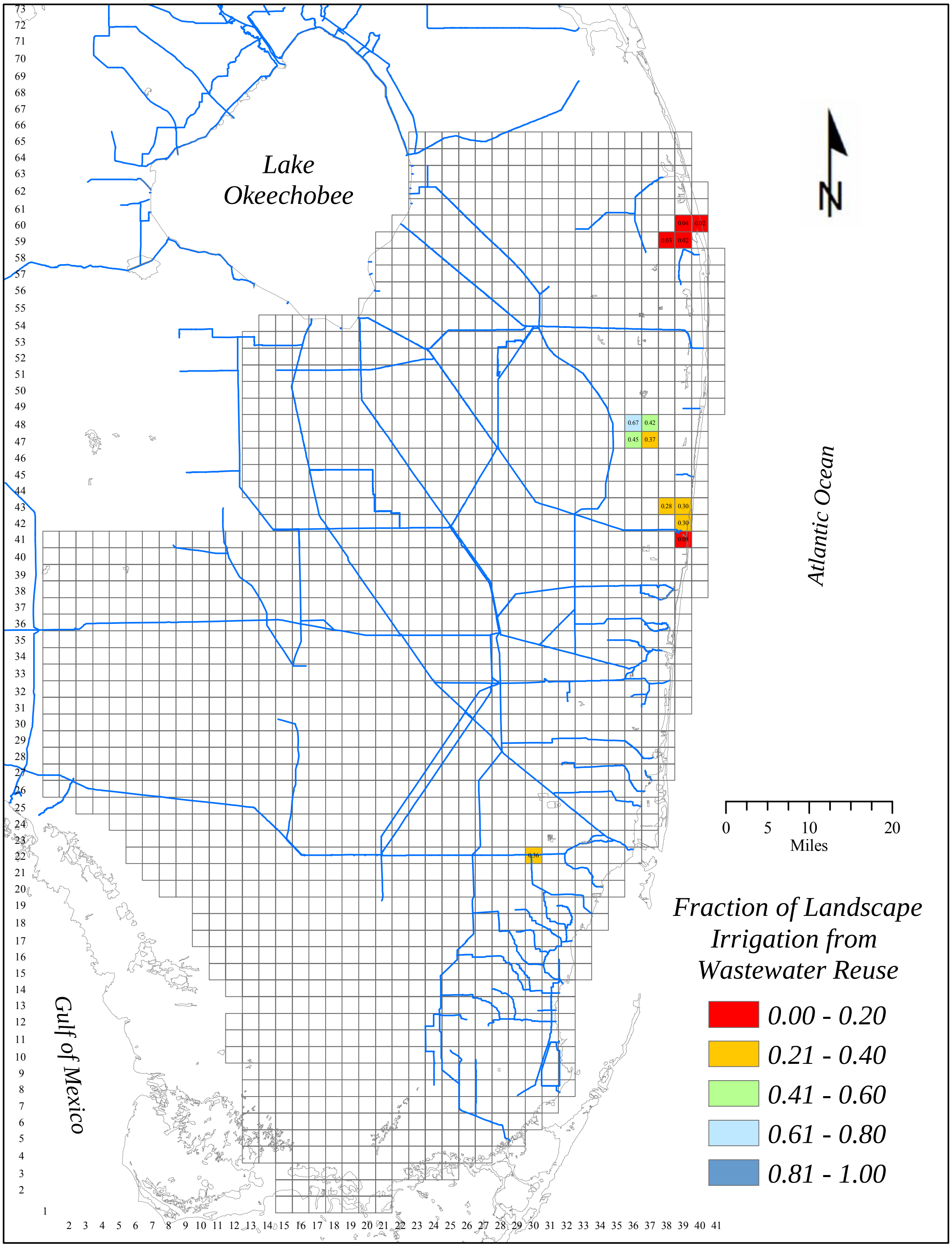

Figure 2.3.5.2 2000 Fraction of Landscape Irrigation from Wastewater Reuse Map

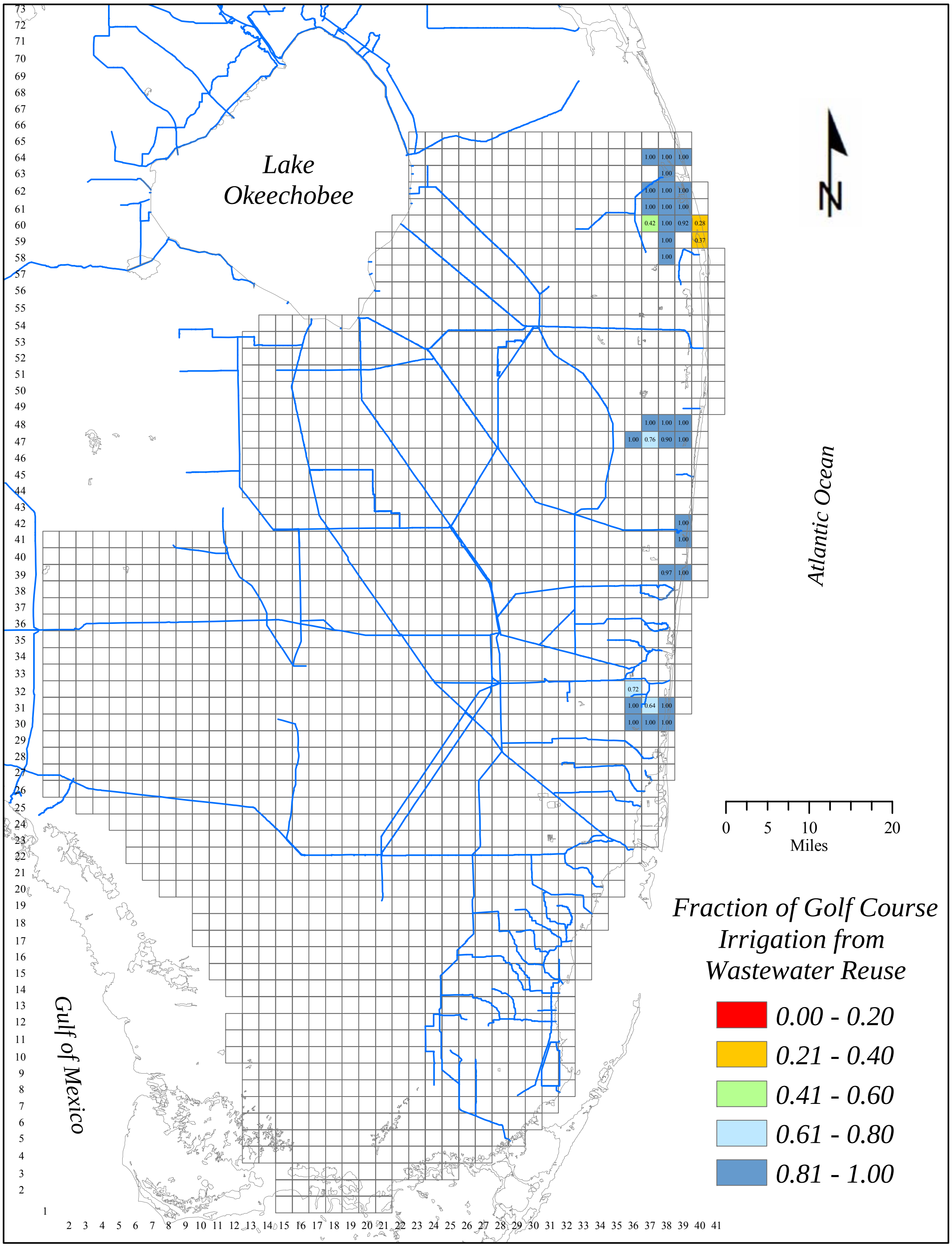

Figure 2.3.5.3 2000 Fraction of Golf Course Irrigation from Wastewater Reuse Map

2.4 OVERLAND FLOW

Overland flow or sheetflow above land surface in the South Florida Water Management Model (SFWMM) involves the movement of surface water either from cell to cell (internodal flow) or from canal to cell and vice versa. This section focuses on the cell to cell flow process involved in moving surface water from one cell to the next. Section 2.6 discusses the mechanics involved in a canal-to-cell or cell-to-canal overland flow. Succeeding references to overland flow in this documentation refer to cell-to-cell overland flow, unless otherwise noted.

2.4.1 Governing Equations

The diffusion flow model (Akan and Yen, 1981) is used to simulate overland flow in the SFWMM. The primary driving force for diffusion flow is the slope of the water surface. Although a diffusion wave model can account for backwater effects through the pressure terms in the momentum equation, the absence of the inertial or acceleration terms prohibit water from traveling opposite head gradients.

Using water depth as a variable, the two-dimensional continuity equation for shallow water flow is:

$$\frac{\partial h}{\partial t} + \frac{\partial (hu)}{\partial x} + \frac{\partial (hv)}{\partial y} - q = 0 \tag{2.4.1.1}$$

where:

- h = water depth [ft];
- u, v = velocity in the x- and y- directions [ft/day]; and
- q = vertical influx which consists of the net effect of rainfall, infiltration and evapotranspiration [ft/day].

Expressing depth of flow as water level above a datum, the momentum equation in the x-direction can be expressed as:

$$\frac{\partial H}{\partial x} + \frac{\tau_{bx}}{\rho g h} = 0 \tag{2.4.1.2}$$

while the momentum equation in the y-direction is:

$$\frac{\partial H}{\partial y} + \frac{\tau_{by}}{\rho g h} = 0 \tag{2.4.1.3}$$

where:

- $H = h + z$ = water level above a given datum [ft NGVD]; the SFWMM uses the National Geodetic Vertical Datum (NGVD) of 1929;
- h = depth of flow [ft]; the hydraulic radius is essentially the depth of flow for wide channels, as in the case for SFWMM;
- z = channel bottom elevation above the datum [ft NGVD];
- τ_{bx}, τ_{by} = bed (bottom and sides) shear stress in the x- and y- directions [lb/ft^2];
- ρ = density of water [slugs/ft^3]; and
- g = acceleration due to gravity [ft/sec^2].

The derivation of the above three partial differential equations assumes the following conditions:

1. Vertical velocities and accelerations are neglected, thus flow is essentially two-dimensional.
2. The fluid is incompressible and has uniform density.
3. Bottom slope is small and the channel bed is fixed (no scouring or deposition).
4. Flow is assumed to vary gradually so that hydrostatic pressure prevails.
5. Manning's equation, which applies to steady uniform turbulent flow, can be used to describe bottom resistance effects or bed shear stress, i.e., the slope of the energy grade line S_f can be approximated by means of a semi-empirical formula valid for steady flow.
6. Coriolis effects, surface resistance (wind) stress and shear stresses due to turbulence are ignored.

The bed shear terms can be defined as:

$$\vec{\tau}_b = \rho g h \vec{S}_f \tag{2.4.1.4}$$

where $\vec{\tau}_b$ is the resultant bed shear stress in the direction of the maximum energy slope S_f.

Applying Manning's equation for wide channels (wetted perimeter ≅ bottom width) in the direction of flow (i.e., direction of maximum energy slope):

$$V = \frac{1.49}{n} h^{\frac{2}{3}} S_f^{\frac{1}{2}} \tag{2.4.1.5}$$

where:

$V = \sqrt{v^2 + u^2}$ = magnitude of the velocity vector;
n = Manning's roughness coefficient; and
$S_f = \sqrt{S_x^2 + S_y^2}$ = magnitude of energy slope.

The direction of flow forms an angle of $\theta = \cos^{-1}\left(\frac{u}{V}\right)$ with the x-axis. Since $\tau_{bx} = \tau_b \cos\theta$ and $\tau_{by} = \tau_b \sin\theta$, and from the preceding two equations, (2.4.1.4) and (2.4.1.5), an expression for the two components of bed shear stress can be stated as:

$$\tau_{bx} = -\frac{\rho g n^2 u V}{(1.49)^2 h^{\frac{1}{3}}} \tag{2.4.1.6}$$

$$\tau_{by} = -\frac{\rho g n^2 v V}{(1.49)^2 h^{\frac{1}{3}}} \tag{2.4.1.7}$$

The negative sign implies that the shear stress goes in the opposite direction to the velocity vector. Based on the third assumption mentioned above, the change in depth of flow with respect to the x or y direction is identical to the change in water level based on the x or y direction. Substituting these equations into the momentum equations yields:

$$u = \frac{1.49 h^{\frac{2}{3}} \sqrt{\frac{\partial H}{\partial x} \cos\theta}}{n} \tag{2.4.1.8}$$

$$v = \frac{1.49 h^{\frac{2}{3}} \sqrt{\frac{\partial H}{\partial y} \sin\theta}}{n} \tag{2.4.1.9}$$

Since the direction of flow expressed in terms of θ in diffusion flow problems goes in the direction of maximum energy slope, its derivation can be based solely on the slopes of the water surface, i.e., θ can be expressed in terms of H.

Assigning the slope of the water surface to the friction slope in the x- and y- directions: $S_x = \frac{\partial H}{\partial x}$ and $S_y = \frac{\partial H}{\partial y}$, the maximum energy slope becomes the resultant water surface slope $S_f = S_n = \sqrt{\left(\frac{\partial H}{\partial x}\right)_x^2 + \left(\frac{\partial H}{\partial y}\right)_y^2}$ and the angle between the flow direction and the x- axis can be calculated as $\theta = \cos^{-1}\left(\frac{S_x}{S_n}\right)$. Therefore, the u and v velocity components can also be expressed as:

$$u = 1.49 \frac{h^{\frac{2}{3}}}{n\sqrt{S_n}} \frac{\partial H}{\partial x} \tag{2.4.1.10}$$

$$v = 1.49 \frac{h^{\frac{2}{3}}}{n\sqrt{S_n}} \frac{\partial H}{\partial y} \tag{2.4.1.11}$$

By substituting Equations (2.4.1.10) and (2.4.1.11) into the continuity equation, (2.4.1.1), the set of three partial differential equations representing overland flow reduces into a single equation with water level or depth of flow as the only unknown variable.

2.4.2 Model Implementation

The SFWMM uses a finite difference approximation of the preceding governing equations to calculate flow velocities in the x- and y- directions, u and v, for each grid cell. The numerical method employed, alternating-direction explicit or ADE, uses the stage values from the previous time step for a particular cell (source cell) and two of its immediate neighboring cells (destination cells): one just to the right (or just to the left) of the source cell, and the other just below (or just above) the same source cell. Two velocities, Equations (2.4.1.10-11), are calculated based on satisfying the diffusion flow model for overland flow. However, violation of the stability condition is avoided by limiting the amount of water across the boundary of two adjacent cells by taking the minimum of (a) the available volume of water from the source cell; (b) flow rate x time; or (c) flow volume required to obtain identical ponding depths between source and destination cells at the end of the time step. The final stages at the source and destination cells are determined by the minimum of the flow volumes resulting from these three conditions. In order to maintain stability and still use the diffusion equation for the majority of the simulation, the model is capable of breaking the standard 1-day time step in the overland flow subroutine into several time slices.

The model uses four six-hour time slices for each day of overland flow calculations. A complete pass of all the grid cells is accomplished for each time slice. Thus, the ponding depth at each cell is updated four times in the course of one day. The difference in the calculations from one time slice to the next is based on the sequence in which source cells are selected and the order in which the two destination cells are selected for each source cell. For the first and third time slices, the left-to-right, top-to-bottom sequence is used. At any given grid cell, the first time slice calculates the flow velocity in the east direction before calculating the flow velocity in the south direction. The order is reversed for the third time slice. In the first and third time slices, the cells immediately to the east and south are referred to as the destination cells and flows across the right and bottom faces of the source cell are calculated. For the second and fourth time slices, the right-to-left, bottom-to-top sequence is used. At any grid cell, the second time slice calculates the flow velocity in the west direction before calculating the flow velocity in the north direction. The order is reversed for the fourth and final time slice. In the second and fourth time slices, the cells immediately to the west and north are referred to as the destination cells and flows across the left and top faces of the source cells are calculated. Figure 2.4.2.1 shows the location of computation (source and destination) cells used in the overland flow subroutine of the SFWMM.

The finite difference approximation of Equation (2.4.1.10) for the horizontal flow velocity becomes:

$$VOF_x = \frac{1.49}{n} h^{\frac{2}{3}} \left(\frac{\Delta H_x}{\Delta x}\right) \left[\sqrt{\left(\frac{\Delta H_x}{\Delta x}\right)^2 + \left(\frac{\Delta H_y}{\Delta y}\right)^2} \right]^{-\frac{1}{2}} \qquad (2.4.2.1)$$

where:

VOF_x = finite difference approximation of u;

h = water depth at the source cell;

ΔH_x = HS - HD_x = difference in water level between source cell S and destination cell D_x in the x-direction;

ΔH_y = HS - HD_y = difference in water level between source cell S and destination cell D_y in the y-direction;

Δx = horizontal distance between the centers of source cell S and destination cell D_x; and

Δy = vertical distance between the centers of source cell S and destination cell D_y.

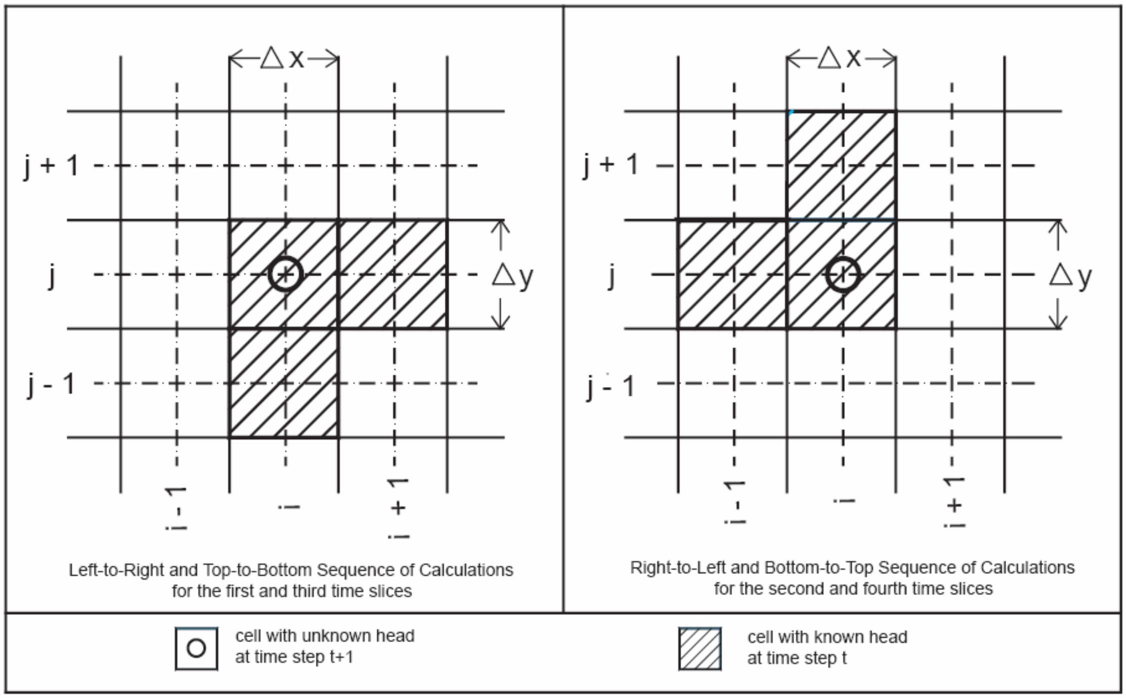

Figure 2.4.2.1 Location of Grid (Source and Destination) Cells Used in Calculating Total Head at Grid Cell (i,j) During Time Step t+1 as Implemented in the Overland Flow Subroutine in the South Florida Water Management Model

Since the model only handles square cells (ΔL = Δx = Δy), hence:

$$VOF_x = \frac{1.49}{n} h^{\frac{2}{3}} \left(\frac{\Delta H_x}{\Delta L}\right)\left(\sqrt{\frac{\Delta H_x^2 + \Delta H_y^2}{\Delta L^2}}\right)^{-\frac{1}{2}}$$

which can be rearranged to:

$$VOF_x = \frac{1.49}{n} h^{\frac{2}{3}} \left(\frac{\Delta H_x}{\Delta L^{\frac{1}{2}}}\right)\left(\frac{1}{\sqrt{\Delta H_x^2 + \Delta H_y^2}}\right)^{\frac{1}{2}} \tag{2.4.2.2}$$

From Equation (2.4.2.2), the flow rate in the x-direction can be calculated as:

$$Q_x = (VOF_x)(h)(WDTHOV_x) \tag{2.4.2.3}$$

where $WDTHOV_x$ is the width of overland flow in the x-direction (Δy).

The solution to the diffusion equation will yield a volume of overland flow in the x-direction:

$$VOLOV_x = Q_x (DTS) \tag{2.4.2.4}$$

where DTS equals the length of a time slice.

To maintain stability, the model limits this volume by the two other parameters discussed earlier. The limiting flow volume becomes the basis for the final stages at the three computational cells. The corresponding flow velocity, flow rate and volume of overland flow in the y-direction are:

$$VOF_y = \frac{1.49}{n} h^{\frac{2}{3}} \left(\frac{\Delta H_y}{\Delta L^{\frac{1}{2}}} \right) \left(\frac{1}{\sqrt{\Delta H_x^2 + \Delta H_y^2}} \right)^{\frac{1}{2}} \tag{2.4.2.5}$$

$$Q_y = \left(VOF_y \right) \left(h \right) \left(WDTHOV_y \right) \tag{2.4.2.6}$$

$$VOLOV_y = Q_y \left(DTS \right) \tag{2.4.2.7}$$

The head at the grid cell denoted by (i,j) at time step t+1 is a function of the head at three adjacent cells evaluated at the previous time step t. The selection of which two cells, in addition to itself, to consider depends on the current time slice, as mentioned earlier. Discussion related to the accuracy of the SFWMM overland flow algorithm is available in two published articles by Lal (1998 and 2000).

Resistance to Sheetflow

Movement of water above land surface, sheetflow or overland flow, is governed by two sets of parameters in the model: detention depth and roughness. Detention depth (DETEN) is the depth of ponding within a grid cell below which no transfer of water from one grid cell to the next is allowed even if a hydraulic gradient exists between the adjacent cells. Detention depth is used to characterize water retained as puddles in small surface depressions that may sporadically exist at varying sizes within a 2-mile by 2-mile model grid cell. The model treats each grid cell as a perfectly horizontal surface. If detention depth is exceeded and a gradient is established between adjacent cells, surface roughness determines the magnitude of flow. The effective roughness parameter (N) used is similar to Manning's roughness coefficient. Lower N values (i.e., less resistance to flow) are associated with larger ponding depths due to an increase in the ratio of water depth to plant height. In the model, N is an exponential function of ponding depth (POND):

$$N = A\ (POND^b) \tag{2.4.2.8}$$

It is computed for each grid cell, every time step based on land use and ponding depth at the grid cell. Figure 2.4.2.2 shows the N values from Equation 2.4.2.8 for Mixed Cattail and Sawgrass (refer to Figure 2.1.2.9 for aerial view of landscape). The effect of micro-topography (non-uniform surface) is somewhat compensated for by computing N as a function of depth. At low water levels in a cell, the flow can be impeded by varying ground levels; as water level in a cell rises, then sheetflow starts to dominate and flow becomes a more uniform function of land cover. Table 2.4.2.1 shows the coefficients A and b used to determine N for the twenty-five land use types in the model. Land use types 7, 8 and 9 are the predominant land use classifications in the Everglades Agricultural Area (EAA). However, since overland flow is not simulated in the EAA (refer to Section 3.2), the coefficients corresponding to these land use types are not used in the model. The Lower East Coast (LEC) protective levees separate the Everglades Protection Areas

(EPAs) from the LEC Service Areas and act as barriers to sheet flow from the natural areas (west of the levees) to the urban areas (east of the levees); refer also to Sections 3.4 and 3.5.

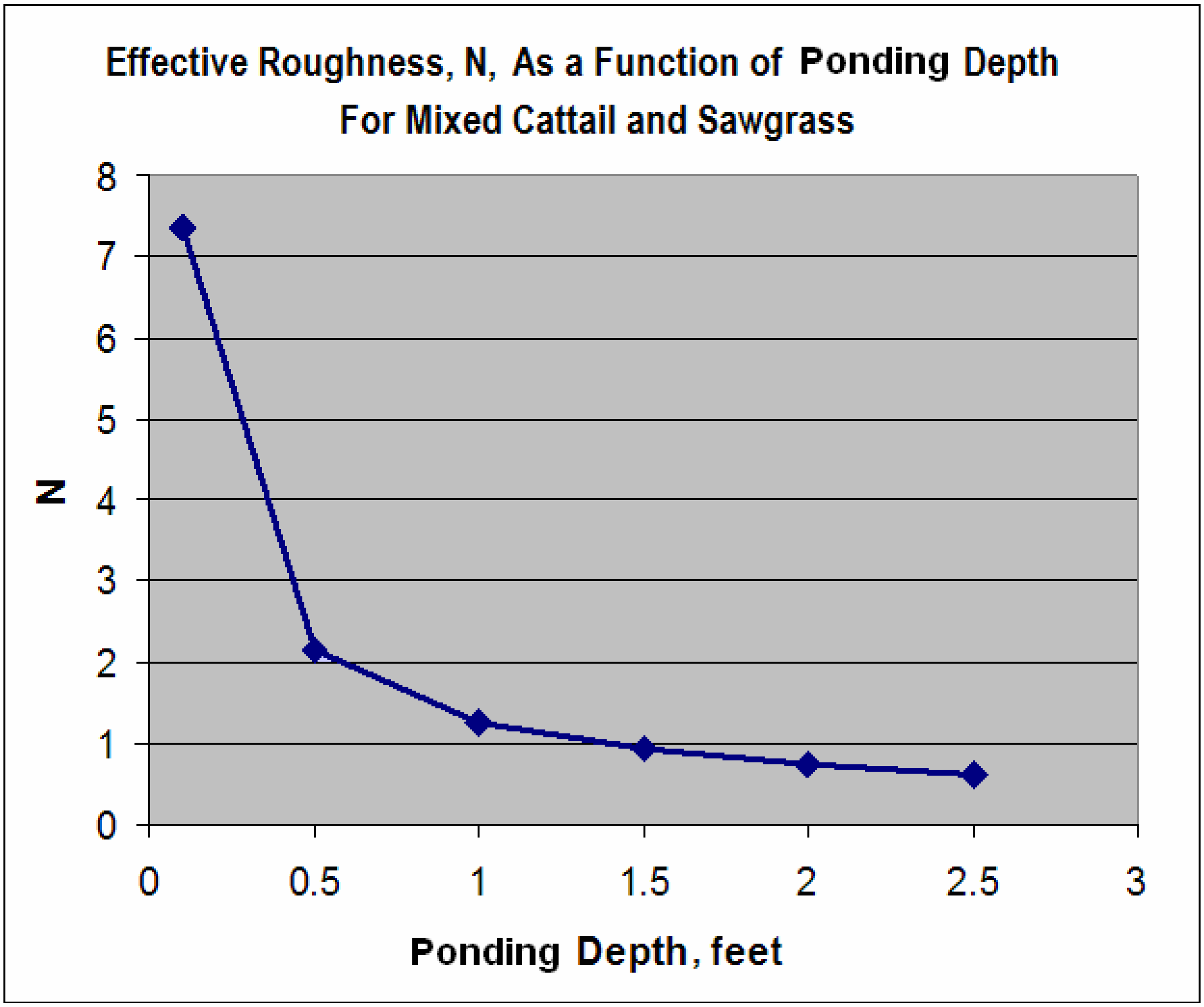

Figure 2.4.2.2 Effective Roughness (N) Values for Mixed Cattail and Sawgrass

Table 2.4.2.1 Overland Flow Coefficients for Effective Roughness as Used in the South Florida Water Management Model (cell-to-cell overland flow)

	Land Use/Description	Effective Roughness, N		Detention Depth (DETEN, ft)	N Value at 2 ft Depth
		A	b		
1	Urban/low density	0.20	0.00	0.60	0.20
2	Agriculture/citrus	0.23	0.00	0.50	0.23
3	Wetland/freshwater marsh	1.10	-0.77	0.10	0.65
4	Wetland/sawgrass plains	1.25	-0.77	0.10	0.73
5	Wetland/wet prairie	0.75	-0.77	0.10	0.44
6	Rangeland/shrubland (scrub and shrub)	1.05	-0.77	0.10	0.62
7	Agriculture/row (or truck) crops	0.23	0.00	0.50	0.23
8	Agriculture/sugar cane	0.23	0.00	0.10	0.23
9	Agriculture/irrigated pasture	0.23	0.00	0.45	0.23
10	Wetland/stormwater treatment area and above-ground reservoir	1.15	-0.77	0.10	0.67
11	Urban/high density	0.08	0.00	0.50	0.08
12	Forest/forested wetlands	0.35	-0.77	0.10	0.21
13	Forest/mangroves	0.55	-0.77	0.15	0.32
14	Forest/melaleuca	0.45	-0.77	0.12	0.26
15	Wetland/cattail	1.20	-0.77	0.11	0.70
16	Forest/forested uplands	0.85	0.00	0.10	0.85
17	Wetland/Ridge & Slough I	0.765	-0.77	0.10	0.45
18	Wetland/marl prairie	0.615	-0.77	0.10	0.36
19	Wetland/mixed cattail / sawgrass	1.25	-0.77	0.11	0.73
20	Water/open water (deep excavated reservoirs)	0.01	0.00	0.10	0.01
21	Wetland/Ridge & Slough II	0.765	-0.77	0.11	0.45
22	Wetland/Ridge & Slough III	0.825	-0.77	0.11	0.48
23	Wetland/Ridge & Slough IV	0.895	-0.77	0.12	0.52
24	Wetland/Ridge & Slough V	1.15	-0.77	0.12	0.67
25	Urban/medium density urban	0.14	0.00	0.55	0.14

Effective roughness, $N = A(h)^b$ where h is ponded depth

2.5 SUBSURFACE FLOW

Subsurface flow in the South Florida Water Management Model (SFWMM) can be divided into four processes: infiltration and percolation, canal seepage, levee seepage and groundwater flow. Infiltration refers to the vertical movement of water across the land surface and percolation is the recharge to the water table. Canal-groundwater seepage describes the movement of canal water into the adjacent soil (and vice-versa) by virtue of the differences between the hydraulic head in the canal and that of the water table. Levee seepage is a process wherein surface water moves across a levee embankment and ends up on a levee borrow canal (e.g., from WCA-3B to L-30 borrow canal). Regional groundwater flow (or simply groundwater flow) corresponds to the horizontal movement of groundwater after all of the above processes have occurred. The following five subsections describe these processes in greater detail.

2.5.1 Infiltration and Percolation

Infiltration is the process by which water on the soil surface enters the soil. Water may come from rainfall and/or irrigation and increases moisture in the unsaturated zone or directly goes to the saturated zone via percolation. Percolation is the recharge to the saturated zone or the amount of water crossing the water table. In South Florida, where unconfined aquifer conditions exist, the location of the water table determines the upper limit of the saturated zone. Ponding exists when the water table elevation exceeds the land surface elevation and the unsaturated zone no longer exists. Infiltration and percolation are assumed to be vertical processes.

The volume of infiltration is taken as the minimum of the following three quantities:

1. available water (above land surface) to infiltrate;
2. infiltration rate multiplied by grid cell area and time step; and
3. available void space between the water table and land surface.

Infiltration rates vary from grid cell to grid cell and range from a value of 9 to 100 ft/day.

Percolation is the amount of water that enters the saturated zone when field capacity (maximum moisture content that can be stored in the unsaturated zone) is exceeded.

2.5.2 Canal-Groundwater Seepage

The interaction of canals with the water table can be modeled by quantifying the exchange of surface water (in the canal) and groundwater (in the aquifer). Although generally referred to as canal seepage, leakance or leakage, water can actually leave and enter a canal depending on the relative stages of the local groundwater and the canal itself, hence the term canal-groundwater seepage or canal-aquifer interaction. Seepage is added (or subtracted) from the recharge term which goes into the solution of the groundwater flow equations [Equation (2.5.4.1)]. The volume of seepage into or out of the canal to or from the aquifer is calculated at each node where the canal passes through for every time step. Canal-groundwater seepage is given by:

$$CGSEEP_{node,t+1} = \left(H_{node,t} - SWL_{node,t}\right)\left(CHHC_{node}\right)\left(DT\right)\left(1.4\right)\left(RCAR_{node}\right) \qquad (2.5.2.1)$$

where:

$CGSEEP_{node,\,t+1}$ = seepage volume [ft^3];

$H_{node,\,t}$ = water level at the node or grid cell through which the canal passes [ft];

$SWL_{node,\,t}$ = canal surface water level at the same nodal location as H_{node} [ft];

$CHHC_{node}$ = canal-aquifer conductivity or connectivity coefficient [ft/day per foot of head difference];

DT = length of one time step [day]; and

$RCAR_{node}$ = length of canal within the node multiplied by the width of the canal [ft^2].

Since RCAR represents the area of the canal bottom, it is necessary to multiply it by a factor of 1.4 in order to approximate the entire bed or wetted area of the canal at the particular node in question (i.e., channel bottom plus side slopes). Seepage is assumed to occur uniformly within the wetted area of the canal. By SFWMM convention, seepage volume is positive if there is inflow to the canal and negative, otherwise. Variable CHHC ranges from 0.01 to 9.00.

2.5.3 Levee Seepage

Levee seepage refers to the movement of groundwater beneath and through a levee, and into the corresponding levee borrow canal or vice versa. Investigations conducted by the Corps and the United States geological Survey (USGS) indicated that significant amounts of seepage occur from the Water Conservation Areas (WCAs) across the major levees to the east.

Figure 2.5.3.1 shows the SFWMM representation of the total groundwater flow beneath a levee. It is the sum of the regional groundwater flow or underseepage (Q_{US}) and levee seepage (Q_{LS}). Prior to version 2.1, groundwater flow was completely characterized in the SFWMM by numerically solving the governing partial differential equation (PDE) for transient flow in a two-dimensional, isotropic, heterogeneous, unconfined aquifer. However, the level of discretization (2 miles x 2 miles) available in the model was considered too coarse for modeling local groundwater phenomena, such as levee seepage. The model's solution to the general groundwater flow equations represents regional groundwater flow while an empirical levee seepage equation is used to solve for levee seepage. The levee seepage algorithm in the SFWMM affords great flexibility because it does not require mixing spatial resolutions within the grid system used in the model (Brion and Guardo, 1991).

The basis for the empirical equations representing levee seepage in the SFWMM is an independent set of computer simulation runs using a two-dimensional (vertical plane) model, SEEP2D (a.k.a. SEEPN). Developed at the U.S. Army Corps of Engineers Waterways Experiment Station, SEEP2D simulates steady-state subsurface flow through a multi-layered aquifer system (confined or unconfined) by solving the Laplace equation using Darcy's Law (Tracy, 1983; Biedenharn and Tracy, 1987).

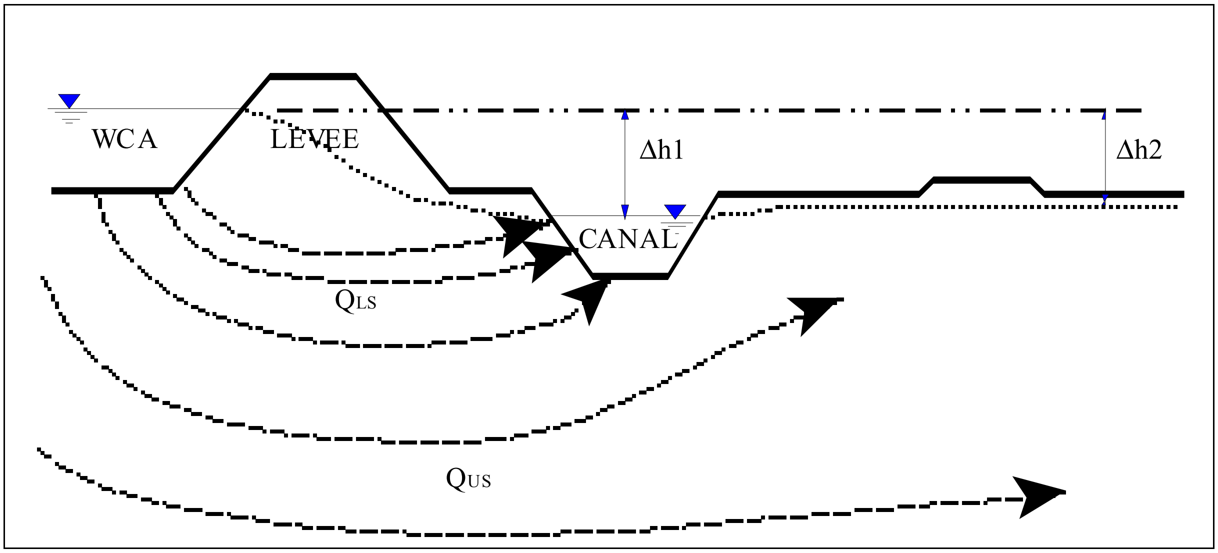

Figure 2.5.3.1 Canal-Levee Configuration Representing a Typical Transect Used in Developing Empirical Levee Seepage Equations in the South Florida Water Management Model

A concise description of the steps taken in establishing the preliminary empirical levee seepage equations is outlined below:

1. Create a 2-D strip model for each selected (based on similar hydrogeologic characteristics) levee cross-section. Based on Corps general and detailed design memoranda, levee configurations and hydrogeologic properties were compiled and reformatted in accordance with requirements of the SEEPN model. The locations of the transects (along L40, L36, L35B, L35, L37, L67, L33, L29, L31N and C-111 levees) used in this analysis are shown in Figure 2.5.3.2.
2. Run SEEPN for different combinations of hydraulic heads and canal stages. Stages in the water conservation areas, borrow canals and areas just east of these canals, that were deemed representative of steady-state conditions (wet, dry, and average) for all transects, were selected as input to the SEEPN model. Model output was summarized to determine the capture rate (amount of total seepage beneath a levee that ends up in the borrow canal) for each model run.
3. Propose empirical equations and derive regression coefficients for the equation relating volume of water captured by borrow canal to total head gradient immediately across the levee (local head gradient) and from cell-to-cell (regional head gradient). Consistent with Darcy's Law, the independent variables in the functional form of the regression equation were chosen as head gradients, instead of absolute stages, since hydraulic gradients are the fundamental physical parameter that determine movement of water in a porous media such as levees.
4. Incorporate regression equations in the SFWMM. This step involves the creation of a function which calculates [Equation (2.5.3.1)] seepage volume from a grid cell to a canal located in an adjacent cell along the alignment of the north-south protective levee.

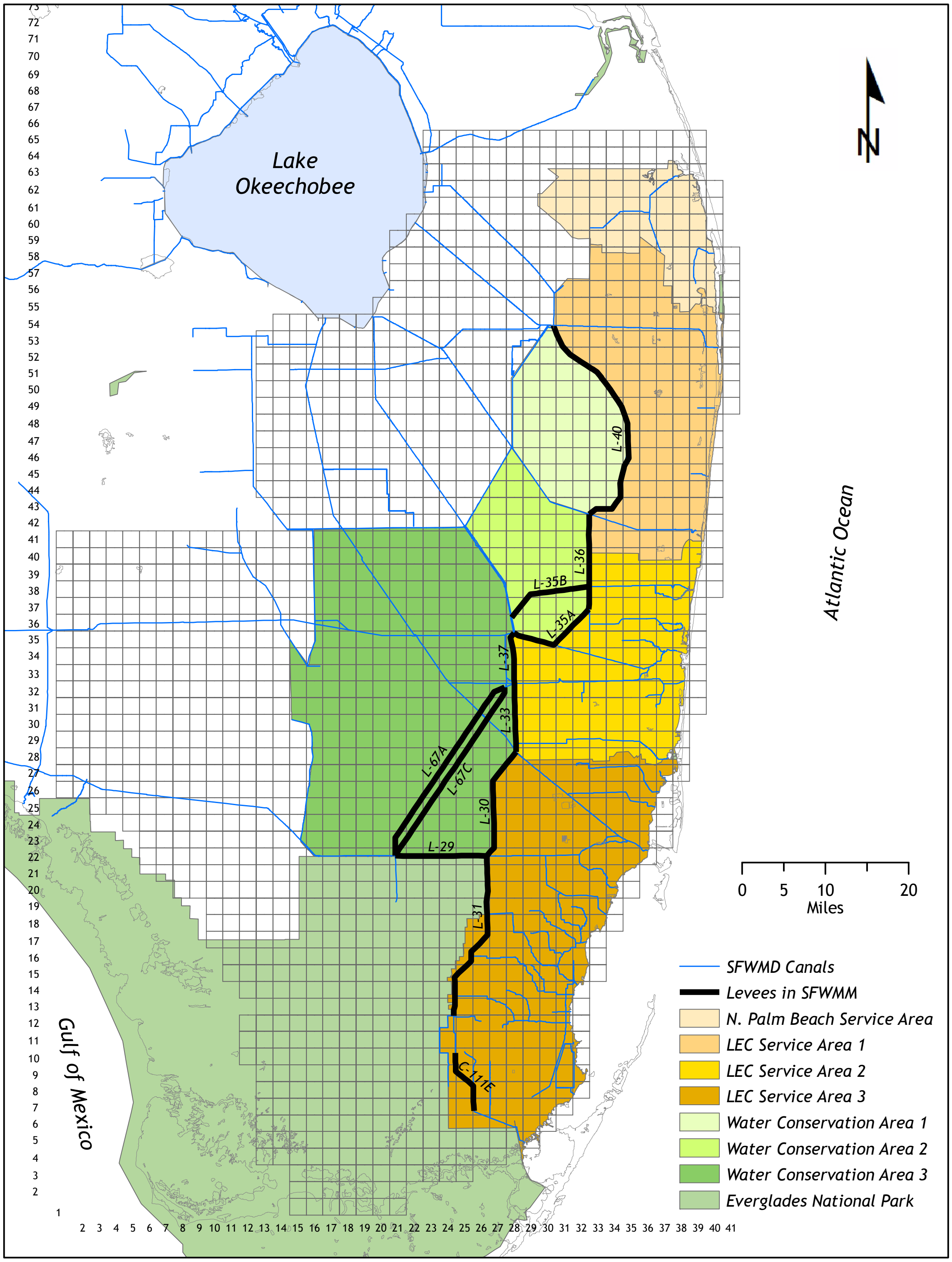

Figure 2.5.3.2 Sections or Transects Across the Major Levees Used to Formulate Levee Seepage Equations in the South Florida Water Management Model

Using step-wise linear regression analysis, sixteen regression equations were established, one for each levee, relating levee seepage and prevailing head gradients. All equations were of the form:

$$Q_{seep} = \beta_0 + \beta_1 \Delta h_1 + \beta_2 \Delta h_2 \tag{2.5.3.1}$$

where:

Q_{seep} = unit levee seepage [cfs/mi];
β_0, β_1, β_2 = regression or levee seepage coefficients;
Δh_1 = head gradient across a levee representing the difference in the water levels inside a water conservation area and a levee borrow canal (local head gradient) [ft]; and
Δh_2 = head gradient across a levee representing the difference in the water levels on opposite sides of a levee borrow canal (regional head gradient) [ft].

During the regression analysis, several cross-sections were found to produce very similar coefficients such that some of them were eventually grouped together and the analysis redone. Regression coefficients derived from this analysis were later referred to as levee seepage coefficients. Table 2.5.3.1 lists the levee seepage coefficients used in the model. These coefficients were optimized during model calibration. Negative values resulting from the use of these regression equations are zeroed out in the model and occur due to the fact that these equations are valid only for certain ranges of head gradients. In addition to the input parameters in Table 2.5.3.1, the user can specify the fraction of levee seepage rate to be applied (srate_frac) and the maximum levee seepage rate (rate_limit).

Table 2.5.3.1 Levee Seepage Coefficients (β_0, β_1, β_2) Used in the South Florida Water Management Model

Levee	β_0	β_1	β_2
L-40E	1.3	-0.7	0.1
L-36N	-1.0	1.8	2.0
L-36S	1.0	-0.2	-0.2
L-35N	2.38	-0.2	0.6
L-35	2.38	-0.2	0.6
L-67AC	134.9	-128.3	-1.8
L-33	13.5	-8.7	0.5
L-30	175.0	-95.6	-8.4
L-29	0.0	0.0	0.0
L-31N	0.0	0.0	0.0
L-31	91.0	-77.9	2.0
C111E	125.0	-77.9	2.0
WPBCT	4.80	0.0	0.0
L-38E	2.0	0.0	0.0
ACMEB	2.0	0.0	0.0
INTRL	4.0	0.0	0.0

Note: In the levee_spg_input.dat file, the coefficient terms are denoted as c_1, c_2, and c_3.

2.5.4 Groundwater Flow

Governing Equations. Regional groundwater flow (or simply groundwater flow) in the SFWMM involves the solution of the partial differential equation (PDE) describing transient flow in a two-dimensional, anisotropic, heterogeneous, unconfined aquifer. The PDE is of the form:

$$\frac{\partial}{\partial x}\left(T\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(T\frac{\partial h}{\partial y}\right)=S\frac{\partial h}{\partial t}-R \qquad (2.5.4.1)$$

where:

x and y = Cartesian coordinates aligned along the major axes of hydraulic conductivity or transmissivity;

T = transmissivity of the aquifer [ft^2 /day];

h = the unknown hydraulic or potentiometric head [ft];

S = unconfined aquifer storage coefficient or specific yield of the porous media; vertically- averaged specific storage; volume of water released or taken into storage per unit cross-sectional area per unit change in the hydraulic head in the

aquifer [dimensionless];
R = recharge; volumetric flux per unit surface area [ft/day]; and
t = time [day].

Equation (2.5.4.1) is strictly valid for confined aquifers only, but is used in the model by allowing T to vary with time as saturated zone thickness changes (Wang and Anderson, 1982) since transmissivity is the product of hydraulic conductivity (assumed to be time-invariant in the model) and aquifer saturated thickness whose value varies as the location of the water table changes from one time step to the next. As mentioned in Section 1.3, one of the unique features of the model domain is a highly permeable surficial aquifer. Assuming full saturation, the variation of transmissivity within the system can be shown using a contour map (Figure 2.5.4.1). The derivation of the governing equation makes the following assumptions:

1. Flow is essentially two-dimensional such that transmissivity, storage coefficient, recharge and hydraulic head can be vertically averaged.
2. The fluid (water) is incompressible.
3. Hydraulic conductivity, as well as transmissivity, is symmetric and the axes can be rotated such that the off-diagonal terms in the tensor are zero. In other words, the coordinate axes are assumed to be aligned with the major trends controlling hydraulic conductivity, in which, for example, flow in the x-direction is a result of the hydraulic gradient only in the x-direction.
4. The momentum equation for an isotropic medium is based on Darcy's Law which relates flow rate to an energy loss gradient by the hydraulic conductivity - Darcy's proportionality constant.
5. Drawdown or water table gradients are small relative to the saturated thickness.

Since the saturated thickness (b) is a function of hydraulic head (h) at any given time, the two-dimensional groundwater flow equation (2.5.4.1) is a nonlinear PDE. It is sometimes called the diffusion equation because it can be derived by performing a mass balance (continuity) and momentum balance to describe the flow in a porous media.

In mathematical terms, Equation (2.5.4.1) is classified as a parabolic PDE that can be solved using a variety of numerical techniques. The SFWMM uses a variation of the Saul'yev method to solve the PDE given boundary and initial conditions (Saul'yev, 1964). The technique is unconditionally stable and explicit (direct) such that no iteration is required within a single time step. Initially, the region to be modeled is subdivided into a block-centered grid network with square and regular grid cells (i.e., $\Delta x = \Delta y =$ constant). The PDE is then transformed to its finite difference approximation: a system of linear algebraic equations written for each grid cell in the network. Lastly, the system of equations is solved sequentially until all nodal heads (average groundwater levels at grid cells) are determined.

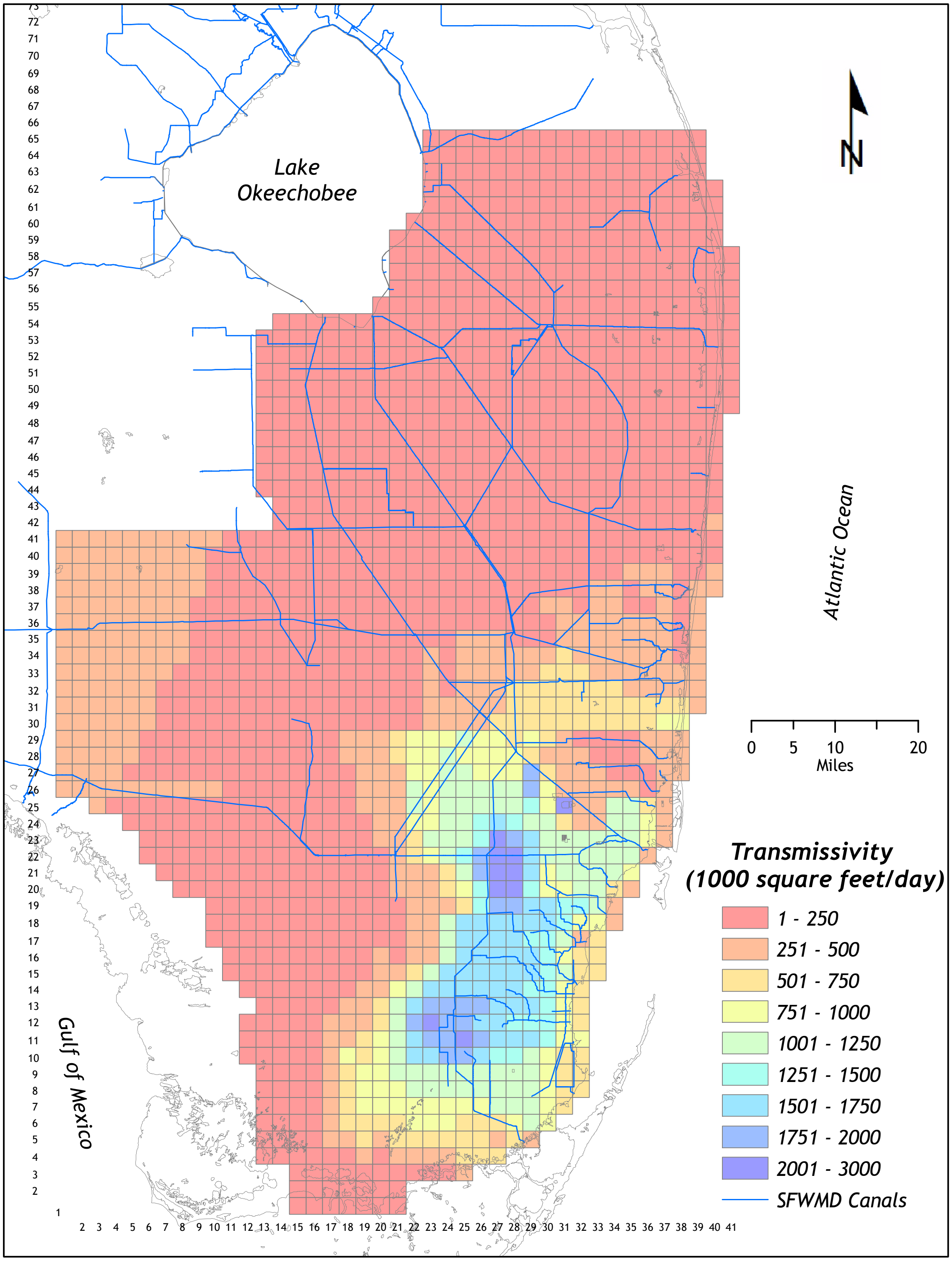

Figure 2.5.4.1 Surficial Aquifer Transmissivity Map for the South Florida Water Management Model (v5.5)

2.5.5 Model Implementation

In order to minimize bias (or error propagation), the system of linear algebraic equations is solved in four different directions in four successive time steps. Unlike the overland flow subroutine, no time slicing is performed in the groundwater flow subroutine. A complete pass of all grid cells in the model domain is accomplished by one of the following directions:

1. left-to-right starting with the southwestern corner cell of the model domain, proceeding from the bottom row to the top row;
2. right-to-left starting with the northeastern corner cell of the model domain, proceeding from the top to the bottom row;
3. bottom-to-top starting with the southwestern corner cell of the model domain, proceeding from the left column to the right column; or
4. top-to-bottom starting with the northeastern corner cell of the model domain, proceeding from the right column to the left column.

Thus, four passes, one of each in the above sequence, through the grid network takes four time steps. Using the Saul'yev method, the finite difference approximation of Equation (2.5.4.1) is varied slightly depending on the direction by which the solution to the PDE is carried out. A basic derivation follows.

Consider the first term in Equation (2.5.4.1). By taking the centered difference at the computational grid cell denoted by reference node (i,j) in terms of the midpoints and using a 0.5Δx spacing, we obtain the following:

$$\frac{\partial}{\partial x}\left(T\frac{\partial h}{\partial x}\right) \cong \frac{1}{\Delta x}\left[\left(T\frac{\partial h}{\partial x}\right)_{i+1/2,j} - \left(T\frac{\partial h}{\partial x}\right)_{i-1/2,j}\right] \qquad (2.5.5.1)$$

Expanding $\frac{\partial h}{\partial x}$ in both terms on the right side yields the following:

$$\left[\frac{\partial}{\partial x}\left(T\frac{\partial h}{\partial x}\right)\right]_{i,j} \cong \frac{1}{\Delta x}\left[T_{i+1/2,j}\left(\frac{h_{i+1,j}-h_{i,j}}{\Delta x}\right) - T_{i-1/2,j}\left(\frac{h_{i,j}-h_{i-1,j}}{\Delta x}\right)\right] \qquad (2.5.5.2)$$

where:

$T_{i+1/2,j}$ = transmissivity between node (i,j) and node (i+1,j); and
$T_{i-1/2,j}$ = transmissivity between node (i-1,j) and node (i,j).

The transmissivity terms can be evaluated at the midpoints of the grid cells. Three commonly used approximations are the arithmetic mean, geometric mean and harmonic mean, all of which produce satisfactory results for most groundwater flow problems (Willis and Yeh, 1987). Average conductivities or transmissivities in the horizontal direction are typically obtained using arithmetic means while those in the vertical direction are obtained using harmonic means. If the spatial distribution of the permeability follows a log-normal distribution, the average permeabilities are calculated using geometric means (de Marsily, 1986). Non-directional averages are best estimated with geometric means. In the SFWMM, transmissivities are evaluated as arithmetic averages of transmissivities from adjacent nodes such that $T_{i+1/2,j}$ = 0.5 ($T_{i+1,j} + T_{i,j}$) and $T_{i-1/2,j}$ = 0.5 ($T_{i,j} + T_{i-1,j}$). If we let T_{x1} = (1/Δx) ($T_{i-1/2,j}$) and T_{x2} = (1/Δx) ($T_{i+1/2,j}$), equality (2.5.5.2) can be simplified into:

$$\left[\frac{\partial}{\partial x}\left(T\frac{\partial h}{\partial x}\right)\right]_{i,j} \cong T_{x1}\left[\frac{h_{i-1,j}-h_{i,j}}{(\Delta x)^2}\right]+T_{x2}\left[\frac{h_{i+1,j}-h_{i,j}}{(\Delta x)^2}\right] \tag{2.5.5.3}$$

Using the same procedure for the y-derivative at node (i,j), we obtain:

$$\left[\frac{\partial}{\partial y}\left(T\frac{\partial h}{\partial y}\right)\right]_{i,j} \cong T_{y1}\left[\frac{h_{i,j-1}-h_{i,j}}{(\Delta y)^2}\right]+T_{y2}\left[\frac{h_{i,j+1}-h_{i,j}}{(\Delta y)^2}\right] \tag{2.5.5.4}$$

where:

$T_{y1} = (1/\Delta y)\,(T_{i,j-1/2})$ and $T_{y2} = (1/\Delta y)\,(T_{i,j+1/2})$.

Next, the forward difference approximation of $\frac{\partial h}{\partial t}$ relative to time t at the same reference node (i,j) is:

$$\left[\frac{\partial h}{\partial t}\right]_{i,j}^{t} \cong \frac{h_{i,j}^{t+1}-h_{i,j}^{t}}{\Delta t} \tag{2.5.5.5}$$

By evaluating all space derivatives in terms of time step t, a simple explicit formulation of PDE (2.5.4.1) results. However, some combinations of Δx and Δt in such a formulation result in numerical errors that could accumulate from one time step to the next, i.e., an explicit formulation is only conditionally stable. The Saul'yev method uses the computational efficiency of an explicit scheme while maintaining stability. This method takes advantage of the direction of calculations in order to produce an explicit scheme based on an implicit formulation. For directions **1.** and **3.**, the solution proceeds in the +x and +y directions. Using Equations (2.5.5.3) through (2.5.5.5), the system of linear algebraic equations approximating Equation (2.5.4.1) takes the form:

$$T_{x1}\left[\frac{h_{i-1,j}^{t+1}-h_{i,j}^{t+1}}{(\Delta x)^2}\right]+T_{x2}\left[\frac{h_{i+1,j}^{t}-h_{i,j}^{t}}{(\Delta x)^2}\right]+T_{y1}\left[\frac{h_{i,j-1}^{t+1}-h_{i,j}^{t+1}}{(\Delta y)^2}\right]+T_{y2}\left[\frac{h_{i,j+1}^{t}-h_{i,j}^{t}}{(\Delta y)^2}\right]$$

$$= S_{i,j}\left[\frac{h_{i,j}^{t+1}-h_{i,j}^{t}}{\Delta t}\right]-R_{i,j}^{t+1} \tag{2.5.5.6}$$

where:

$$T_{x1}=\frac{T_{x_{i-1,j}}+T_{x_{i,j}}}{2} \qquad T_{x2}=\frac{T_{x_{i+1,j}}+T_{x_{i,j}}}{2}$$

$$T_{y1}=\frac{T_{y_{i,j-1}}+T_{y_{i,j}}}{2} \qquad T_{y2}=\frac{T_{y_{i,j+1}}+T_{y_{i,j}}}{2} \tag{2.5.5.7}$$

Equation (2.5.5.6) is implicit because $h_{i-1,j}^{t+1}$, $h_{i,j}^{t+1}$, and $h_{i,j-1}^{t+1}$ appear simultaneously in the formulation. However, since the scheme proceeds in the +x and +y directions, then all values to the left (e.g., $h_{i-1,j}^{t+1}$) and below (e.g., $h_{i,j-1}^{t+1}$) the current cell (i,j) are known from a previous calculation during the same time step t+1. The method takes advantage of the direction of calculations in space. Thus, the unknown head $h_{i,j}^{t+1}$ is solved in terms of head values from the previous time step (old heads) and head values from the previous calculations (known or boundary heads) at the nodes surrounding (i,j).

Similarly, for directions **2.** and **4.**, the solution proceeds in the -x and -y directions; $h_{i+1,j}^{t+1}$ and $h_{i,j+1}^{t+1}$ are known from previous calculations, and the corresponding system of linear algebraic equations to be solved is:

$$T_{x1}\left[\frac{h_{i-1,j}^{t}-h_{i,j}^{t}}{(\Delta x)^2}\right]+T_{x2}\left[\frac{h_{i+1,j}^{t+1}-h_{i,j}^{t+1}}{(\Delta x)^2}\right]+T_{y1}\left[\frac{h_{i,j-1}^{t}-h_{i,j}^{t}}{(\Delta y)^2}\right]+T_{y2}\left[\frac{h_{i,j+1}^{t+1}-h_{i,j}^{t+1}}{(\Delta y)^2}\right]$$

$$=S_{i,j}\left[\frac{h_{i,j}^{t+1}-h_{i,j}^{t}}{\Delta t}\right]-R_{i,j}^{t+1} \tag{2.5.5.8}$$

Finally, $h_{i,j}^{t+1}$ can be solved via:

$$h_{i,j}^{t+1}=\frac{D+C-Bh_{i,j}^{t}}{A} \tag{2.5.5.9}$$

where the following applies to directions **1.** and **3.**:

$$A=\frac{T_{x1}}{(\Delta x)^2}+\frac{T_{y1}}{(\Delta y)^2}+\frac{S_{i,j}}{\Delta t} \tag{2.5.5.10}$$

$$B=\left[\frac{T_{x2}}{(\Delta x)^2}+\frac{T_{y2}}{(\Delta y)^2}\right] \tag{2.5.5.11}$$

$$C=\frac{S_{i,j}}{\Delta t}h_{i,j}^{t-1}+R_{i,j}^{t} \tag{2.5.5.12}$$

$$D=\left[\frac{T_{x1}h_{i-1,j}^{t+1}+T_{x2}h_{i+1,j}^{t}}{(\Delta x)^2}\right]+\left[\frac{T_{y1}h_{i,j-1}^{t+1}+T_{y2}h_{i,j+1}^{t}}{(\Delta y)^2}\right] \tag{2.5.5.13}$$

while the following applies to directions **2.** and **4.**:

$$A=\frac{T_{x2}}{(\Delta x)^2}+\frac{T_{y2}}{(\Delta y)^2}+\frac{S_{i,j}}{\Delta t} \tag{2.5.5.14}$$

$$B = \left[\frac{T_{x1}}{(\Delta x)^2} + \frac{T_{y1}}{(\Delta y)^2} \right] \tag{2.5.5.15}$$

$$C = \frac{S_{i,j}}{\Delta t} h_{i,j}^{t-1} + R_{i,j}^{t} \tag{2.5.5.16}$$

$$D = \left[\frac{T_{x1} h_{i-1,j}^{t} + T_{x2} h_{i+1,j}^{t+1}}{(\Delta x)^2} \right] + \left[\frac{T_{y1} h_{i,j-1}^{t} + T_{y2} h_{i,j+1}^{t+1}}{(\Delta y)^2} \right] \tag{2.5.5.17}$$

Figure 2.5.5.1 shows a typical computational grid used in the groundwater flow subroutine. The head at the grid cell denoted by (i,j) at time step t+1 is a function of the head at five adjacent cells, including itself, evaluated at time steps t and t+1. The selection of which cells to evaluate at a particular time step depends on the current direction of calculations.

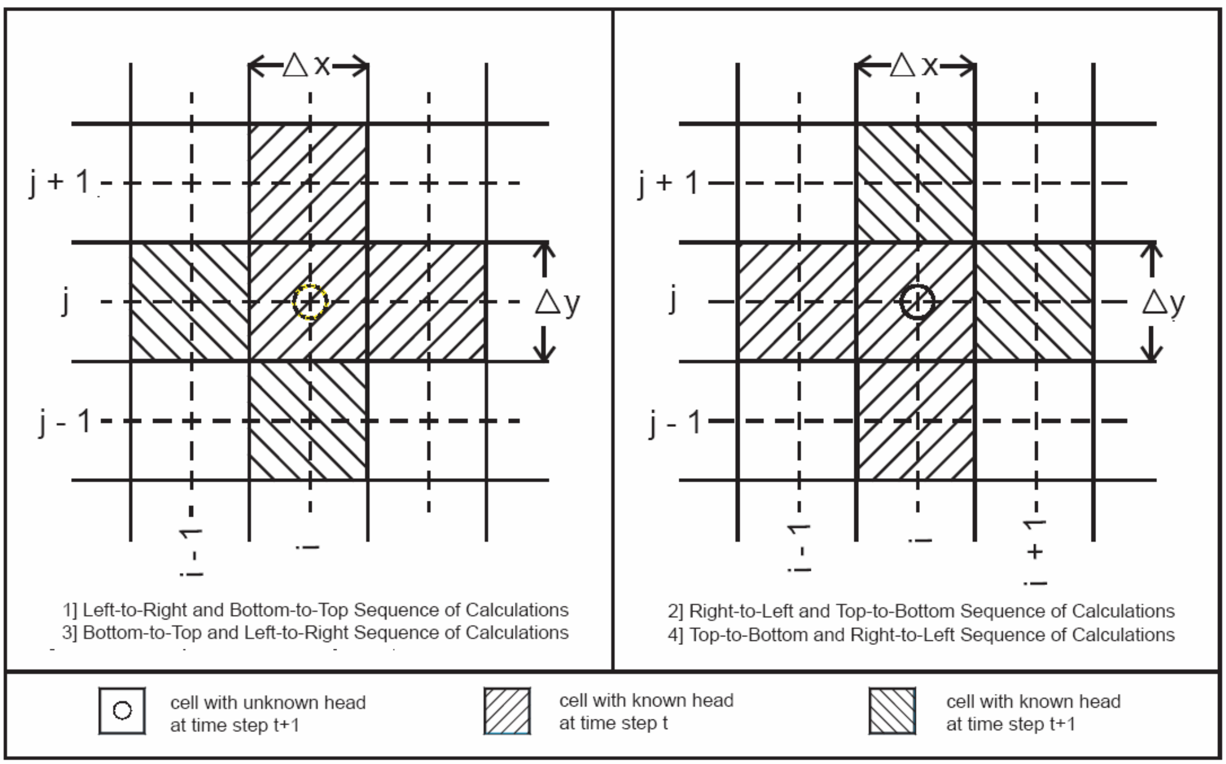

Figure 2.5.5.1 Location of Grid Cells Used in Calculating Total Head at Grid Cell (i,j) During Time Step t+1 as Implemented in the Groundwater Flow Subroutine in the South Florida Water Management Model

Coupling of Groundwater and Surface Water

The solution to the governing groundwater flow equations [Equation (2.5.4.1)] assumes a vertically homogenous (i.e., constant hydraulic properties) soil column and applies only to the saturated portion of the aquifer. The thickness of the saturated zone is assumed to be "unbounded" during the solution of the groundwater flow equations. If the water surface elevation goes above ground level, the assumption of homogeneity is violated; ponded water above land surface and saturated water in the aquifer have different hydraulic properties. The coupling of groundwater and surface water is further complicated by the existence of an intervening zone of aeration (unsaturated zone). The thickness of the unsaturated zone varies as the location of the water table fluctuates from one time step to the next. The model maintains mass balance for the unsaturated zone as its control volume changes with time. However, detailed physical processes such as lateral subsurface flow within this zone and capillary rise from the water table into the root zone are not modeled in the SFWMM.

Part of the algorithm used in the groundwater flow subroutine is the adjustment of the hydraulic or potentiometric heads just before and after the solution to the groundwater flow equations in order to account for differences in aquifer and ponded water hydraulic properties. These adjustments are often done in the wetland areas, e.g. WCAs and ENP, where occasional drying and rewetting of model grid cells occur. In these areas, the SFWMM assumes that the soil column is dry above the water table and below land surface, i.e. the unsaturated zone is assumed nonexistent. In the irrigated areas of the LEC region, an unsaturated zone moisture accounting procedure is performed (refer to Section 3.5). Moisture is assumed to be uniformly distributed within the unsaturated zone and always available for root uptake and plant transpiration.

A brief description of the variable names pertinent to the current discussion is as follows:

- ells = elevation of land surface [ft NGVD];
- h = hydraulic or potentiometric head; elevation of groundwater; location of the water table within the soil column relative to a datum [ft NGVD]. For modeling purposes, this variable has a maximum value of land surface elevation;
- infilt = infiltration; equivalent depth of water crossing land surface; typically water movement from ponding to the unsaturated zone [ft];
- perc = percolation; equivalent depth of water crossing the water table; typically water movement from the unsaturated zone to the saturated zone [ft];
- pond = depth of ponding [ft];
- S = storage coefficient for a confined aquifer; equivalent to the specific yield for an unconfined aquifer or the fraction (by volume) of water in a soil column released from (or gained into) storage per unit area of aquifer per unit decline [or increase]; in head [dimensionless];
- solmc = soil moisture content in the unsaturated zone [ft];
- t = time step [day]; and
- whc = water holding capacity in the unsaturated zone, [dimensionless]; equivalent to field capacity or the drained upper limit or the fraction (by volume) of water in a soil column above which water will percolate past the root zone and into the saturated zone.

Prior to the solution of the groundwater flow equations, if ponding exists, the hydraulic heads over the groundwater are reset to include the additional head provided by the ponded water:

$$h_t = h_t + pond_t \tag{2.5.5.18}$$

A residual ponding term, whose value is equal to (1.0 - S) (pond), is assumed not to take part in the solution to the groundwater flow equations. It is, however, added back to the computed heads in order to maintain mass balance for each computational cell. The SFWMM also assumes that moisture in the unsaturated zone will not affect the solution to the groundwater flow equations. Moisture in this zone is updated at the end of the calculations if the computed heads encroach upon the unsaturated zone.

If the computed head, h_{t+1}, goes above land surface, the final ponding depth is updated to include residual ponding and/or unsaturated zone moisture content.

$$pond_{t+1} = (h_{t+1} - ells)\ S + (1.0 - S)\ (pond_t) + solmc_t \tag{2.5.5.19}$$

The final head is equal to land surface elevation, i.e., h_{t+1} = ells.

If the computed head goes below land surface, residual ponding and unsaturated zone moisture are added back to the aquifer. Ponding depths and final heads are updated appropriately if the combined effects of residual ponding and unsaturated zone moisture content are to saturate the entire soil column. Otherwise, the final ponding depth becomes zero and the unsaturated zone mass balance is performed. For accounting purposes, additional infiltration and percolation will occur if residual ponding exists and percolation will increase if the water table encroaches upon the unsaturated zone to an extent that will bring the moisture content in this zone at field capacity.

One of the strengths of the SFWMM is its ability to simultaneously describe the state of the surface water and groundwater systems within the model domain. This state is defined in terms of ponding depths, unsaturated zone water content, and groundwater levels. The formulation of the recharge term (the combined effect of percolation, evapotranspiration, canal-groundwater seepage, and aquifer withdrawal for domestic, industrial and irrigation purposes) in Equation (2.5.4.1), levee seepage, and the procedure outlined in the preceding discussions comprise the vertical coupling of groundwater and surface water in the model.

Figure 2.5.5.2 shows a block diagram of the physical processes simulated in the model for surface and subsurface systems. Rainfall is a process that moves water from the atmosphere into surface storage. Evapotranspiration is the movement of water from both surface and subsurface systems into the atmosphere. A canal, which is essentially a special form of surface storage, exchanges water with ponding and the saturated zone storage through runoff/overbank flow and canal-groundwater seepage, respectively. Lastly, levee seepage is a localized flow phenomenon that describes the movement of water from the aquifer across a major levee and into a borrow canal.

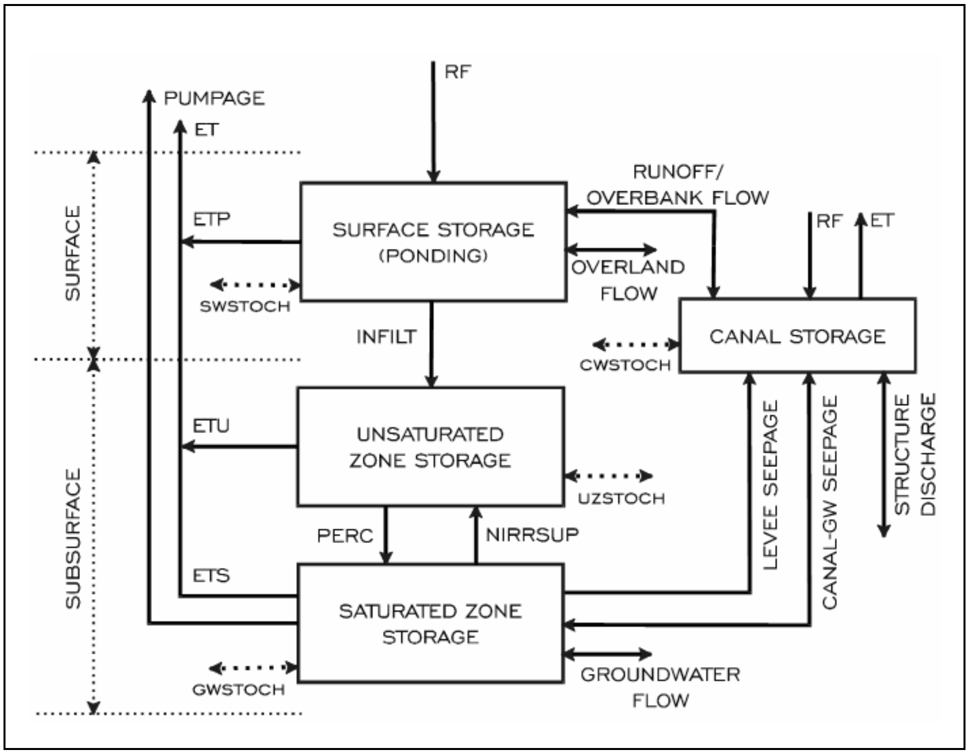

Figure 2.5.5.2 Generalized Block Diagram of Surface-Subsurface Interaction in the South Florida Water Management Model

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2.6 CANAL ROUTING

2.6.1 Basic Principles

Canal or channel flow routing in the South Florida Water Management Model (SFWMM) uses a mass balance approach to account for any changes in storage within a canal reach given beginning-of-day canal stage, canal and structure properties, and calculated or specified inflows and outflows. The mass balance is performed every time step (1 day) for each canal reach and involves grid cells through which each canal reach passes. The SFWMM assumes that the width of a canal is constant along its entire length. The model includes the ability to assume either a constant wedge-shaped longitudinal canal profile or a dynamic (daily) wedge-shaped longitudinal canal profile. In both cases, the approximating channels are assumed to be rectangular with a linear slope. In the case of the dynamic slope, the slope in the canal is calculated on a daily basis. The two cases will be discussed later in this section [refer to Section 2.6.2 Profile Slopes in Canals].

The components of the canal water budget are rainfall, evapotranspiration (ET), overland flow (cell-to-canal or canal-to-cell), canal seepage, and structure inflows and outflows. Because some of these components are functions of canal stage, an iterative procedure is used to calculate the end-of-day canal stage.

Rainfall into a canal reach varies by grid cell. The volume of precipitation within a canal segment is equal to the depth of rainfall assigned to a particular grid cell multiplied by the surface area of the canal reach located within the grid cell. Evaporation depth within the canal segment is equal to the product of reference crop ET rate and open-water coefficient (KMAX) assigned to the grid cell. A canal segment is that portion of a canal reach that falls entirely within a grid cell while a canal reach is a series of canal segments bounded by the canal's primary inlet and outlet structures.

Canals also interact with freewater or ponded water within the grid cell. In contrast to cell-to-cell overland flow, this interaction provides a means for the model to direct runoff from individual grid cells into canals or to account for overbank flow. Runoff enters the canal as lateral sheetflow, and in situations where excessive canal stages occur, water overtops its banks and becomes part of ponded water. The same approach used to model the resistance to flow for overland flow (refer to Section 2.4) is used to calculate the exchange of canal water with ponded water. Assuming that the canal bisects a grid cell into two 1-mile by 2-mile strips, the slope of the energy grade line, which runs perpendicular to either side of the canal, is assumed to be equal to the ponding depth (not the difference between the stage in the canal and the average stage in the grid cell) divided by one-half of the short side of either strip. This rough approximation, which is equal to one-fourth the length of one side of a grid cell (or 0.5 mile) yields satisfactory results. As in the case for cell-to-cell overland flow, the effective roughness coefficient (N) in cell-to-canal (or vice versa) overland flow is expressed as a function of ponding depth at the grid cell where a canal segment is located. N varies considerably with vegetation or land use type. Based on these flow characteristics, different values of parameter A and b can be used as in Equation 2.4.2.8. N may also vary as a function of channel properties such as sediment distribution and riverbank irregularities. This level of detail is not accounted for in the model.

The SFWMM v5.5 has the capability to specify overland flow coefficients which are unique to a specific canal and override those specified in Table 2.6.1.1 which are based on grid cell land use.

Table 2.6.1.1 Values of Parameter A and b Used to Define Effective Roughness Coefficient (N) for Cell-to-Canal or Canal-to-Cell Overland Flow in the South Florida Water Management Model

	Land Use/Description	**A** **Cell-to-canal**	**b** **Cell-to-canal**	**A** **Canal-to-cell**	**Threshold Ponding Depth for SW-Canal Interaction to Start**
1	Urban/low density	0.50	0.0	0.25	0.50
2	Agriculture/citrus	0.50	0.0	0.30	0.45
3	Wetland/freshwater marsh	2.00	-0.77	0.30	0.20
4	Wetland/sawgrass plains	2.00	-0.77	1.10	0.25
5	Wetland/wet prairie	2.00	-0.77	1.20	0.25
6	Rangeland/shrubland (scrub and shrub)	2.00	-0.77	1.50	0.25
7	Agriculture/row (or truck) crops	0.50	0.0	0.30	0.45
8	Agriculture/sugar cane	0.50	0.0	0.25	0.09
9	Agriculture/irrigated pasture	0.35	0.0	0.25	0.40
10	Wetland/stormwater treatment area and above-ground reservoir	2.00	-0.77	1.30	0.20
11	Urban/high density	0.25	0.0	0.20	0.45
12	Forest/forested wetlands	2.00	-0.77	0.50	0.10
13	Forest/mangroves	2.00	-0.77	1.00	0.10
14	Forest/melaleuca	2.00	-0.77	0.60	0.25
15	Wetland/cattail	2.00	-0.77	1.00	0.25
16	Forest/forested uplands	1.50	0.0	1.00	0.25
17	Wetland/Ridge & Slough I	2.00	-0.77	1.00	0.25
18	Wetland/marl prairie	2.00	-0.77	0.75	0.25
19	Wetland/mixed cattail / sawgrass	2.00	-0.77	1.00	0.25
20	Water/open water (deep excavated reservoirs)	0.01	0.0	0.01	0.00
21	Wetland/Ridge & Slough II	2.00	-0.77	0.80	0.10
22	Wetland/Ridge & Slough III	2.00	-0.77	1.50	0.25
23	Wetland/Ridge & Slough IV	2.00	-0.77	1.50	0.25
24	Wetland/Ridge & Slough V	2.00	-0.77	1.50	0.25
25	Urban/medium density urban	0.45	0.0	0.25	0.48

Note: The b values for canal-to-cell flow are zero.

Land use types 7, 8 and 9 are the three predominant land use classifications in the EAA. Since overland flow is not simulated in the EAA (refer to Section 3.2), the coefficients corresponding to these land use types are not used in the model.

Canal seepage describes the interaction of canals with the water table (refer to Section 2.5). The operations of structures are site-specific and are discussed, as necessary, throughout this documentation.

2.6.2 Profile Slopes in Canals

The most common assumption in the model for a canal water surface slope is a constant longitudinal profile such that a constant offset or head drop (HDC) occurs along the entire length of each canal (Figure 2.6.2.1). In South Florida, both the beginning and end stage of the canals are often monitored and a representative slope can be determined from observed conditions. The constant offset can be considered as a pre-defined slope in the hydraulic grade line that represents the average or long-term difference between the stage in the canal at its upstream end and at its downstream end. For a given constant slope canal, HDC is specified as two values, one for the dry season and one for the wet season. This slope remains unchanged from one time-step to the next (except for the change from season to season) and is independent of the discharge in the channel. When a canal reach spans more than one model grid cell, the total head drop over the canal reach is assumed to be evenly distributed across the cells (i.e. change in head from one segment to another is uniform).

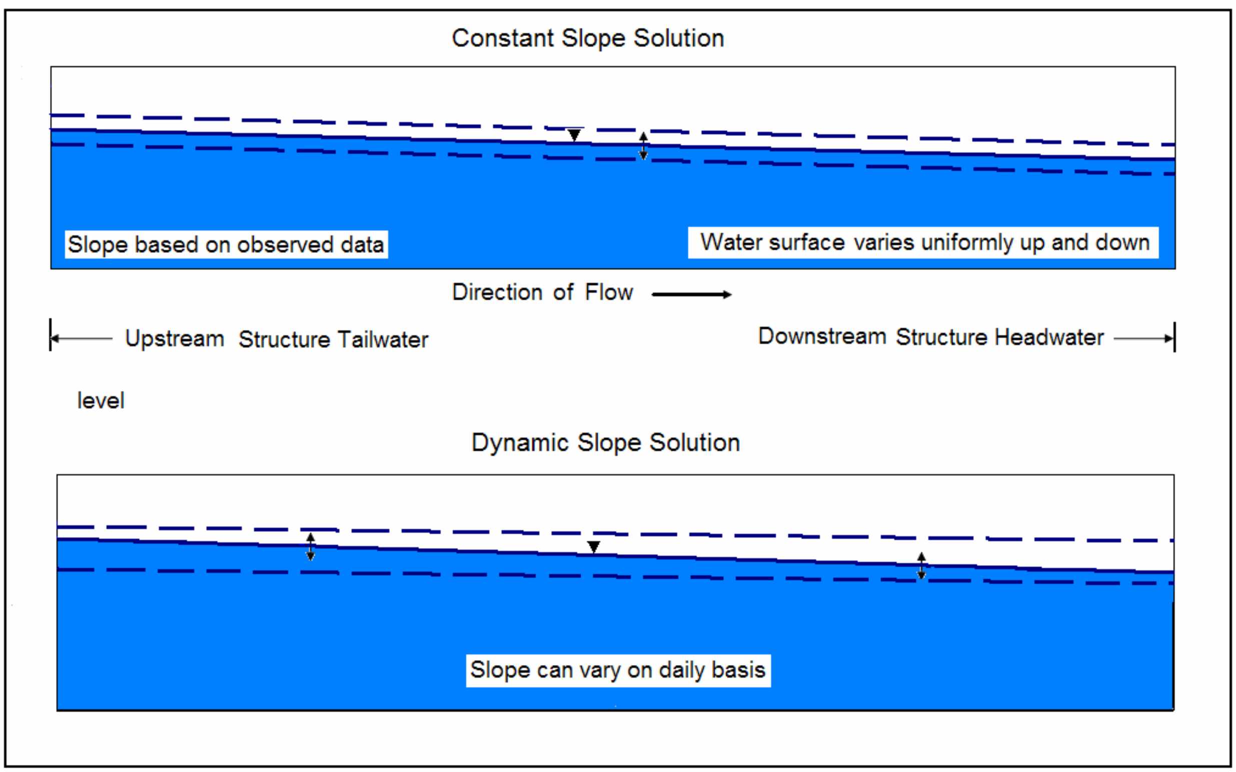

Figure 2.6.2.1 Canal Profiles Showing Head Drop

In some of the LEC canals where sufficient calibration data exists, a dynamic longitudinal profile is calculated such that a time varying offset or head drop occurs over the length of the canal reach (Figure 2.6.2.1). Alternately stated, this means the total HDC varies from one time-step to the next. For any given time step, head drop from segment to segment within the reach still remains uniform. The targeted canals are mostly in the LEC, urban, more-developed areas that are less affected by ponded cell water than canals in other parts of the system.

In calculating dynamic canal slopes, the effects of both inflow and outflow (and where it occurs along the canal) are considered as part of the calculation of head drop. In order to apply Manning's equation when calculating slope, one representative effective flow value must be derived for the entire reach. To calculate this flow term, the SFWMM uses a simplified weighting scheme in which all of the in-line and lateral inflows and outflows are considered. This methodology is graphically illustrated in Figure 2.6.2.2. First, the canal is divided in half and then the accounting of flows is completed for both segments by applying Equation (2.6.2.1). The most downstream segment is considered to be the first segment. The sign conventions of inflows and outflows play an important role in determining the canal slope. The upstream in-line and lateral flows coming in and the downstream in-line and lateral flow going out have a positive sign meaning that they would contribute to increasing the slope.

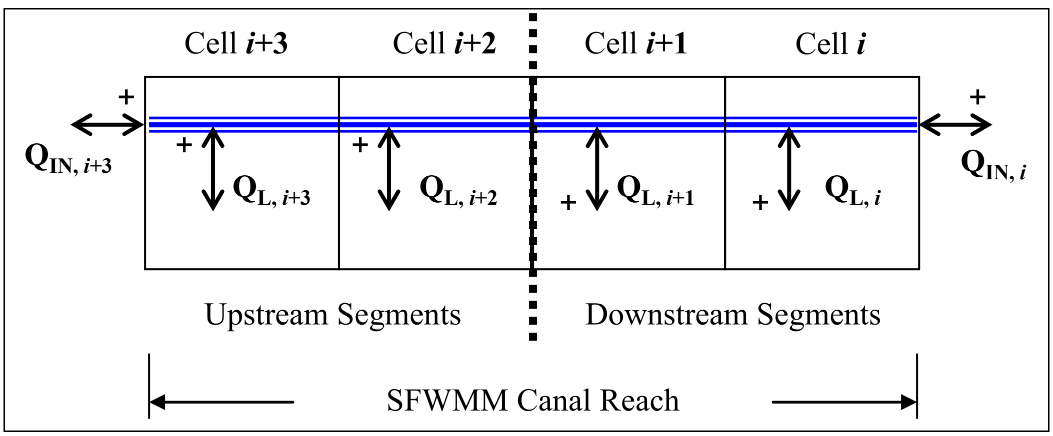

Figure 2.6.2.2 Conceptual Diagram for Calculating Flow in Determination of Slope for Dynamically Dimensioned Canal Reaches

$$Q_{EFF} = (Q_{IN,n} + Q_{IN,1})/2 + (\sum_{1}^{n/2} Q_L)/4 + (\sum_{n/2+1}^{n} Q_L)/4 \tag{2.6.2.1}$$

where:

Q_{EFF} = Effective flow to be used in determination of canal slope;
n = Number of segments forming the canal reach;
Q_{IN} = Net in-line structural flow term calculated at most upstream and downstream segments; and
Q_L = Net lateral flow term calculated at all canal segments. This term is made up of overland and groundwater (seepage) flows as well as structure flows.

Because the head-drop calculation is part of the iteration of the canal solution, a limit is specified on both the maximum and minimum head-drop (based on historical values) so the solution for flow and stage does not iterate beyond normal operating limits. The dynamic canal slope method has limited applicability because it does not work well in areas where: (1) it is difficult to quantify the inflows and outflows; and (2) where ponding occurs in the canal most of the time (e.g. in WCAs).

In addition to the dynamic slopes, another major influence on how canals respond to different flow situations is linked to actual observed structure operations. At each structure a single set of on/off triggers is normally relied upon for operations, but lower gate settings may be used in certain high flow events that are triggered by local rainfall accumulation. Input to the model allows canals to respond to different rainfall thresholds. If the 14-day average rainfall exceeds a threshold input, then the gate would be lowered by a slight deviation from the normal operations. If the 14-day average rainfall exceeds a second threshold input, then the gate would be lowered by an additional amount. These short term operational deviations help modeled canal profiles to respond in a manner similar to that of observed field data.

2.6.3 Canal Water Budget

The beginning-of-day stage CHDEP0 at the most downstream node of a canal reach is set equal to the stage at the end of the previous time step. The canal stage at the most upstream node of the same canal reach is set equal to the stage at the most downstream node plus offset HDC. Canals defined at intermediate nodes are assumed to have stages proportional to their relative distances from the extreme nodes of the reach.

The initial estimate of the end-of-day or equilibrium stage ($CHDEP^0$) at the downstream node is assumed to be equal to beginning-of-day stage. Initial change in storage $CHSTOR^0$ is, therefore, zero. Rainfall and evapotranspiration are calculated for each canal segment using methods described above. Discharge at the downstream structure (typically a weir or pump) is calculated as a function of its headwater (= $CHDEP^0$). Discharges elsewhere within the reach (other outlet structures, canal seepage and overland flow) are either prescribed (e.g., historical) or calculated as a function of $CHDEP^0$ adjusted for their location relative to the most downstream node of the reach and the slope of the assumed constant or dynamic hydraulic grade line HDC. In dynamic slope canals, the previous end-of-day HDC is assumed as the initial estimate of slope prior to iteration.

The net inflow or accumulation $ACVOL^0$ is calculated using the following:

$$ACVOL^0 = Q_{in} - Q_{out} \tag{2.6.3.1}$$

where:

$$Q_{in} = RF + OVLNF_{in} + SEEP_{in} + QSTR_{in}; \text{ and}$$
$$Q_{out} = ET + OVLNF_{out} + SEEP_{out} + QSTR_{out}.$$

It should be noted that $OVLNF_{in}$, $OVLNF_{out}$, $SEEP_{in}$, and $SEEP_{out}$ are functions of the assumed end-of-day stage. Therefore, they are implicit functions of the unknown stage. $QSTR_{in}$ and $QSTR_{out}$ may or may not be implicit functions. Solving for the change in storage based on beginning- and assumed end-of-day stages yields the following:

$$CHSTOR^0 = [(CHDEP^0 + HDC^0/2) - (CHDEP0 + HDC0/2)] (CAREA) \quad (2.6.3.2)$$

where CAREA is the surface area of the canal reach equal to the product of the width and the length of the canal reach. Note that in constant slope canals, where HDC^0 equals HDC0, slope cancels out of the change in storage calculation.

By definition, canal water budget indicates that the change in storage $CHSTOR^i$ must be equal to the net inflow or accumulation $ACVOL^i$, where i denotes the i th iteration within the same time step. The difference is the estimation error given by:

$$ERROR^0 = CHSTOR^0 - ACVOL^0 \quad (2.6.3.3)$$

Eliminating this error is the objective in establishing a canal water budget. The objective is met by iteratively assuming the end-of-day stage. A positive error implies that the assumed end-of-day stage is overestimated. In order for the canal to experience a change in storage $CHSTOR^0$ due to $CHDEP^0$ more inflow (or less outflow) should have resulted from the same $CHDEP^0$. Thus, if $CHSTOR^0 > ACVOL^0$, the new estimate at the downstream stage is made lower than the previous estimate, $CHDEP^1 < CHDEP^0$. Conversely, if $CHSTOR^0 < ACVOL^0$ the new estimate is raised, $CHDEP^1 > CHDEP^0$. $CHDEP^0$ is incremented (decremented) to $CHDEP^1$ based on the magnitude of the error, an initial increment value (INC^0) of 1.0 ft is used. Therefore:

$$CHSTOR^1 = CHSTOR^0 + INC^0 \quad (2.6.3.4)$$

The calculations enter an iteration loop where Equations 2.6.3.4 and 2.6.3.1 through 2.6.3.3 are solved and a stopping criterion is tested. If the value of ERROR changes sign, the magnitude of the increment in stage is halved to prevent oscillation between successive stage estimates. (The number of iterations is the number of times the assumed equilibrium stage is updated.) The iteration loop is terminated when either of the following stopping criteria is met: (a) the absolute value of ERROR/CAREA becomes less than the convergence value (0.01 ft); or (b) the maximum allowable number of iterations (110) has been reached. In general, constant slope canals are able to converge more quickly than dynamic slope canals due to the above mentioned simplification of Equation 2.6.3.2. In the event that a dynamic slope canal is unable to converge within forty iterations, downstream segment stage is fixed (this term habitually becomes stable by this number of iterations) and slope is determined using a standard-step iterative approach based on the last two HDC values as calculated by the Manning's approximation. This method determines the final HDC subject to the above mentioned global constraints.

The above calculations are performed for all canal reaches in the model domain except for those in the EAA. Conveyance considerations in the major EAA canals are discussed in Section 3.2.

2.7 INITIAL AND BOUNDARY CONDITIONS

2.7.1 Initial Conditions

Initial conditions throughout the system are prescribed in the form of stages. For Lake Okeechobee, historical stage is used at the start of a simulation. For grid cells or nodes, initialization water levels are interpolated from observed historical data at each grid cell using a local averaging method on a basin-by-basin basis. The stage at a grid cell is the distance-weighted average of all observed data falling within a ring with whose inner radius is the distance to the nearest neighbor and thickness equal to half the grid cell diagonal. This method assures exact interpolation at the gage locations, provides zones with similar stage values, and provides smooth transition between these zones. For canals, the initial water level is assumed to be equal to the maintenance level or the historical headwater level at the downstream structure. The choice between the two options does not really make any difference as far as inferences drawn from the model output since the model is intended to be run on a long-term (several years) basis.

2.7.2 Boundary Conditions for Lake Okeechobee

Boundary conditions refer to the time series of flows or stages at the peripheral grid cells of the model. For the SFWMM, boundary conditions are applied to both the lumped representation of Lake Okeechobee and to the distributed system of grid cells forming the majority of the model domain. This section will describe the boundary conditions applied to Lake Okeechobee as seen in Figure 2.7.2.1, while Section 2.7.3 will detail the assumptions related to boundary conditions in the gridded portions of the model.

Kissimmee River Basin

Kissimmee River Basin inflow enters the north-central region of Lake Okeechobee through the S-65E structure (see Figure 2.7.2.1). The contributing basin includes the Upper Kissimmee, which covers a chain of nine managed lakes (Lakes Alligator, Myrtle, Hart, Gentry, East Tohopekaliga, Tohopekaliga, Cypress, Hatchineha and Kissimmee), and the Lower Kissimmee, which encompasses both canalized and restored reaches of the Kissimmee River. The flows through S-65E represent about 25 percent of the total Lake Okeechobee inflow.

In order to account for inflow contributions from this basin to Lake Okeechobee, a daily time-series of flows at S-65E is input into the SFWMM. This time-series is the aggregation of two independently determined values: 1) the discharge at the S-65 structure which represents the outflow for the entire Upper Kissimmee Chain of Lakes and 2) the runoff contribution provided by the Lower Kissimmee between the S-65 and S-65E structures. The means of developing these terms are discussed hereafter in greater detail.

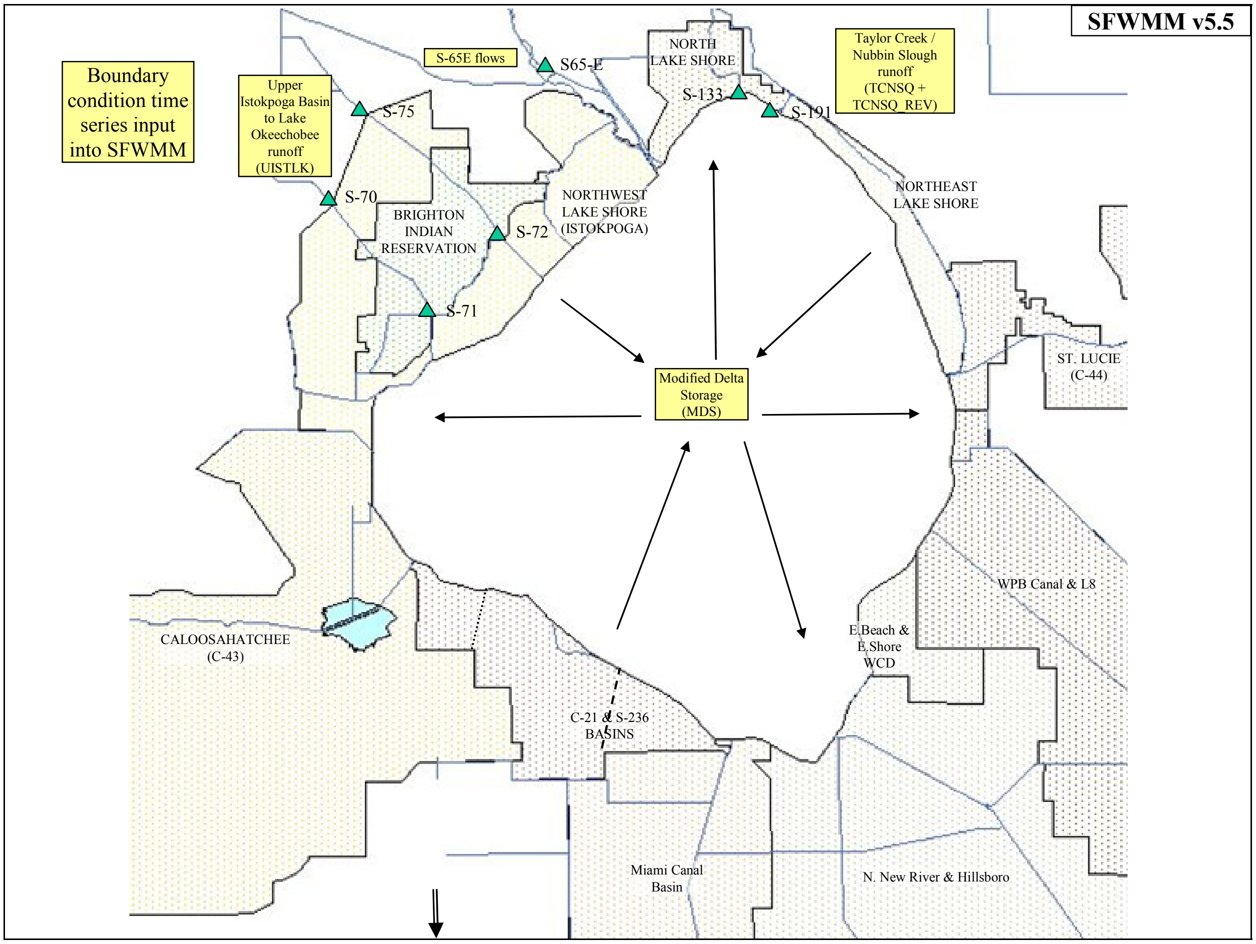

Figure 2.7.2.1 Lake Okeechobee Boundary Conditions, South Florida Water Management Model v5.5

The Upper Kissimmee Chain of Lakes Routing Model (UKISS) computer model was developed to simulate the operation of the Upper Kissimmee Basin (Figure 2.7.2.2). The model serves as a management tool to predict the lake conditions so that alternative management schemes, aimed at achieving specific objectives, can be evaluated. A Technical Memorandum for the UKISS model is provided in Appendix N. The primary output from this model from the perspective of the SFWMM is a daily time series of simulated flow at the S-65 structure.

The major assumptions and limitations of the model are presented below:

1. A primary assumption of the routing model is that level pool conditions exist. The assumption is valid as long as the flow through the lake is small relative to the storage. The assumption is reasonable under normal flow conditions but is slightly violated under heavy discharge conditions.
2. The model simulates the management of the system according to a set of management rules. These rules are expressed in regulation schedules, gate operation criteria, and established rules governing the operation of the structures. As long as the operation follows the established rules, the simulation of the management is possible. Under unusual conditions, the operation may differ from the established rules and thus explains the inability of the routing model to simulate those events.

3. The model runs in daily time steps and generates daily average flows and stages. The time step resolution is adequate for most applications except for extreme storm events where instantaneous peak stages and flows are important. Nevertheless, an examination of the recorded lake hydrographs suggests that, due to the large size of the lakes, the instantaneous stages are not significantly different from the daily averages. The errors introduced are probably small in comparison to random fluctuation of the lake stages due to wind effects and other disturbances.
4. For certain applications where only the management variables change, historical rainfall and inflow data are used. The implicit assumption is that a change in the management will not change the historical hydrologic variables.

Runoff in the Lower Kissimmee Basin is based on historical or adjusted hydrological conditions for the period of simulation. Observed runoff is computed as the difference of historical flow measurements for S-65E and S-65 flows. This calculated difference can then be added to the simulated S-65 discharges from the UKISS model to develop a time series of flow at S-65E. In some simulations, regression modeling techniques are used to adjust the historical runoff in order to account for changes in system management or climatologic conditions.

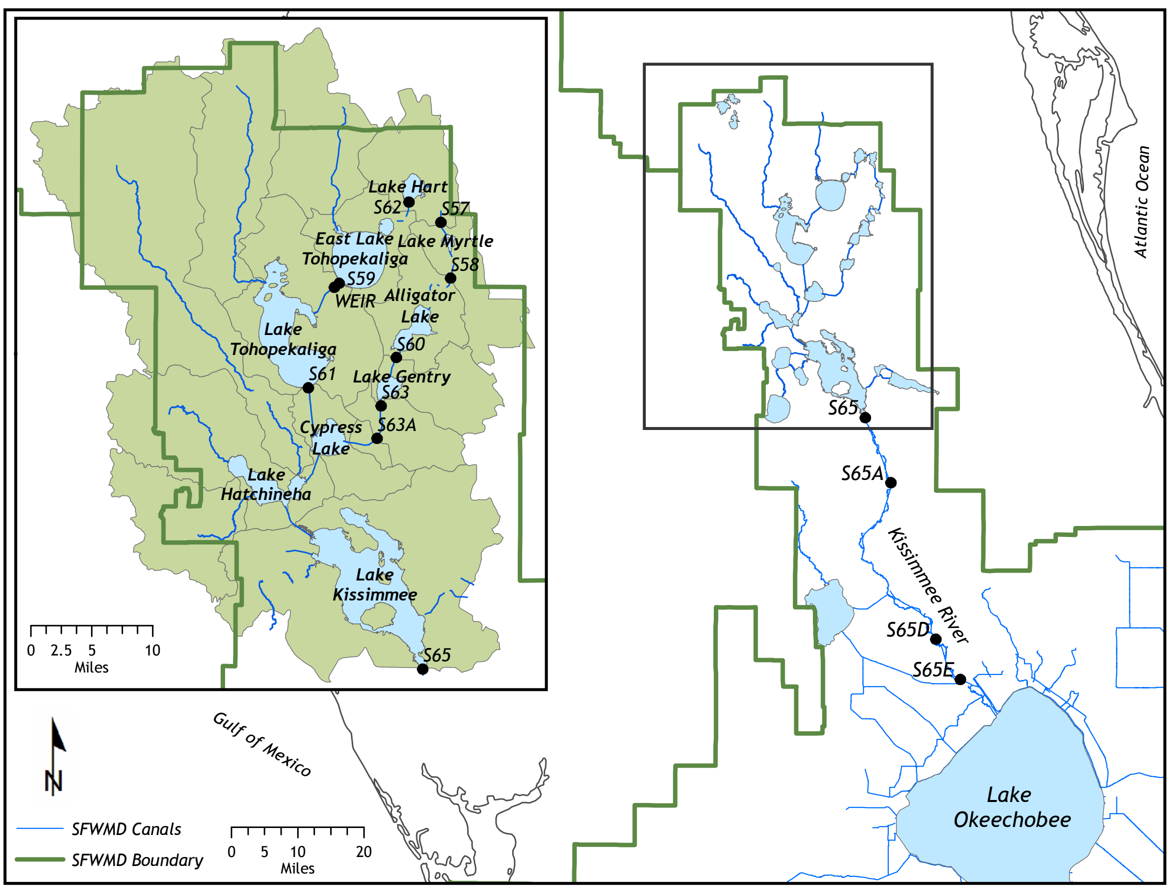

Figure 2.7.2.2 Upper Kissimmee River Basin

Upper Istokpoga Basin

The Upper Istokpoga Basin (above S70 and S75) contributes runoff into Lake Okeechobee via S70/S75 through S71/S72. This volume is made up of primarily upper basin runoff from both irrigated and non-irrigated lands in conjunction with some contribution from flood control releases out of Lake Istokpoga. In order to quantify the historical contribution of the Upper Istokpoga Basin to LOK term in the SFWMM, historical flow data for the S70, S71, S72, and S75 structures were collected. To account for lag effects between releases at the upstream structures of S70/S75 and releases at S71/S72, a monthly volumetric analysis was performed. Upper Istokpoga to Lake Okeechobee (UISTLK) contribution was quantified as the minimum of monthly combined S70/S75 and monthly combined S71/S72 flows. This calculation is sufficient to capture the flow-through contribution from the upper basin to Lake Okechobee. Once the historical monthly volumes were calculated, these volumes were temporally distributed within a given month based on the distribution observed at S71/S72 (flowing to and directly affecting the Lake). Periods of missing data (only observed at S70/S75) in the historical record were patched using a monthly regression dependent on combined current month S71/S72 flow and both current and previous month Lower Istokpoga Basin average rainfall. Regression results are presented in Figure 2.7.2.3.

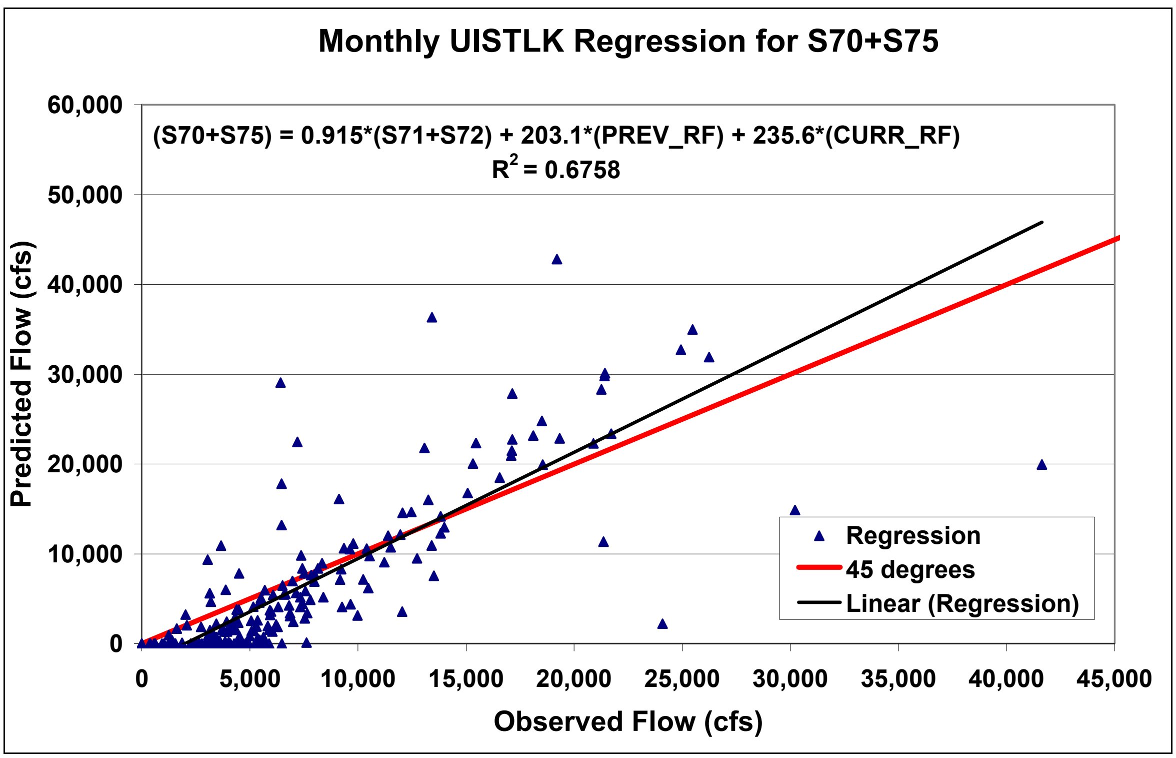

Figure 2.7.2.3 Monthly Upper Istokpoga to Lake Okeechobee Flow Regression Analysis

Taylor Creek/Nubbin Slough

The Taylor Creek/Nubbin Slough (TCNSQ) inflow term is calculated as the sum of historically observed flow at S133 and S191. In order to patch missing periods of data in the 1965-2000 period of record, a two-level analysis was performed. First, a monthly volumetric regression analysis was performed correlating TCNSQ flow to S-65E flow and both current and previous month Taylor Creek/Nubbin Slough/S133 Basin average rainfall. Once the historical monthly volumes were calculated, these volumes were temporally distributed within a given month based on a daily regression model utilizing moving averages of S-65E flow and independent average rainfall from the Taylor Creek, Nubbin Slough and S133 Basins. These moving averages were selected based on the expected response time associated with each element of the regression model (e.g. rainfall from more upstream basins would have a longer moving average than a more downstream basin). Regression results for the monthly and daily regressions are presented in Figure 2.7.2.4 and Figure 2.7.2.5, respectively. While there is not a very high correlation in the daily regression model and it tends to over-predict low flow events and under-predict high flow events, this is acceptable since its purpose is only to distribute within the volumes predicted by the more reliable monthly regression model.

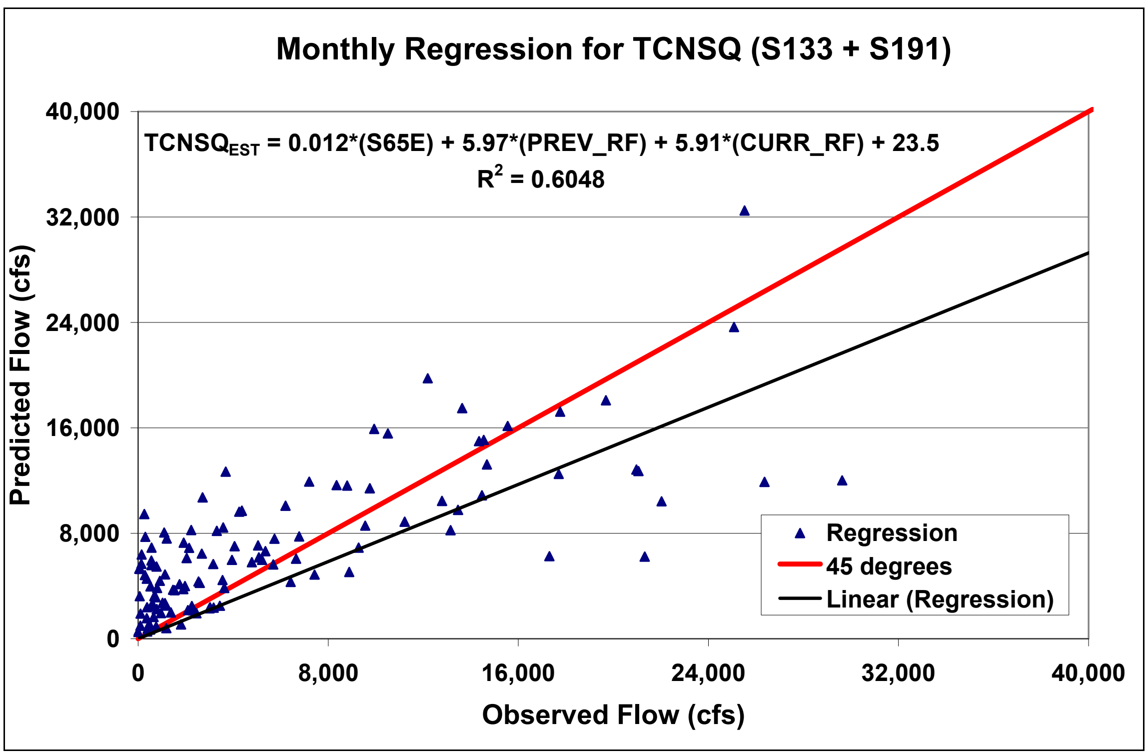

Figure 2.7.2.4 Monthly Taylor Creek/Nubbin Slough Flow Regression Analysis

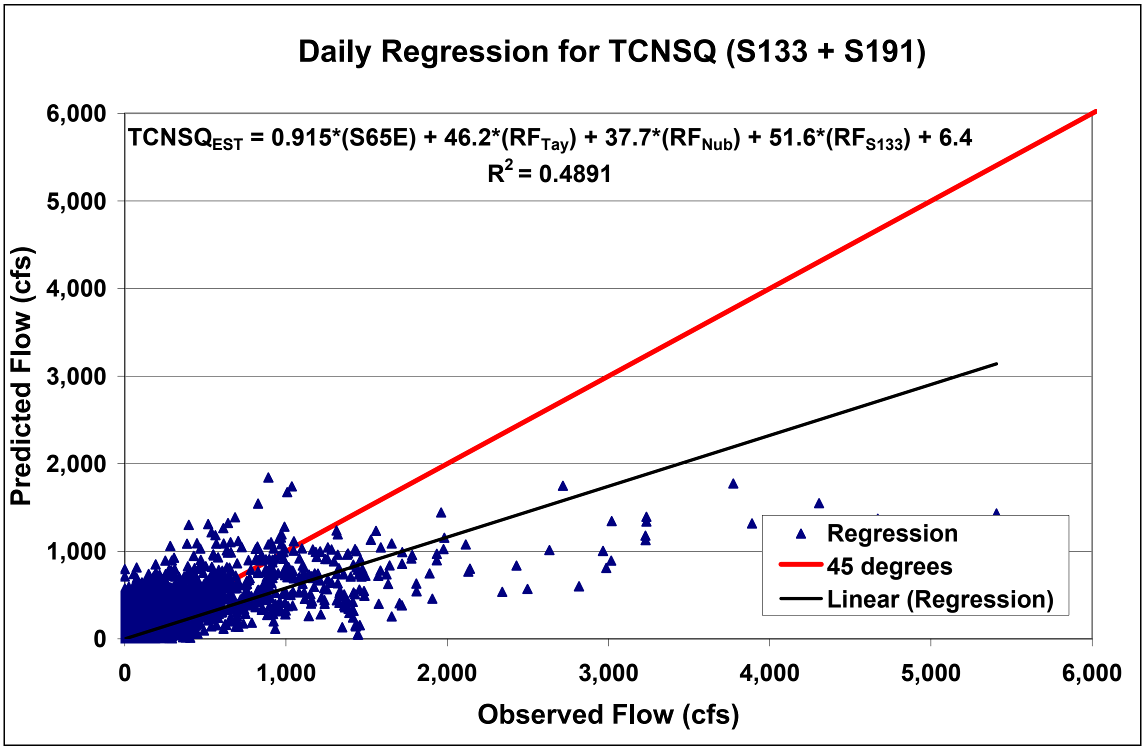

Figure 2.7.2.5 Daily Taylor Creek/Nubbin Slough Flow Regression Analysis

Lake Okeechobee Modified-Delta-Storage

One of the primary difficulties facing any tool attempting to simulate Lake Okeechobee is the challenge of representing changes in stage and corresponding storage in the absence of known or measured influence across many of the boundary conditions along the Lake perimeter. Historical data is scarce or non-existent for several structures or flow ways that represent parts of the Lake water budget. For example, runoff from Fisheating Creek is the second largest single tributary (next to Kissimmee Basin inflows) into the Lake. One difficulty in estimating inflows at this location is that measured flows are only available at the Palmdale monitoring station which is located on the upper Fisheating Creek Basin, several miles upstream of the confluence of the creek to the Lake. In order to address the inherent difficulty of accounting for flows (like Fisheating Creek) without appropriate historical boundary data, the SFWMM simulates Lake Okeechobee (as a water budget approach outlined in Section 3.1) utilizing a modified-delta-storage methodology (Trimble, 1986). A brief discussion of the approach follows, however a more detailed explanation was provided in an earlier document (SFWMD, 1999).

The modified-delta-storage (MDS) term represents the arithmetic sum of all Lake historical water budget components that: 1) are not accounted for in another simulated term on Lake Okeechobee and 2) are assumed not to change from what happened historically [Equation (2.7.2.1)]. The MDS term is calculated as follows:

$$MDS = RF^{hist} + qin^{hist} - qout^{hist} \qquad (2.7.2.1)$$

where:

q = total structural flow aggregated over the current time step; and
RF = rainfall volume over the current time step.

Due to the data issues already identified, it is easier to calculate this term using knowledge of historical daily stage (storage) change and historical flow at structures that will be simulated in the SFWMM at run-time. Net levee seepage and regional groundwater movement in the Lake are assumed to be small relative to the other hydrologic components of the Lake water budget and are, therefore, not considered in the calculation of MDS. By back-calculating the MDS term as in Equation (2.7.2.2) to Equation (2.7.2.4), the historical Lake Okeechobee water budget is preserved. It is possible to begin with the historic water budget definition for the Lake (excluding seepage and regional groundwater movement):

$$delS^{hist} = RF^{hist} + qin^{hist} - qout^{hist} - ET^{hist} \tag{2.7.2.2}$$

where:

$delS = S_{t+1} - S_t$ = change in storage from the current to the next time step; and
ET = evapotranspiration volume over the current time step.

This can be expanded to form the following equation in which some components will not change for any anticipated management/operational scenario to be evaluated in the future (subscript *NC*) and some components will change given the same scenario (subscript *C*):

$$(delS^{hist})_C = [(qin^{hist})_{NC} + (qin^{hist})_C + (RF^{hist})_{NC}] - [(qout^{hist})_{NC} + (qout^{hist})_C + (Et^{hist})_C] \tag{2.7.2.3}$$

Rearranging this equation and substituting Equation 2.7.2.1 gives:

$$(delS^{hist} - qin^{hist} + qout^{hist} + ET^{hist})_C = (RF^{hist} + qin^{hist} - qout^{hist})_{NC} = MDS \tag{2.7.2.4}$$

Note that the equation above illustrates the ability to calculate the MDS term using an aggregation of historically observed Lake storage change, structure flow for stations that will be simulated (subscript *C*) and historical ET measurement. All of these terms can be easily obtained or estimated.

The static nature of components that are retained in the MDS term can be attributed to the following factors:

1. the management/operational scenario being analyzed may not significantly impact, if at all, those particular components;
2. even if they do, the components themselves may be too small in magnitude in comparison with the others such that neglecting them may be a reasonable assumption; and
3. there may be no means of quantifying them within reasonable certainty.

2.7.3 Other Boundary Conditions

The boundary conditions applied to the gridded portions of the SFWMM are graphically illustrated in Figure 2.7.3.1. The general southeasterly direction of both natural (overland and groundwater flow) and man-controlled (structure discharge) flows in South Florida allows the northern boundary condition of the distributed mesh to be defined in terms of historical or independently simulated flows depending on the scenario simulated. On the northeastern boundary along the Martin-Palm Beach County line a no-flow boundary condition is assumed for both overland flow and groundwater flow. A no-flow boundary condition for both surface water and groundwater is imposed on the northwestern and midwestern boundaries except for basins tangent to the SFWMM which provide single point inflows into the model. These basins, collectively known as the Western and Feeder Canal Basins, are presented in additional detail below.

The southwestern portion of the model domain, where the model cuts the western portion of the Everglades National Park (ENP), is defined as a no-flow boundary as far as groundwater movement is concerned. On the surface, a uniform overland flow condition is imposed, as the hydraulic gradient or water surface profile is always assumed to be parallel to land surface. At the later part of this section, the tidal boundary is discussed. Along the eastern seaboard several tidal stations had adequate data available. Along the southern rim of the ENP, tidal boundary data were developed. The tidal boundary data values are passed to the groundwater subroutine as known head boundary conditions.

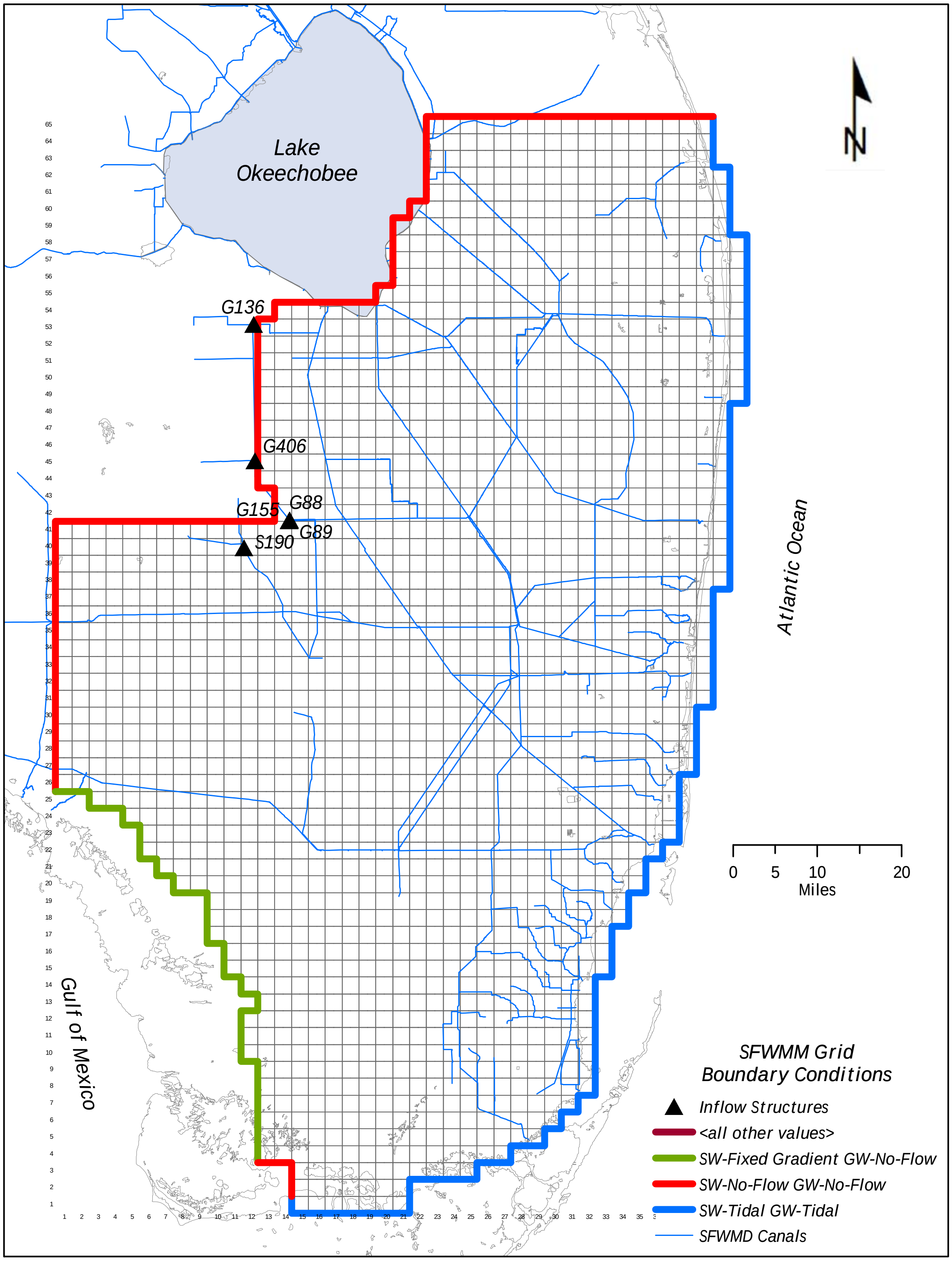

Figure 2.7.3.1 South Florida Water Management Model Gridded Boundary Conditions

Western Boundary Flows at the L-1 and L-3 Canals

This section describes the criteria and procedures followed to obtain Western Boundary flows at the L-1 and L-3 Canals for the SFWMM simulations. These flows are not always intended to reflect historical values, but rather flows that would be obtained if climatological conditions for the period 1965-2000 were repeated, given infrastructure and operations of the system that were in place circa 2000 (Cadavid and Brion, 2002). Flow time series needed as boundary inflows to the SFWMM are given at several different locations (see Figure 2.7.3.2):

- G-136, representing flows from the L-1 Canal and the C-139 Basin. These flows will be directed to the EAA and will not enter STA-5.
- G-406, representing flows from the C-139 Basin. These flows will be potentially diverted into STA-5, depending on model user input.
- Historical flows from the L-3 are prescribed at three locations: G-88, G-155 and G-89. Flow routing at these structures is dependent on user input and simulated features. G-88 flows can be directed to the EAA, STA-5 or STA-6. G-155 flows are sent to STA-5 or into northwest WCA-3A. G-89 flows are directed to STA-5 or to central WCA-3A via the L-4 Canal and the S-140 structure.

Other flow monitoring locations playing an important role in this analysis are the L3DF and L3BRS UVM (Ultrasonic Velocity Meter) locations. L3DF is located on the L-3 Canal slightly downstream of the current G-406 location. L3BRS is located on the L-3 Canal just upstream of the G-88, G-155 and G-89 structures.

The flow data set used in the C-139 Basin Rulemaking process is comprised of flows at G-136 and G-406 locations for the period October 1978 through April 2000 (Walker, 2000a). The input data set for SFWMM simulations is comprised of flows at the locations described above for a longer period of time: January 1965 through December 2000 (December 1995 for the ECP simulations). The construction of flow time series for the SFWMM follows closely the procedures applied for assembling the flow data set for the C-139 Basin Rule (Walker, 2000a, 2000b).

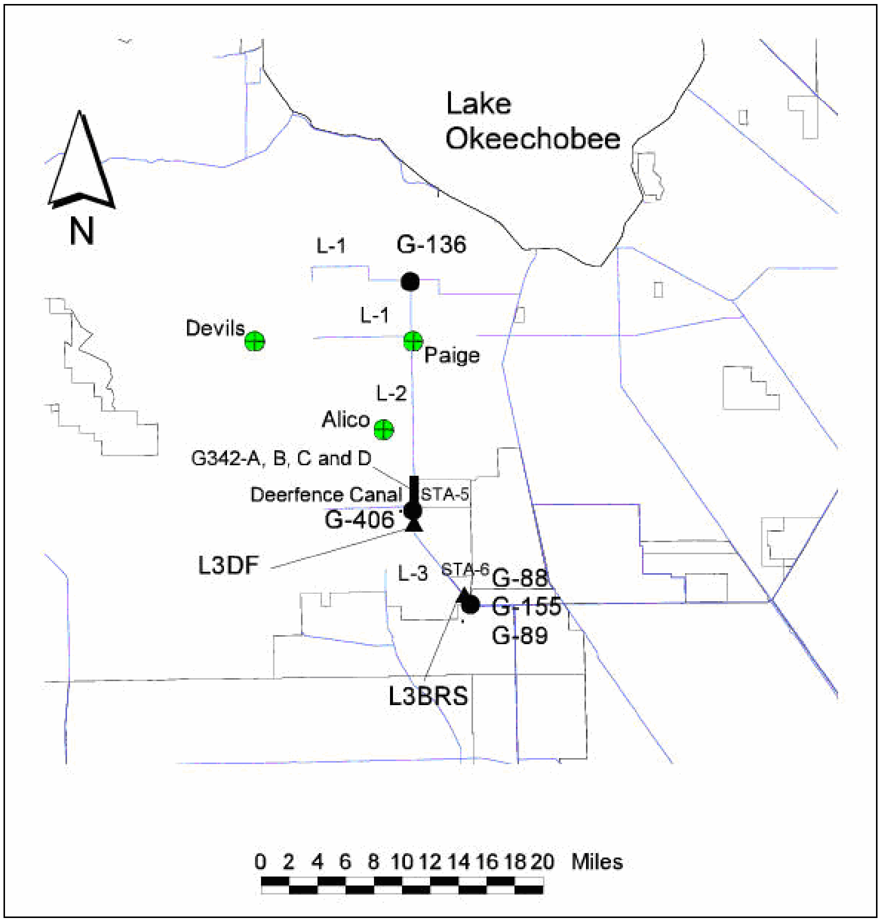

Figure 2.7.3.2 Schematic Representation of the L-3 Flow Locations

S190 Boundary Flows

South of the Western Basins, an additional structural boundary flow into the SFWMM is applied at structure S-190. A 36-year continuous time series (1965-2000) of daily runoff at S-190 is estimated using the AFSIRS/WATBAL model on the Feeder Canal Basin. It is assumed that local runoff from the Feeder Canal Basin could potentially be routed to the S-190 structure located downstream of Seminole Big Cypress Reservation irrigated lands (Figure 2.7.3.3). On a daily time step, the projected Reservation irrigation demands are compared to the estimated S-190 flows. If there is available water in the Feeder Basin, it is used to meet Reservation demands and the boundary discharge at S-190 is decreased accordingly to preserve the water budget. The revised estimated S-190 flows are a boundary condition in the SFWMM, while the revised supplemental demand time series are input to the model to be met by the regional system. On an average annual basis over the 36 year period, approximately 3,870 ac-ft/yr of water is captured upstream of S-190 for use by Seminole Big Cypress lands. This volume represents roughly 13.6% of the total 28,510 ac-ft/yr of supplemental irrigation demand.

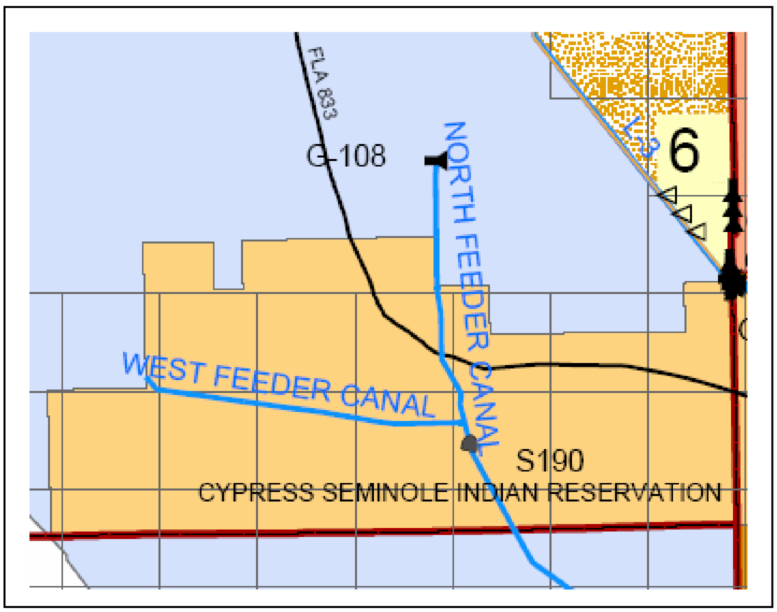

Figure 2.7.3.3 S-190 in Relationship to Seminole Big Cypress Reservation

Tidal Boundaries

Raw data from the National Oceanic and Atmospheric Administration/National Ocean Service (NOAA/NOS) were selected to create the tidal data set for the SFWMM. This sub-section presents a quick overview on how the data has been collected and treated.

To develop and evaluate the tidal data needed, the following steps were taken: (1) Collect historical data available to create tidal boundary file for SFWMM; (2) use NOAA/NOS Products and Services Division coefficients to simulate tidal data for secondary stations where historical data are not available (Table 2.7.3.1 and Figure 2.7.3.4); and (3) transform NOAA/NOS four historical daily values and hourly values to mean monthly. The 36 years (1965 to 2000) of daily data sets for each station were reduced to 12 monthly average values. The final data sets used to define the SFWMM tidal boundary for the east coast and the south east region of the model domain are shown in Figure 2.7.3.5. The model interpolates daily values from the monthly values, then the daily tidal data are passed to the groundwater routine as known head boundary conditions.

Table 2.7.3.1 Constants from the National Ocean Service Products and Services Division used to Compute Water Level for the Secondary Stations

Tidal station	**Time**		**constant**
	High Water	**Low Water**	
Flamingo Bay	3 hr 5 min	4 hr 28 min	0.837
Main Key	1 hr 3 min	1 hr 58 min	0.163
Virginia	Reference station		
Hollywood Beach	8 min	15 min	1.017
Delray Beach	53 min	1 hr 16 min	1.243
Palm Beach	-41 min	-35 min	1.365
Stuart	1 hr 44 min	2 hr 41 min	0.483

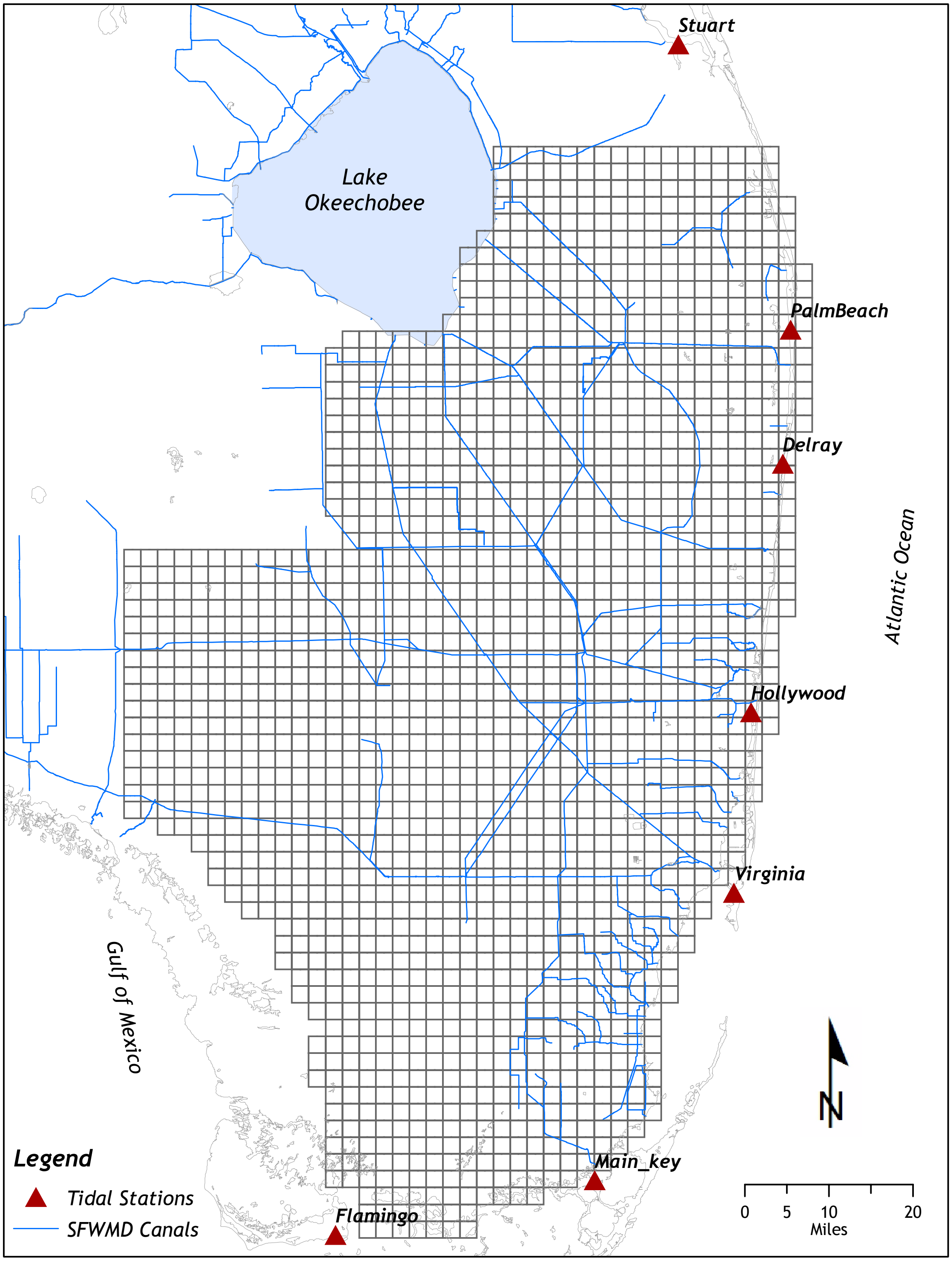

Figure 2.7.3.4 Tidal Stations Used to Define Coastal Boundary Conditions for the South Florida Water Management Model

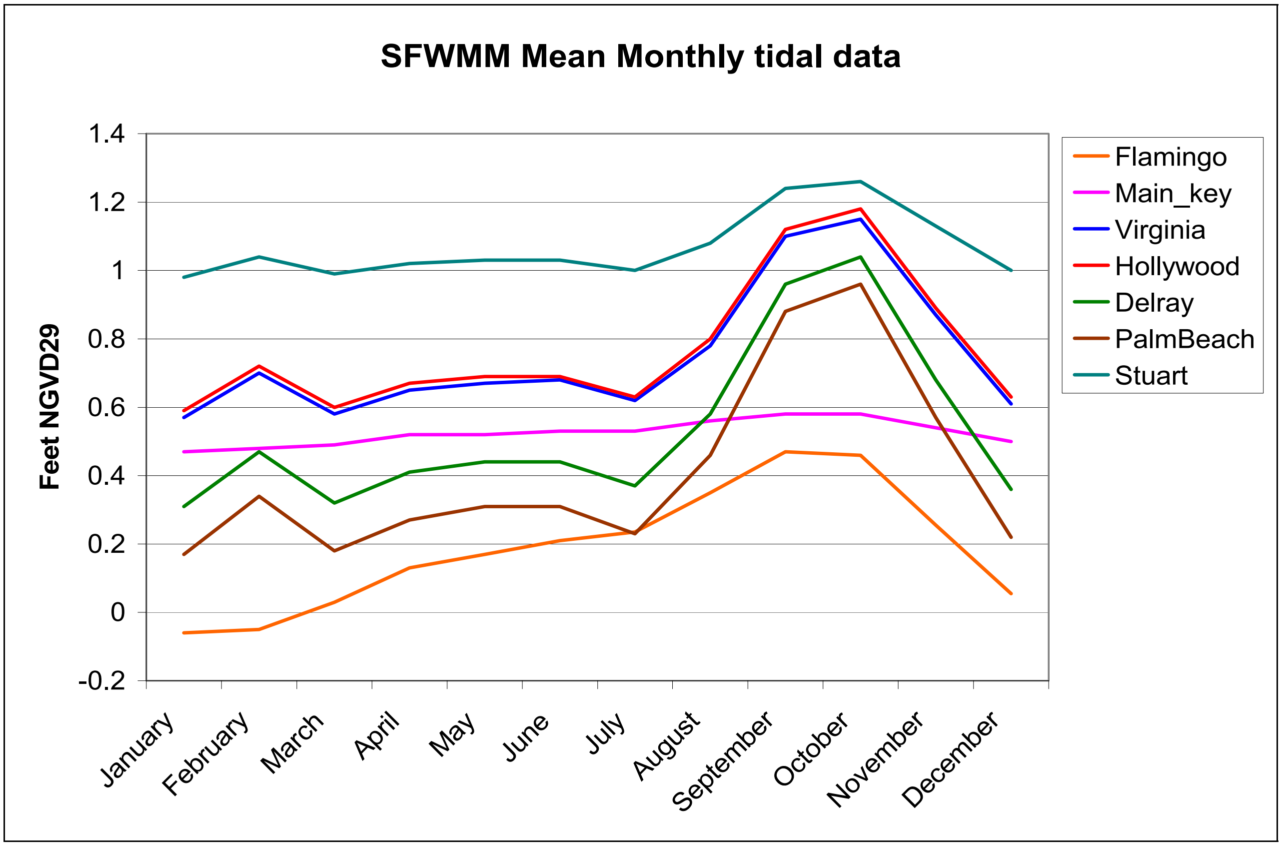

Figure 2.7.3.5 Mean Monthly Tidal Data used to define the South Florida Water Management Model Tidal Boundary

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3 POLICY AND SYSTEM MANAGEMENT COMPONENTS

In general, the task of describing the policy and system management components as implemented in the South Florida Water Management Model (SFWMM) is a difficult one. The complexity evident in the makeup of the South Florida regional system in combination with the specificity often associated with local and regional system management policies can lead to an overwhelming amount of detail in the description of the system and the means by which it is modeled. This complexity, in conjunction with the consideration that the SFWMM must be able to simulate all aspects of current and future proposed operational and infrastructure alternatives, makes it difficult to balance the desire to achieve both a comprehensive and a concise representation of SFWMM modeling capability. To help address this limitation, the approach utilized in the subsequent sections is to provide a general explanation of model methodologies and features on an area-by-area basis followed by specific examples of how model capabilities are applied to real-world or hypothetical examples.

3.1 LAKE OKEECHOBEE

3.1.1 Introduction

The name "Okeechobee" was derived from the Seminole Indian words "Oki" (water) and "Chubi" (big), and appropriately translates into "big water." Lake Okeechobee (LOK), the second largest freshwater lake lying entirely within the continental United States of America, occupies a surface area of approximately 728 square miles and has an average depth of 9 feet. Figure 3.1.1.1 shows the stage-area-storage relationships for Lake Okeechobee. The primary uses of Lake Okeechobee water include: (1) agricultural water supply to the Lake Okeechobee Service Area (LOSA); (2) backup water supply and prevention of saltwater intrusion to the Lower East Coast Service Areas (LECSAs); (3) water supply to adjacent municipalities (Belle Glade, Pahokee, Clewiston and Moore Haven; (4) use as a bird and wildlife feeding ground; (5) recreational uses (e.g., fishing and boating); and (6) environmental water supply to downstream ecosystems including the Caloosahatchee and St. Lucie Estuaries and the remnant Everglades. Lake stages are controlled for the purpose of: (1) environmental protection and enhancement of the Lake littoral zone (vegetation zone along the peripheral Lake areas) and the Everglades; (2) flood protection of adjacent areas; (3) water supply to agricultural and urban users; and (4) protection of the St. Lucie and Caloosahatchee Estuaries. The primary inflows to Lake Okeechobee are the Kissimmee River, Fisheating Creek, Taylor Creek/Nubbin Slough, Indian Prairie and Harvey Pond Canals. Its primary outlets are the Caloosahatchee River, St. Lucie River, Miami Canal, North New River Canal, Hillsboro Canal, West Palm Beach Canal and L-8 Canal.

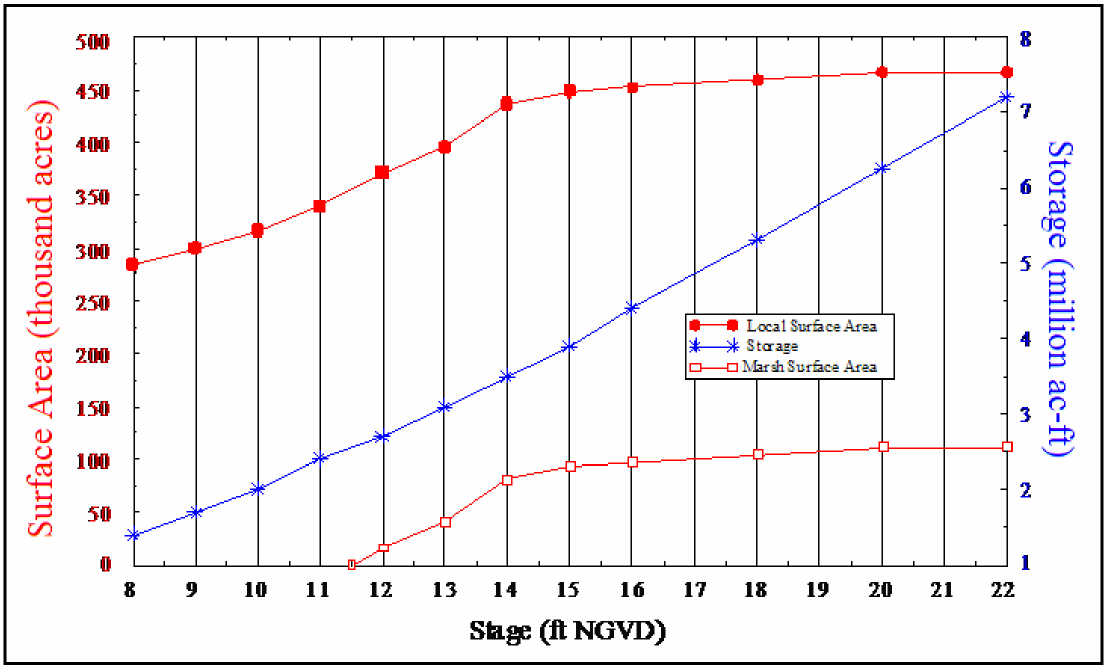


Figure 3.1.1.1 Lake Okeechobee Stage-Area-Storage Relationships

3.1.2 Lake Okeechobee Water Budget

In the SFWMM, Lake Okeechobee is simulated as a lumped hydrologic system as contrasted to the majority of the model domain where a distributed system of 2-mile by 2-mile grid cells is used (refer to Section 1.3). There is only one water level that is associated with the Lake at any given time step. For each daily time step the water budget equation is solved for Lake Okeechobee. This equation relates the change in storage within the Lake as a control volume, and incoming and outgoing flows for the same control volume. Mathematically, Lake hydrologic components (rainfall, evapotranspiration and seepage) and managed flows (structure discharges) account for changes in Lake storage. Rainfall and evapotranspiration are discussed in detail in Sections 2.2 and 2.3. Net levee seepage and regional groundwater movement in the Lake are assumed to be small relative to the other hydrologic components of the Lake water budget and are, therefore, not calculated in the model. Studies by Meyer and Hull (1969), and Shaw (1980) indicate that seepage rates range from 0.1 to 0.9 cfs/mile/ft. Runoff inflows generated from surrounding tributary drainage basins are discussed in Section 2.7. A generalized form of the Lake Okeechobee storage change equation (neglecting levee seepage and regional groundwater flow) can be written as:

$$S_{t+1} = S_t + Inflows_t - Outflows_t \tag{3.1.2.1}$$

where:

S_{t+1} = storage in the Lake at the next time step [ac-ft];
S_t = storage in the Lake at the current time step [ac-ft];
$Inflows_t$ = volume flux into the Lake (e.g. rainfall, structure discharge) during the current to the next time step [ac-ft]; and
$Outflows_t$ = volume flux out of the Lake (e.g. evapotranspiration, structure discharge) during the current to the next time step [ac-ft].

Management rules or operational policies dictate the amount, spatial distribution and timing of discharges through all Lake water control structures which, in turn, determine the variation of Lake storage. Given Lake storage, the corresponding Lake water stage and surface area can be obtained via the stage-area-storage relationship previously presented.

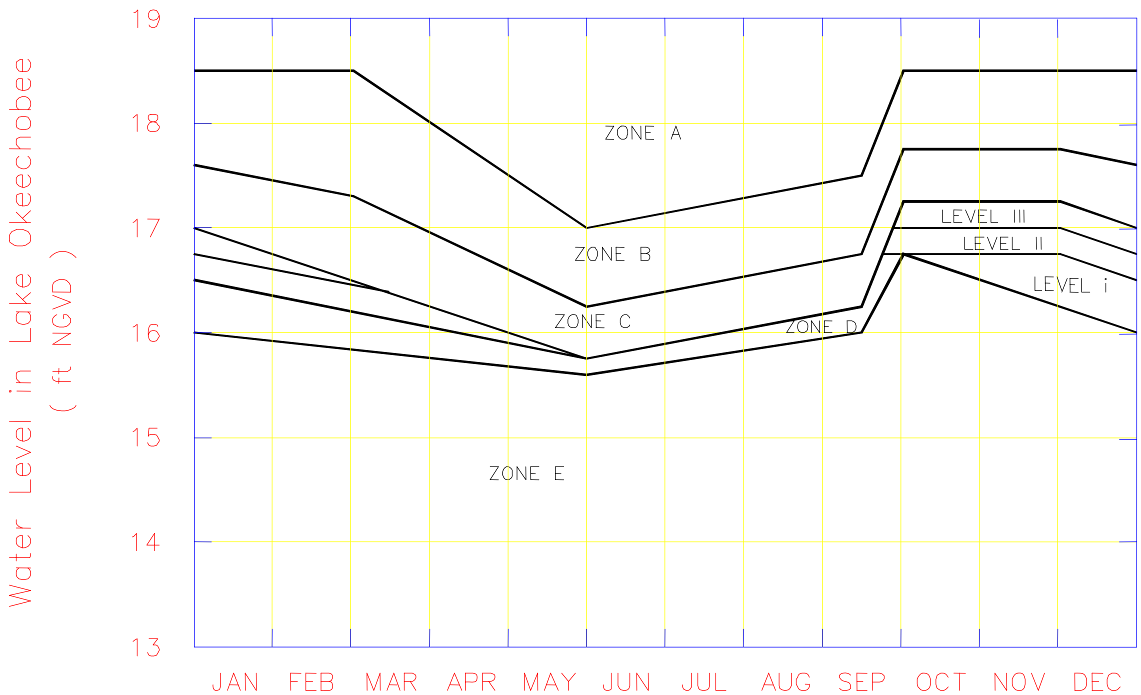
3.1.3 Lake Management Processes

High water levels in Lake Okeechobee are managed through regulatory and non-regulatory releases. Regulatory releases are made according to a calendar-based regulation schedule, established by the U.S. Army Corps of Engineers (USACE) in conjunction with the SFWMD and other public entities, to ensure that the integrity of the peripheral levee encompassing Lake Okeechobee is not compromised due to high water levels. The regulation schedule is designed to have minimum impact on the downstream ecological systems whenever possible while continuing to meet the flood control criterion. Regulatory releases can be made through the St. Lucie Canal and/or the Caloosahatchee River to tide water or through the Water Conservation Areas (WCAs), if this can be accomplished with minimum impact to the Everglades natural systems. Non-regulatory releases are sent to areas of the system for a myriad of purposes including irrigation, saltwater intrusion control, domestic water supply and environmental enhancement. Additionally, in the future, Lake Okeechobee discharges will be made to many of the proposed storage features (including above ground reservoirs and Aquifer Storage and Recovery facilities) to be constructed in the vicinity of the Lake. Several regulation schedules have been used for Lake Okeechobee in the past and flexibility is incorporated in the SFWMM to simulate several different alternatives.

In general, there are two distinctly different approaches to Lake management available in the SFWMM. The first type represents a time-dependent, trip-line operation where management decisions are on-off and clearly defined. The second type represents a time-dependent, climate-based operation where operational flexibility is included to account for predicted weather patterns. The use of climate forecasts in the simulation is achieved by pre-processing time series inputs of non-perfect forecasts of Lake inflow aggregated over various prediction windows (e.g. six to twelve months). The simulation checks the forecast daily, but the forecast is updated monthly. The forecast is produced using one of several estimation methodologies that rely on regional, global, and solar indicators which are useful tools for assisting operations, and for estimating inflows to Lake Okeechobee (Trimble, et al. 1997).

As an illustrative means of demonstrating the types of capabilities that are available to SFWMM users, two different regulation schedules will now be outlined. The Run 25 Regulation Schedule represents the trip-line type of operation – if the Lake level passed a regulation line, action was taken. It was used for real-time operations from 1993 to 2000. In 2000, the Water Supply and Environment (WSE) Regulation Schedule was implemented for Lake Okeechobee which represents a broader scope in determining operations. Climatic influences, both local and global, were included in WSE (Trimble, et al. 1998).

Under Run 25, water levels in Lake Okeechobee are managed through regulatory (flood control) and non-regulatory (primarily water supply) releases. The regulatory level for Lake Okeechobee ranges from 15.65 ft NGVD in late May to 16.75 ft NGVD on October 1. Table 3.1.3.1 summarizes the generalized operational rules governing Lake Okeechobee as implemented in the model for Run 25. The order by which the release type is presented in this table determines the sequence of deliveries as simulated in the model. The summary of the Run 25 regulatory rules as set forth by the USACE is given in Figure 3.1.3.1. As shown in the figure, regulatory releases are primarily conditioned on Lake stage falling above one of the calendar-based trigger lines.



ZONE	AGRICULTURAL CANALS (2)	CALOOSAHATCHEE RIVER (2)	ST. LUCIE CANAL
A	PUMP MAX. PRACTICABLE TO WCAs	UP TO MAX. CAPACITY AT S-77	UP TO MAX. CAPACITY AT S-80
B (1)	MAX. PRACTICABLE TO WCAs	6,500 CFS AT S-77	3,500 CFS AT S-80 (3)
C (1)	MAX. PRACTICABLE TO WCAs	UP TO 4,500 CFS AT S-77	UP TO 2,500 CFS AT S-80 (3)
D	MAX. PRACTICABLE TO WCAs	MAX. NON-HARMFUL DISCHARGES TO ESTUARY WHEN STAGE RISING	MAX. NON-HARMFUL DISCHARGES TO ESTUARY WHEN STAGE RISING
E	NO REGULATORY DISCHARGE	NO REGULATORY DISCHARGE	NO REGULATORY DISCHARGE

NOTES: (1) RELEASES THROUGH VARIOUS OUTLETS MAY BE MODIFIED TO MINIMIZE DAMAGES OR OBTAIN ADDITIONAL BENEFITS.
(2) SUBJECT TO FIRST REMOVAL OF LOCAL RUNOFF.
(3) EXCEPT WHEN EXCEEDED BY LOCAL INFLOW.

Figure 3.1.3.1 Lake Okeechobee Run 25 Regulation Schedule
(Adapted from U.S. Army Corps of Engineers).

Table 3.1.3.1 Lake Okeechobee Operations in the SFWMM

RELEASE TYPE	TRIGGER	ACTION/S	DESTINATION	EXCEPTION/S	SUBROUTINES USED
CONSUMPTIVE USE WATER SUPPLY:					
EAA, L8 and S236	Volumetric based on crop ET requirement	LOK supplements rainfall and local storage to meet total ET requirements	EAA and L8 via S354, S351, S352 & Culvert 10A; S236 Basin via S236	Delivery is subject to supply side management criteria (Section 3.3) and structure capacity limitations	AGAREA –SSM EAACOR EAA_FLOW_DIST_CAPAC_SETUP ALLOC_TO_EAA CANL_DEP_STRUC_PARAM_SETUP GEN_DEP_STRUC_CAPAC_SETUP SPEC_CANL_DEP_ STRUC_FLW
Lower East Cost (Domestic use, Industrial use, Agricultural use)	Net LECSA demands minus WCA contribution	LOK as back-up source. Delivery occurs when available water in WCA is less than demand in LEC service area	LEC service areas via EAA and WCA conveyance systems	If runoff from EAA sufficient to meet LEC demands	AGAREA WSNEEDS LAKE_NONREG_WCA LAKE_REG_WCA
Other LOSA basins including C43, C44, S4, etc…	Demand time series	Volumetric transfer from LOK considering conveyance limitations	Other LOSA Basins	Delivery is subject to supply side management criteria (Section 3.3) and structure capacity limitations	SSM CALOOS STLUCIE LAKE_NONREG_WCA LAKE_REG_WCA
ENVIRONMENTAL WATER SUPPLY:					
Everglades	NSM (or other) stage targets	If simulated stage at trigger location(s) is less than target stage(s), deliver water at maximum available capacity	▪ WCA-3A via Miami Canal first, then NNRC if desired ▪ WCA-2A via Hillsboro Canal ▪ WCA-1 via WPB Canal		MAIN LAKE_NONREG_WCA LAKE_REG_WCA
Estuary	Demand time series	If estuary demands exceed local basin runoff, supplement water to meet remaining demand	Caloosahatchee and/or St. Lucie Rivers		CALOOS STLUCIE
STORAGE INJECTION (ASR OR ABOVE-GROUND RESERVOIRS)*:					
	LOK stage adjusted for water supply releases, and adjusted LOK stage compared with storage injection line	Deliver water to associated reservoir &ASR(s)	Appropriate reservoir, Deliver to RES/ASR systems in EAA first, then to Caloosahatchee, St. Lucie or North Storage.	If LOK stage below user input storage injection schedule line; subject to conveyance capacity and available storage in reservoir / ASR.	ASR-INPUT LARGER_RESERV_STOR LAKE_NONREG_WCA LAKE_REG_WCA RESOUT ASR RESASR_SIM

Table 3.1.3.1 (*cont.*) Lake Okeechobee Operations in South Florida Water Management Model

RELEASE TYPE	TRIGGER	ACTION/S	DESTINATION	EXCEPTION/S	SUBROUTINES USED
REGULATORY:					
	LOK stage adjusted for water supply releases, and storage injection, if applicable	Delivery of water according to schedule operational rules	1. WCAs Basins: a. WCA-1 via WPB Canal b. WCA-1 via Hillsboro Canal c. WCA-2A via NNR Canal d. WCA-3A via Miami Canal (sequence can be specified by user) 2. Caloosahatchee & St. Lucie Estuaries	Dry Season (to the south): if demand in EAA exceeds conveyance capacity; or if runoff exceeds operational capacity Wet Season (to the south): if runoff or demand in EAA exceeds operational capacity or Everglades does not need water (optional)	LAKE_NONREG_WCA LAKE_REG_WCA

* Only used when these components are simulated in a particular scenario.

The WSE schedule shares many of the same features as Run 25 from the perspective of non-regulatory water supply and storage injection considerations. Many of the fields outlined in Table 3.1.3.1 are applicable to both schedules. However, in contrast to Run 25, the WSE schedule requires several additional criteria checks besides Lake stage in determining whether conditional regulatory releases are made. Figure 3.1.3.2 illustrates the WSE Operational Schedule. Figures 3.1.3.3 and 3.1.3.4 delineate operational decision trees that detail the implementation of the WSE schedule. Additional decision criteria that are part of the WSE schedule (diamonds in the decision tree; Figure 3.1.3.3 and 3.1.3.4) and their modeled implementation can be described as follows:

Lake Okeechobee Water Level Criteria – Lake water levels are checked against the defined operational zones. Depending on which zone simulated Lake stages fall after adjusting for water supply and storage injection discharges, the additional criteria as defined in the decision tree are applied.

Tributary Hydrologic Conditions – This index helps to determine when there is an opportunity to 'hedge' water management practices. For example, if tributary conditions are wetter than normal (and as a corollary higher inflow to Lake Okeechobee is expected), it may be appropriate to more aggressively release regulatory discharges in order to minimize the potential of adverse impacts later (e.g. high Lake stages). Two measures of the tributary hydrologic conditions are included within the design of the operational decision tree: 1) Lake Okeechobee tributary basin excess or deficit of net rainfall (rainfall minus evapotranspiration) during the past thirty days and, 2) the average S-65E inflow for the past two weeks. Each measure is updated on a weekly basis. Table 3.1.3.2 summarizes the ranges of the net rainfall and two-week average flow as they were selected in the original WSE Environmental Impact Study (EIS) to represent the various hydrologic regimes. The wettest classification of the two regional hydrologic indicators is selected to represent the hydrologic conditions in the tributary basin to ensure that flood protection criteria are being met. Therefore, if net rainfall indicates wet conditions but S-65E flow indicates normal conditions, the operational condition will be taken to be 'wet'. In the SFWMM, weekly pre-processed time series data is input and user input options define the thresholds for classification of tributary conditions. It is interesting to note that during the development of the WSE schedule, the SFWMM was one of the primary tools for testing classification schemes to determine the best threshold values for meeting regional hydrologic performance measures.

Table 3.1.3.2 Classification of Tributary Hydrologic Regimes

Tributary Condition	**Net Rainfall (inches past 4 weeks)**	**S-65E Flows (cfs-2 week average)**
Very Dry	less than ¯3.00	less than 500
Dry	¯3.00 - ¯1.01	500 - 499
Normal	¯1.00 - 1.99	1,500 - 3,499
Wet	2.00 - 3.99	3,500 - 5,999
Very Wet	4.00 - 7.99	6,000 - 8,999
Extremely Wet	greater than 8.0	greater than 9,000

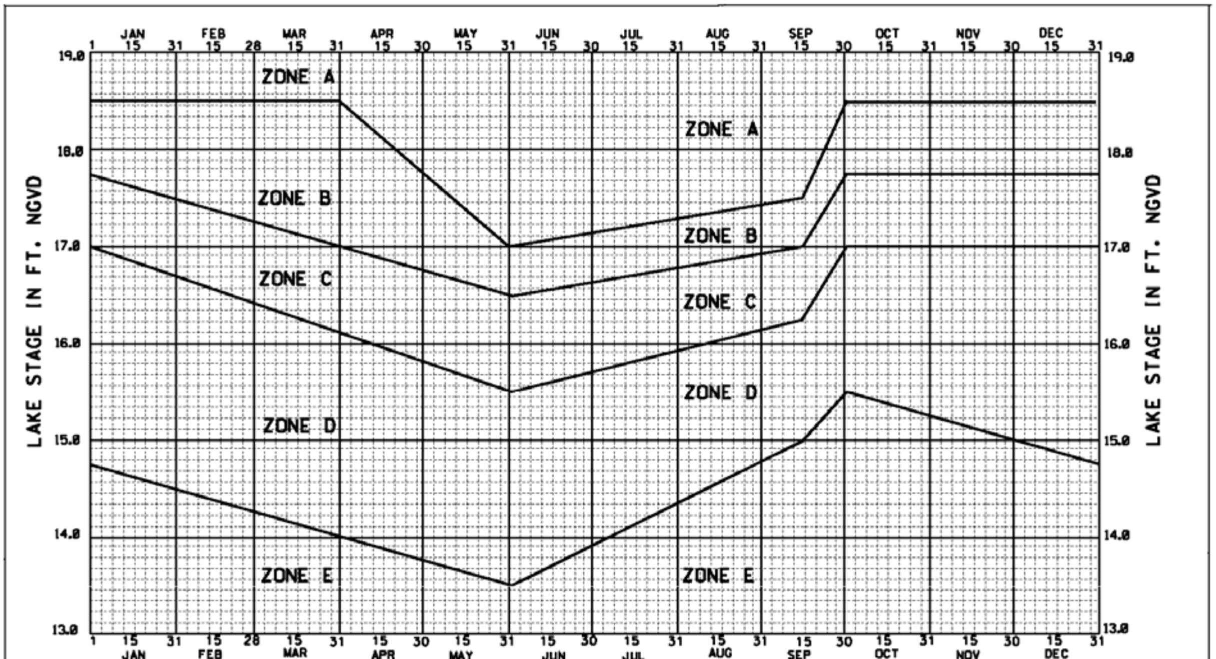
Climatic and Meteorologic Outlooks – While tributary conditions provide a good short-term indicator of potential trends in the magnitude of Lake Okeechobee inflows, the ambient conditions in the tributary basins are not the only contributing factor. Climatic and meteorological forecasts consider several longer-term (up to twelve month) regional, global, and solar indicators in helping to estimate the potential volume of water that can be expected to flow into Lake Okeechobee. As with the tributary conditions, information provided by these indices helps to determine when there is an opportunity to 'hedge' water management practices. The decision tree operational guidelines for WSE utilize three different outlooks in the decision making process: meteorologic forecast, seasonal outlook and multi-seasonal outlook. Each of these measures has an associated classification scheme for determining hydrologic regimes. In the SFWMM, monthly pre-processed non-perfect hind-cast data is input and user options define the thresholds for classification of outlooks (Table 3.1.3.3). An additional simplifying assumption is made in the model in which the meteorologic forecast is not considered and the seasonal forecast is assumed to apply in both decision boxes. This assumption is necessary due to the difficulty in deriving hind-cast meteorologic forecasts over the 1965-2000 period of simulation.

Table 3.1.3.3 Classification of Seasonal and Multi-Seasonal Inflow Predictions

	Seasonal Inflow Prediction	**Multi-Seasonal Inflow Prediction**
Condition	**(Equivalent LOK Depth** in feet)**	**(Equivalent LOK Depth** in feet)**
Dry	< 1.1	< 1.1
Normal	1.1 – 2.1	1.1 - 3.2
Wet	2.11 – 3.2	3.21 - 4.3
Very Wet	greater than 3.2	greater than 4.3

**Volume-depth conversion based on average Lake surface area of 467,000 acres

Determination of Discharges – Examining the WSE “Part 2” decision tree outcomes for discharges to tide, considerable flexibility can be observed in the final determination of discharge volumes. Several of the outcome boxes indicate releases “up to” a determined level. In real-time operations, this allows water managers to optimize the performance of the competing considerations when making regulatory discharges. In the SFWMM, simplifying assumptions are made that enable users to retain some flexibility in determining the operations associated with the decision tree outcome. For boxes that dictate a release “up to” maximum discharge or a determined steady flow, the model will always simulate the maximum allowable flow rate. In the case of decision boxes that indicate “up to maximum pulse release”, users have the option of specifying which of the three levels of pulse discharges to make to both the St. Lucie and Caloosahatchee Estuaries. Pulse releases are designed to mimic the flow pattern associated with naturally occurring rainfall events and as such should result in less impact to the estuary ecology by allowing time for recovery of the salinity envelope prior to resuming high discharge rates. Once a 10-day outflow pulse is initiated by the schedule, the release rule is continued to completion even if Lake stage drops below that pulse level. After a 10-day period is completed, the need for additional releases is re-evaluated. The pulse level values are shown in Table 3.1.3.4.



RELEASE THROUGH OUTLETS AS INDICATED

ZONE	AGRICULTURAL CANALS To WCAs (1,2)	CALOOSAHATCHEE RIVER AT S-77 (1,2,4)	ST. LUCIE CANAL AT S-80 (1,2,4)
A	PUMP MAXIMUM PRACTICABLE	UP TO MAXIMUM CAPACITY	UP TO MAXIMUM CAPACITY
B (3)	MAXIMUM PRACTICABLE RELEASES	RELEASES PER DECISION TREE (THESE CAN RANGE FROM MAXIMUM PULSE RELEASE UP TO MAXIMUM CAPACITY)	RELEASES PER DECISION TREE (THESE CAN RANGE FROM MAXIMUM PULSE RELEASE UP TO MAXIMUM CAPACITY)
C (3)	MAXIMUM PRACTICABLE RELEASES	RELEASES PER DECISION TREE (THESE CAN RANGE FROM NO DISCHARGE UP TO 6500 CFS)	RELEASES PER DECISION TREE (THESE CAN RANGE FROM NO DISCHARGE UP TO 3500 CFS)
D (3,5)	AS NEEDED TO MINIMIZE ADVERSE IMPACTS TO THE LITTORAL ZONE WHILE NOT ADVERSELY IMPACTING THE EVERGLADES. (SEE NOTE 5.)	RELEASES PER DECISION TREE (THESE CAN RANGE FROM NO DISCHARGE UP TO 4500 CFS)	RELEASES PER DECISION TREE (THESE CAN RANGE FROM NO DISCHARGE UP TO 2500 CFS)
E	NO REGULATORY DISCHARGE	NO REGULATORY DISCHARGE	NO REGULATORY DISCHARGE

NOTES:
(1) SUBJECT TO FIRST REMOVAL OF RUNOFF FROM DOWNSTREAM BASINS
(2) GUIDELINES FOR WET, DRY AND NORMAL CONDITIONS ARE BASED ON: 1) SELECTED CLIMATIC INDICES AND TROPICAL FORECASTS AND 2) PROJECTED INFLOW CONDITIONS. RELEASES ARE SUBJECT TO THE GUIDELINES IN THE WSE OPERATIONAL DECISION TREE, PARTS 1 AND 2.
(3) RELEASES THROUGH VARIOUS OUTLETS MAY BE MODIFIED TO MINIMIZE DAMAGES OR OBTAIN ADDITIONAL BENEFITS. CONSULTATION WITH EVERGLADES AND ESTUARINE BIOLOGISTS IS ENCOURAGED TO MINIMIZE ADVERSE EFFECTS TO DOWNSTREAM ECOSYSTEMS.
(4) PULSE RELEASES ARE MADE TO MINIMIZE ADVERSE IMPACTS TO THE ESTUARIES
(5) ONLY WHEN THE WCAs ARE BELOW THEIR RESPECTIVE SCHEDULES

CENTRAL AND SOUTHERN FLORIDA
INTERIM REGULATION SCHEDULE
LAKE OKEECHOBEE

DEPARTMENT OF THE ARMY, JACKSONVILLE DISTRICT
CORPS OF ENGINEERS, JACKSONVILLE, FLORIDA
DATED: 5 NOVEMBER 1999

WSE (WITH CLIMATE OUTLOOK)

Figure 3.1.3.2 WSE Regulation Schedule

Table 3.1.3.4 Pulse Release Hydrographs for the Three Levels of Zone D Regulation Schedule for Lake Okeechobee

DAY	St. Lucie I	St. Lucie II	St. Lucie III	Caloos. I	Caloos. II	Caloos. III
1	1,200	1,500	1,800	1,000	1,500	2,000
2	1,600	2,000	2,400	2,800	4,200	5,500
3	1,400	1,800	2,100	3,300	5,000	6,500
4	1,000	1,200	1,500	2,400	3,800	5,000
5	700	900	1,000	2,000	3,000	4,000
6	600	700	900	1,500	2,200	3,000
7	400	500	600	1,200	1,500	2,000
8	400	500	600	800	800	1,000
9	0	400	400	500	500	500
10	0	0	400	500	500	500

note: All values in cubic-feet per second.

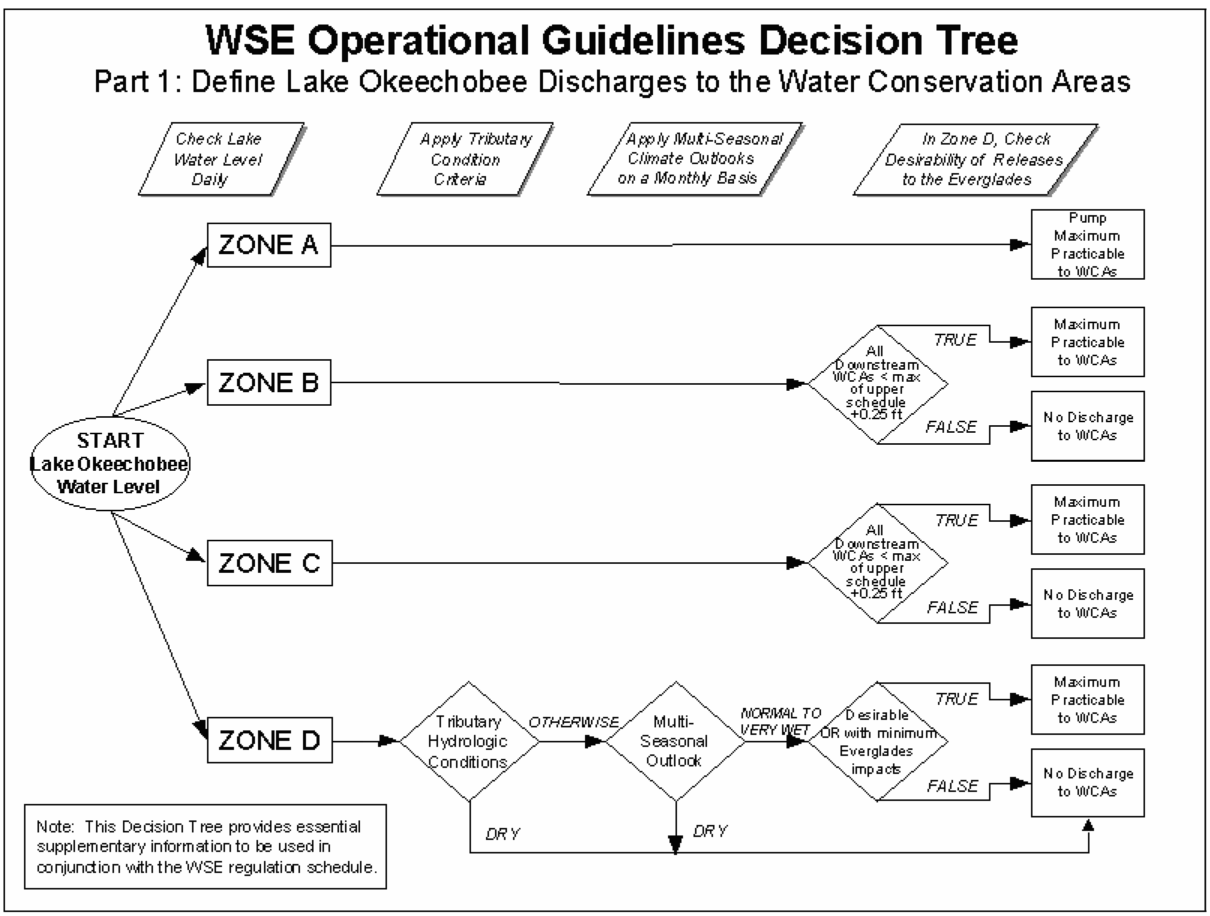


Figure 3.1.3.3 WSE Decision Tree for Lake Okeechobee Discharges to WCAs

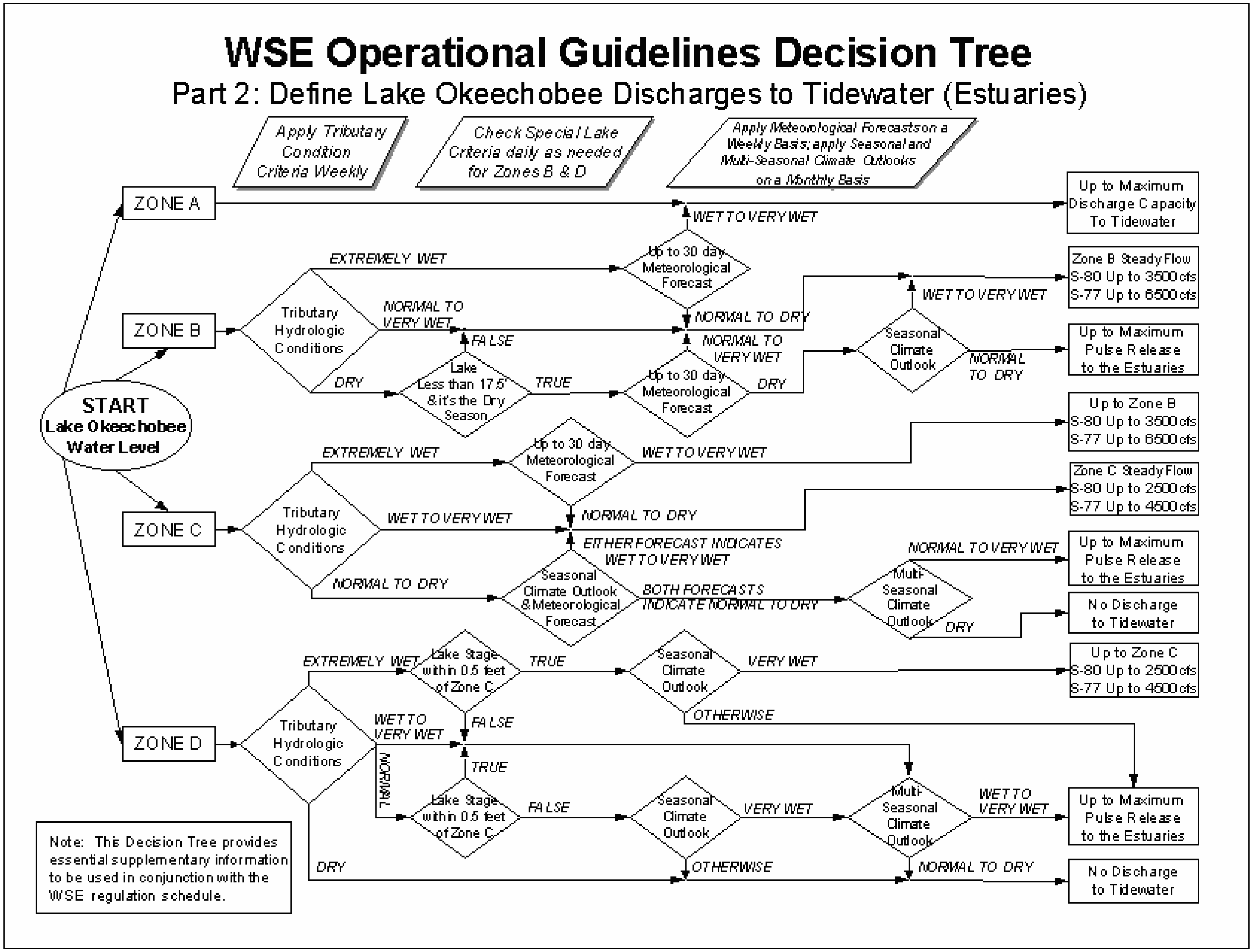


Figure 3.1.3.4 WSE Decision Tree for Lake Okeechobee Discharges to C-43 and C-44s

In addition to evaluations of different regulation schedules, the SFWMM has been used as a guide for shorter-term (less than 6 months) planning for Lake Okeechobee operations. For short-term planning, operational rules that deviate from normal may be implemented to meet short-term objectives. Flexibility is incorporated into the SFWMM so that changes in operations for Lake Okeechobee for defined periods throughout the calendar year can be simulated. These input options can be used to simulate several types of deviations, including varying the level of pulse releases and modifying breakpoints for classification for climate forecasts.

3.1.4 Lake Interaction with the C-43 and C-44 Basins/Estuaries

As explained in the text above, the Lake uses the C-43 Canal (Caloosahatchee River) and the C-44 Canal (St. Lucie River) as conduits for releasing regulatory flows west and east of the Lake, respectively. Additional considerations within these basins include the consumptive use water supply needs of agricultural users (as explained in Section 3.3) and the environmental water supply or flow attenuation needs of the downstream estuaries. Figure 3.1.4.1 shows a schematic diagram of the C-43 Basin/Estuary simulation module. Terms used in Figure 3.1.4.1 are defined as follows:

LOK2RES	=	Regulatory flood control release from Lake Okeechobee (LOK) to C-43 Reservoir through S-77.
LOK2BSN	=	Water supply deliveries from LOK to the C-43 Basin through S-77.
LOK2EST	=	Releases from LOK to Caloosahatchee Estuary through S-77/S-79. Includes LOK regulatory flood control releases and environmental water supply from LOK to meet estuarine demands.
RF	=	Rainfall into C-43 Reservoir.
ET	=	Evapotranspiration from C-43 Reservoir.
SEEPAGE	=	Seepage from C-43 Reservoir.
SPILLOVER	=	Spillover from C-43 Reservoir during extreme wet conditions. This excess volume is assumed to be discharged into the Caloosahatchee Estuary through S-79.
RES2LOK	=	Backpumping of C-43 Reservoir runoff to Lake Okeechobee. Only allowed if LOK stage is below a certain threshold (typically 13.0 ft). S77 backflow can also occur if Lake Okeechobee is below 11.1 ft.
RES2BSN	=	Water supply from C-43 Reservoir to C-43 Basin.
BSN2RES	=	Runoff from C-43 Basin routed to C-43 Reservoir.
RES2EST	=	Environmental water supply from C-43 Reservoir through S-79 to meet demand in Caloosahatchee Estuary. This demand is calculated at S-79 based on a prescribed flow distribution that would lead to desirable salinity envelopes within the Estuary.
RES2ASR	=	Injection from C-43 Reservoir into aquifer storage and recovery facilities (ASR).
ASR2EST	=	Environmental water supply from C-43 ASR through S-79 to meet demands in Caloosahatchee Estuary.
INJECTION LOSS	=	ASR efficiency loss (usually assumed to be 30%).
ASR2BSN	=	Water supply from C-43 ASR to C-43 Basin.
BSN2EST	=	Runoff from C-43 Basin routed to the Caloosahatchee Estuary through

S-79 (may meet estuarine demands or may be excess).

S235 = S-4 Basin runoff that is routed to the C-43 Basin through S-235.

S4D2CAL = S-4 Basin runoff from the Diston Water Control District that is routed to the C43 Basin via the 9-mile Canal.

CAL2S4D = C-43 Basin runoff routed to the S-4 Basin (Diston Water Control District) via the 9-mile Canal.

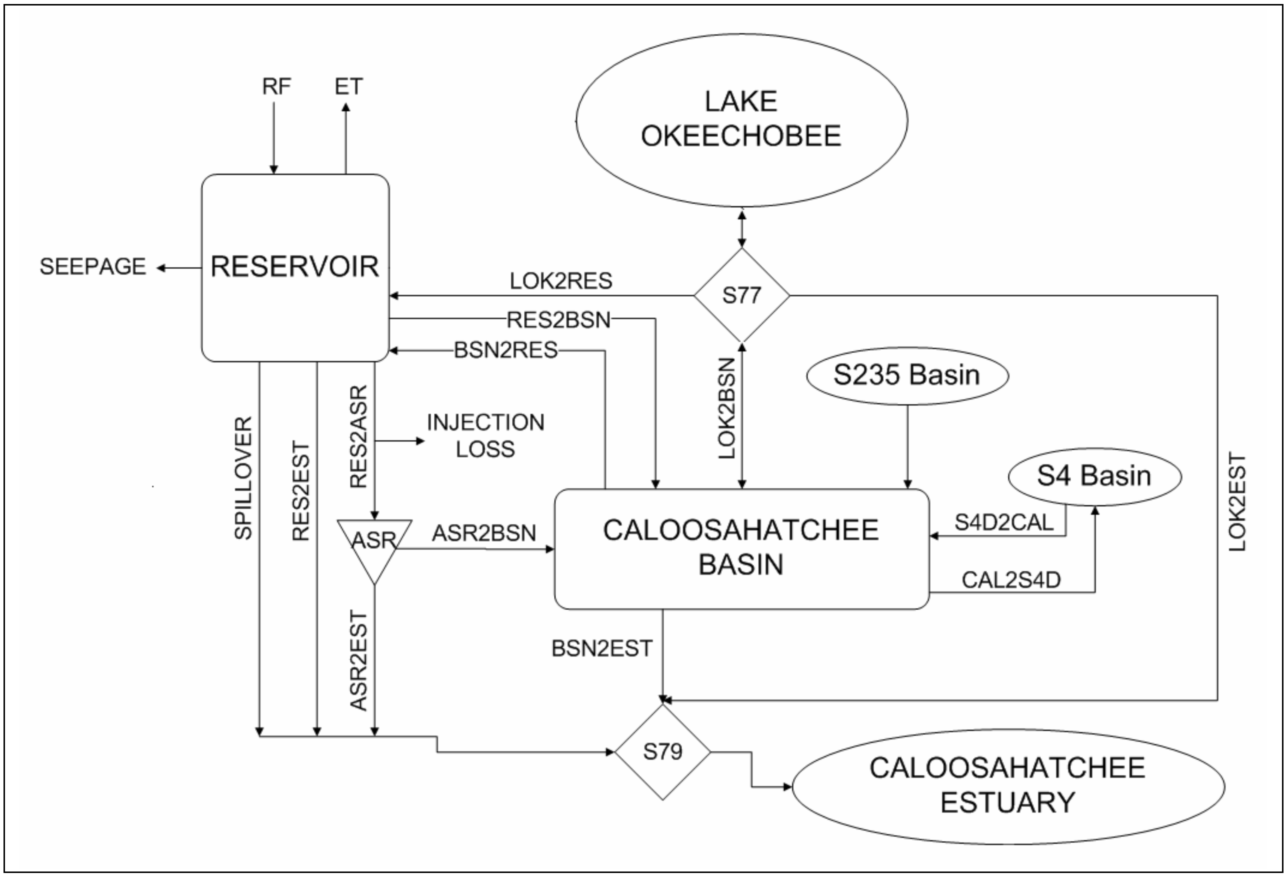


Figure 3.1.4.1 Schematic Diagram of Caloosahatchee Basin/Estuary Simulation Module

Figure 3.1.4.2 displays a schematic diagram of the C-44 Basin/Estuary simulation module. Terms used in Figure 3.1.4.2 are defined as follows:

LOK2RES = Regulatory flood control release from LOK to C-44 Reservoir through S-308.

LOK2BSN = Water supply deliveries from LOK to the C-44 Basin through S-308.

LOK2EST = Releases from LOK to St. Lucie Estuary through S-308/S-80. Includes LOK flood control regulatory releases and environmental water supply from LOK to meet estuarine demands.

RF = Rainfall into C-44 Reservoir.

ET = Evapotranspiration from C-44 Reservoir.

SEEPAGE = Seepage from C-44 Reservoir.

SPILLOVER = Spillover from C-44 Reservoir during extreme wet conditions. This excess volume is assumed to be discharged into the St. Lucie Estuary through

S-80.

RES2BSN = Water supply from C-44 Reservoir to C-44 Basin.

BSN2RES = Runoff from C-44 Basin routed to C-44 Reservoir.

RES2EST = Environmental water supply from C-44 Reservoir through S-80 to meet minimum demand in St. Lucie Estuary. This demand is calculated at S-80 based on a prescribed flow distribution that would lead to desirable conditions (identified as salinity envelopes and biological indicators for oysters and sea grasses) within the estuary.

RES2ASR = Injection from C-44 Reservoir into ASR facilities.

ASR2EST = Environmental water supply from C-44 ASR through S-80 to meet demands in St. Lucie Estuary.

INJECTION LOSS= ASR efficiency loss (usually assumed to be 30%).

ASR2BSN = Water supply from C-44 ASR to C-44 Basin.

BSN2EST = Runoff from C-44 Basin routed to the St. Lucie Estuary through S-80 (may meet estuarine demands or may be excess).

SLTRIB = Runoff from tributaries of the St. Lucie Estuary including the following basins: C25, C23/C24, Ten Mile Creek, South Fork, Tidal Basin. The runoff may meet estuarine demands or may be excess.

Storage facilities such as ASRs and reservoirs in the Caloosahatchee and St. Lucie Basins currently do not exist but can be simulated as options in the model. For the Caloosahatchee and St. Lucie Basin/Estuary, the purposes of these facilities are:

1. to attenuate regulatory flows from the Lake through structure S-77 (or S-308) which would otherwise be harmful to the basin (flooding) and to the estuary (sudden lowering in salinity);
2. to provide backup source of water for satisfying irrigation needs in the basin which otherwise comes exclusively from Lake Okeechobee; and
3. to regulate inflows to the estuary from the local basin which may be deemed harmful to the ecology in the area.

The detailed steps describing the interaction between the Lake and the C-43 and C-44 Basin/Estuaries, as calculated by the SFWMM v5.5, are presented in Appendix E.

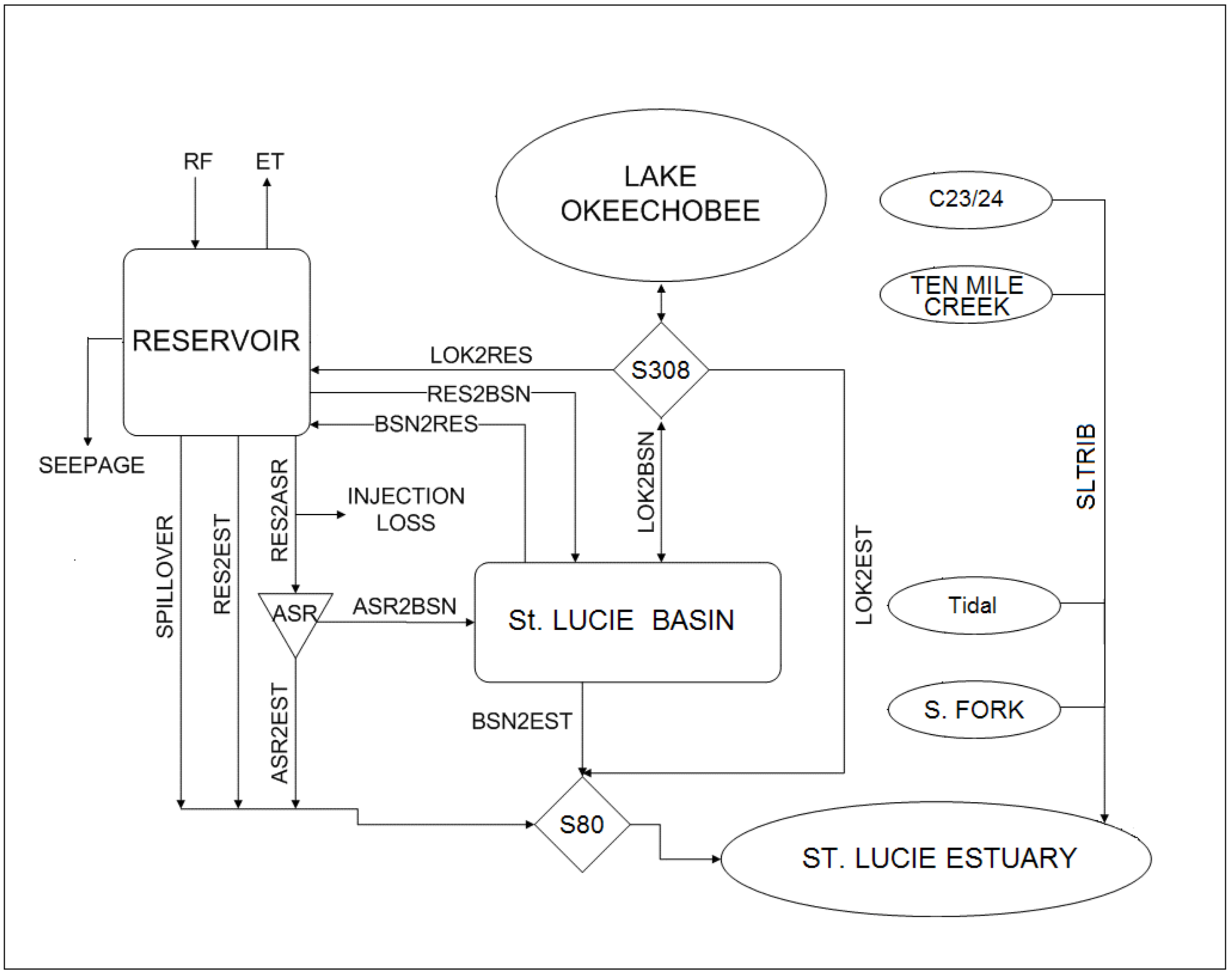


Figure 3.1.4.2 Schematic Diagram of St. Lucie Basin/Estuary Simulation Module

3.1.5 Lake Management Algorithm

The overall algorithm for simulating water releases from Lake Okeechobee is given in the following pseudo-code format. This text illustration is intended to provide insight into the way that the SFWMM internally implements the complexities associated with the management alternatives described up to this point.

1. Define key gages (monitoring point and/or canal) in WCAs/ENP and corresponding reference/trigger stages. These user-input locations and values will be used in the determination of water supply need in the WCAs/ENP and in assessing the potential impacts of regulatory discharges in the WSE decision trees.
2. Compute conveyance limitations for Lake release locations. EAA canal conveyance calculations are outlined in Section 3.2. For most other structures, pump capacities or gravity discharge based on headwater/tailwater (HW/TW) are considered. All subsequent steps involving water releases from the Lake are subject to the appropriate conveyance limitations.
3. Calculate required supplemental irrigation demands (consumptive use water supply) for the entire Lake Okeechobee Service Area. Demands may be calculated within the distributed portion of the model or read-in as pre-processed data (Trimble, 1992a and

1992b). The means of calculating supplemental demands within the SFWMM are described in Sections 3.2 and 3.3. Any water shortage cutbacks to deliveries will be applied in this step.

4. Execute St. Lucie module in order to determine portion of C-44 runoff that goes into St. Lucie Estuary and into Lake Okeechobee as backflow, release from Lake Okeechobee to satisfy C-44 Basin demand and minimum St. Lucie Estuary demand, if any. In general, the module can set priorities between:
   A. satisfying minimum St. Lucie Estuary demand (environmental delivery);
   B. routing runoff, if any, from C-44 Basin to the Lake or to the St. Lucie Estuary or satisfying C-44 Basin demand from the Lake.

5. Execute Caloosahatchee module in order to determine portion of C-43 runoff that goes into Caloosahatchee Estuary and into Lake Okeechobee as backflow, release from Lake Okeechobee to satisfy C-43 Basin demand and minimum Caloosahatchee Estuary demand, if any. In general, the module can set priorities between:
   A. satisfying minimum Caloosahatchee Estuary demand (environmental delivery);
   B. routing runoff, if any, from C-43 Basin to the Lake or to the Caloosahatchee Estuary or satisfying C-43 Basin demand from the Lake.

6. Calculate non-regulatory environmental deliveries to WCAs/ENP and/or water supply flows through WCAs to LECSAs. The means of calculating water supply needs to the WCAs/ENP and the LECSAs will be discussed in detail in Sections 3.4 and 3.5, respectively. In addition to conveyance limitations as outlined in step 2, minimum water levels at canals or nodal locations can be specified in the model in order to not “overdrain” Lake Okeechobee. This option restricts the timing of releases from the Lake such that water will not be made available at a downstream location if the stage in the Lake falls below specified levels. Another option exists in the model to deliver water only on user specified days of the week (e.g. LEC on Sunday and Thursday and EAA on Monday and Friday). This option is intended to reflect the practices of system operators during periods of shortage when allocated water is delivered to individual users on specified days in order to prevent competition for water supply.

7. Update and check intermediate Lake Okeechobee stage. If stage is within the regulatory zones as input for flood control purposes (i.e., stage in Zone A, B, C or D), then make regulatory releases as dictated by trip-line operation or decision trees. Due to the magnitude of a regulatory discharge through a single conveyance canal, Lake stage may drop to a level so as to significantly influence the amount of discharge through the next conveyance canal. For this reason, the model updates Lake stage before it calculates the necessary release for the next conveyance canal in the list. The order by which releases into WCAs are made is input by the user. Structure and conveyance capacities are reduced by the amount of water already discharged for non-regulatory purposes as defined in steps 3, 4, 5 and 6. If Lake stage is within the "normal operating zone" (i.e., "Zone E"), this flood control step is skipped.

8. Update final Lake Okeechobee stage and return to main program.

3.2 EVERGLADES AGRICULTURAL AREA

3.2.1 Introduction

The entire area whose primary supplemental water supply needs are met by Lake Okeechobee is collectively known as the Lake Okeechobee Service Area (LOSA). This area is comprised of several major basins including the Everglades Agricultural Area (EAA), the Caloosahatchee River (C-43) Basin, the St. Lucie River (C-44) Basin and many other, smaller basins located around the Lake (Figure 3.2.1.1). In this region, the majority of the supplemental demands on the regional system are for the purpose of agricultural irrigation. In the SFWMM, LOSA basins are handled using two distinct modeling approaches as dictated by data availability issues. The EAA, L-8 and S-236 Basins are modeled as part of the distributed 2-mile x 2-mile gridded system. In contrast, the C-43, C-44, S4 and other basins are modeled using a lumped system water-budget approach. This section focuses on the methodologies used in simulating the EAA within the SFWMM while Section 3.3 covers the simulation of lumped LOSA basins and management policies that affect LOSA as a whole. Topics addressed in this section include: calculation of ET in the EAA as it relates to the estimation of runoff and demand (irrigation requirement) in the EAA; routing of runoff within the EAA, including canal conveyance considerations; and additional complexities of system components and management in the EAA, primarily above ground storage features including reservoirs and Stormwater Treatment Areas (STAs).

General Description

The Everglades Agricultural Area (EAA) encompasses an area south and southeast of Lake Okeechobee (Figure 3.2.1.2), covering approximately 593,000 acres of land of which 468,000 acres are in agricultural production (1988 land use cover information). A strong interaction exists between the hydrologic and management processes in the Everglades Agricultural Area. Of the area in agricultural production, about eighty percent is sugar cane. The four primary conveyance canals within the EAA are the Miami, North New River, Hillsboro and West Palm Beach Canals. They are used both for water supply and flood control purposes. The major structures in the EAA are S-3/S-354, S-2/S-351, S-352, S-5A, S-6, S-7, and S-8 (Figure 3.2.1.2).

The Rotenberger Tract and Holey Land, although part of the Miami Canal Basin, are separated from the irrigated areas by levees, and thus, are treated as separate subbasins in the model. The following discussion will focus on the Miami, North New River/Hillsboro and West Palm Beach Canal Basins. The 298 Districts, as shown in Figure 3.2.1.2, will be discussed in Section 3.3.8. Figure 3.2.1.3 conceptualizes inflows and outflows from the EAA. The SFWMM simulates discharges at all inlet and outlet structures shown in Figure 3.2.1.3 except G-88 and G-136 at which discharges are estimated separately (refer to Section 2.7).

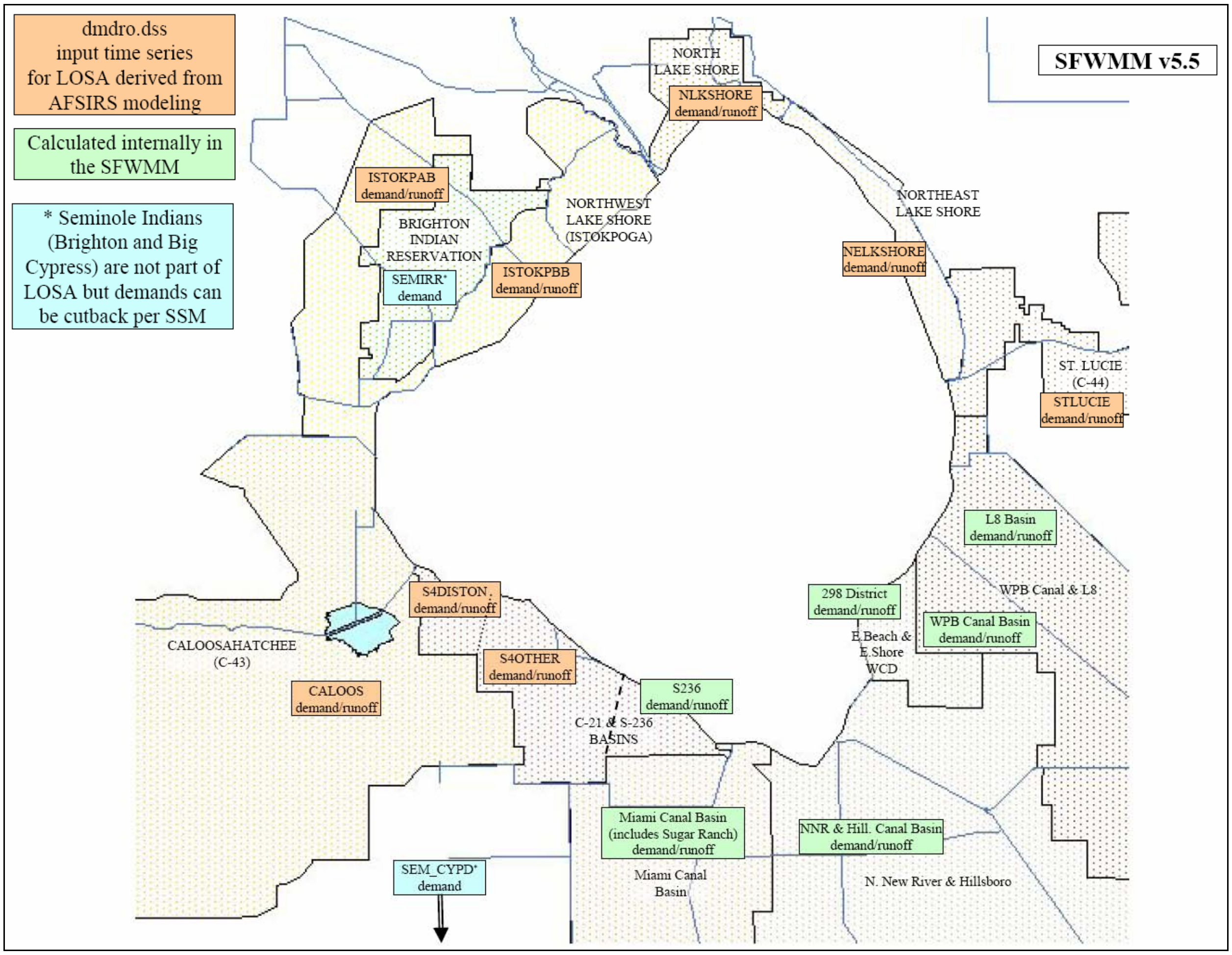


Figure 3.2.1.1 LOSA Basins around Lake Okeechobee

The unique characteristics of the EAA are as follows:

1. Extensive field-scale management operations within the EAA are simplified such that they fit within the regional-scale modeling framework of the SFWMM. Water levels within the EAA are well-maintained below land surface due to seepage irrigation. Thus, overland flow is not calculated between grid cells within the EAA although infiltration, evapotranspiration and groundwater flow are still simulated as distributed processes within the same area.
2. Discharges from the Lake into the EAA and into the WCAs through the EAA canals are influenced by operating rules in the EAA, as well as by those in Lake Okeechobee and the Water Conservation Areas.
3. The amount of water that can flow through the EAA is constrained by EAA canal conveyance characteristics, and local runoff and demand conditions.
4. Flow-through capacity along an EAA canal, i.e., the amount of Lake water that can be delivered south into the Water Conservation Areas, depends on EAA canal conveyance characteristics. The latter, in turn, is a function of the EAA canal water surface profile. Therefore, a hydrodynamically-based routing procedure where the water surface profile and corresponding discharge is calculated for the EAA is necessary in order to account for the daily variation of EAA flow-through capacity. This procedure is different from the

water budget approach applied to non-EAA canals where a hydraulic grade line with time-invariant slope is assumed.

5. Limited or sparse stage data exists for the interior part of the EAA such that calibration by matching historical stages is not possible at this point in time.

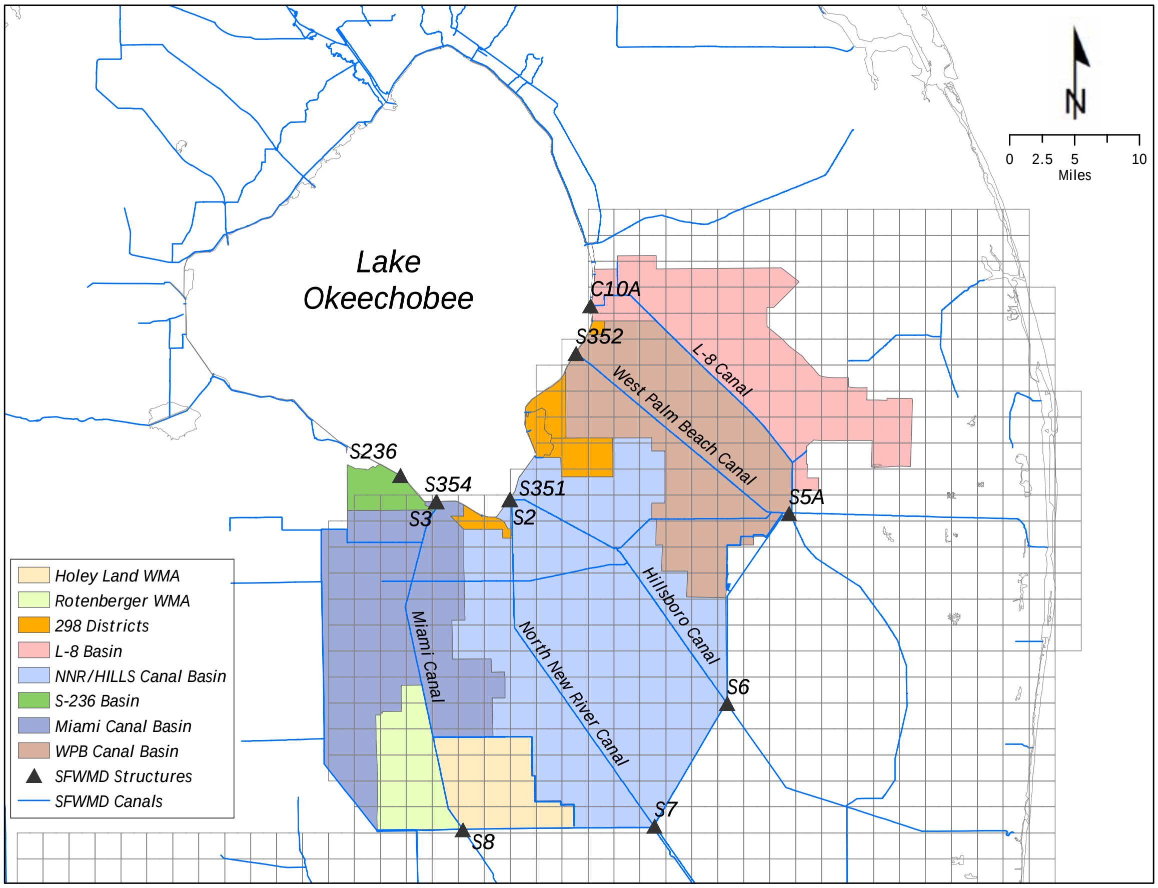


Figure 3.2.1.2 SFWMM Grid Superimposed on Major Basins in the EAA

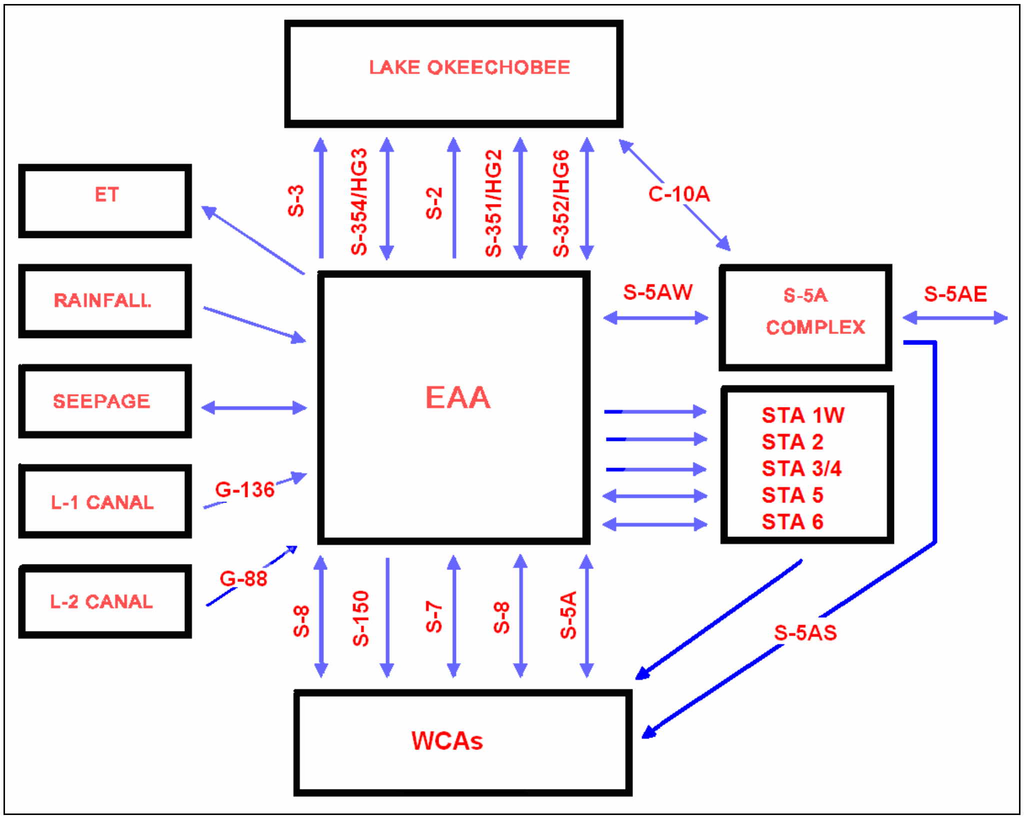


Figure 3.2.1.3 Conceptual Diagram of the Hydrologic System in the EAA as Represented in the SFWMM (Adapted from Abtew and Khanal, 1992).

3.2.2 Simulation of Everglades Agricultural Area Runoff and Demand

The EAA is a system with limited storage capacity. Runoff occurs in times when rainfall exceeds storage capacity and irrigation requirements in the area. Irrigation requirement, on the other hand, is the amount of water in excess of rainfall needed to satisfy evapotranspiration requirements within the EAA. In the soil moisture balance model discussed in the EAA report by Abtew and Khanal (1992), the entire area of the EAA in production was assumed to have a uniform depth to water table equal to 1.5 feet below land surface. This is consistent with the level at which the water table is maintained in the EAA during seepage irrigation, the type of irrigation used for the predominant crop type in the area, sugar cane. Within this narrow band of soil, referred to as the soil column (A in Figure 3.2.2.1), a desired range of moisture contents is maintained. The lower and upper limits of this range (C and D in Figure 3.2.2.1) expressed in terms of equivalent depths of water are SOLCRT and SOLCRNF, respectively.

Therefore, the EAA is simulated in the model such that the natural fluctuation of total soil moisture above the water table is within SOLCRT and SOLCRNF. Also, the water table is maintained at 1.5 feet below land surface.

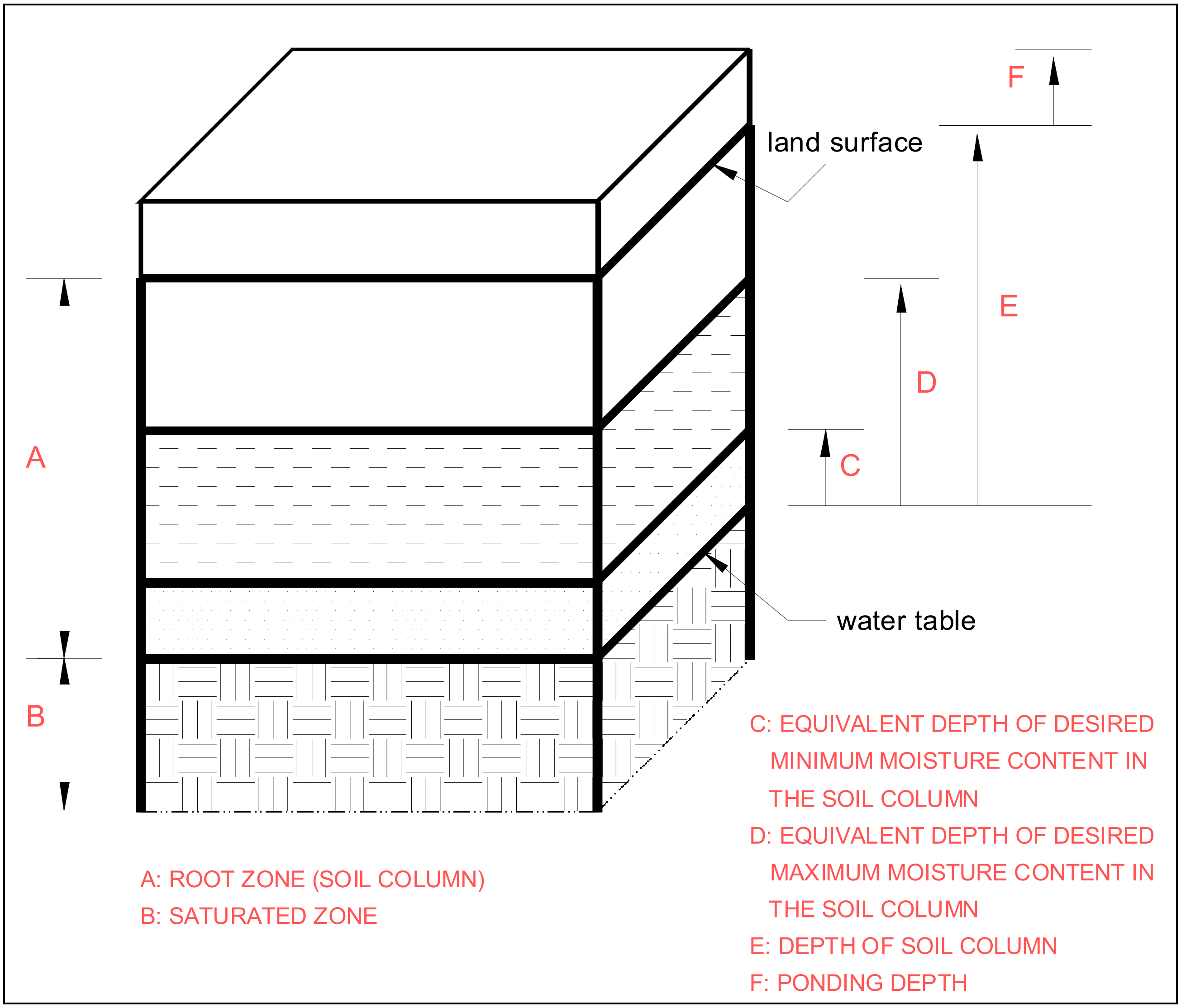


Figure 3.2.2.1 Conceptual Representation of an EAA Grid Cell in the SFWMM

A definition of some pertinent variables used in simulating runoff and irrigation requirements in the EAA is given below:

DPH	=	depth of irrigation requirement;
depth_soil_eaa	=	assumed distance between land surface and the water table; thickness of the soil column; equal to 1.5 ft;
DPTHRNFF	=	potential depth of runoff initially equal to the sum of POND and SOLMX in excess of SOLCRNF;
ELLS	=	land surface elevation relative to NGVD;
ET	=	total evapotranspiration from ponded water, and moisture in the unsaturated and saturated zones;
	=	ETP + ETU + ETS;
fracdph_max	=	ratio of maximum equivalent depth of water that can be stored in the soil column and equivalent depth of desired maximum moisture content in the same soil column; used as a calibration parameter (refer to Chapter 4);

fracdph_min = ratio of minimum equivalent depth of water that can be stored in the soil column and equivalent depth of desired minimum moisture content in the same soil column; used as a calibration parameter (refer to Chapter 4);
GDAR = grid cell area;
GWMAXDP = equivalent depth of water required to fill the storage space below the base of the soil column to the water table plus meeting anticipated saturated zone evapotranspiration;
H = head; location of the water table relative to NGVD;
PERC = water that goes to the saturated zone from ponding and excess moisture in the soil column used to raise the water table up to the base of the soil column;
PERC_IRRIG = water that goes to the saturated zone from irrigation used to raise the water table up to the base of the soil column;
POND = ponding depth;
RAIN = depth of rainfall;
S = storage coefficient; typically 0.20;
SOLCRNF = equivalent depth of desired maximum moisture content in the soil column a calibration parameter that varies with month of year;
= (fracdph_max)(depth_soil_eaa)S;
SOLCRT = equivalent depth of desired minimum moisture content in the soil column; trigger for irrigation requirements to be met from outside sources (e.g., LOK); a calibration parameter that varies with month of year;
= (fracdph_min)(depth_soil_eaa)S;
SOLMDPH = maximum equivalent depth of water that can be stored in the soil column; storage capacity of the soil column;
= (depth_soil_eaa)S;
SOLMX = equivalent depth of soil moisture in the soil column;
VOL_IRRIG = volume of irrigation requirement for an EAA grid cell equal to the product of DPH and GDAR; and
VOL_EXCESS_WATER = volume of excess water that runs off from an EAA grid cell equal to the product of DPTHRNFF and GDAR.

The following sequence of calculations is performed for each EAA grid cell at each time step. Evapotranspiration is calculated first. Assuming unrestricted supply of water at all times, either through available moisture in the root zone, rainfall or irrigation, the theoretical crop requirement is given by:

$$ETMX = (KCALIB)(KVEG)(PET_0) \tag{3.2.2.1}$$

where:

PET_0 = depth of potential evapotranspiration for a reference crop (wet marsh) calculated using SFWMD Simple Method;
KVEG = theoretical crop coefficient which are monthly averaged values; KVEG was based on an earlier study (Abtew and Khanal, 1992). In the EAA, only the predominant crop type: truck crops, sugar cane or irrigated pasture is assigned to each cell; and

KCALIB = adjustment/calibration parameter which varies from month to month; KCALIB was created to take into account differences between modeling approaches, specifically modeling scale, used in the soil moisture balance model by Abtew and Khanal (1992) and the South Florida Water Management Model.

The monthly variation of theoretical crop coefficient KVEG for the three predominant crop types in the EAA was given in Table 2.3.4.3 (land uses 7, 8, and 9). Note that the final/calibrated KVEG values for the EAA correspond to the product of the theoretical KVEG and the adjustment/calibration parameter KCALIB discussed in this section.

Total evapotranspiration depth, on the other hand, is given by:

$$ET_0 = (KFACT)(PET_0) \tag{3.2.2.2}$$

where KFACT is an adjustment factor that takes into account vegetation/crop type and location of the water table relative to land surface. Table 3.2.2.1 shows the adjustment factor KFACT as a function of depth. Note that ETMX corresponds to ET_0 evaluated at land surface down to the depth to shallow root zone. A definition of some variables introduced in Table 3.2.2.1 follows the table.

Table 3.2.2.1 Variation of KFACT in the Equation for Theoretical Total Evapotranspiration as a Function of Depth

Depth from Land Surface to Water Line DWT: water table condition (below ground) PND: ponding condition (above ground)	**Adjustment Factor, KFACT**
DWT ≥ DDRZ	0.0
DSRZ < DWT < DDRZ	(KCALIB)(KVEG)[(DDRZ – DWT) / (DDRZ – DSRZ)]
0.0 ≤ DWT ≤ DSRZ	(KCALIB)(KVEG)
0.0 < PND ≤ OWPOND	(KCALIB)(KVEG) + [KMAX – (KCALIB)(KVEG)](PND / OWPOND)
PND > OWPOND	KMAX

The table variable definitions are as follows:

OWPOND = ponding depth above which open-water ET exists; transpiration by plants submerged at depths equal to or more than OWPOND no longer contribute to evapotranspiration, and evapotranspiration is equal to open-water evaporation; OWPOND is assigned a value of 12 inches in the model;

DSRZ = depth from land surface to the bottom of the shallow root zone; depth below which the root system of a crop will experience increased difficulty in extracting water from the saturated zone; equal to 18 inches;

DDRZ = depth from land surface to the bottom of the deep root zone; depth below which the root system of a crop can no longer extract water from the saturated zone; assumed to be between 36 to 46 inches;

PND = depth of ponding;
DWT = distance of water table below land surface; and
KMAX = conversion factor from PET_0 to open water ET; assumed to be equal to 1.1.

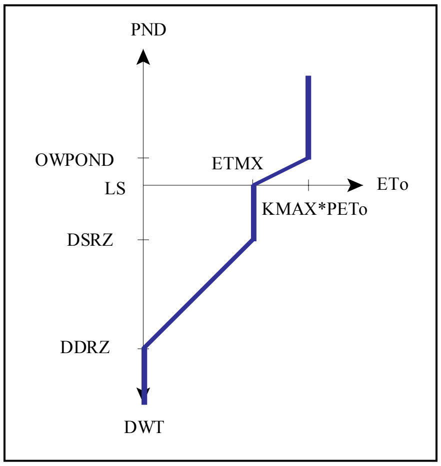


Figure 3.2.2.2 Variation of Total Evapotranspiration, ET_0, as a Function of Depth

Figure 3.2.2.2 is a diagram of the total evapotranspiration as it varies with depth. The actual total evapotranspiration (ET) is the sum of three components: ETS from the saturated zone, ETU from the unsaturated zone, and ETP from free water zone or ponding. The model assumes that evapotranspiration is extracted from the unsaturated zone first, and the free water zone last. Initially, ponding and rainfall are assumed to increase moisture in the soil column. Unsaturated zone evapotranspiration then becomes the lesser value between the theoretical crop requirement [Equation (3.2.2.1)] and the total moisture in the soil.

$$ETU_t = \min(ETMX_t, POND_{t-1} + RAIN_t + SOLMX_{t-1}) \tag{3.2.2.3}$$

The remaining theoretical requirement, $ETMX_t - ETU_t$, if any, will be met from the water table. This amount is limited by the remaining theoretical total evapotranspiration. The anticipated evapotranspiration from the saturated zone is:

$$ETS_t = \min(ETMX_t - ETU_t, ETS_0) \tag{3.2.2.4}$$

where ETS_0 is the theoretical saturated zone ET. It is essentially the same as ET_0 defined at depths below land surface (LS in Figure 3.2.2.2). Evapotranspiration from ponding becomes:

$$ETP_t = \min(ET_0 - ETMX, POND_t) \tag{3.2.2.5}$$

For accounting purposes, the following equalities are assumed for ponding and non-ponding conditions:

1. If ponding exists:
   $ET = ET_0$, $ETU = ETMX$, $ETS = 0.0$, and $ETP = ET - ETU$
2. If there is no ponding:
   ETU from Equation (3.2.2.3),
   ETS from Equation (3.2.2.4), $ETP = 0.0$, and $ET = ETU + ETS$

The soil moisture content expressed in terms of equivalent water depth above the base of the soil column is calculated next:

$$SOLMX_t = SOLMX_{t-1} + POND_{t-1} + RAIN_t - (ETU_t + ETP_t)$$

If the updated soil moisture content exceeds the storage capacity of the soil column, SOLMDPH, ponding will result at the end of the time step and soil moisture have to be reevaluated. Thus:

$$POND_t = \max(\ SOLMX_t - SOLMDPH,\ 0.0\ ) \tag{3.2.2.6}$$

$$SOLMX_t = SOLMDPH \quad \text{if} \quad POND_t > 0.0 \tag{3.2.2.7}$$

The potential depth of runoff, DPTHRNFF, equals the ponding depth plus any soil moisture beyond the equivalent depth of the desired maximum moisture content in the soil column, SOLCRNF. (NOTE: SOLCRNF ≠ SOLMDPH).

$$DPTHRNFF_t = \max(\ POND_t + SOLMX_t - SOLCRNF,\ 0.0\ )$$

So far, this amount of potential runoff assumes that the water table is already at 1.5 feet below land surface elevation. An assumption in the simulation of the EAA in the SFWMM is that ponded water and moisture in the unsaturated zone percolates into the saturated zone up to the base of the soil column, if necessary, before runoff actually occurs. DPTHRNFF is reduced by the amount of percolation or the amount of water needed to bring the water table at 1.5 feet below land surface. In other words, if the water table is below the base of the soil column, the potential depth of runoff will be used to fill the available storage in the form of percolation. The concept of maintaining the water table at 1.5 feet below land surface, and the specification of the desired minimum and maximum moisture content (in terms of equivalent depth) above the water table are key modeling techniques used to simulate runoff and quantify irrigation requirements (demands) in the EAA module of the SFWMM.

Actual percolation is the lesser value between what could potentially runoff, DPTHRNFF, and the amount of water necessary to bring the water table up to the base of the soil column, GWMAXDP. Assuming that the water table is below the base of the soil column, GWMAXDP represents the available storage between the base of the soil column and the water table plus anticipated saturated zone ET. It can be calculated as follows. The vertical distance between the water table and the base of the soil column, WT_TO_BSC, is given by:

$$WT\_TO\_BSC_t = (ELLS - SOLMDPH \div S) - H_t$$

Note that SOLMDPH ÷ S is equal to 1.5 ft, and WT_TO_BSC is greater than zero if the base of the soil column is above the water table.

$$EQUIV\_DEPTH\_SOIL\_COL\_TO\_WT_t = \max[\ (WT\_TO\_BSC_t)(S),\ 0\ ]$$

$$GWMAXDP_t = EQUIV\_DEPTH\_SOIL\_COL\_TO\_WT_t + ETS_t$$

$$PERC_t = \min(\ DPTHRNFF_t,\ GWMAXDP_t\ ) \tag{3.2.2.8}$$

The updated potential depth of runoff becomes:

$$DPTHRNFF_t = DPTHRNFF_t - PERC_t \tag{3.2.2.9}$$

while the remaining storage below the base of the soil column that needs to be filled in from other sources (specifically, via irrigation) is:

$$GWMAXDP_t = GWMAXDP_t - PERC_t \tag{3.2.2.10}$$

It should be noted that $GWMAXDP_t$ can be positive only if $DPTHRNFF_t = 0.0$ after Equation (3.2.2.9). In other words, DPTHRNFF and GWMAXDP are mutually exclusive, i.e., they cannot be non-zero at the same time.

The model assumes that the portion of the potential depth of runoff that comes from ponding percolates below the soil column before soil moisture in excess of SOLCRNF does. Therefore, if the amount of water that percolates is greater than $POND_t$, then, all ponding is assumed to percolate and soil moisture is reduced. $SOLMX_t$ and $POND_t$ are updated within the current time step t:

$$SOLMX_t = SOLMX_t - (PERC_t - POND_t) \tag{3.2.2.11}$$

$$POND_t = 0.0 \tag{3.2.2.12}$$

Otherwise, $POND_t$ is reduced while $SOLMX_t$ remains the same:

$$POND_t = POND_t - PERC_t \tag{3.2.2.13}$$

If, at this point in the algorithm, the updated potential depth of runoff, $DPTHRNFF_t$ in Equation (3.2.2.9), is still positive, it implies that the water table is already at the base of the soil column and no irrigation is required for this EAA grid cell. $DPTHRNFF_t$ will, indeed, leave the grid cell and the final ponding above land surface and final soil moisture in the soil column are computed using the following three equations:

$$SOLMX_t = SOLMX_t + POND_t - DPTHRNFF$$

$$POND_t = \max(SOLMX_t - SOLMDPH, 0.0) \tag{3.2.2.14}$$

$$SOLMX_t = SOLMX_t - POND_t \tag{3.2.2.15}$$

And the volume of excess water leaving the grid cell becomes:

$$VOL\_EXCESS\_WATER = (DPTHRNFF)(GDAR) \tag{3.2.2.16}$$

If, on the other hand, the updated potential depth of runoff, $DPTHRNFF_t$, is zero, it implies that: (1) ponding is zero; (2) irrigation may be required to bring the water up to the bottom of the soil column and/or maintain an equivalent depth of minimum moisture content SOLCRT in the soil column; and (3) the water table may still be below the base of the soil column (NOTE: SOLCRT $\leq$ SOLCRNF).

The irrigation requirement is calculated next. The total required storage depth for irrigation is:

$$\text{TOTAL\_DEPTH} = \text{GWMAXDP} + \text{DEPTH\_BELOW\_MIN} \qquad (3.2.2.17)$$

The first term in the above equation, GWMAXDP, represents the equivalent depth of water required to maintain the saturated zone. The second term, DEPTH_BELOW_MIN, is the equivalent depth of water required to maintain minimum moisture content in the unsaturated zone. It is calculated as:

$$\text{DEPTH\_BELOW\_MIN} = \max(\ \text{SOLCRT} - \text{SOLMX}_t\ ,\ 0.0)$$

By definition, the depth of irrigation requirement, DPH, is equal to the lesser value between the net theoretical crop evapotranspiration requirement, max(ETMX – RAIN_t,0), and the total required storage depth for irrigation.

$$\text{DPH} = \min[\ \max(\ \text{ETMX} - \text{RF}_t\ ,\ 0.0\ )\ ,\ \text{TOTAL\_DEPTH}\ ] \qquad (3.2.2.18)$$

The model assumes that irrigation brings the soil moisture content in the soil column (unsaturated zone) up to the minimum level SOLCRT before percolation occurs. Percolation, at this point in the discussion, is the process by which water is introduced below the soil column via irrigation in order to bring the water table 1.5 feet below land surface. Therefore, the anticipated increase in soil moisture in the unsaturated zone, after irrigation, will be equal to the lesser of values between the depth of irrigation requirement and irrigation required to bring the soil content in the soil column to equivalent depth SOLCRT:

$$\text{SOLMX}_t = \text{SOLMX}_t + \min(\ \text{DPH}\ ,\ \text{DEPTH\_BELOW\_MIN}\ ) \qquad (3.2.2.19)$$

Finally, anticipated percolation due to irrigation can be calculated as that portion of DPH in excess of DEPTH_BELOW_MIN:

$$\text{PERC\_IRRIG} = \max(\ \text{DPH} - \text{DEPTH\_BELOW\_MIN}\ ,\ 0.0\ ) \qquad (3.2.2.20)$$

For a given EAA grid cell, the volume of irrigation requirement is given by:

$$\text{VOL\_IRRIG} = (\text{DPH})(\text{GDAR}) \qquad (3.2.2.21)$$

3.2.3 Routing of Excess Runoff

The above calculations are done for all cells in each EAA basin. On any given day, a grid cell may either have excess water or irrigation requirement but not both. The total net excess volume of water for a given basin j is given by the formula:

$$NET\_EXCESS\_VOL_j = \sum_{i=1}^{nnodes_j} \left(VOL\_EXCESS\_WATER_i - VOL\_IRRIG_i \right) \quad (3.2.3.1)$$

where:

j = 1 for Miami Canal Basin;
= 2 for North New River/ Hillsboro Canal Basin; and
= 3 for West Palm Beach Canal Basin.

A positive total net excess volume of water for an EAA basin j is equal to what could potentially leave the basin. Thus, for a given time step, runoff from some cells are used to meet irrigation requirements in the other cells within the same basin and any net excess volume of water (potential excess runoff) can be routed out of the basin and into storage areas such as Lake Okeechobee and the Water Conservation Areas. The intrabasin transfer of the volume of excess water is not done based on the traditional channel routing or overland flow procedures but is performed by direct transfer of water. It is assumed that secondary and tertiary canal systems in the EAA have sufficient capacity to move this volume of water from appropriate cells into cells within the same basin that require irrigation within one time step.

In reality, the system may not be able to remove the entire net excess volume of water from a given EAA basin due to the following constraints:

1. Attenuation and lag effects in the secondary and tertiary canal systems cause actual excess runoff leaving a basin to be less than the potential excess runoff for the same day. Based on a comparison of simulated daily excess water with historical runoff from all EAA basins for the period 1983 through 1990, the actual excess runoff can be calculated as a fraction of the potential excess runoff which, in turn, is equal to the net excess volume calculated in Equation (3.2.3.1). In effect:

actual excess runoff = (FRACT)(NET_EXCESS_VOL) (3.2.3.2)

The reduction factor, FRACT, is a fraction that varies with the magnitude of potential excess runoff.

2. The design capacity of outlet structures limits the amount of excess runoff that can be removed from an EAA basin. Table 3.2.3.1 shows the operational constraints used in removing excess runoff for each EAA basin on a daily basis as implemented in the SFWMM. The empirical equations in the table are a result of a statistical analysis of available flow records for the major EAA structures.

Rotenberger Tract and Holey Land, although part of the Miami Canal Basin, are separated from the irrigated areas by levees, and are treated as separate basins in the model. Any net runoff in excess of structure design capacities is returned uniformly to all grid cells within the appropriate basin. Currently, interbasin transfers of runoff within the EAA through the Cross and Bolles Canals are not simulated in the model.

Table 3.2.3.1 Operational Constraints Used in the SFWMM for Removing Excess Runoff from EAA Basins

EAA Basin	Flood Control Back Pumping (BP) to LOK	Routing of Remaining EAA Runoff
Miami Canal Basin	BP = 80% of 7-day running mean daily runoff from basin in excess of 3200 cfs Note: Back Pumping is done through S-3. (S-3 capacity* = 2,600 cfs).	A maximum daily rate of 750 cfs to Holey Land, depending on Holey Land's stage relative to its schedule. The remainder goes to WCA-3A through S-8. (S-8 capacity* = 4,200 cfs).
North New River-Hillsboro Canal Basin	BP = 80% of 7-day running mean daily runoff from basin in excess of 4500 cfs Note: Back Pumping is done through S-2. (S-2 capacity = 3,600 cfs).	10% of runoff goes through S-150 into WCA-3A; 50% of runoff goes through S-7 into WCA-2A (S-7 capacity = 2,500 cfs); and 40% of runoff goes through S-6 into WCA-1 (S-6 capacity* = 2,900 cfs)
West Palm Beach Canal Basin	None	100% of runoff goes through S-5A pumps into WCA-1 (S-5A capacity = 4,800 cfs)

* rounded-off to the nearest 100 cfs

Meeting Irrigation Requirements

If the total net excess volume of water for any EAA basin is negative, then an irrigation requirement for the basin has to be met from storage areas outside the basin. Currently, only Lake Okeechobee is used to meet irrigation requirements in the EAA. Deliveries to meet irrigation requirements are limited by conveyance capacities of the primary canals in the EAA. Likewise, water shortage policies as outlined in Section 3.3 may be imposed during periods of low Lake levels. Any irrigation requirement not met, due to conveyance limitations and/or limits set by management policies, will result in a uniform reduction in water levels for all grid cells in the appropriate destination EAA basin(s). On a given day, all EAA basins may not have irrigation requirements simultaneously. The discussion of EAA canal conveyance is given next.

EAA Canal Conveyance

Deliveries from Lake Okeechobee through the infrastructure in the EAA to the Everglades and/or LEC are subject to constraints, as discussed in the following sub-sections.

Downstream Constraints in the Everglades. If stages in the Everglades are sufficiently high that releases from Lake Okeechobee could do further harm, releases are discontinued. The conditions for which releases from the Lake for environmental water supply or flood control are discontinued are dictated by the simulated management criteria for both the Lake and the EPA. Examples of such constraints would be stage-based rainfall driven operation targets for the EPA (to be discussed in Section 3.4) and checks against criteria as outlined in Part 1 of the WSE decision tree operations for Lake Okeechobee (as shown in Section 3.1).

Conveyance Constraints on Releases from Lake Okeechobee to Everglades and/or LEC. In the EAA, canal constraints are not just a function of design capacities and hydraulic conductivities, but also a function of day-to-day operational concerns. An analysis of historical flows through the major EAA canals (Miami, North New River, Hillsboro and West Palm Beach) reveals that the actual amount of regulatory flows released from the Lake and the actual magnitude of agricultural runoff removed from the EAA were rarely close to the design capacity of the canals (Trimble, 1995b). In order to establish realistic allowable flows through these canals consistent with historical data, a seasonal average percentage of design discharge (Q_{design}) is used to define each EAA canal conveyance capacity in the model (Table 3.2.3.2). Due to the nature of wet season rainfall which often occurs in sudden heavy outbursts, the percentages associated with the wet season are stricter than those for the dry season. Lateral inflows (runoff from EAA basins) are pumped as necessary into the major canals from farm-scale pumps. Although the lateral inflows are greater during the wet season, they also occur in dry seasons. The values shown in Table 3.2.3.2 are reevaluated from time to time by analyzing more recent historical flow data at the major inlet and outlet structures in the EAA.

Table 3.2.3.2 Allowable Percentage of Design Discharge through the Major EAA Conveyance Canals

EAA Conveyance Canal	Q_{design} [cfs]	Dry Season Percentage	Wet Season Percentage
Miami Canal	2,000	75%	50%
NNR-Hillsboro Canal	2,400	80%	50%
West Palm Beach Canal	950	65%	50%

There are several type of uses for the canal conveyance and a priority has been established where canal constraints are limiting factors. The priority of flow volumes in using EAA canal/structure conveyance is as follows:

1. EAA basin runoff/demand;
2. Water supply deliveries to STAs;
3. Runoff from 298 drainage districts;
4. Water supply to Big Cypress Seminole Reservation and Holey Land WMA;
5. Environmental (rain-driven) water supply to Everglades and water supply to the LEC;
6. BMP Makeup water;
7. Excess water to proposed reservoirs, if applicable;
8. Regulatory releases from Lake Okeechobee to WCAs.

Conveyance for the major EAA canal systems for flow through are calculated each time step based upon the HEC-2 look up tables for the "neutral case" condition (USACE, 1990). The neutral case refers to the flow through capacity during no lateral flow conditions (no runoff *and* no demand) within the EAA. Given an EAA conveyance canal with upstream and downstream controls, there exists a unique combination of upstream stage, downstream stage and canal profile that responds to the maximum flow of water from the source (LOK) to the destination (WCA or STA). The maximum headwater stage in the canals for flow through releases from Lake Okeechobee to STAs, WCAs and/or LEC is assumed to be 12.0 ft NGVD.

The percentages from Table 3.2.3.2 are then applied to Lake water pass-through/flow-through calculations in the following manner. During the wet season, when Lake stage is above regulation, the maximum amount of water Q_{max} that can be released from the Lake and delivered south to the WCAs via EAA canals can be calculated as

$$Q_{max} = \min[\text{neutral\_case}, (\text{percent\_wet})(\text{design\_discharge})] - (\text{runoff} + \text{existing flow from LOK}) \tag{3.2.3.3}$$

Flow calculations for the neutral_case are defined a little later in this section. Flow-through capacity during water supply conditions, on the other hand, can be defined as

$$Q_{max} = \min[\text{neutral\_case} - (\text{demand} + \text{existing flow from LOK}), (\text{percent\_wet})(\text{design discharge})] \tag{3.2.3.4}$$

During the dry season, two other empirical relationships can be defined for regulatory release and water supply release conditions:

$$Q_{max} = \min[\text{neutral\_case} - (\text{runoff} + \text{existing flow from LOK}), (\text{percent\_dry})(\text{design\_discharge})] \tag{3.2.3.5}$$

and

$$Q_{max} = \max\{ \text{neutral\_case} - [\text{demand} + (\text{existing flow from LOK})], 0.0 \} \tag{3.2.3.6}$$

It must be emphasized that the above formulas for computing maximum allowable flows through the major EAA conveyance canals are empirical in nature. They reflect the field operators' preferences as they adapt to real day-to-day hydrologic conditions. Therefore, Equations (3.2.3.3) through (3.2.3.6) include the subjectivity involved in operating major structures in the EAA.

The neutral_case refers to the pass-through/flow-through capacity during no lateral flow conditions (no runoff and no demand) within the EAA. Given an EAA conveyance canal with upstream and downstream controls, e.g. S-354/Miami Canal/S-8, there exists a unique combination of upstream stage (S-354_HW), downstream stage (S-8_TW) and canal profile (along the Miami Canal) that corresponds to the maximum flow of water from the source (Lake Okeechobee) to the destination (WCA-3A). To determine the maximum flow for each major canal reach, a steady-state backwater analysis was conducted (Gee and Jenson, 1995) and the rating curve information was identified for all types of configurations structures that would occur for the same canal reaches. Then a separate solution routine was written for canals in the EAA where neutral case conveyance calculations are performed. Figure 3.2.3.1 shows all types of configurations where neutral case conveyance calculations are performed in the model. Italicized words refer to the specific program subroutines or functions that perform the calculations. For example, given Lake stage and S-8 pump headwater, a subroutine solving the configuration like Figure 3.2.3.1(c) would be executed when the pass-through discharge along the existing Miami canal is required. If the canal configuration is modified to include an intervening diversion structure (e.g. STA3/4 flows) along the Miami canal, then the subroutine solving the configuration like Figure 3.2.3.1(d) would be executed. The key assumption in this approach is that a known water surface profile provides a unique discharge through a specific canal reach-structure configuration. Since the model is not concerned with what happens internally within the EAA, specification of headwater (Lake stage) and tailwater (downstream of EAA) conditions is sufficient to determine neutral_case flows. The model adjusts the headwater and tailwater conditions at appropriate canal reaches and intermediate structures in response to runoff or demand conditions in the EAA.

In summary, the neutral_case (no-runoff or no-demand condition) discharges or conveyance capacities are obtained in the model as a series of look-up tables generated from multiple HEC-2 runs for each canal, covering a wide range of flows, and upstream and downstream stages. Table 3.2.3.3 lists some properties of the nine EAA canal reaches where look-up tables were generated for and used in calculating conveyance capacities through the EAA.

Table 3.2.3.3 Some Physical Properties of the Eight EAA Canal Reaches Used in Calculating Conveyance Capacities through the EAA

EAA Canal	Upstream Reference Stage	Downstream Reference Stage	Length [mi]
Miami	LOK stage	S8_TW	26.2
North New River	LOK stage	S7_TW	28.6
Hillsboro	S351_TW	S6_HW	23.7
West Palm Beach	S352_TW	S5A_HW	20.8
Miami* (upper reach)	S354_TW	S8NEW_HW	19.3
North New River* (upper reach)	S351_TW	S7NEW_HW	24.6
North New River* (lower reach)	S7NEW_TW	S7_HW	4.0
Miami* (lower reach)	S8NEW_TW	S8_HW	6.9

*Refers to future base scenario with proposed Stormwater Treatment Areas in operation, and the Miami and North New River canals are both split into upper and lower reaches. NOTE: Variables in parentheses are known or fixed values.

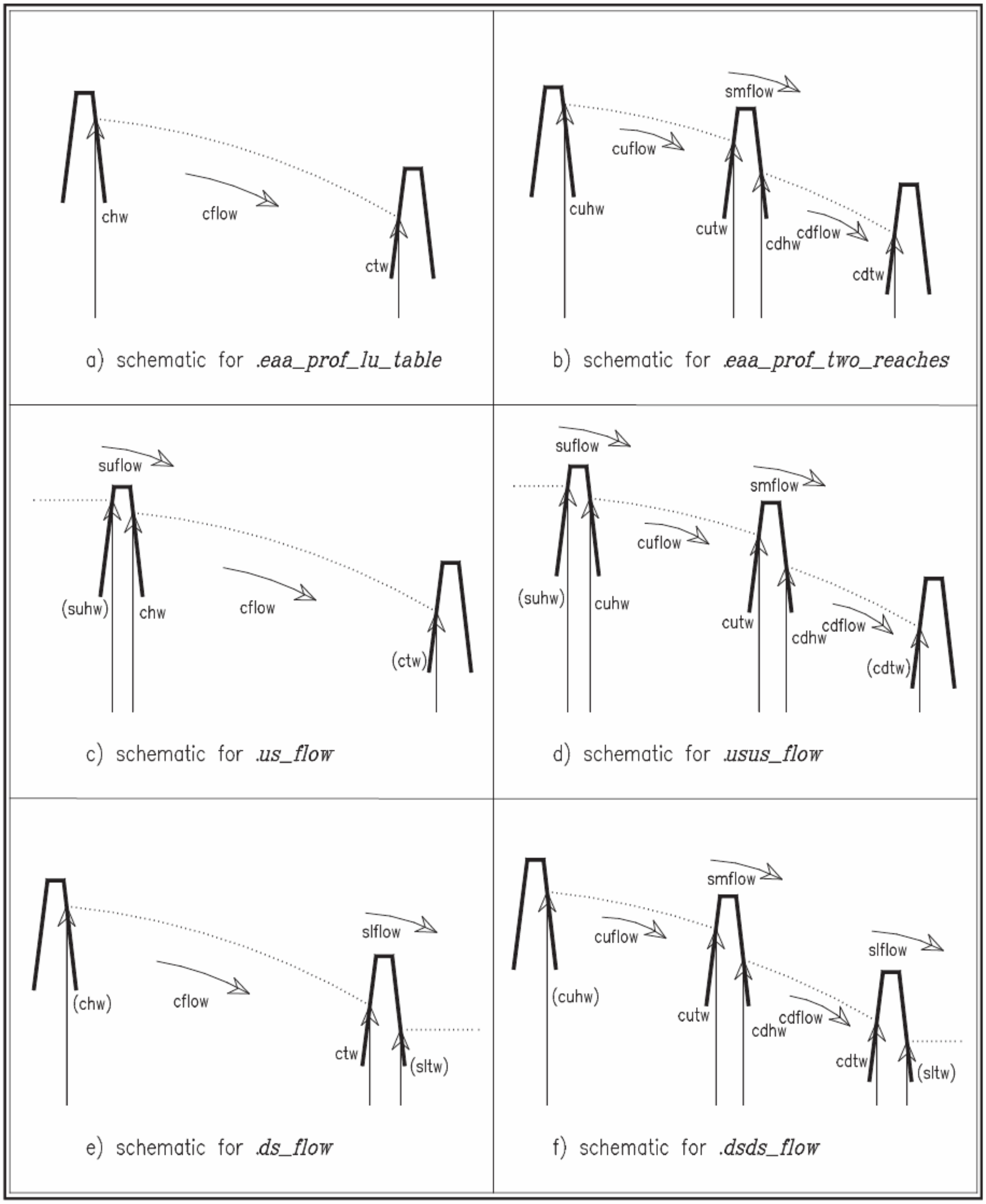


Figure 3.2.3.1 Canal-Structure Configurations Used in Calculating Canal Conveyance Capacities for the EAA Algorithm in the SFWMM

Conveyance Constraints on Water Supply Releases to Everglades and/or LEC. When making water supply deliveries, there are additional operational concerns. The following discussion refers to the operations related to environmental deliveries (refer to Section 3.4) with a full build-out of the STAs. Deliveries for Best Management Practices and makeup water are handled separately from the water supply deliveries. Environmental releases made from Lake Okeechobee (LOK) through the major EAA canals for water supply purposes are two fold: To deliver water to the Everglades (WCAs/ENP) when stages at specific target locations are sufficiently low; and to deliver water to LEC service areas when canal stages in WCAs are at or below floor elevation or marsh stages (if applicable) at specific locations in WCAs are below criteria for minimum flows and levels.

The maximum possible water supply release from Lake Okeechobee through each major EAA canal is defined by the following:

$$max\_flow\_through\_ws = \max[\,(CNCC)(CF) - (other\_flows)\,,\,0.0]$$

where:

CNCC = current neutral case capacity based on HEC-2 lookup tables;
CF = EAA canal conveyance multiplier (1.0 represents the current system, greater than 1.0 represents increased capacity); and
other_flows = EAA_canal_basin_runoff_or_demand + LOK_ws_to_sta + LOK_ws_to_Big_Cypress_Seminoles + LOK_ws_to_Holeyland_WMA

Decisions must be made in the SFWMM in the way demands are being met when demands on Lake Okeechobee exist in both the Everglades and the LEC service areas. Decisions are made on volumes of water treated by STAs that go to meet environmental needs in the Everglades and volumes of water untreated that go directly to meet LEC demands. The flowchart shown in Appendix F2 depicts the flexibility in the SFWMM for managing Lake Okeechobee releases and/or EAA basin runoff in meeting Everglades and/or LEC service demands.

In order to establish realistic allowable flows through these canals consistent with historical data, a seasonal average percentage of design discharge (MFC) is used as an additional limit to conveyance capacity. If needs exist in the Everglades, the maximum flow to the Everglades from Lake Okeechobee is as follows:

1. During wet season and when runoff from EAA canal basin is greater than zero:
   $$max\_LOK\_to\_Glades = \max\{\,\min[\,(CNCC_{PSTA})(CF)\,,\,oper\_capac] - other\_flows\,,\,0.0\}$$
2. During the dry season or when runoff from EAA canal basin is zero:
   $$max\_LOK\_to\_Glades = \min(CNCC_{PSTA} - other\_flows\,,\,oper\_capac)$$

where:

$CNCC_{PSTA}$ = neutral case capacity of canal when water is pumped into STA since the water going to meet environmental needs is treated by STA oper_cap:

a. Under current conditions (with CF = 1.0):
   $$oper\_capac = (CNLD_{cap})(MFC)$$
b. For future proposed conditions (with CF > 1.0):
   $$oper\_capac = (CNLD_{cap})(MFC) + [\,CNLD_{cap}\,(CF - 1.0)\,]$$
   The additional capacity due to increased conveyance goes to meet Everglades needs (MFC is not applied to this additional capacity);

$CNLD_{cap}$ = current design capacity for canal system;
MFC = maximum fraction of current design capacity delivered from LOK to Everglades.

If there are LEC service area demands only, then the release from Lake Okeechobee is as follows:

flow_through_ws = min(max_flow through_ws , LEC service area demands met by LOK)

3.2.4 L8 Basin, S236 Basin and 298 Districts

The discussion to this point of irrigation requirements and runoff routing within the distributed mesh portion of the SFWMM has focused on the primary EAA basins. A similar methodology is applied to the L8 and S236 Basins and to the 298 Districts (also known as Water Control Districts), located on the southeastern rim of Lake Okeechobee (refer to Figure 3.2.1.1). These basins follow the same methodology for estimation of net supplemental irrigation requirement and excess runoff as that previously outlined in this section. All three of these basins can receive water supply from Lake Okeechobee. Runoff routing options are handled differently, however. Excess water from the L-8 Basin can be sent to Lake Okeechobee or to the S-5A complex on the northern edge of the Everglades Protection Area where it can be diverted into either WCA-1 or LECSA-1. The S236 Basin runoff can be directed either into the Lake or to the Miami canal if Stormwater Treatment Area 3&4 is being simulated (additional detail provided in Section 3.2.5). For the 298 districts, the majority of runoff is returned to the Lake. Additional options exist within the model to redirect fractional contributions of runoff into the appropriate canal basins (West Palm Beach, North New River, and Miami River Canals) as shown in Table 3.3.8.1. These options are used in routing water associated with operational criteria associated with the Everglades Construction Project (ECP).

Table 3.2.4.1 Fractional Contributions of 298 Districts to Major EAA Canals

298 District Name	Pump Station	Receiving Canal	Max pump size [cfs]	Fraction of 298 Total RO*
South Shore Drainage Dist.	SSDD	Miami	178	19%
South Florida Conservation Dist.	SFCD P5E	Miami	120	16%
East Beach Water Control Dist.	EBWCD #3	West Palm Beach	338	36%
East Shore Water Control Dist.	ESWCD PS2	Hillsboro	439	29%

* Remaining fraction of Total RO flows into Lake Okeechobee

3.2.5 Everglades Agricultural Area Reservoirs and Storage Components

Water-holding facilities or reservoirs serve a variety of functions within the EAA. The Holey Land can be considered as an above-ground reservoir that acts as a wetland preserve. Additional examples of above-ground reservoirs in the EAA are the Stormwater Treatment Areas (STAs) whose function is to improve the water quality of runoff generated from the EAA as well as releases from Lake Okeechobee. Proposed EAA Storage Reservoirs are examples of above-ground reservoirs which are intended to store Lake water or EAA runoff for later use. These uses include: 1) to meet EAA water supply needs (primarily irrigation) during drier times within the EAA and 2) to pass Lake Okeechobee regulatory flows to the EPA. The Holey Land and partially constructed STA reservoirs currently exist in the EAA, while design and construction work on EAA Storage is underway.

In general, the means by which reservoirs are modeled is discussed in Section 3.6. The following text will describe how the STA storage features are handled in the vicinity of the EAA as a means of showing the capabilities of the SFWMM. The objectives of STAs (Figure 3.2.5.1) are summarized as follows:

1. To reduce long-term average concentration of total phosphorus from EAA runoff to the Everglades Protection Area to an ultimate goal of 10 ppb.
2. To restore the hydroperiod in the northern areas of WCA-2A and WCA-3A.
3. To increase quantity and improve quality of water retained in the Everglades system through redirection of runoff from C-51W Basin.
4. To restore the hydroperiod in the Rotenberger Tract with water of suitable quality.
5. To reduce localized water quality problems in Lake Okeechobee associated with discharges from special drainage districts adjacent to the Lake such as the 298 Districts.

In order to illustrate the level of complexity that can be obtained using the model, Figure 3.2.5.2 shows a schematic of how the SFWMM depicts the operation of the system within and around the EAA area after all proposed STAs are in place (circa 2010 Base Condition).

The specific operation of STAs within any given model simulation will vary with other options modeled (e.g. Rain Driven Operations where optimal environmental deliveries to the EPA are considered). However, the general assumptions used in implementing STAs in the SFWMM are:

1. A mass balance approach using minimal input data is used in calculating discharge in and out of STAs. These discharges are subject to structure and canal conveyance capacity constraints.
2. EAA Best Management Practices when included are simulated by increasing the upper limit of the soil moisture storage in the unsaturated zone for the cells in the EAA. This maximum is determined by trial and error.
3. STAs can be treated as multi-compartment reservoirs.
4. In general, the operational water depths are as follows: minimum depth = 0.5 ft; desired mean depth = 2.0 ft; depth at which outflow begins = 1.25 ft; and maximum depth = 4.5 ft.
5. Water supply releases from Lake Okeechobee to LEC bypass STAs and are, thus, untreated.
6. Inflows vary by location and condition. All inflows are subject to canal conveyance capacities and/or structure capacities.

A summary of the general operating considerations for STAs in the EAA is given in Table 3.2.5.1.

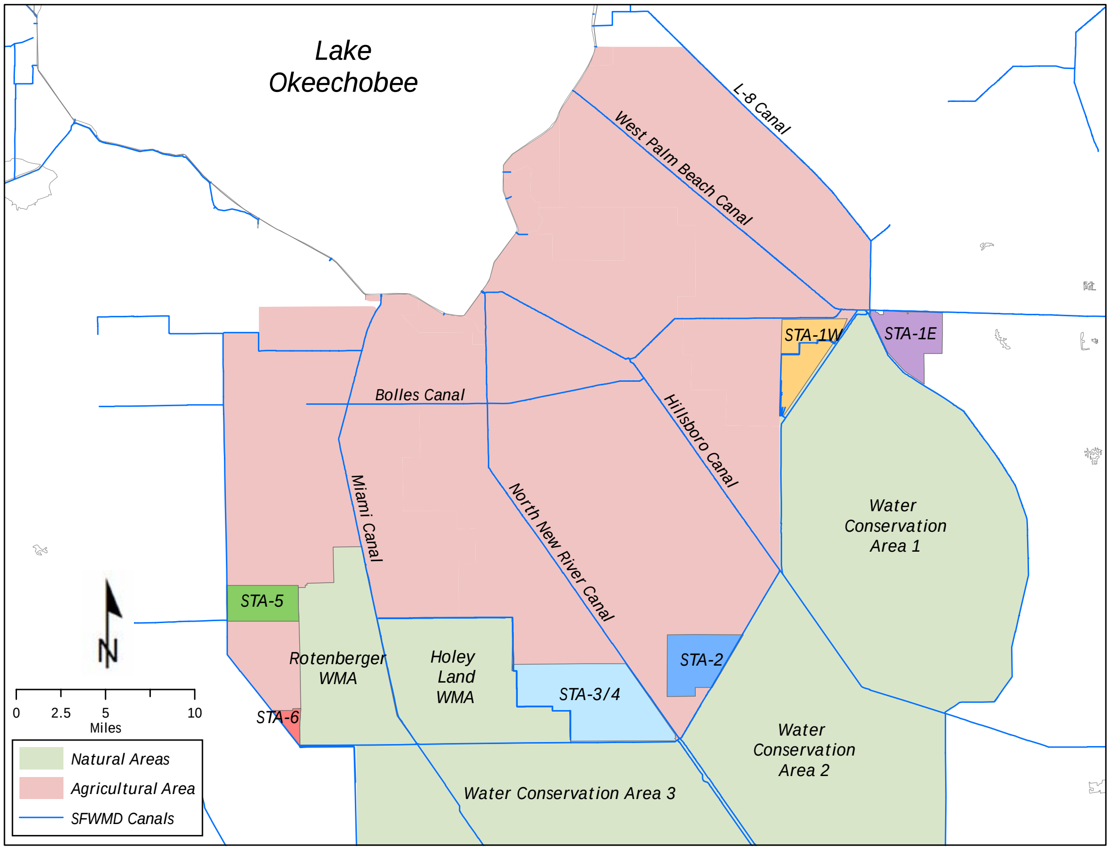


Figure 3.2.5.1 Location of Stormwater Treatment Areas

Table 3.2.5.1 General Operating Considerations for STA-type Reservoirs in the EAA Simulation within the SFWMM

Purpose	Source of Water	Rule for Outflow
Stormwater treatment to reduce phosphorus loading into Everglades Hydroperiod enhancement in WCAs by improvement of volume, timing, and distribution of flow to the Everglades	EAA or other basin runoff LOK regulatory releases LOK environmental water	Regulate outflow such that average depth of water in the Stormwater Treatment Area is approximately equal to 1.25 to 2.0 ft

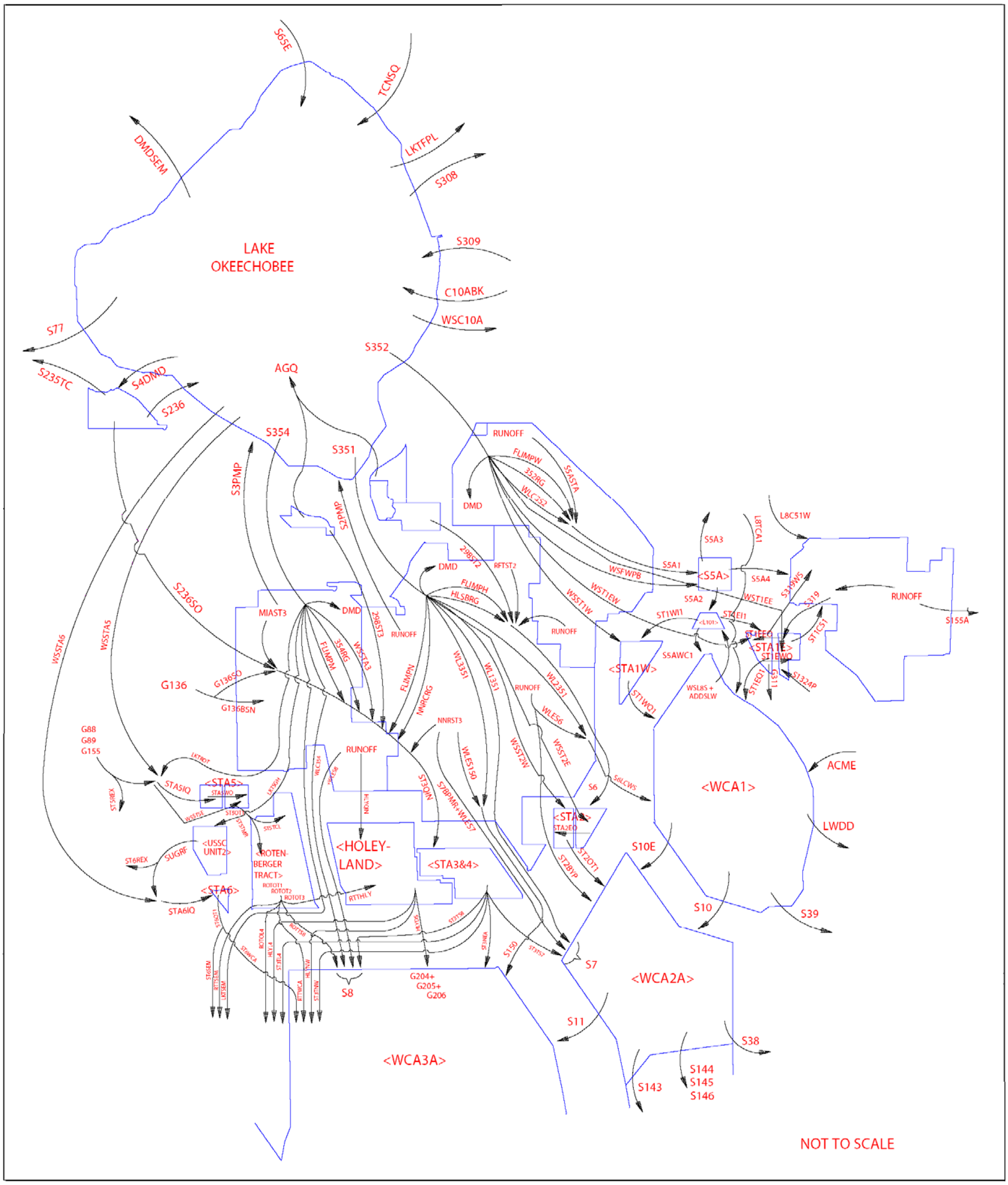


Figure 3.2.5.2 Flow Distribution within and Around the EAA with STAs Fully Constructed

Two options exist in the SFWMM that affect the volume of water treated in STA-3&4, STA-2 and STA-1W. These options refer to the way demands are being met in the Everglades and urban areas. The operations of Lake Okeechobee, EAA, Water Conservation Areas, and Lower East Coast are closely related. Although this section focuses on the EAA, a discussion of some operational rules applicable to the WCAs as well as the Lower East Coast may be necessary at this point in order to explain various options in the model. These options are:

1. "No Priority" Option:

   Under this option, the Everglades will receive (for environmental restoration purposes) all available EAA runoff ahead of the Lower East Coast (for water supply purposes) by virtue of the Everglades' closer proximity to the EAA. The amount to be delivered to the Everglades is limited by the canal conveyance capacities within the EAA as well as operational constraints associated with intervening retention/detention areas such as STAs, if any. Of course, such deliveries will only occur in the model if some stage (or flow) targets are defined by the user for the Everglades; otherwise, all available EAA runoff will be used to meet water supply needs in the LEC.

   The first source of water that meets LEC demands are the Water Conservation Areas. If the runoff generated from the EAA exceeds the remaining LEC demands after the appropriate Water Conservation Area has made its release, all EAA runoff is pumped into the appropriate STA, subject to conveyance constraints. EAA runoff in excess of the STA pump capacity and conveyance capacities within the EAA bypasses the STAs, remains untreated, and is still routed south to alleviate flooding within the EAA.

   If the runoff generated from the EAA is less than or equal to the remaining LEC demands, i.e. after the appropriate WCA has made its release, all EAA runoff bypasses the appropriate STA and is subject to EAA conveyance constraints. Water sent south to meet LEC Service Areas demands is all untreated.

2. Everglades/LEC Priority Option:

   In this option, the user specifies a fraction, FRCT, of the total volume of water available from EAA runoff that will be used directly, i.e., untreated, to meet LEC service area demands as required. This fraction can range from 0.0 to 1.0; environmental demands get priority with a fraction equal to 0.0 while LEC service area demands get priority with a fraction equal to 1.0. In general, what bypasses the STAs and meets LEC service area demands equals FRCT multiplied by the total available water. Conversely, what gets treated by the STAs and meets environmental demands equals (1.0 – FRCT) multiplied by the total available water.

If an STA and a non-STA reservoir both exist in same EAA basin, the model assumes that the non-STA reservoir receives excess runoff/Lake Okeechobee regulatory releases first; the remainder of the excess water goes to the STA reservoir for treatment.

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3.3 LAKE OKEECHOBEE SERVICE AREA

3.3.1 Introduction

The Lake Okeechobee Service Area (LOSA) basins that are modeled as lumped basins in the SFWMM are simulated in a very different manner than that used for the gridded portions of the model (refer to Section 3.2). This section will detail the water budget approach utilized in the SFWMM by describing the implementation of the AFSIRS/WATBAL pre-processing tool (Wilcox and Novoa, 2003b). Additional topics in this section include special considerations on a basin-by-basin basis for the various non-EAA LOSA areas and an overview of the regional supply-side management policy that applies to the entire LOSA.

The AFSIRS/WATBAL model is used for LOSA in order to provide a consistent means of estimating supplemental irrigation requirements and excess runoff for the portions of the South Florida System that are not part of the distributed mesh portion of the SFWMM and subject to the Lake Okeechobee Supply Side Management protocol. The use of AFSIRS/WATBAL in this role is considered to be appropriate for several reasons including:

1. AFSIRS/WATBAL has been successfully applied to basins in the LOSA in previous efforts (e.g. Caloosahatchee Water Management Plan, 2000);
2. The model outputs of daily supplemental demand and runoff are consistent with the required inputs to the SFWMM;
3. Input data for running AFSIRS/WATBAL, including climate data, land use, soil data, etc. is available or can be readily estimated; and
4. Model run-times are short enough to allow for modeling long-term periods of record (36 years).

3.3.2 AFSIRS/WATBAL Model Overview

In the SFWMM, a consistent modeling approach is used to estimate lumped basin demands and runoff in all non-gridded portions of the SFWMM (refer to Figure 3.2.1.1). The AFSIRS/WATBAL model is the pre-processing tool used for this task. The model was developed for the Caloosahatchee Water Management Plan (CWMP) to estimate basin-scale, current and future water demand, and runoff (SFWMD, 2000). The model is based on and built around the Agricultural Field Scale Irrigation Requirements Simulation (AFSIRS) model (Smajstrla, 1990). A short discussion of AFSIRS is presented in Appendix S. The generalized approach of this tool applies a water budget methodology to determine the "edge-of-basin" impact of a lumped area on the regional system. The primary components of the hydrologic budget including rainfall, ET, internal basin transfers and storage change (both in the soil column and detention storage) are all considered. The combined influences of these components are then translated on a daily basis to a net basin-scale runoff (source) or demand (sink) term, which is accepted by the SFWMM.

As an illustration of the AFSIRS/WATBAL modeling concept, consider an irrigated field in which soil moisture is at field capacity and no other local storage (ditches, etc.) outside of soil storage exists. In the successive time step, if rainfall occurs, this will be translated (after meeting local crop ET needs) to a volumetric discharge at the edge-of-field and will be resolved as “runoff”. The excess water leaving the field would no longer be available within the control volume, but would impact the adjacent area (the regional system). Likewise, if there is no rainfall during the successive time step, crop ET will result in a depletion of water in the soil column. In order to maintain optimal plant yield, this deficit will be made up by pumping water from outside of the control volume into the field. This practice would be resolved as “demand” (a water-supply sink from the regional system). This simple example can be assumed to occur in several individual fields throughout a basin. The interactions between these individual fields takes place through a series of interconnected canals and detention areas in which carryover storage, transmission losses and incidental irrigation all become important factors in the water budget. All of these field scale and basin scale features are considered in the AFSIRS/WATBAL model. To further illustrate the tool’s design, a conceptual diagram of the field-scale process representation, as applied in AFSIRS/WATBAL, is shown in Figure 3.3.2.1. The conceptual model for how individual field scale land uses is translated into basin scale demand and runoff is provided in Figure 3.3.2.2.

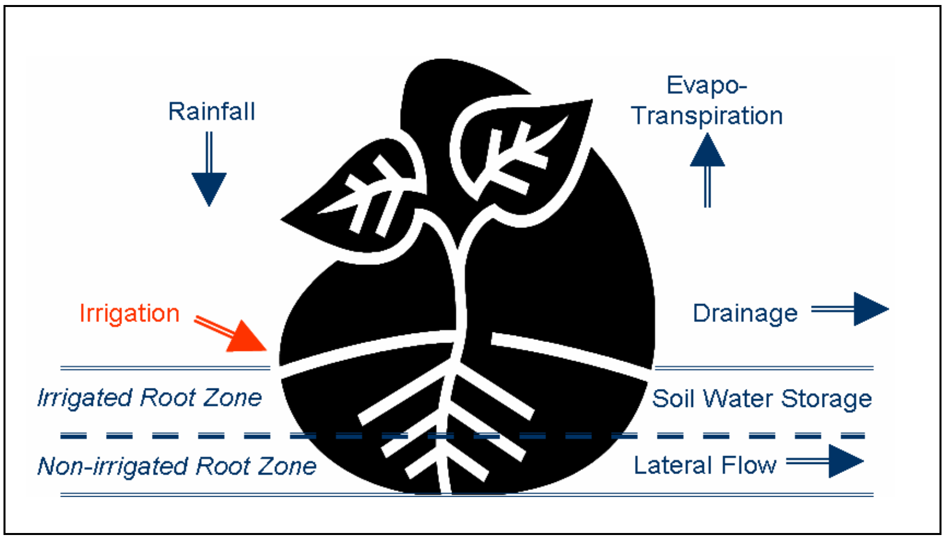

Figure 3.3.2.1 AFSIRS/WATBAL Conceptualization at Field Scale

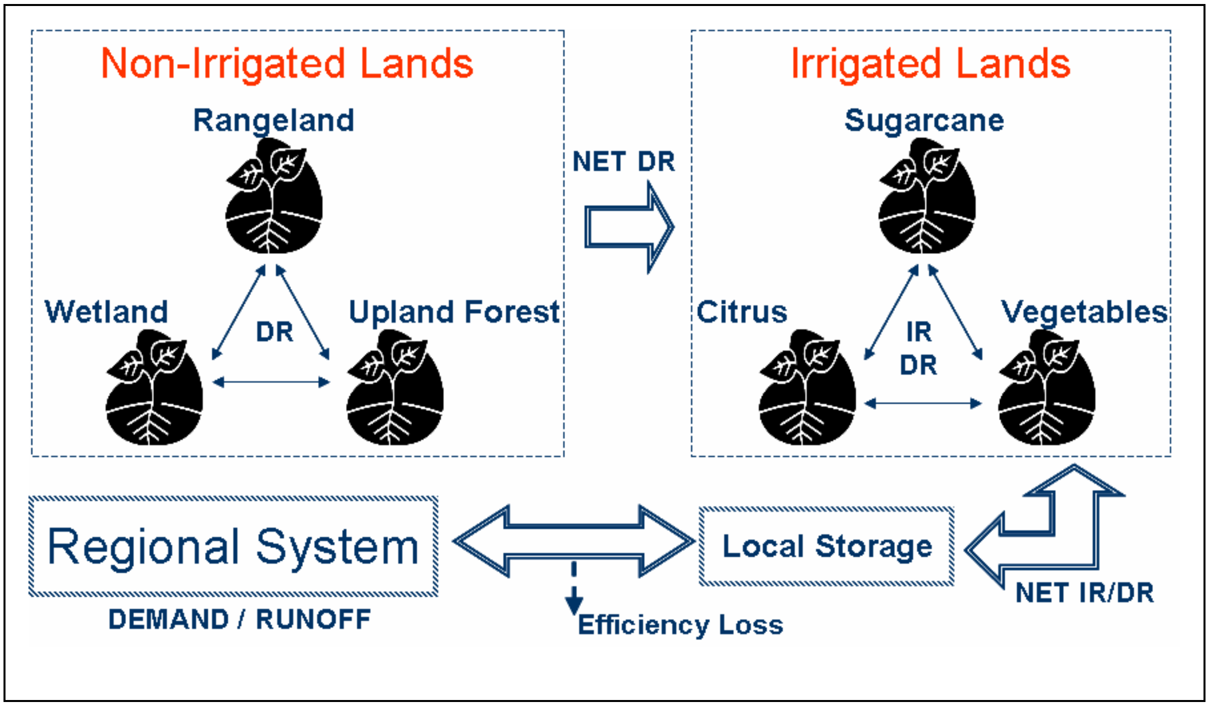

Figure 3.3.2.2 AFSIRS/WATBAL Conceptualization of Field Scale to Basin Scale Translation (DR = Field Scale Drainage, IR = Field Scale Irrigation)

The water budget equation for Figure 3.3.2.1 is:

$$\Delta Sto = Rain + IRR - ET_c - DR - LF \qquad (3.3.2.1)$$

where:

ΔSto = change in soil moisture;
Rain = effective rainfall;
IRR = irrigation requirement, including crop-specific efficiency loss;
ET_c = total ET for a particular crop types;
DR = drainage from the soil column; and
LF = lateral flow groundwater lost from root zone (assumed to be zero).

The drainage term, although illustrated in a manner that implies surface runoff in Figure 3.3.2.1, is in fact a quantification of the excess water that leaves the root zone. The physical methods by which this may occur include surface runoff, ditch or local storage capture and groundwater recharge. Recalling that AFSIRS on the field scale is a water budget accounting of the root zone, this drainage term is accounted as excess water and is treated as a loss term, regardless of destination or transmission means. The lateral flow is assumed to be zero since unsaturated zone flows are negligible and saturated zone flows are highly variable depending on local conditions. The inclusion of efficiency in the irrigation term is assumed to account for any lateral flows not considered. During wet periods, the soil moisture will increase to a point (SMAX) where rain, in excess of crop ET, will become runoff. During dry times, the soil moisture will decrease to a point (SMIN) were irrigation, supplemental to any rainfall, is required to meet the crop ET.

The AFSIRS/WATBAL water budget modeling for a given basin has three primary components (Figure 3.3.2.3): AFSIRS, WATBAL and AFSIRS Water Budget, as well as a central location for common data (RF_PET_LU_inputs). AFSIRS calculates irrigation requirements for cropland. The AFSIRS Water Budget spreadsheet was developed to calculate and route runoff and groundwater components for AFSIRS. The WATBAL spreadsheet calculates the hydrology of nonirrigated land. Further details related to each of these components are available in the appendix to the Caloosahatchee Water Management Plan (CWMP). Depending on whether the model is applied as a single basin implementation or a multiple basin implementation, additional complexity can be added in the form of additional spreadsheets to control the routing from one basin to another.

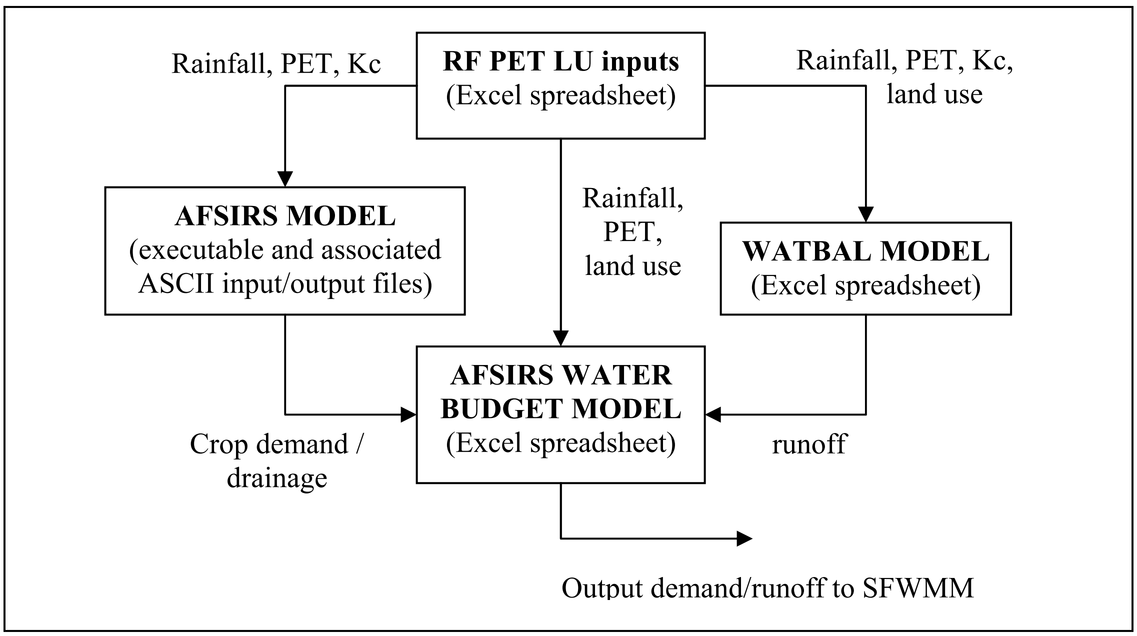

Figure 3.3.2.3 Single Basin Implementation of AFSIRS/WATBAL

3.3.3 Caloosahatchee and S4 Basins

The Caloosahatchee implementation of the AFSIRS/WATBAL model is conceptualized as a four basin model covering the lands between S-77/S-235 and S-79 that influence the regional system. These basins are defined as East Caloosahatchee-groundwater irrigated (ecal-gw), East Caloosahatchee-C43 irrigated (ecal-d), West Caloosahatchee-groundwater irrigated (wcal-gw), and West Caloosahatchee-C43 irrigated (wcal-d). The break between the "East" and "West" basins occurs at S-78. As previously mentioned, the multi-basin conceptualization of the model requires the addition of spreadsheets to handle the routing between basins. In addition to this need, the Caloosahatchee Basin has the supplementary consideration of public water supply withdrawals from the Caloosahatchee Canal (Lee County and Fort Meyers) and deliveries from the regional system [Lake Okeechobee, Caloosahatchee reservoir, ASR, etc.] to supplement agricultural and public water supply withdrawals. The final model conceptualization accounting for all of these considerations is presented in Figure 3.3.3.1. Calibration results for this basin are presented in Section 4.3.

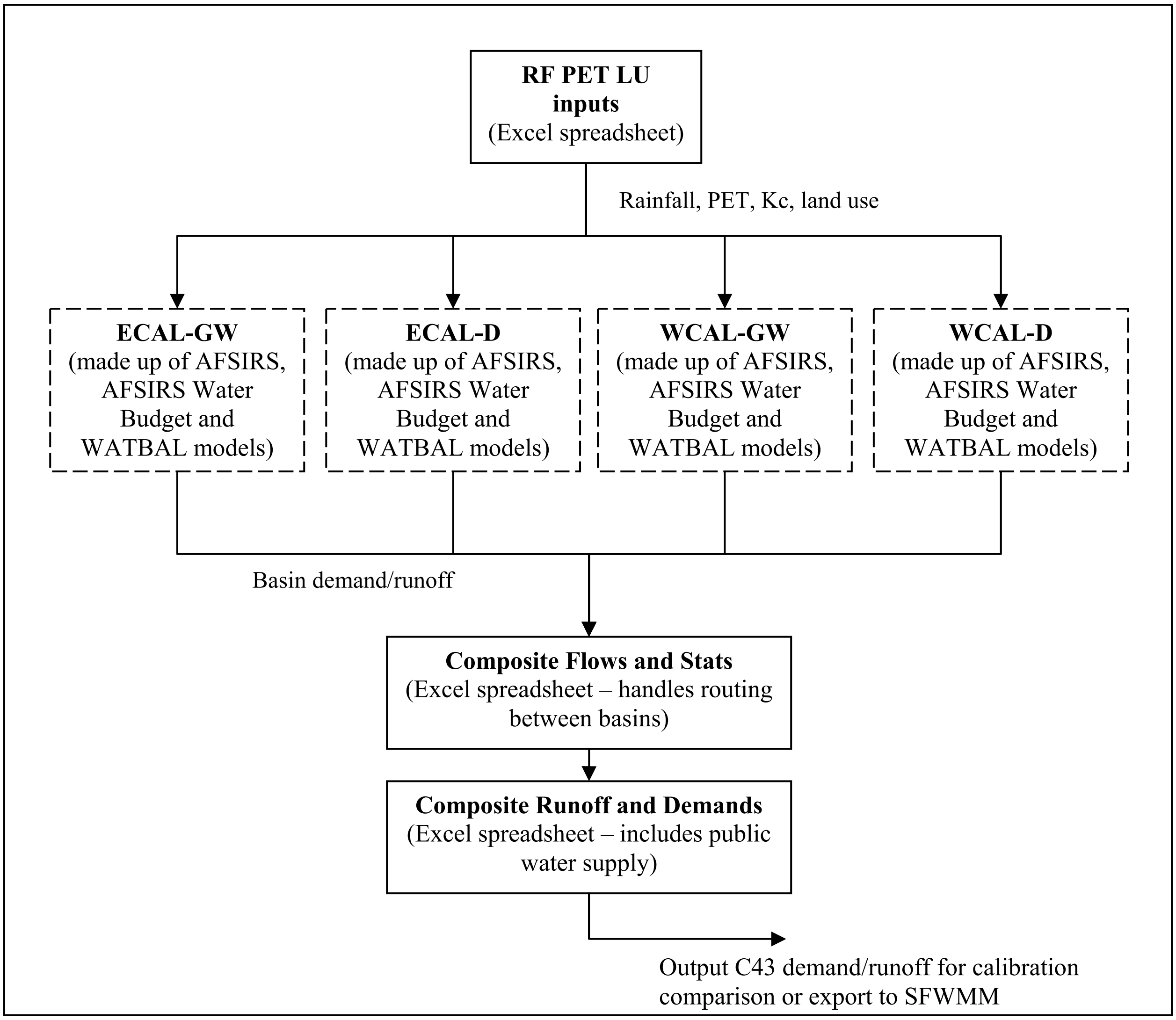

Figure 3.3.3.1 Caloosahatchee (C-43) Basin Implementation of AFSIRS/WATBAL

In the SFWMM, the S4 Basin is treated in a manner similar to the Caloosahatchee – as an external "bucket" to Lake Okeechobee. An additional level of complexity is also added due to the fact that a physical connection exists between the S4 and Caloosahatchee Basin via the S235 structure and the 9-mile canal (to Lake Hicpochee). In order to give users the flexibility to model impacts due to these connections, the S4 Basin is modeled as the combination of two separate basins: S4_Diston (portion of S4 Basin that has a physical connection to the Caloosahatchee Basin) and S4_Other. Input options give flexibility in modeling the interaction between the S4 Basin and the Caloosahatchee Basin via both S235 and the 9-mile canal, allowing the user to input appropriate routing and conveyance limitations depending on the scenario to be modeled.

As illustrated in Section 3.1, the SFWMM has the capability to simulate proposed storage features in the C-43 Basin. These components interact with the demand/runoff values generated by AFSIRS/WATBAL to help simulate regional routing of basin water. Excess runoff can be captured by the C-43 reservoir and then later released to meet water supply needs in the basin or the downstream estuary. Above-ground storage features are simulated as independent features with their own water budgets including rainfall and ET processes. In order to preserve the overall basin water budget, AFSIRS/WATBAL estimates of excess runoff are lowered when above-ground reservoirs are simulated. This area-weighted adjustment helps to avoid double accounting of rainfall and ET within the overall basin boundary.

The calibration of the Caloosahatchee Basin is presented in Section 4.1. The calibrated parameters for the Caloosahatchee Basin were used for the other LOSA basins. The results are also presented in Chapter 4.

3.3.4 St. Lucie Basin and Florida Power & Light Reservoir

For consistency's sake, modeling of the St. Lucie Basin demand/runoff time series was estimated using the AFSIRS/WATBAL model as with the other LOSA basins outside the gridded SFWMM domain. Similar capabilities exist in the SFWMM for simulating reservoir and ASR interactions with the St. Lucie as those previously outlined for the Caloosahatchee Basin. Explicit accounting of Lake Okeechobee deliveries is also considered in the SFWMM to maintain stages in the Florida Power & Light (FPL) Reservoir at Indiantown. While in reality, flows to the FPL reservoir are sent from Lake Okeechobee through the S-308 structure (and the C-44 canal) into the S-153 Basin, the SFWMM assumes that these deliveries are made directly from Lake Okeechobee to the FPL reservoir. This simplifying assumption is made since the magnitude of these discharges is very small relative to the capacity of the S-308 structure.

3.3.5 Lower Istokpoga Basins

In SFWMM v5.5, the Lower Istokpoga Basin is split into two basins. These basins are defined as: Lower Istokpoga Above Brighton (ISTOKPAB) and Lower Istokpoga Below Brighton (ISTOKPBB). This is necessary due to the fact that the Lower Istokpoga Above Brighton Basin is subject to the combined conveyance limitation of the G207 and G208 pump capacities (270cfs). These pumps serve both the Brighton Seminole Reservation and the agricultural land above S71/S72 and below S70/S75. In the SFWMM, Brighton Tribal demands have first priority in water supply deliveries. Unmet demands in the Lower Istokpoga Above Brighton Basin accrue from one time step to the next until sufficient conveyance exists to make deliveries. Demand/runoff time series are estimated using AFSIRS/WATBAL for these basins.

3.3.6 North and Northeast Lake Shore Basins

The North and Northeast Lake Shore Basins have relatively small areas of irrigated lands compared to several of the other LOSA basins. However, in order to account for all LOSA agriculture, it was necessary to explicitly model these basins in SFWMM v5.5. The North Lake Shore Basin (NLKSHORE), as modeled, has a relationship to the previously described Taylor Creek/Nubbin Slough (TCNSQ) inflow term. NLKSHORE demand/runoff goes through either S-133 (only runoff) and/or S-193. This is an issue since a portion of the runoff that goes through S-133 is already quantified in the SFWMM as part of the TCNSQ (S133 + S191) inflow term. To avoid any double accounting, an additional term TCNSQ_REV was derived. TCNSQ_REV is defined as the portion of the TCNSQ term which comes from tributary basins upstream of the North Lake Shore. This upstream flow enters the North Lake Shore Basin and is effectively reduced (on days with NLKSHORE demands) or increased (on days with NLKSHORE runoff), resulting in the "at Lake" TCNSQ observed flow. This relationship is illustrated in Figure 3.3.6.1. At run-time, SFWMM v5.5 reads both the TCSNSQ and TCNSQ_REV term and then internally adjusts the NLKSHORE demand and runoff terms to ensure that the Lake Okeechobee budget is correctly accounted and that model output reflects TCNSQ as it is read in from the input file.

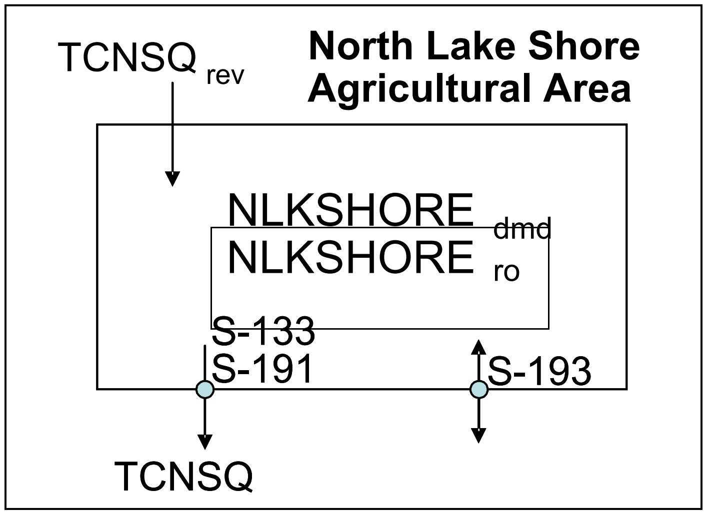

Figure 3.3.6.1 Relationship between TCNSQrev, TCNSQ and NLKSHOREdmd/ro

3.3.7 Seminole Brighton and Big Cypress Reservations

The Seminole Brighton Reservation is located in the Lower Istokpoga Basin northwest of Lake Okeechobee. Following the Brighton/Istokpoga calibration exercise outlined in Section 4.3, the AFSIRS/WATBAL model was run using calibrated parameters and land use as defined in the Work Plan authorization outlined in the letter from Lewis, Longman & Walker (2000). These demand estimations as modeled were consistent with water rights compact entitlement volumes protected by Florida state law. A daily time series of Brighton Reservation demand was calculated for the period 1965-2000. This time series was then modified by a rescaling program which imposes a daily maximum of 530 ac-ft (the combined conveyance of the G207 and G208 pump stations) and attempts to obtain an annual average of 28,500 ac-ft over the period of simulation (consistent with release volumes over the last several years). While the impact of this rescaling was large in previous modeling efforts, the calibration exercise for the Brighton/Istokpoga area reduced the impact of rescaling, effectively making the program only a check on conveyance limitations. Results of the rescaled time series are presented in Table 3.3.7.1. As can be seen the 2/10 monthly demand in the time series is in agreement with (and actually exceeds) the entitlement delivery requirement for the Brighton Reservation.

The Seminole Big Cypress Reservation is incorporated in the SFWMM using a pre-processed lumped water-budget modeling approach consistent with the modeling of other non-EAA LOSA basins and the Seminole Brighton Reservation. Following the basin calibration exercise outlined in Section 4.3, a 36-year continuous time series (1965-2000) of daily basin-scale irrigation demands is estimated using the AFSIRS-WATBAL basin-scale water budget model with 2000 Work Plan landuse estimates provided by the Seminole Tribe. Deliveries to meet estimated supplemental Seminole Big Cypress demands come from several regional sources. In order of priority, regional water is available from STA 6, Rotenberger Wildlife Management area and Lake Okeechobee via the Miami canal/G404. Results of the Seminole Big Cypress demand estimation effort are presented in Table 3.3.7.2.

Table 3.3.7.1 Comparison of Modeled Demands to Work Plan Entitlement for Seminole Brighton Reservation

As modeled with AFSIRS/WATBAL and rescaling for 1965-2000 period.	Average Annual Demand [ac-ft]	28,500
	Max Monthly Demand [ac-ft]	10,348
	Max Monthly Demand [mgm]	3,374
	Monthly 2/10 Demand [mgm]	**2,383**
From Work Plan 2/10 Demand [mgm]		**2,262**

Table 3.3.7.2 Comparison of Modeled Demands to Work Plan Entitlement for Seminole Big Cypress Reservation

As modeled with AFSRIS/WATBAL for 1965-2000 period.	Average Annual Demand [ac-ft]	28,509
	Max Monthly Demand [ac-ft]	10,694
	Max Monthly Demand [mgm]	3,486
	Monthly 2/10 Demand [mgm]	**2,659**
From Work Plan 2/10 Demand [mgm]		**2,606**

While estimated supplemental demands for the Seminole Reservations, and therefore deliveries, for every month of simulation do not equate to monthly entitlement quantities, tribal rights to these quantities are preserved.

3.3.8 Supply-Side Management for Lake Okeechobee Service Area

The guiding policy for implementation of agricultural water shortage restrictions for the Lake Okeechobee Service Area (LOSA) is the Supply-Side Management (SSM) plan. In contrast to the WSE schedule, SSM is used to manage lower stages in Lake Okeechobee (Figure 3.3.8.1). The zone below the "SSM Trigger Line" identifies when water shortage restrictions will be imposed within LOSA. Under the SSM methodology, the amount of water available to users of Lake Okeechobee water is defined as allocable volume and is a function of available storage within the Lake in conjunction with expected net losses. The allocable volume of water is dependent on both expected climatic conditions and on a projected Lake stage at the end of the dry season, known as the Reference Elevation. Temporal allocation of water under SSM is designed to avoid Lake levels lower than the reference elevation at the end of the dry season, although this may not be prevented depending upon the severity of the drought.

Supply-Side Management represents a complicated calculation scheme with consideration for many factors. Included in the determination of SSM output are terms that consider LOSA current and projected demand, the deliveries made to non-LOSA water supply users, temporal distribution patterns of demand through the calendar year, and projected changes in Lake storage (from net rainfall and inflows).

The end result of the SSM algorithm during periods of water shortage is a cutback fraction that is applied by default to all LOSA basins and that can be optionally applied to Seminole Tribal demand and environmental water supply depending on user input. This cutback fraction will allow only a portion of the supplemental irritation demand for a basin to be delivered. The model has the capability to apply a global maximum cutback fraction (e.g. provide a minimum level of service to consumptive users not to exceed 50% cutback) or to impose a phased cutback approach based on drought severity (e.g. apply a maximum cutback of 15% for a mild drought or 60% for a severe drought). Details on how the SFWMM handles the specifics of the SSM calculation are available in Appendix F1.

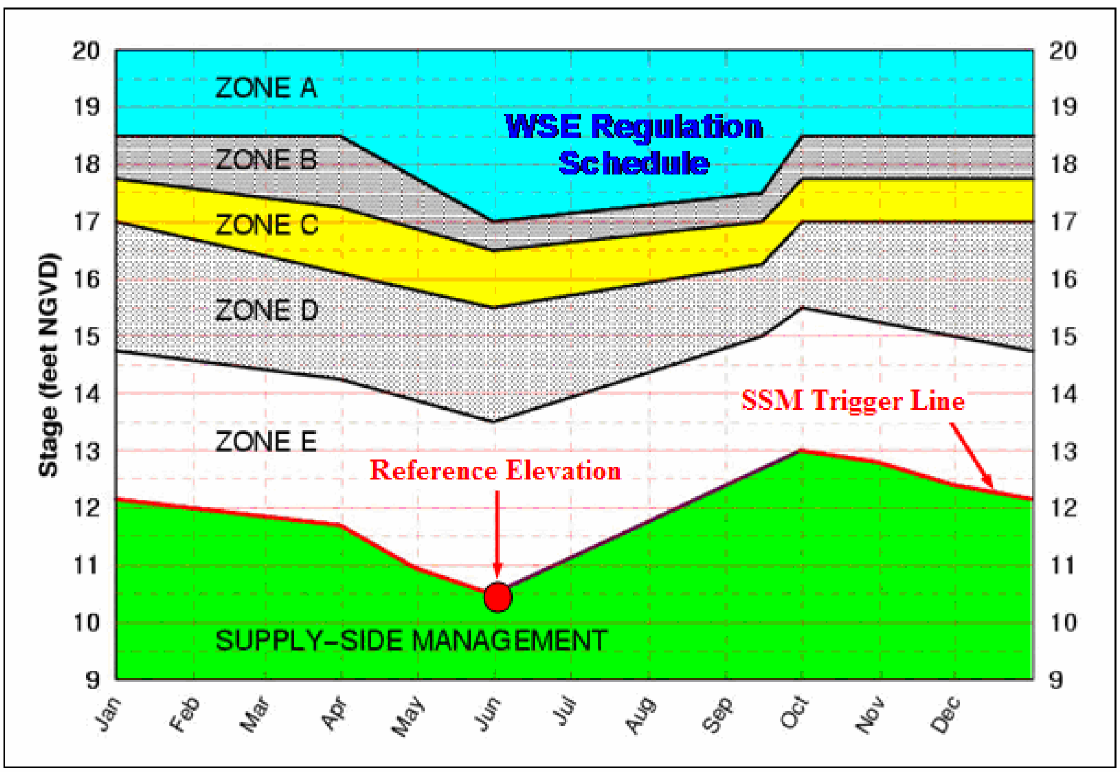

Figure 3.3.8.1 Lake Okeechobee Regulation Schedule with Supply-Side Management Line

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3.4 EVERGLADES PROTECTION AREA

3.4.1 Introduction

The Water Conservation Areas (WCA-1, WCA-2A, WCA-2B, WCA-3A and WCA-3B) comprise five surface water management basins in the Everglades. Bounded by the Everglades Agricultural Area (EAA) on the north and the Everglades National Park Basin on the south, the WCAs are confined by levees and water control structures that regulate the inflows and outflows to each conservation area. In general, they were designed: (1) to provide viable wetland habitat; (2) to receive excess water from the EAA; (3) to receive regulatory releases from Lake Okeechobee; (4) to prevent flood water from accumulating in the Everglades and from flooding urban and agricultural lands in eastern coastal areas; (5) to recharge regional groundwater; (6) to store water for dry season water deliveries to eastern Miami-Dade, Broward, and Palm Beach counties for agricultural and municipal water supply; and (7) to control saltwater intrusion into the groundwater. All WCAs are jointly owned by the state and the District. The U.S. Fish and Wildlife Service (USFWS) manages WCA-1 while the District and the Florida Game and Freshwater Fish Commission (FGFWFC) jointly manage WCA-2 and WCA-3. The Everglades National Park (ENP), on the other hand, is operated by the National Park Service and is located on the southern tip of the Florida peninsula. A schematic diagram showing the boundaries of the WCAs and the ENP is shown in Figure 3.4.1.1. The WCAs and the ENP region are commonly known as the Everglades Protection Area (EPA) (SFWMD, 1992).

Water Conservation Area 1 is part of the Arthur R. Marshall Loxahatchee National Wildlife Refuge. It has an area of 227 square miles and is located entirely within south-central Palm Beach County. Figure 3.4.1.2 shows a schematic of the WCA-1 Basin boundary, canals and water control structures. WCA-1 has six primary functions (Cooper and Roy, 1991). They are:

1. to provide viable wetland habitat;
2. to detain and store flood and drainage water during the wet season for water supply during the dry season;
3. to prevent water accumulating in the Everglades from overflowing into urban and agricultural lands in eastern Palm Beach County;
4. to receive and store releases from Lake Okeechobee;
5. to provide conveyance of water supply releases from Lake Okeechobee to the Hillsboro Canal Basin; and
6. to supply water to eastern Palm Beach and Broward Counties.

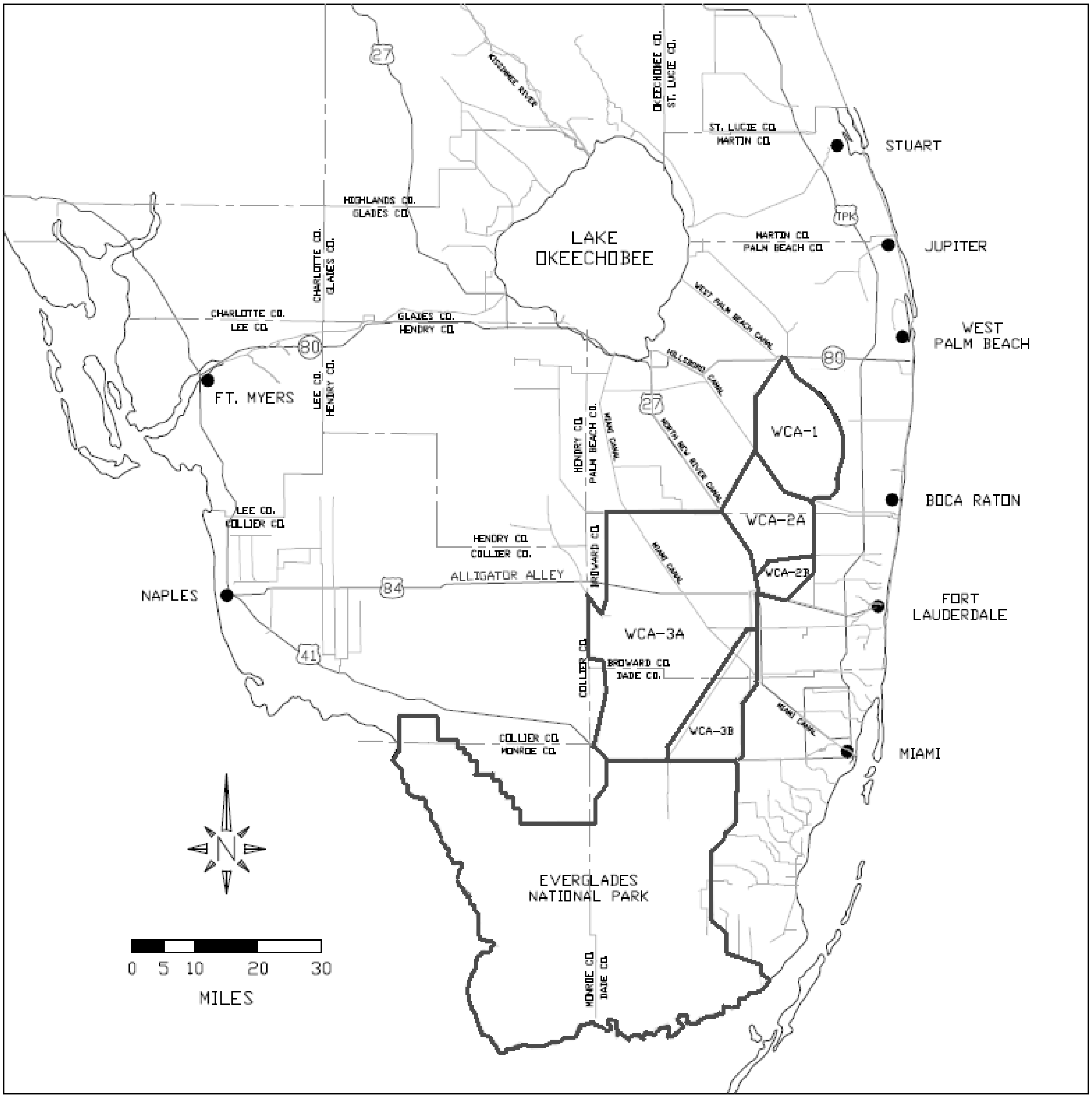


Figure 3.4.1.1 Surface Water Management Basins in the Everglades: WCAs and ENP (Adapted from Cooper and Roy, 1991).

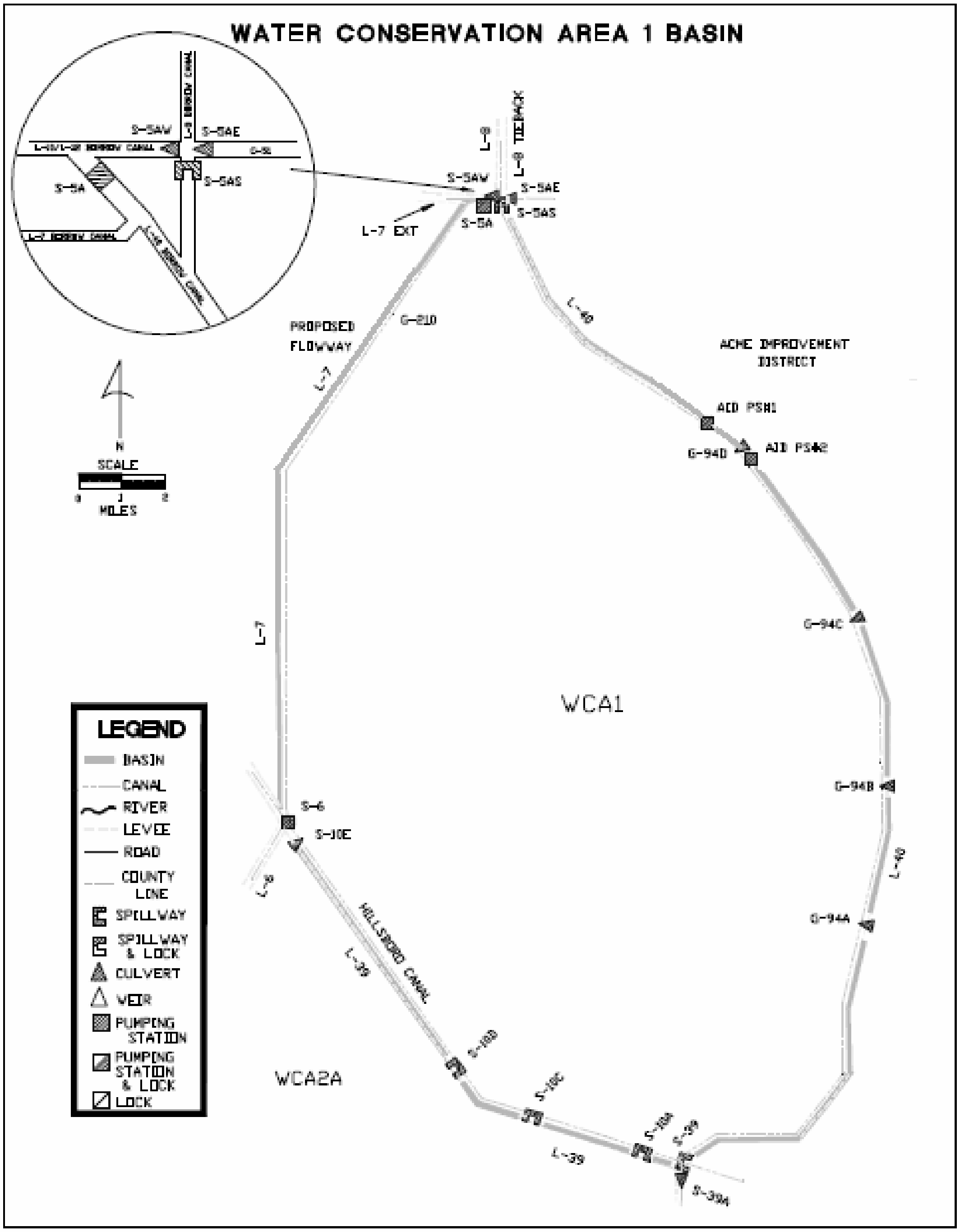


Figure 3.4.1.2 WCA-1 Basin Boundary, Canals and Water Control Structures (Adapted from Cooper and Roy, 1991).

Water Conservation Area 2 (WCA-2A and WCA-2B, also known as Sawgrass Recreational Area), comprising 210 square miles, is located immediately south of WCA-1. Originally constructed as a single area, this WCA was divided by a levee, L-35B, constructed in 1961 to allow better control of water levels, and as a consequence, reducing seepage losses out of the entire area. WCA-2B occupies an area of significant recharge to the Biscayne Aquifer. Water supplied to the aquifer by way of WCA-2B is important in maintaining groundwater levels in coastal areas to the east (Cooper and Roy, 1991). WCA-2A has an area of 173 square miles and is located in the south-central portion of Palm Beach County and the north-central portion of Broward County. It has ground elevations ranging from 13 ft NGVD in its northern tip to around 7 ft NGVD at its southern end. Water levels in WCA-2A are normally regulated between 13.0 and 14.5 ft NGVD as of the early 1980's. Water enters the area across the Hillsboro Canal from WCA-1 on the northeast side and across the North New River Canal on the northwest side. Water is discharged from WCA-2A through structures into Cypress Creek Canal (C-14), North New River Canal, and WCA-2B. This water conservation area has five primary functions (Cooper and Roy, 1991). They are:

1. to provide viable wetland habitat;
2. to store flood and drainage water during the wet season for subsequent use during the dry season;
3. to prevent floodwater accumulating in the Everglades from flooding urban and agricultural lands in eastern Broward County;
4. to receive and store regulatory releases from Lake Okeechobee and WCA-1; and
5. to provide conveyance for water supply releases from Lake Okeechobee to eastern Broward County.

Figure 3.4.1.3 shows a schematic of the WCA-2A Basin boundary, canals and water control structures.

WCA-2B has an area of 37 square miles and is located in the central portion of Broward County. It has ground elevations ranging from 9.5 ft NGVD in the northern portions down to about 7.0 ft NGVD in the southern portions of the area. Long term storage of water in this water conservation area is not possible due to high seepage rates. Releases from WCA-2B are not normally done. This water conservation area has five primary functions (Cooper and Roy, 1991). They are:

1. to provide viable wetland habitat;
2. to recharge regional groundwater (in the Biscayne Aquifer);
3. to supply water to adjacent basins in Broward County;
4. to receive and store regulatory discharges from WCA-2A; and
5. to prevent floodwater accumulating in the Everglades from flooding urban and agricultural lands in eastern Broward County.

Figure 3.4.1.4 shows a schematic of the WCA-2B Basin boundary, canals and water control structures.

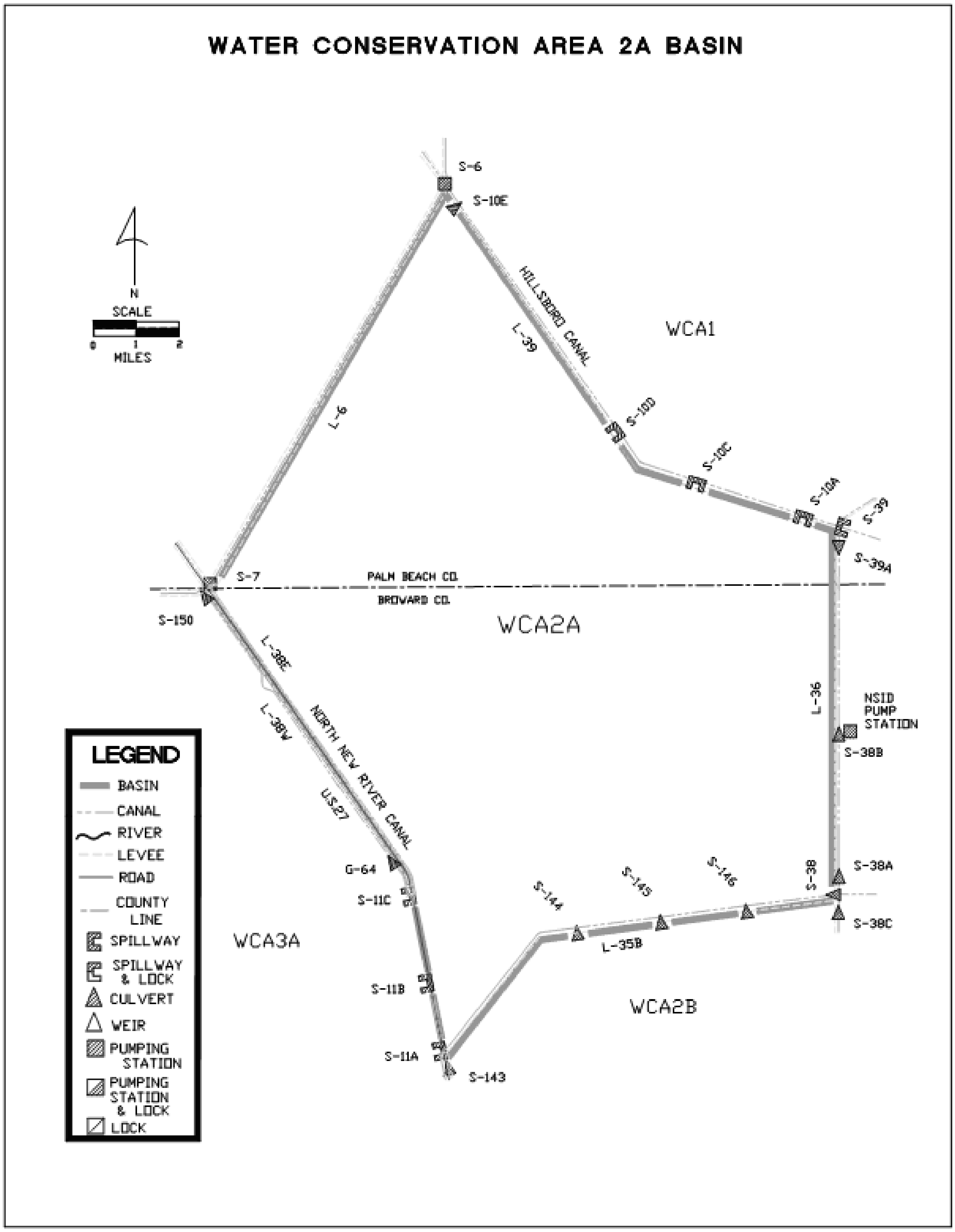


Figure 3.4.1.3 WCA-2A Basin Boundary, Canals and Water Control Structures (Adapted from Cooper and Roy, 1991).

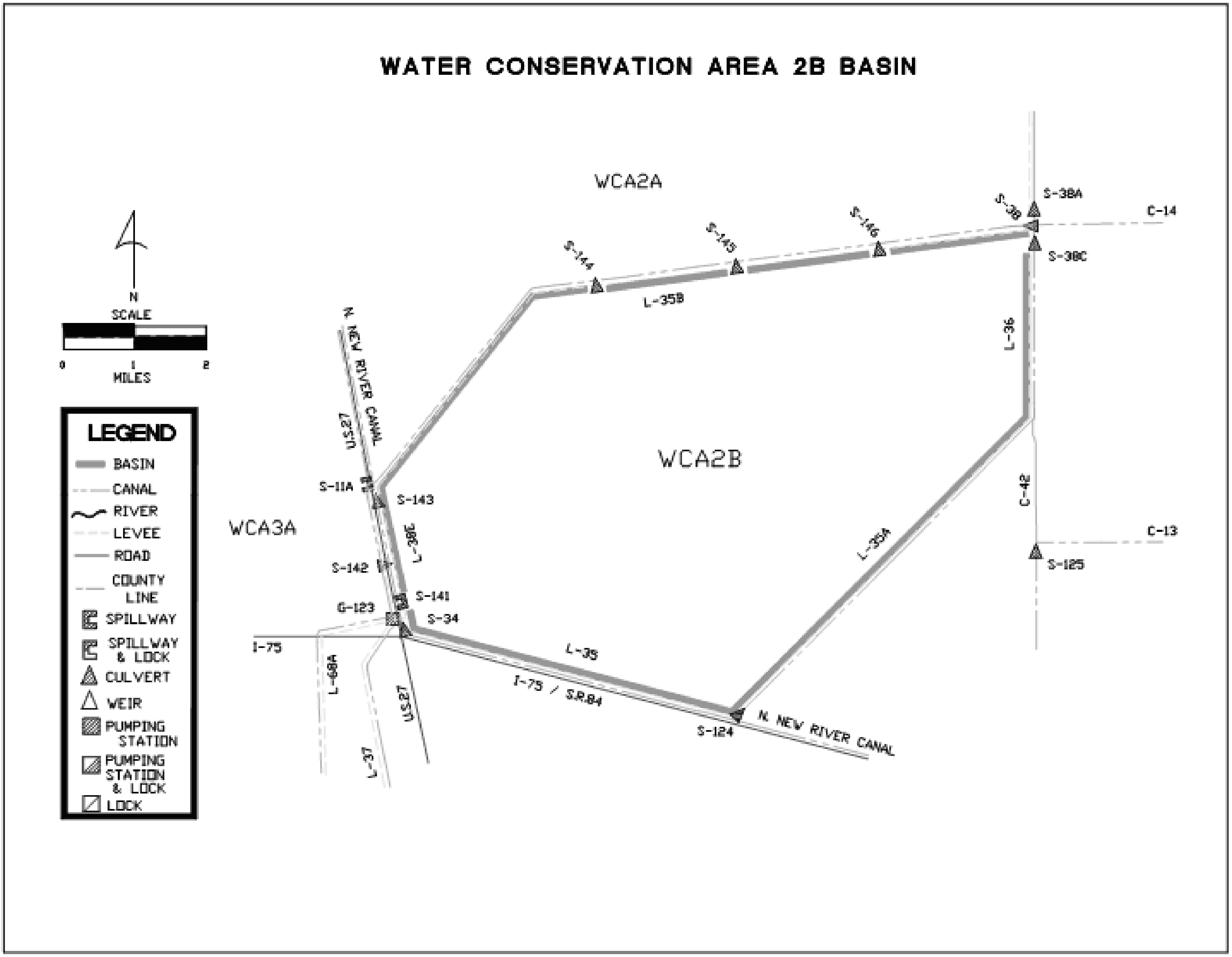


Figure 3.4.1.4 WCA-2B Basin Boundary, Canals and Water Control Structures
(Adapted from Cooper and Roy, 1991).

Water Conservation Area 3 (WCA-3A and WCA-3B) consists of 914 square miles. It is divided into two subareas by the L-67 borrow canals which run northeast to southwest cutting across the Broward-Dade County line. WCA-3A has an area of 786 square miles and is located in western Broward County and in northwestern Miami-Dade County. The ground elevations in this area range from about 13 ft NGVD in the northern section to around 7 ft NGVD in the southern portion. Water levels are normally regulated between 9.5 and 10.5 ft NGVD by releases from structures along the southern border of the area. Inflow to this water conservation area comes from several northern basins and canals. WCA-3A has five primary functions (Cooper and Roy, 1991). They are:

1. to provide viable wetland habitat;
2. to store flood and drainage water during the wet season for water supply for subsequent use in the dry season;
3. to prevent floodwater accumulating in the Everglades from flooding urban and agricultural lands in eastern Miami-Dade and Broward Counties;
4. to receive and store regulatory releases from Lake Okeechobee and WCA-2A; and
5. to provide conveyance for water supply releases from Lake Okeechobee to eastern Dade County and the Everglades National Park via the South Dade Conveyance System (SDCS). Like WCA-2B, WCA-3B (a.k.a. Francis Taylor Wildlife Management Area) has no regulation schedule due to its high seepage rate. Figures 3.4.1.5 and 3.4.1.6 show schematics of WCA-3A and WCA-3B Basin boundaries, canals and water control structures, respectively.

The surface water management basin defined by the Everglades National Park has an area of 1,684.5 square miles. The extent of the ENP covers three counties in the state: Dade County (886.5 sq mi), Monroe County (773.9 sq mi), and Collier County (24.1 sq mi). The peripheral structures around the basin are primarily used for water supply to the basin. The Rainfall Plan for ENP (Neidrauer and Cooper, 1989) was developed to allow a more "natural" passage of overland flow into the park. It was based on a statistical model, developed by the District and in cooperation with the Corps and ENP, which correlates upstream weather conditions to the amount, timing and distribution of flows to the ENP. Figure 3.4.1.7 shows a schematic of the ENP Basin boundary, canals and water control structures.

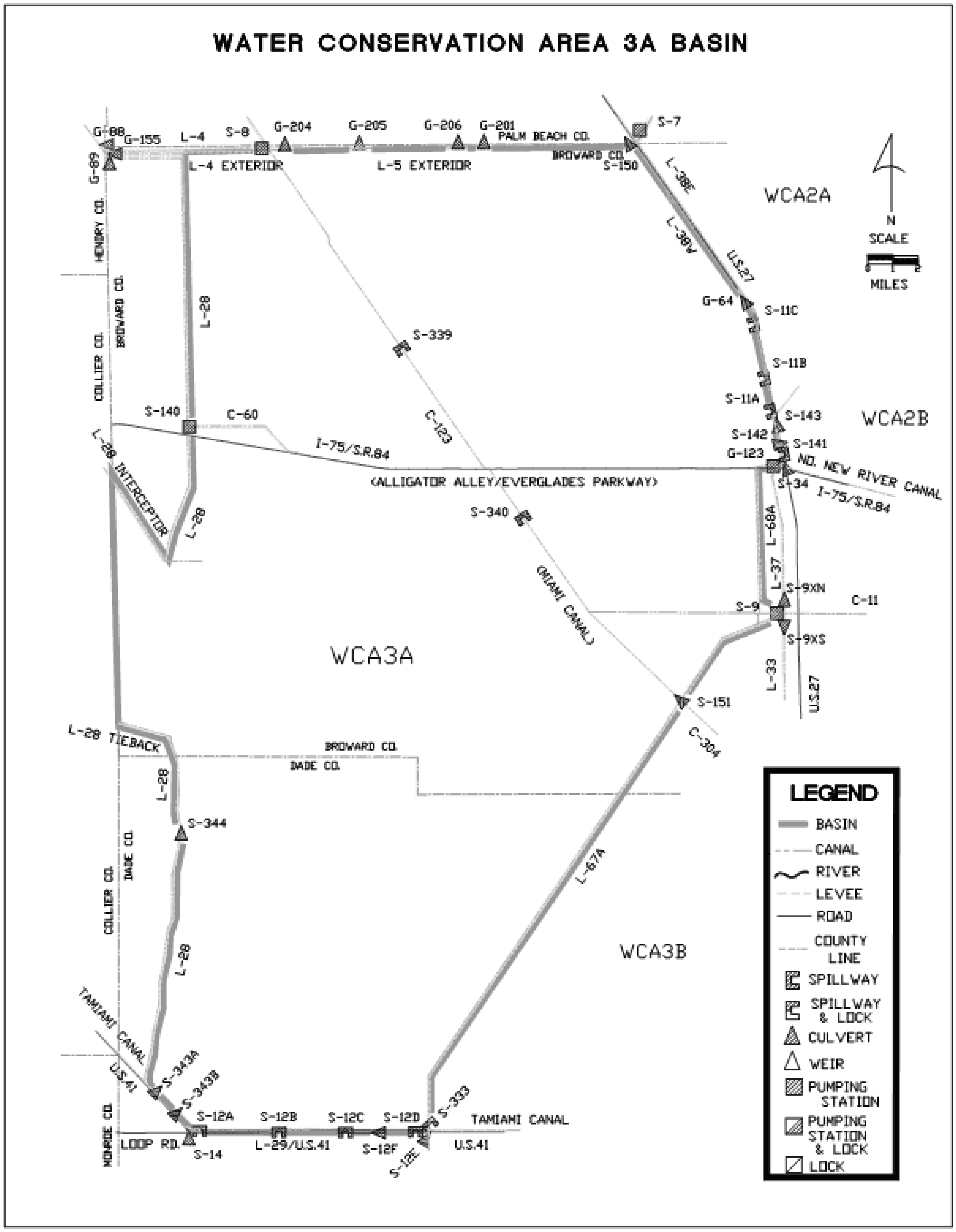


Figure 3.4.1.5 WCA-3A Basin Boundary, Canals and Water Control Structures (Adapted from Cooper and Roy, 1991).

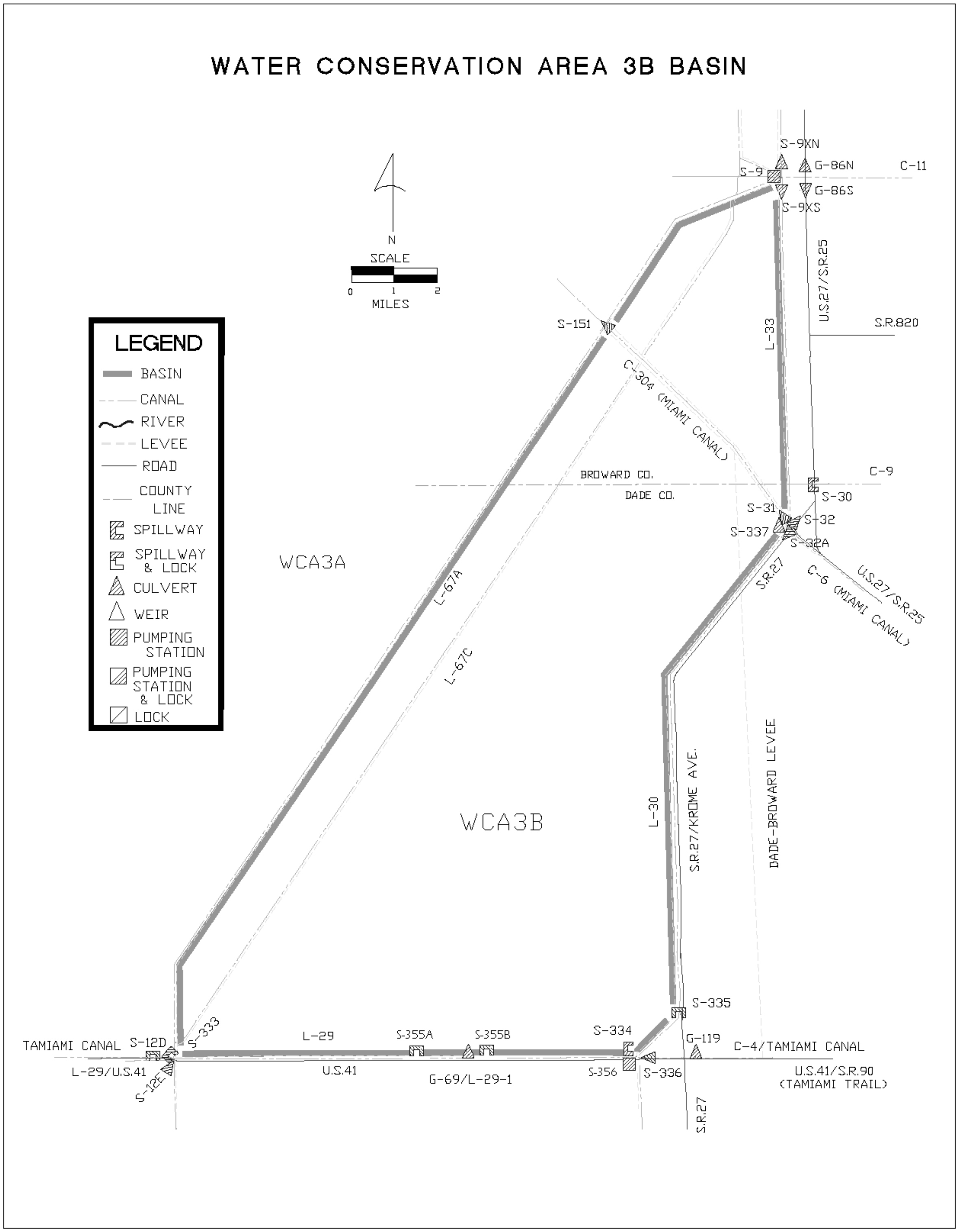


Figure 3.4.1.6 WCA-3B Basin Boundary, Canals and Water Control Structures (Adapted from Cooper and Roy, 1991).

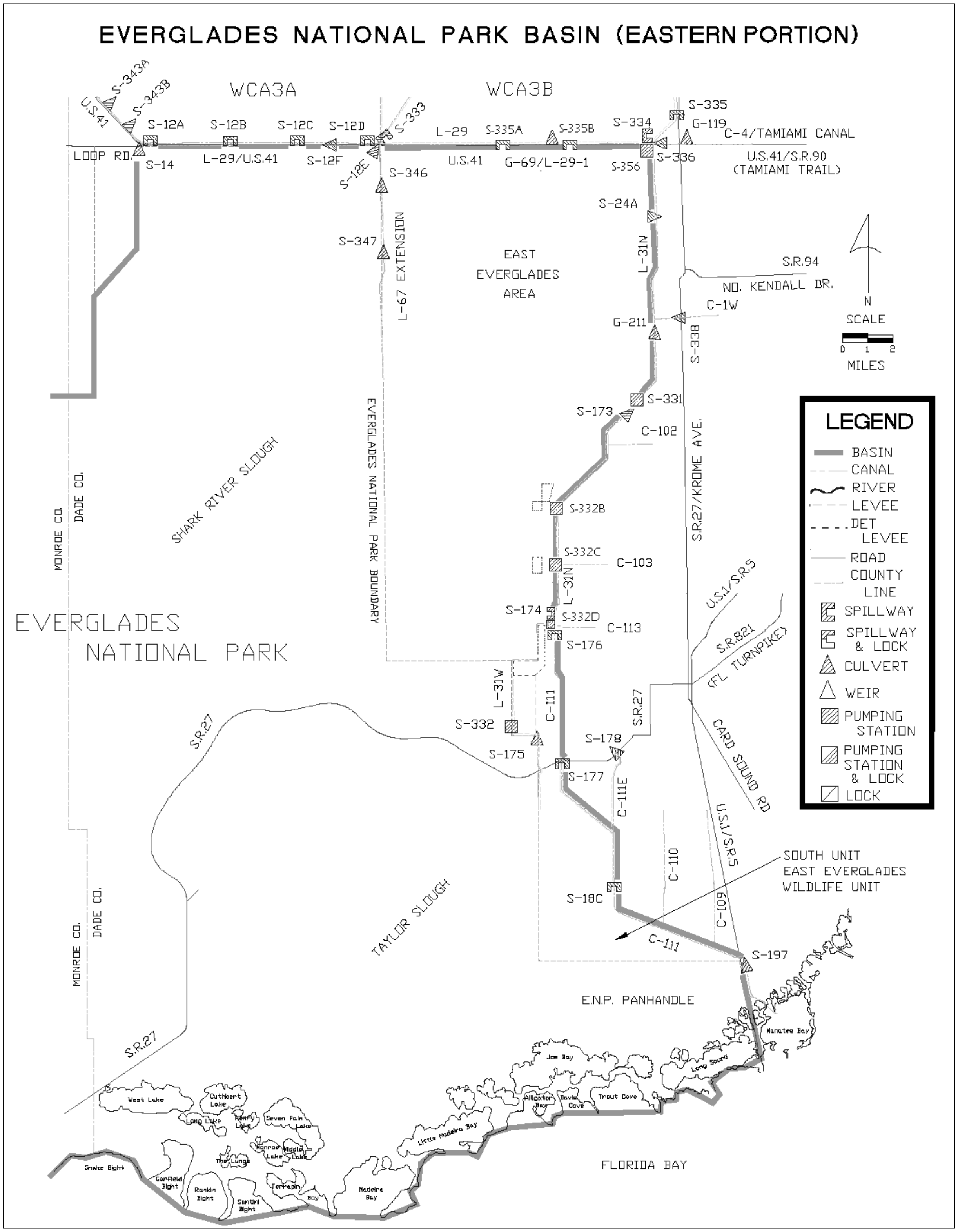


Figure 3.4.1.7 ENP Basin Boundary, Canals and Water Control Structures (Adapted from Cooper and Roy, 1991).

3.4.2 Model Implementation

The model represents the Everglades Protection Area as a system of homogeneous 2-mile by 2-mile grid cells. The grid network shown in Figure 3.4.2.1 delineates the five WCAs and Eastern Everglades National Park from the rest of the model. Separate water budgets can be prepared for the six water management basins shown in this figure. A comparison between the actual and modeled areas in the WCAs is depicted in Table 3.4.2.1.

Table 3.4.2.1 Comparison between Actual and Modeled Areas in the WCAs

Everglades Protection Area Water Budget Basin	Area as Defined in the Everglades SWIM Plan, 1992 (sq mi)	Area as Modeled in the SFWMM v5.5 (sq mi)
WCA-1	227	224
WCA-2A	173	164
WCA-2B	37	44
WCA-3A	786	768
WCA-3B	128	108

Water budgets are generated as part of the model output as well as in post processing of the initial output as outlined in Appendix A. Not only are water budgets created for the above areas, but also many other area and reservoir water budgets are also generated. On the SWFMM-CERP modeling site (http://modeling.cerpzone.org/pmviewer/index.jsp), about 50 water budgets that are used for various evaluations are presented.

3.4.2.1 Operating Rules

Similar to Lake Okeechobee, the operating rules governing the management of the Water Conservation Areas may be classified into three categories: regulatory (flood control) , water supply (exclusively to LEC service areas) and environmental (proposed flow and/or stage targets in the Water Conservation Areas). Water used for environmental purposes can sometimes be classified under water supply. The rules governing these types of releases are closely related. Initially, a list of outlet and inlet structures will be given as a function of release type (regulatory or water supply). Then, a discussion of structure operations for both release types will be presented. Lastly, the proposed environmental release rules will be summarized.

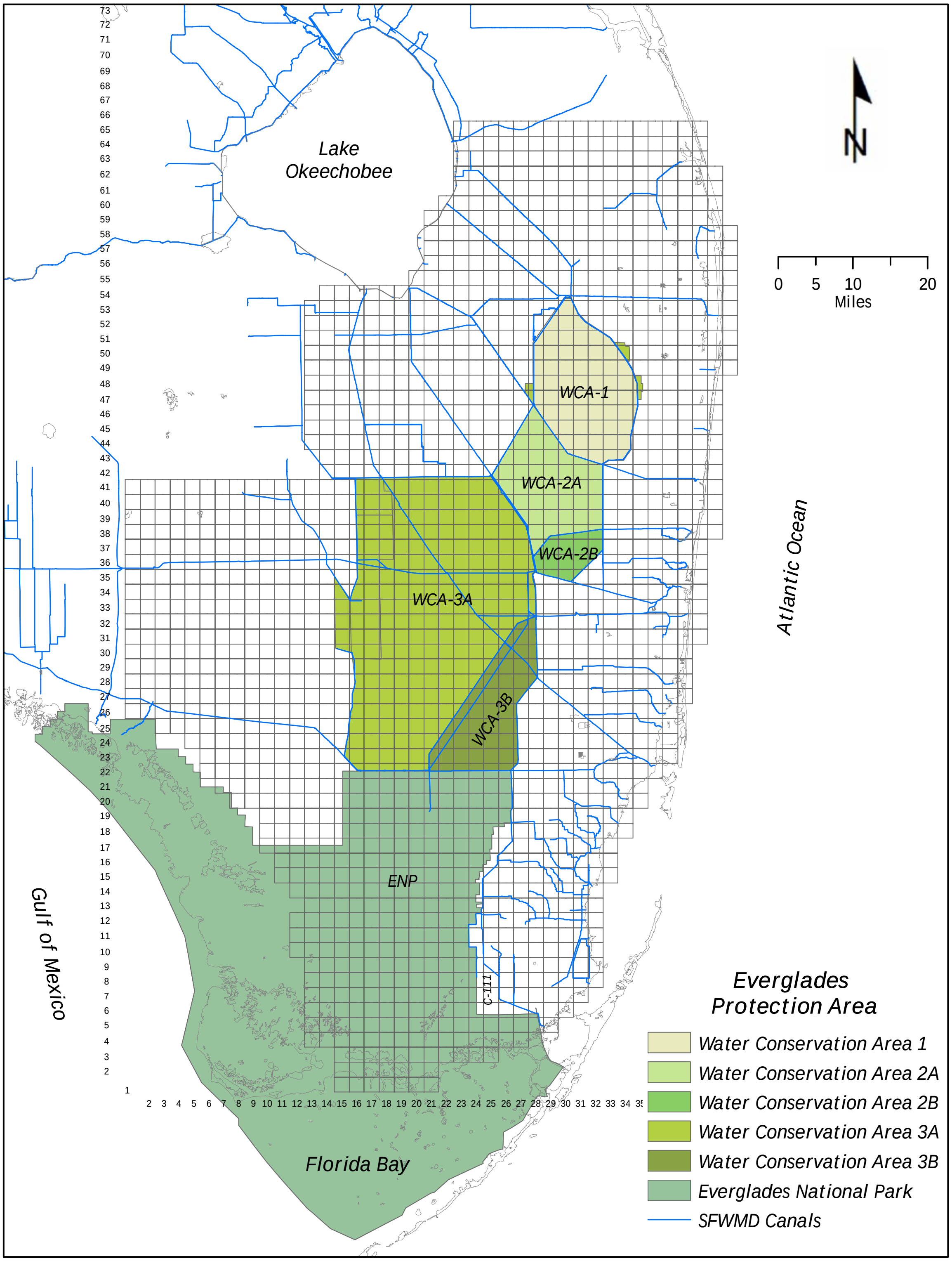


Figure 3.4.2.1 SFWMM Grid Cell Network with Model Boundary

WCA Structures for Regulatory and Water Supply Discharges

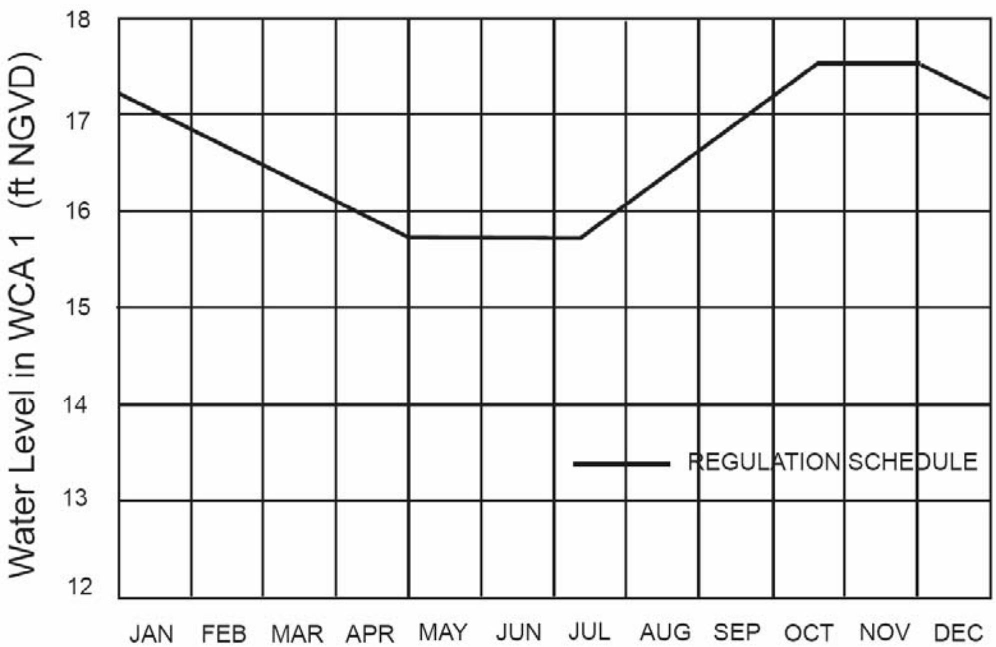
A WCA outlet structure, in general, can be identified either as a water supply or flood control structure. In the SFWMM, water supply releases are made first before flood control releases are done. Figures 3.4.2.2 through 3.4.2.4 show the current regulation schedules for WCA-1, WCA-2A and WCA-3A, respectively. Figure 3.4.2.5 is a composite display of the three regulation schedules. A common feature among these schedules is that certain monitoring points (observation wells or canals) trigger flood control releases when the stage exceeds a certain threshold or maximum level. A summary of the trigger locations including model grid cell locations in (x,y: column, row) coordinates for regulatory releases from WCAs is listed in Table 3.4.2.2.

Table 3.4.2.2 Trigger Locations for Regulatory Releases from WCAs as Used in the SFWMM

Water Conservation Area	Trigger Location
WCA-1	Arithmetic average of 1-8T (x,y: 34,47), 1-7 (x,y: 31,48) and 1-9 (x,y: 33,46) when the average simulated stage of location is greater than land surface elevation for the 3 gages + 0.5 ft; 1-8C (L-40 borrow canal stage), otherwise.
WCA-2A	2A-17 (x,y: 29,40) when simulated stage at location is greater than land surface elevation for the gage + 0.5 ft; L-38 borrow canal, otherwise.
WCA-3A	3-gage average, i.e., arithmetic average of 3A-3 (x,y: 25,37), 3A-4 (x,y: 21,29) and 3A-28 (x,y: 19,24).

In the model, some regulation zones are collapsed into single zones, thus simplifying the implementation of flood control releases from the WCAs. Flood control releases out of WCA-2A and WCA-3A closely follow the schedules prescribed by the Corps (Figures 3.4.2.3 and 3.4.2.4). The flood control or regulatory release rules for WCA-3A are shown in Figure 3.4.2.4. Also, a tabular summary of the structure operations in the WCAs for regulatory discharges as implemented in the model is shown in Table 3.4.2.3.

In a given day, the model normally assumes that the amount of regulatory discharge out of a particular water conservation area is limited by the volume of water above a certain level (typically the schedule itself) within the corresponding peripheral borrow canal. For WCA-1 and WCA-2A, the maximum drawdown of the canal for regulatory releases is assumed to be 0.5 feet below regulation schedule if the stage in the gaging station is used as trigger. On the other hand, if canal stage is used as a trigger, then the maximum drawdown of the canal stage is assumed to be equal to the regulation schedule.

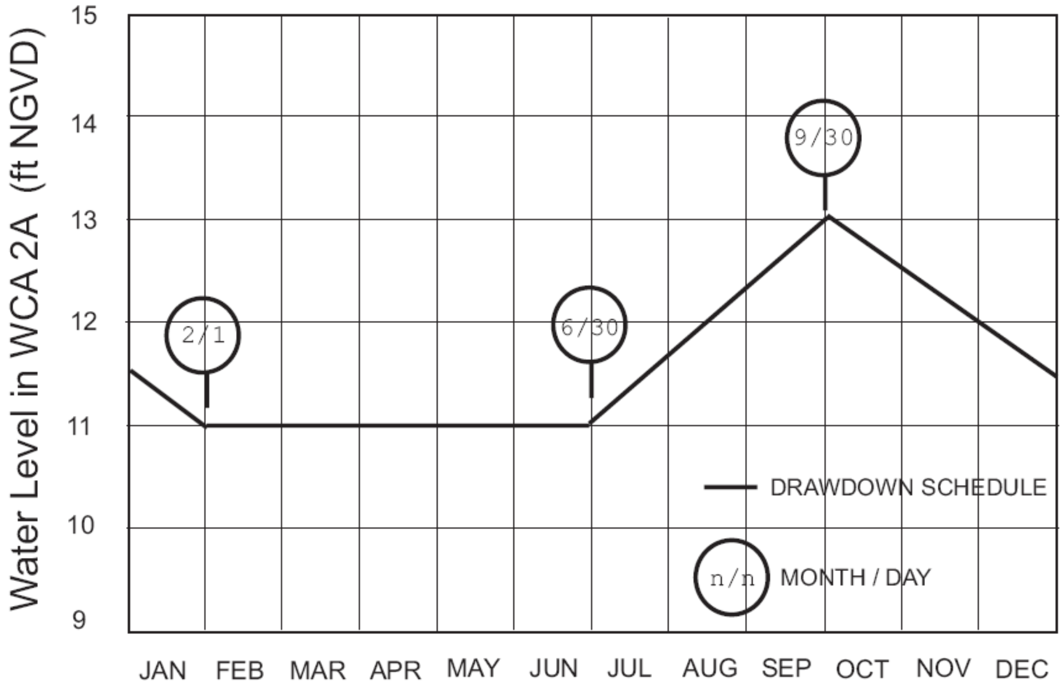


Notes: (1) NO RELEASES PERMITTED WHEN 1-8 CANAL GAGE READING FALLS BELOW ELEVATION 11.0, UNLESS WATER IS SUPPLIED FROM ANOTHER SOURCE.

(2) INDICATOR GAGES FOR REGULATION WILL BE AS FOLLOWS: TRANSITION TO BE MADE WHEN POSSIBLE TO AVOID DISCONTINUTIES.

DATES	USE GAGE	CONDITIONS
1 JAN - 30 JUN	1-8 CANAL	ALL
1 JUL - 31 DEC	1-8 CANAL	EXCEPT AS NOTED BELOW
	AVG. 1-7, 1-8 1-9	DURING RISING STAGES WHEN CANAL STAGE EXCEEDS AVERAGE

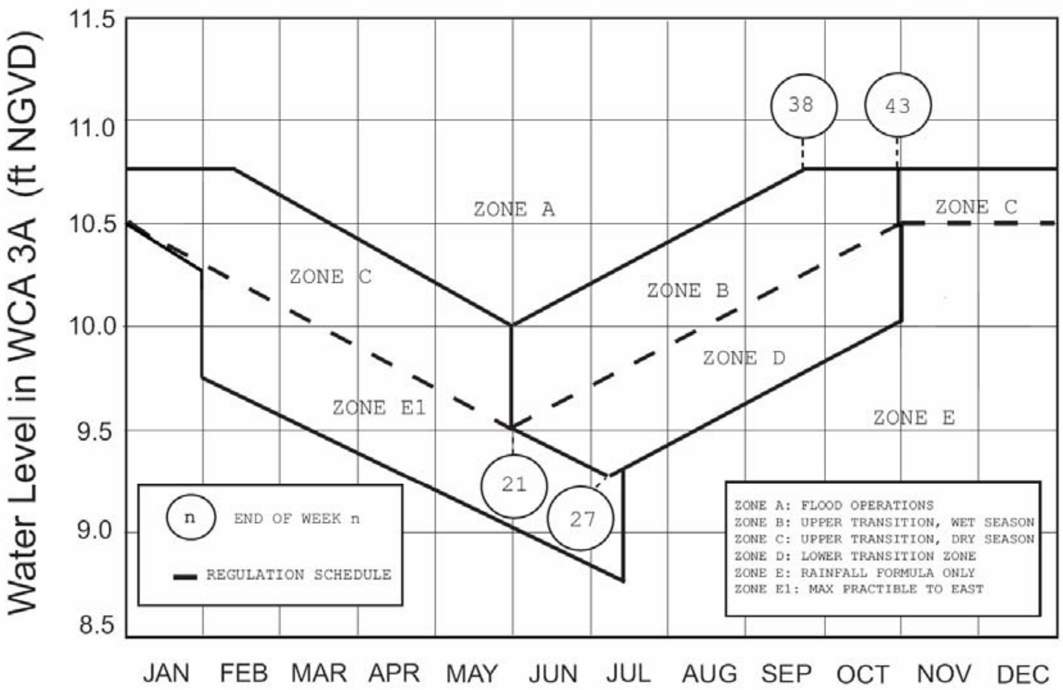
Figure 3.4.2.2 Regulation Schedule for WCA-1



ZONE	RELEASES
A	UP TO MAXIMUM AT S-11; MAXIMUM CAPACITY AT S-144, S-145 AND S-146, MAXIMUM PRACTICABLE AT S-143 AND S-38 WHEN REQUESTED BY THE CORPS OF ENGINEERS, BUT NOT TO EXCEED 11.0 FT NGVD IN POOL 2B. L-35B AND L-38 BORROW CANALS SHOULD NOT BE DRAWN DOWN BELOW 10.5 FT NGVD.
B	WATER SUPPLY L-35B AND L-38 BORROW CANAL SHOULD NOT BE DRAWN DOWN BELOW 10.5 FT NGVD UNLESS WATER SUPPLIED FROM ANOTHER SOURCE

DATES	USE GAGE	CONDITIONS
1 JAN - 31 JAN	2-17	IF 2-17 STAGE RECEEDS TO 11.5 FT NGVD SWITCH TO S-11B HEADWATER STAGE
1 FEB - 30 JUN	S-11B	ALL
1 JUL - 31 DEC	2-17	ALL

Figure 3.4.2.3 Regulation Schedule for WCA-2A



Zone	S-12	S-333	S-151	L-28 Culverts
A	Open Full	Maximum allowable discharge	Maximum diversion to 3B when 3B stage is less than 8.5 ft.	S-343A & B and S-344 open full if no problems downstream
B	S-333 Open: Discharge 45% of target flow S-333 Closed: Discharge 73% of target flow(up to 100% if desired by ENP)	Discharge up to 55% of target flow when permitted by this agreement	Same as Zone A whether S-333 is open or closed	Same as Zone A whether S-333 is open or closed
C	S-333 Open: Discharge 45% of target flow S-333 Closed: Discharge 45% of target flow plus all or part of S-333's amount if desired by ENP	Same as Zone B	May be used to divert water from WCA-3A whether S-333 is open or closed	May be used to divert water from WCA-3A whether S-333 is open or closed
D	S-333 Open: Discharge 45% of target flow S-333 Closed: Discharge 45% of target flow plus all or part of S-333's amount if desired by ENP	Same as Zone B	Used only for water supply deliveries to Miami-Dade County	Closed
E	Discharge 45% of target flow whether S-333 is open or closed	Same as Zone B	Used only for water supply deliveries to Miami-Dade County	Closed
E1	Closed	May be used to divert water form WCA-3A	May be used to divert water form WCA-3A	Closed

Figure 3.4.2.4 Regulation Schedule for WCA-3A

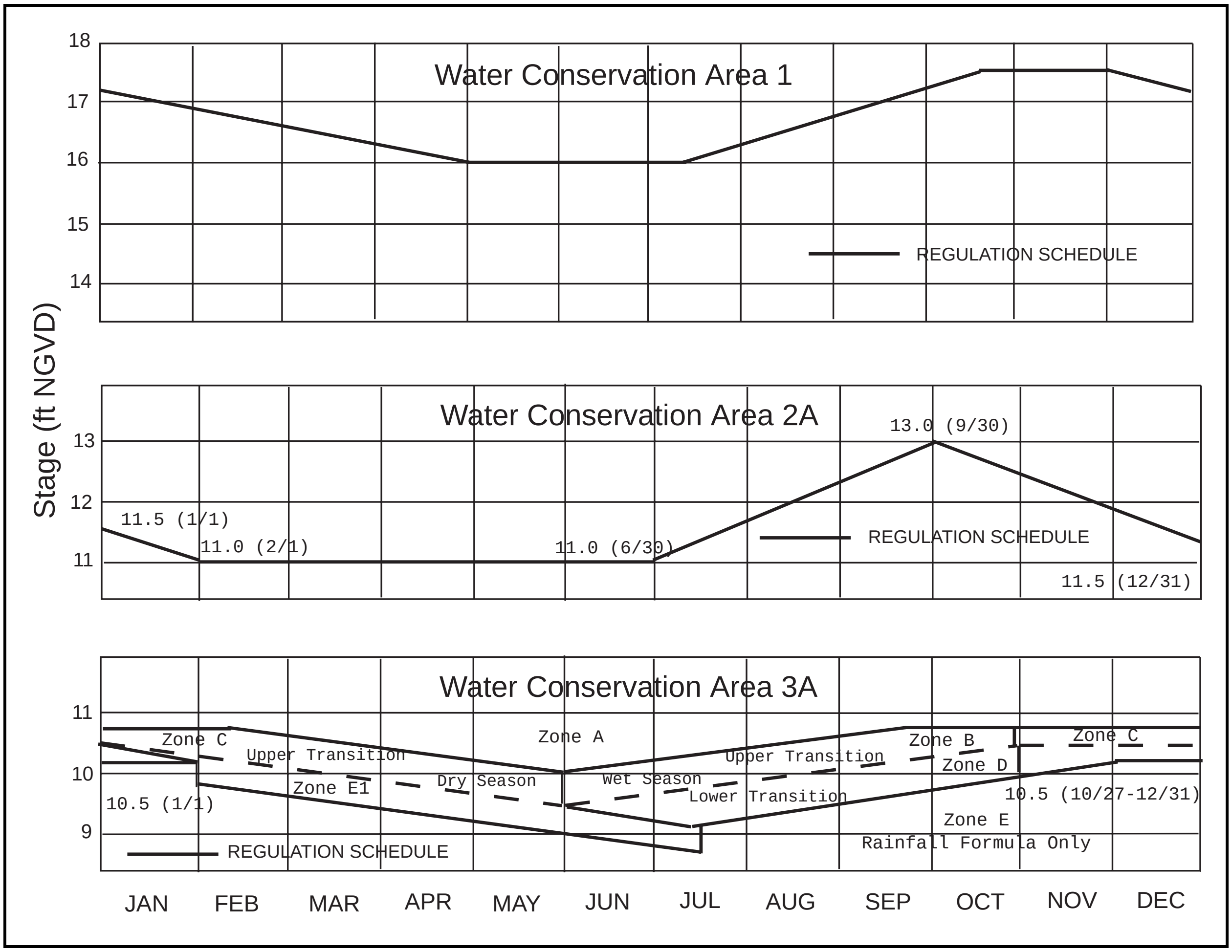


Figure 3.4.2.5 Regulation Schedule for WCAs 1, 2A, and 3A

Table 3.4.2.3 Structure Operations Associated with Regulatory Discharges in the WCAs

Water Conservation Area	Structure	Headwater, HW	Tailwater, TW	Maximum Capacity (cfs)	Destination	Operations	Exceptions
WCA-1*	S-10E	Rim canal stage in WCA-1	Stage at col 28 row 46	$438\ (hw-tw)^{0.5}$	WCA-2A	According to operational schedule	
	S-10A C D	Rim canal stage in WCA-1	Stage in WCA2A rim canal	$1800\ (hw-tw)^{0.5}$	WCA-2A	According to operational schedule	
WCA-2A*	S-144	L-35B borrow canal stage	Stage at col 29 row 37	$140\ (hw-tw)^{0.5}$	WCA-2B	According to operational schedule	
	S-145	L-35B borrow canal stage	Stage at col 30 row 37	$140\ (hw-tw)^{0.5}$	WCA-2B	According to operational schedule	
	S-146	L-35B borrow canal stage	Stage at col 31 row 38	$140\ (hw-tw)^{0.5}$	WCA-2B	According to operational schedule	
	S-11A-C	L-38 borrow canal stage in WCA-2A	Stage in WCA3A rim canal	$1800\ (hw-tw)^{0.5}$	WCA-3A	According to operational schedule	

Table 3.4.2.3 (*cont.*) Structure Operations Associated with Regulatory Discharges in the WCAs

Water Conservation Area	Structure	Headwater, HW	Tailwater, TW	Maximum Capacity (cfs)	Destination	Operations	Exceptions
WCA-3A	S-151	WCA-3A conveyance canal stage	Stage C-304 in WCA-3B	1050 $(hw\text{-}tw)^{0.5}$	WCA-3B	According to operational schedule	When stage in WCA-3B at 3B-71 (col 24 row 26) > 8.2 ft
	S-333	WCA-3A conveyance canal stage	L-29 borrow canal in NESRS	1909 $(hw\text{-}tw)^{0.5}$	NESRS	According to Modified Rainfall Deliveries	When stage at G3273 (col 24 row 17) > 6.8 ft NGVD[3]
	S-12ABCD	WCA-3A conveyance canal stage	L-29 borrow canal in western ENP	45700 $(hw\text{-}tw)^{0.5}$ max. flow = 4800 cfs consider tailwater constraints	Western ENP	According to Modified Rainfall Deliveries	
	S - 343AB	WCA-3A conveyance canal stage	Stage at col 15 row 22	390 cfs	col 15 row 22 in BCNP	Seasonal according to operational schedule	
	S - 344	L-28 borrow canal	Stage at col 15 row 27	135 cfs	col 15 row 27 in BCNP	Seasonal according to operational schedule	

* The maximum volume of water the outlet structures can discharge in a day for flood control purposes is the volume required to lower the upstream conveyance canal stage to the maximum of:

1. the bottom elevation of Zone A for current time step minus 0.5 ft and
2. the minimum elevation of Zone A for regulation schedule: 15.0 ft for WCA-1; 11.0 ft for WCA-2A
3. the model uses 7.05 ft in cell (24,17) to represent the G3273 gage site stage of 6.8 ft.

A definition of a WCA floor elevation was given in Section 3.1. Stated differently, floor elevation is typically the level triggered by a WCA canal at which the source of water supply to the Lower East Coast Service Area switches from the WCA to another upstream source (e.g., Lake Okeechobee). It is sometimes referred to as WCA minimum level or the level at which discharges are made from WCA to supply water to LEC service areas only if an equal amount is discharged from an upstream source into the WCA. The floor elevations of the different WCAs are shown in Table 3.4.2.4. Floor elevations are user-input to the model.

Table 3.4.2.4 WCA Floor Elevations Used in the SFWMM

WCA	**Trigger Location**	**Trigger Stage** (ft NGVD)
WCA-1	S10 headwater (same level as 1-8C gage)	14.0
WCA-2A	S11B headwater (same as L-35B stage)	10.5
WCA-3A	S12 headwater	7.5

In order to quantify the total volume of water available from a WCA to meet LEC needs in a given day, the following calculation is done in the model.

$$Vol\_Avail = \max(A + B + C, 0.0) + D \qquad (3.4.2.1)$$

where:

A = total net groundwater seepage into WCA conveyance canal assuming canal stage at floor elevation (minimum);
B = total net overland flow into WCA conveyance canal assuming canal stage is at its minimum;
C = local canal storage above the floor elevation; and
D = upstream inflow into WCA conveyance canal.

Table 3.4.2.5 summarizes the operations associated with WCA outlet structures related to water supply deliveries to LEC service areas.

Table 3.4.2.5 Structure Operations for Water Supply Releases from WCAs to LECSAs

Water Conservation Area	Upstream Inflow	Service Area	Outlet Structures	Maximum Capacity (cfs)
WCA-1	EAA runoff through S5A from WPB Canal Basin L8 Basin runoff through S-5Aw and S-5A EAA runoff through S6 from Hillsboro Canal Basin Supplemental LOK releases through S351 and S6, if needed Supplemental LOK releases through S352 and S5A, if needed	Service Area 1 (Eastern Palm Beach County and Northern Broward County)	S5AS into L-8 and M Canal S5AE into C-51 G94 A, B, & C into Lake Worth Drainage District S39 into Hillsboro Canal & Deerfield Agricultural District	2,000 700 223 566 $(hw-10.5)^{0.5}$
WCA-2A	EAA runoff through S7 from North New River Canal Basin	Service Area 2 (Eastern Broward County)	S38 into C-14 S-143 and S34 into North New River Canal between S-34 and G-54	302 $(hw-tw)^{0.5}$ 225 $(hw-tw)^{0.5}$
WCA-3A	EAA runoff through S8 from Miami Canal Basin Western Basins runoff through S-140A EAA runoff through S150 from NNRC Basin Back-pumped flow through S9 from C-11 in Eastern Broward County	Service Area 3 (Eastern Dade County)	S151	1050 $(hw-tw)^{0.5}$

Two options exist in the model for making water supply releases through multiple WCA outlet structures into a particular LEC service area. They are:

A. "No Priority" or "equal adversity" option
In this option, water is delivered proportional to the demands. For each service area, ratio_ws equals fraction of LEC_demand to be met from a particular outlet structure such that
ratio_ws = min[(tot_volume_of_water_available)/(tot_demand_in_service_area),1.0)]
and,
flow_through_outlet_structure = min[(ratio_ws)(LEC_demand), structure capacity].

B. Priority option
In this option, the order in which outlet structures are input specifies the priority: structures listed first get higher priority. This option can be presented in pseudo-code:

```
start_loop: i = 1, number of outlet structures
        flow_through_outlet_structure(i)
        = min[total volume of water available(i), total volume of water needed,
                flow capacity of structure(i)]
        total_volume_of_water_available(i+1)
        = total_volume_of_water available(i+1) - flow_through_outlet_structure(i)
end_loop
```

3.4.2.2 Environmental Deliveries

In addition to operations for passing regulatory flows and deliveries for public water supply, operations for passing water to and through natural areas are also included. Historically, the operational schedules for the Water Conservation Areas have been calendar-based which repeat every year. The schedules typically specify the release rules for a Water Conservation Area and are based on the water level at one or more key gages. However, there are two basic operations for environmental deliveries as a function of rainfall or natural pattern with annual fluctuations. These are: the Rainfall Plan (RFP) and the Rain-Driven Operations (RDO). The RFP analyzes the rainfall and passes a fraction of that amount from the WCAs to ENP – this is basically a “push” (export) type of operation. The RDO analyzes the stage at key points and either passes water out of an area or causes water to pass to an area – it operates as either an export or “pull” (import) type of operation. As of this writing the RFP is part of the C&SF system operations and the RDO is planned for the Comprehensive Everglades Restoration Plan (CERP). The operation of both is described below.

Rainfall Plan for the Everglades National Park

The rainfall plan is a water management plan designed to benefit ENP by attempting to mimic natural hydrology within the major slough in the park (Shark River Slough or SRS). Specifically, the plan has three objectives: (1) to base the amount and timing of water deliveries to SRS on recent weather conditions (rainfall and evaporation) upstream of the slough, i.e., from WCA-3A; (2) to moderate the sudden changes in flow that were caused by strictly following the regulation schedule for WCA-3A; and (3) to restore flow to the eastern section of North East Shark River slough, thus redistributing flow across the entire slough. The plan has been in effect since a two-year field test conducted from July 1985 to July 1987 revealed positive results. The model includes plan provisions as part of its base run.

In order to accomplish the above objectives, a statistically-based equation to calculate total target flow from WCA-3A to ENP was formulated (Neidrauer and Cooper, 1989). The plan calls for a total target flow equal to the sum of a rainfall-driven component and a regulatory component.

$$Q_{target}(t) = Q_{rfd}(t) + Q_{trans}(t) \tag{3.4.2.2}$$

The rainfall-driven component was formulated based on a statistical analysis of hydrologic data prior to man-made changes to both spatial and temporal distribution of surface flow into the slough. It relates the current week's flow rate to the previous week's flow rate and the rainfall and evaporation in each of the previous ten weeks. Due to limitations in data availability, the 1941-1952 period of record was selected. The multiple regression formula that resulted from the analysis contains variables expressed in terms of deviations from their respective means.

$$\begin{aligned} q(t) = CQ\left[q(t-1)\right] &+ CR_1 \sum_{j=1}^{2}\left[r(t-j) - Ke(t-j)\right] \\ &+ CR_2 \sum_{j=3}^{6}\left[r(t-j) - Ke(t-j)\right] \\ &+ CR_3 \sum_{j=7}^{10}\left[r(t-j) - Ke(t-j)\right] \end{aligned} \tag{3.4.2.3}$$

where:

$q(t)$ = $[Q(t) - Q_{mean}(t)]$;
$Q(t)$ = discharge into SRS during week t [cfs];
$Q_{mean}(t)$ = historical mean discharge to SRS for week t [cfs];
CQ = lagged flow coefficient [dimensionless];
CR_1, CR_2, CR_3 = lagged rainfall excess coefficient [cfs/in.];
$r(t)$ = $[RF(t) - RF_{mean}(t)]$;
$RF(t)$ = rainfall during week t [in.];
$RF_{mean}(t)$ = historical mean rainfall for week t [in.];
K = pan evaporation coefficient;
$e(t)$ = $[EVP(t) - EVP_{mean}(t)]$;
$EVP(t)$ = pan evaporation during week t [in.];
$EVP_{mean}(t)$ = historical mean pan evaporation for week t [in.]; and
t = weekly time step.

Therefore, the rainfall-driven component is given by:

$$Q_{rfd}(t) = Q_{mean}(t) + q(t) \tag{3.4.2.4}$$

The regulatory component, on the other hand, is a refinement of the existing schedule for WCA-3A (broken line in Figure 3.4.2.4). Thus, transition zone D was included in the schedule as part of the Rainfall Plan. The amount of regulatory discharge prescribed within this transition zone is given by the formula:

$$Q_{trans}(t) = 2{,}500\ [S(t) - S_{min}(t)];\ Q_{trans} \geq 0 \qquad (3.4.2.5)$$

where:

$Q_{trans}(t)$ = regulatory component of discharge when the water level in WCA-3A is in the transition zone [cfs];

$S(t)$ = water level (WCA-3A 3-gage average) at the beginning of week t [ft NGVD]; and

$S_{min}(t)$ = water level at the bottom of the transition zone (Zone D in WCA-3A regulation schedule) at the beginning of week t [ft NGVD].

The coefficient 2,500 in Equation 3.4.2.5 represents the discharge from WCA-3A at, or near, the capacity of the outlet structures by the time the water level in WCA-3A has reached Zone A.

Everglades Rain-Driven Operations

The rain-driven operational (RDO) concept includes rules for importing water from upstream sources such as EAA runoff, EAA Storage Area, and/or Lake Okeechobee, to the appropriate Water Conservation Areas, and importing/exporting water from the appropriate WCA in order to mimic a desired target stage hydrograph at key locations within the Everglades system. Key locations are entered in the SFWMM as row, column values. Rotenberger and Holey Land Wildlife Management Areas (WMAs) are also operated under the rain-driven concept. Target stage hydrographs, based on an estimate of the pre-drainage water level response to rainfall using the Natural System Model (NSM), or variations thereof, were used as operational targets for achieving hydrologic restoration of the Everglades in the simulation of the restoration alternatives. The target stage hydrographs mimic an estimate of the more natural water level response to rainfall. Thus, the RDO rules are intended to improve the timing and spatial distribution of water depths in the Water Conservation Areas (WCAs) and Everglades National Park and to restore more natural hydropatterns. Modifications to the operational schedules for Water Conservation Areas 2A, 2B, 3A, 3B, and the current rain-driven operations for Everglades National Park, will be made to implement rain-driven operations for all of these areas.

The term “trigger” refers to a gaged or ungaged location whose water level is used to trigger action at an upstream or downstream structure. The term "trigger level" means the water level used to trigger action at an upstream or downstream control structure. These trigger levels are related to the pre-processed target stage hydrographs by simple offsets, which range from +/- 1.0 feet. There is usually one trigger level for the import rules, and two trigger levels associated with the exportation of water. The two export trigger levels define two release zones. The lower zone is a conditional release zone such that releases are made only if the downstream area has a “need.” The upper zone is an unconditional, or flood control release zone, such that releases are made in this zone even if the downstream area does not “need” the water. The trigger levels were adjusted during the modeling process via trial-and-error in order to maximize the matching of the simulated hydropatterns to the natural (pre-drainage) hydropatterns.

In addition to adjusting the trigger levels, adjustments to the preprocessed input hydrographs can be made. Once a target hydrograph has been generated (for example, from NSM), three adjustments of the hydrograph can be made:

1. Translations – Adjustment to target depth prior to the application of any minimum or maximum depth criteria; e.g. increase the target depth by 0.2 ft;
2. Truncations – Apply a maximum or minimum threshold depth to the target location; e.g. target depth not to exceed 1.5 ft depth (any depths that are greater are set to 1.5 ft); and
3. Offsets – Adjustment to target depth following the application of truncation criteria; e.g. increase the truncated target depth by 0.2 ft.

Translations can either be positive or negative, but every value of the time series target will be adjusted similarly (Figure 3.4.2.6). The truncation values are applied to the time series hydrograph after any translations have been applied. Truncations can be made to the maximum depth or minimum depth of the pre-preprocessed hydrograph and will cause the maximum and/or minimum value to be used (instead of the pre-preprocessed hydrograph values) during a period when the hydrograph exceeds the truncation values (also shown in Figure 3.4.2.7).

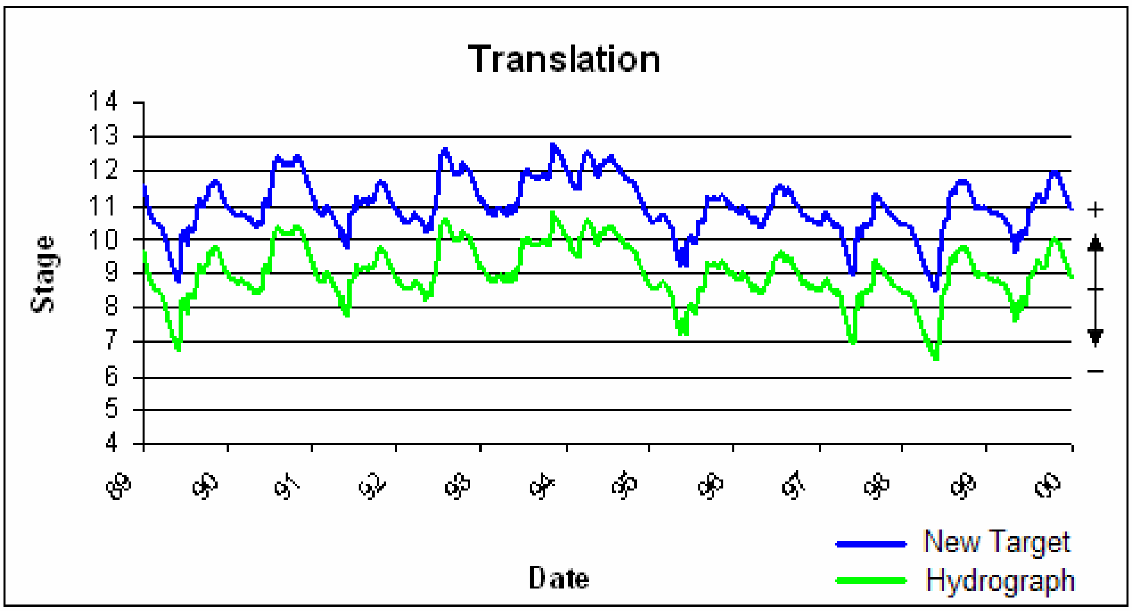


Figure 3.4.2.6 Example of Translation for Hydrograph Targets

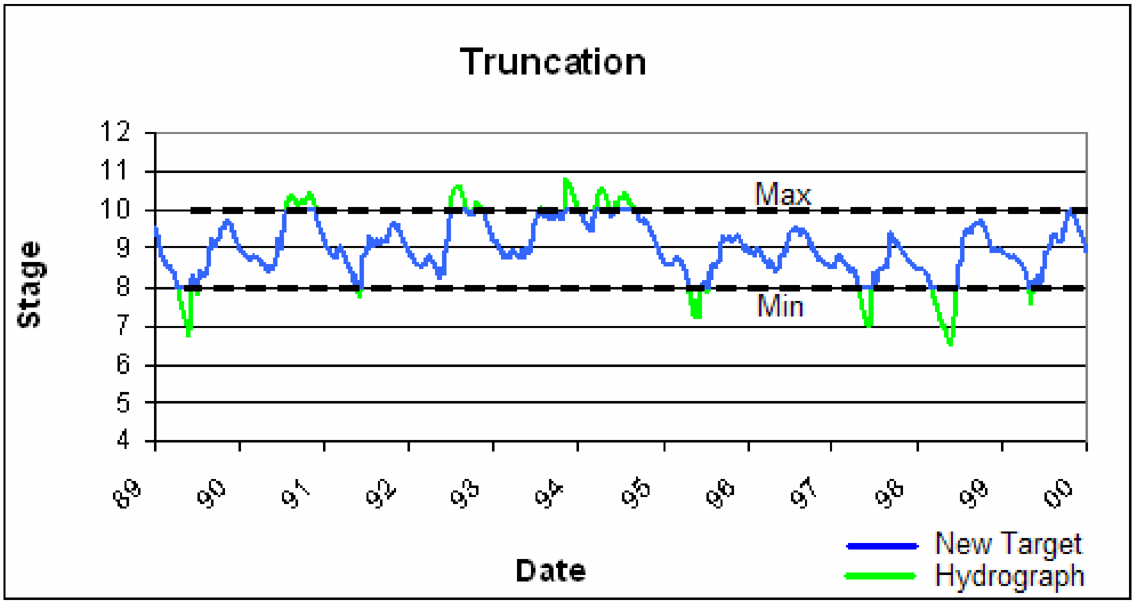


Figure 3.4.2.7 Example of Truncation for Hydrograph Targets

There are two different kinds of offsets: one for importing water (Figure 3.4.2.8) and one for exporting water. The import offset triggers water supply from the upstream water body. The export offset triggers a flood control discharge which sends water to the downstream location. The general purpose of the offset is to allow smoother operations than would be present if the model was trying to hit a finite value, which would result in oscillating operations. If the predicted stage is approximately equal to the pre-preprocessed time series stage, then no operation would be necessary. Only during times when the two stages diverge (beyond the offset limits) would a structural operation be needed. For example, if the import offset is 0.25 feet, then no operations would be needed until the predicted stage is greater than the target stage by 0.25 feet.

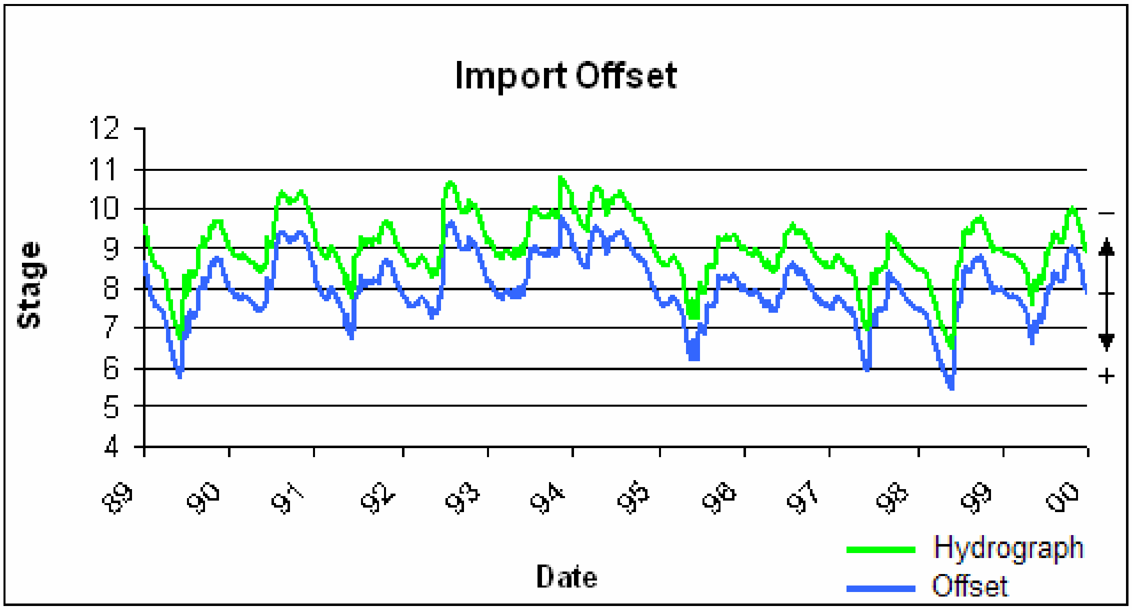


Figure 3.4.2.8 Examples of Import Offset for Hydrograph Targets

A location can have more than one import trigger. Multiple triggers can be used to set the priority of the source of water. The first water-source priority that the model would use, to attempt to meet the need, would be a source associated with the smallest import offset. For example, the gage G1502, located in Northeast Shark River Slough (NESRS) (row 17, column 24), could have an import offset which is associated with the Lake Belt water source in the model. It can also be entered as WC2BP (see Table 3.4.2.6) and water would come from a different source (WCA-2B in this case). By having an offset in WC2BP different than the offset for G1502 (refer to Table 3.4.2.6, "-9" compared to "0"), the priority could be to receive water from WC2BP first. In this example, close attention must be paid to the sign convention. The import trigger is positive in the down direction, therefore an import value of "-9" means the model will try to achieve a depth of 9 feet above the pre-preprocessed target hydrograph. In effect, that means there is almost always a demand for importing water from WC2BP.

The export offset has two values that act as thresholds (Figure 3.4.2.9). The first threshold, offset 1, creates a conditional release of water – if the receiving area is within import limits. The second threshold, offset 2, creates an unconditional release of water – regardless of the downstream condition. The export sign convention is positive (up). If the predicted stage is lower than the target hydrograph plus offset 1, then no structure operations are initiated. If the predicted stage is greater than the target hydrograph plus offset 1, but lower than the target hydrograph plus offset 2, then a structural release will be triggered ONLY IF the downstream area is below its import target. If the predicted stage is above the target hydrograph plus offset 2, then a structural release will be made regardless of downstream conditions.

Table 3.4.2.6 Examples of Import and Export Triggers

Targets Location	Import /Export	ROW COL	Translation	Upper Truncation	Lower Truncation	Offset 1	Offset 2
G1502	import	17 24	0.5	0.2	-99	0	n/a
HOLYL	export	42 20	0	0.9	-1	-0.19	-0.08
LKHIY	import	45 18	0	1.3	-1.4	0.54	n/a
LOXSL	import	59 36	0	99	-99	0.60	n/a
NESRS	import	20 22 21 25	0	99	-99	0	n/a
ROTEN	import	43 16 46 15	0	1.4	-0.4	0.21	n/a
ROTEN	export	43 16 46 15	0	1.6	-0.9	-0.16	0
ST3HL	import	45 18	0	1.3	-1.4	0.22	n/a
WC2BP	import	17 24	0.5	0.2	-99	-9.00	n/a

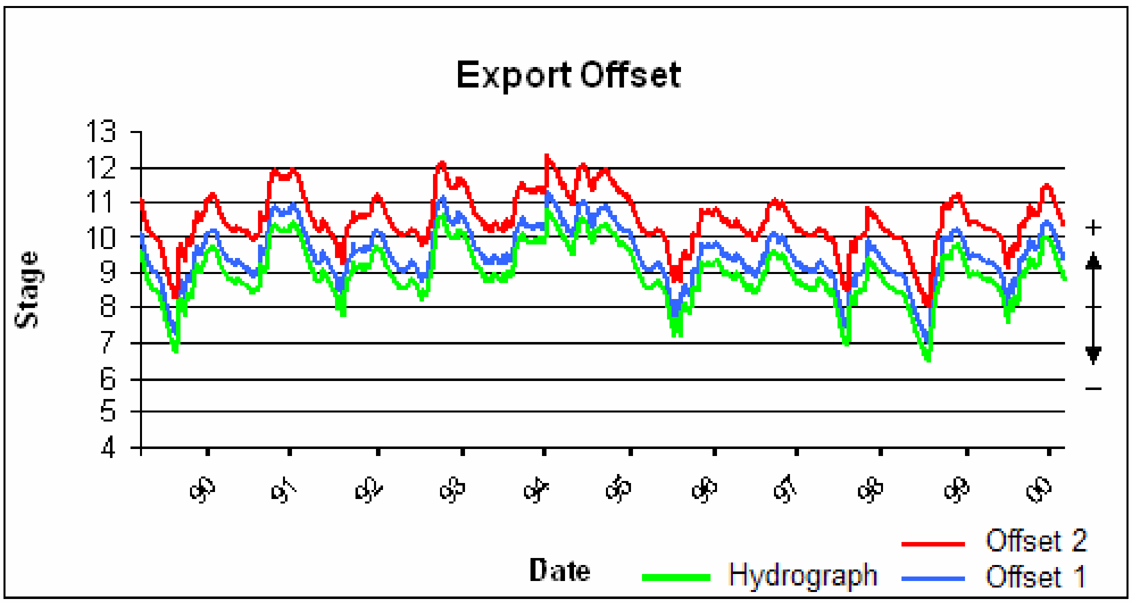


Figure 3.4.2.9 Examples of Export Offsets for Hydrograph Targets

Target hydrographs are typically associated with NSM predictions of stage. However, other time series hydrographs can be used – particularly if scientific evidence or ecological preference would indicate otherwise. The potential sites for NSM triggers are shown in Figure 3.4.2.10. The model provides the flexibility to allow a correlation between any cell and a particular structure. Care should be given to only correlate sites that have direct correlations and do not have intervening structure flows.

A SFWMM representation of the rain-driven operations within the Everglades system is presented in Figure 3.4.2.11. Deliveries from upstream sources (EAA runoff, EAA Storage area, and/or Lake Okeechobee) are routed through the STAs prior to release into the WCAs or the WMAs. The distribution of STA outflow is designed to improve hydropatterns.

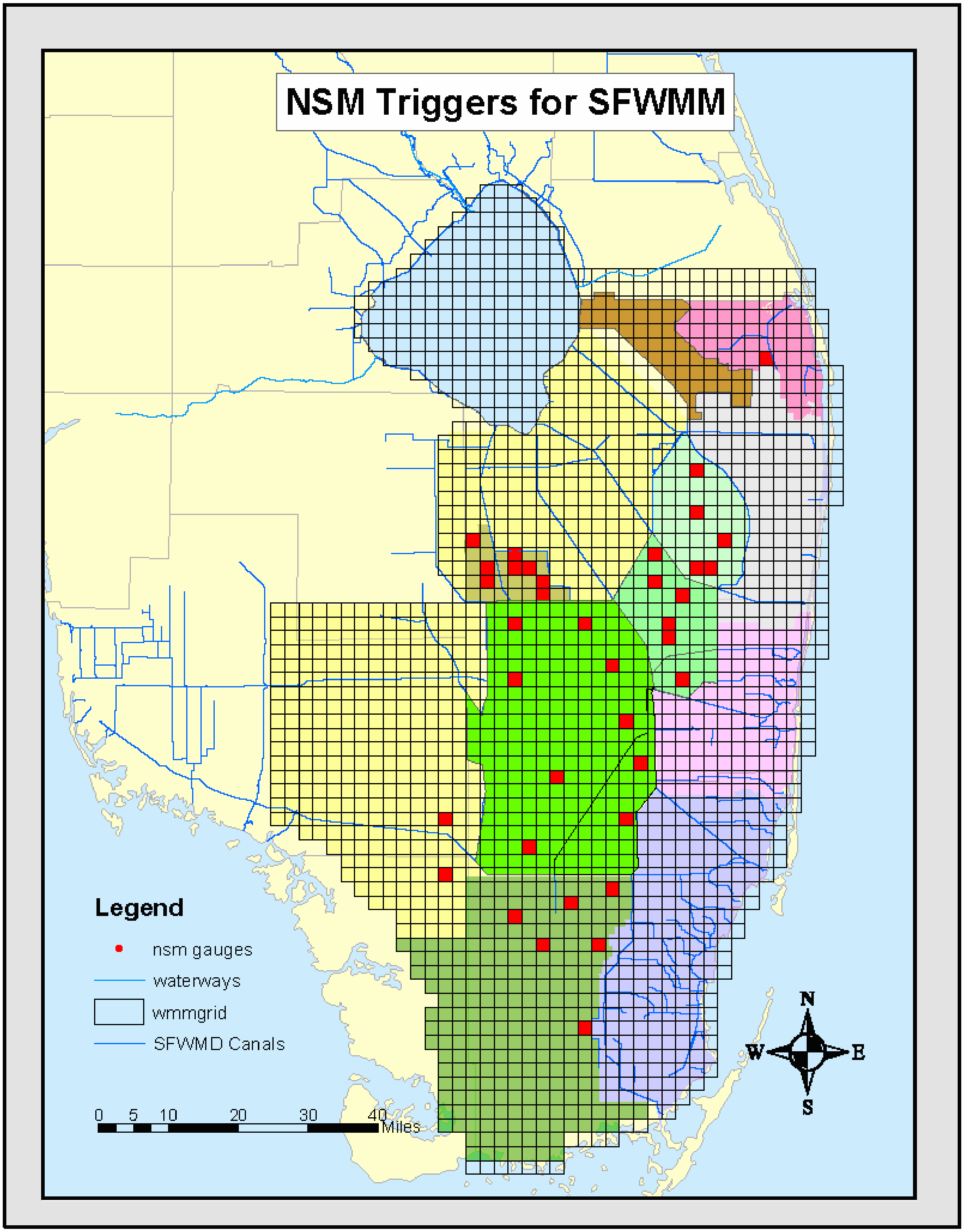


Figure 3.4.2.10 Potential Import and Export Trigger Sites from NSM

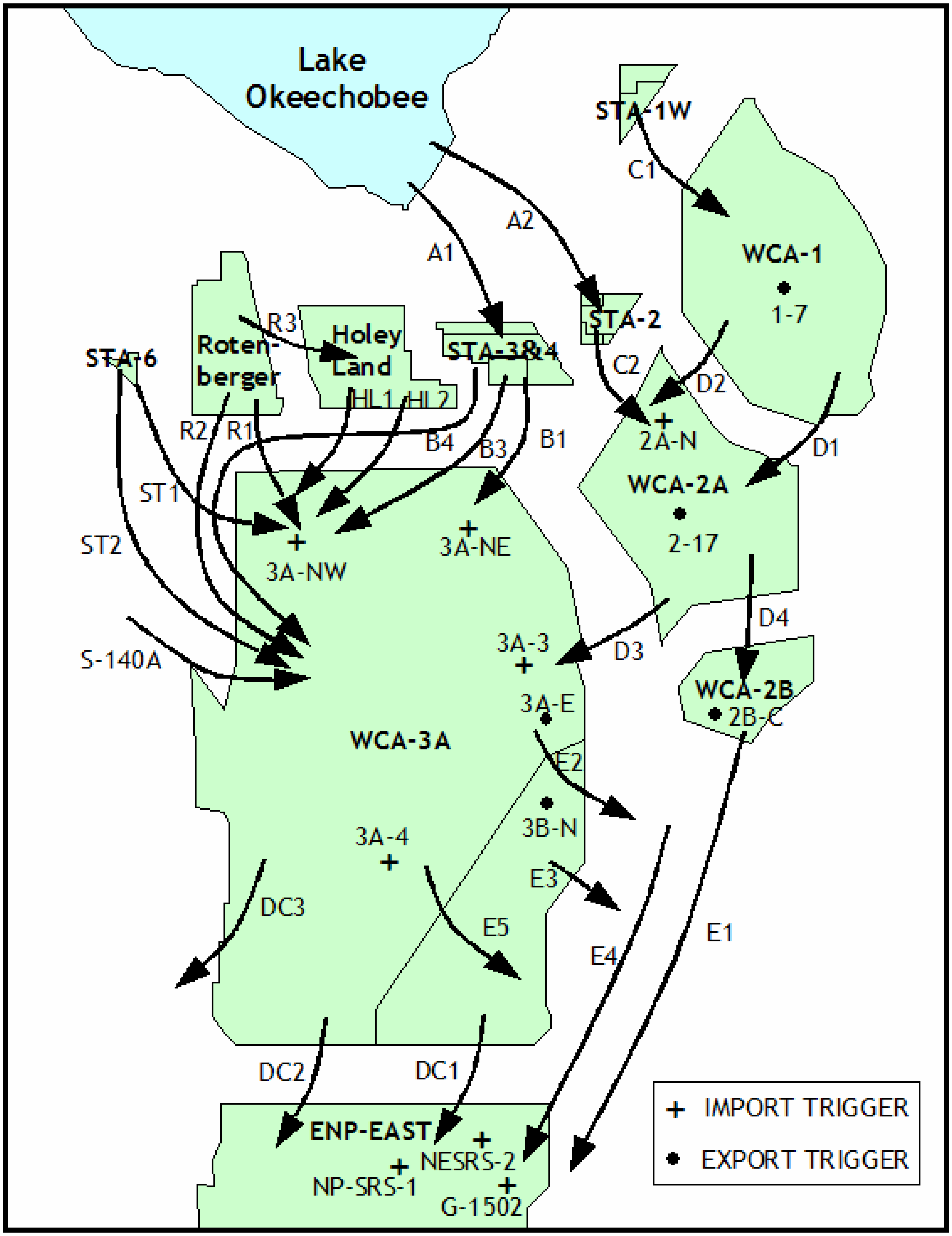


Figure 3.4.2.11 Flow Routing Associated with Example Triggers for Everglades Rain-Driven Operations

3.5 SIMULATION OF THE LOWER EAST COAST OF SOUTH FLORIDA

3.5.1 Introduction

An important management option available in the model is its ability to impose short-term water restrictions on the various water users within the Lower East Coast (LEC) of South Florida. Sources for water consumption within the LEC can be broken down into three categories: (1) wellfield withdrawals made to meet public water supply needs; (2) irrigation used to satisfy supplemental requirements (in addition to rainfall) of different LEC urban water use types (landscape, nursery, agriculture, and golf course); and (3) regional deliveries made to maintain LEC canals at desired levels. These desired levels, also referred to as maintenance levels, are necessary to prevent saltwater intrusion from the eastern seaboard, and to some extent, to satisfy agricultural needs within the LEC. The first two categories use groundwater from the surficial aquifer, primarily the Biscayne aquifer, while the third category utilizes surface water available from the Water Conservation Areas and Lake Okeechobee.

Sections 3.1 and 3.2 explain the rules involved in limiting/restricting water deliveries from the Water Conservation Areas (via "floor" elevations) to the LECSAs and from Lake Okeechobee (via supply-side management) to the LOSAs, respectively. The objective of this section is three-fold: (1) to explain how the model estimates the amount of water necessary to keep the LEC canals at their maintenance levels and how it is eventually met; (2) to show the unsaturated zone accounting procedure as it relates to the pre-processed quantities (PET, ETU, IRRIG, etc.) generated from the ET-Recharge model (refer to Section 2.3); and (3) to explain the trigger and cutback mechanisms in the model as applied to the different water use types in Lower East Coast.

3.5.2 Water Supply Needs Calculations

Lower East Coast (Figure 3.5.2.1) water supply needs on the regional surface water system (WCAs and LOK) are defined as the surface water deliveries required from outside the LEC service areas necessary to maintain the LEC canals at desired levels. LEC water supply needs can also be referred to as water use requirements or surface water requirements. Deliveries from the regional system are needed especially during dry periods when LEC groundwater levels are at their lowest and the potential for saltwater intrusion is greatest.

A canal network is a system of canals served by one or more outlets of a storage area or a reservoir. It may consist of a single canal reach or a complicated system of canal reaches. A canal reach is a continuous section of canal bounded by control structures, and can contain numerous inflow and/or outflow points. The procedure used to estimate the water supply needs will be explained by way of an example (Figure 3.5.2.2).

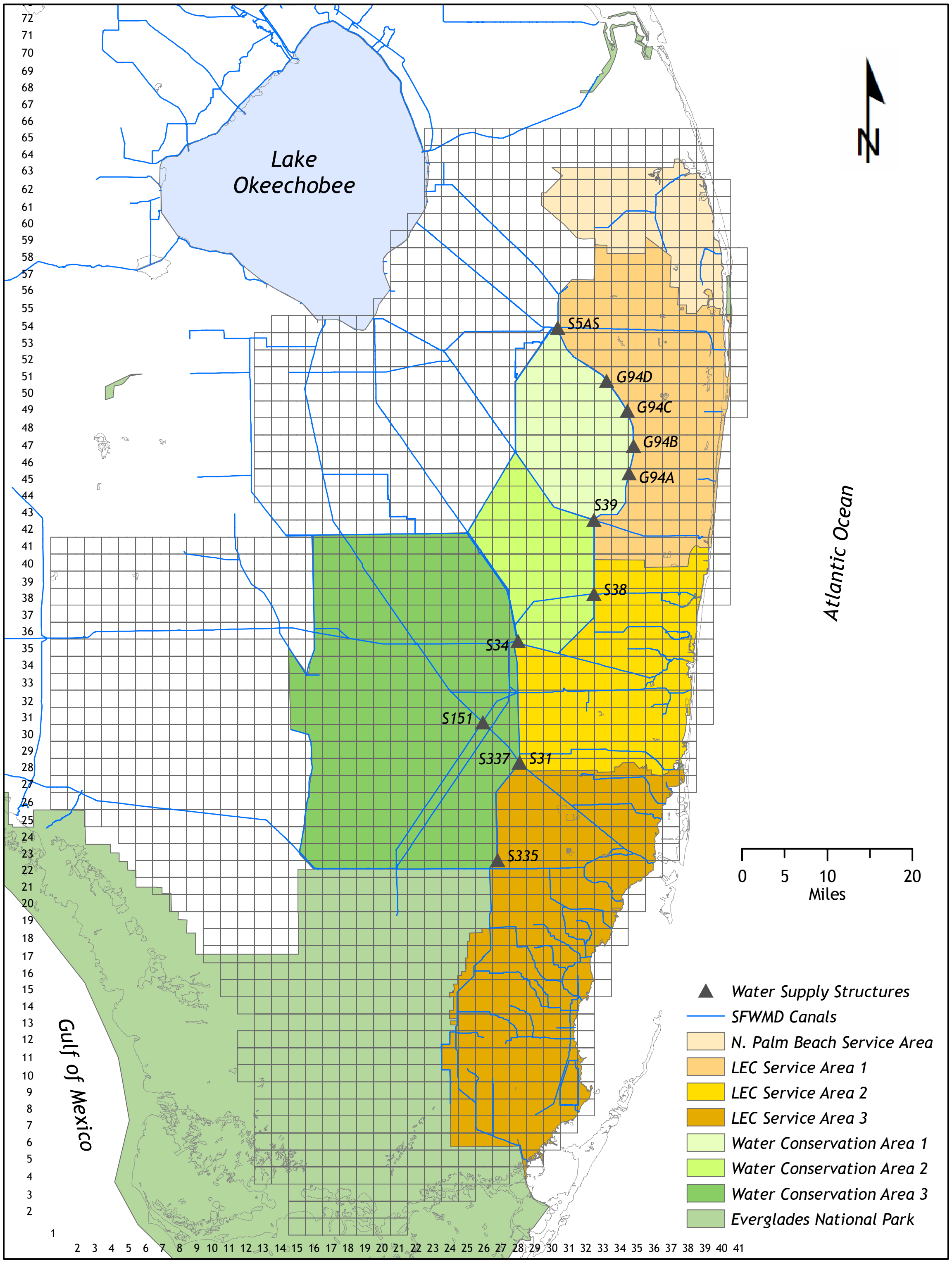


Figure 3.5.2.1 Primary Structures Used in Making Water Supply Deliveries to the Three Service Areas within the LEC

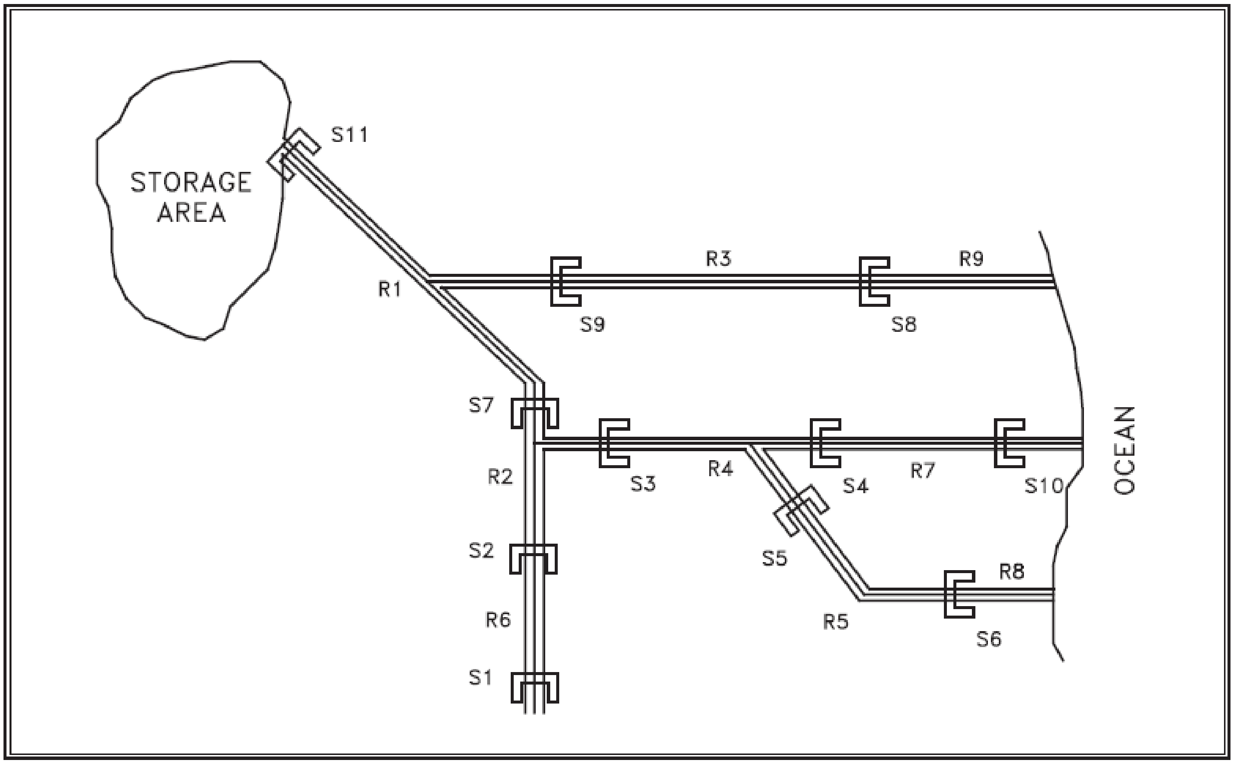


Figure 3.5.2.2 Hypothetical Canal Network Used to Explain Water Supply Needs Calculations in the SFWMM

Hypothetical Example

For each canal reach in a canal network, the following information is needed by the model in order to estimate surface water requirements (Table 3.5.2.1):

1. Total head drop (from upstream end to downstream end) of water surface. This quantity is assumed to remain constant throughout the simulation for this example (in SFWMM v5.5, the ability to simulate some canals with varying slopes is added).
2. Canal maintenance level. A value of -9.5 means that the water level in a canal reach is not maintained.
3. Average width of the canal reach.
4. Canal-aquifer conductivity or connectivity coefficient which is used to calculate canal-groundwater seepage or in general, canal-groundwater interaction.
5. Name of upstream canal reach discharging into the canal reach of interest.
6. Number of downstream outflows simulated for water supply, which is less than or equal to the number of outflows in canal.

To route water from a storage area through the canal system for water supply purposes, the following input information is also required for each canal reach (Table 3.5.2.2):

1. total number of outlet structures simulated in a canal reach;
2. names of downstream structures; and
3. names of canals receiving the water from each outlet structure.

Table 3.5.2.1 Canal Definition Data for Example Hypothetical Canal Network

Canal Reach Name	Head Drop (ft)	Average Width of Canal Reach (ft)	Canal-Aquifer Conductivity Coefficient (ft/day/ft-head)	Maintenance Level (ft NGVD)	Upstream Canal Reach to be Maintained	Number of Outlet Structures for Water Supply: 1 for no outlets
R1	0.3	80	3.0	5.0	none	2
R2	0.2	80	5.0	4.0	R1	2
R3	0.1	60	2.0	4.5	R1	1
R4	0.0	50	6.0	3.0	R2	2
R5	0.1	100	10.0	2.0	R4	1
R6	0.1	80	10.0	3.0	R2	1
R7	0.0	100	5.0	2.0	R4	1
R8	0.0	100	5.0	-9.5	R5	1
R9	0.0	100	3.0	-9.5	R3	1

Table 3.5.2.2 Routing Information for Example Hypothetical Canal Network

Canal Reach Name	Number of Outlet Structures	Names of Downstream Structures	Receiving Canals Corresponding to Each Outlet Structure
R1	2	S7 S9	R2 R3
R2	2	S2 S3	R6 R4
R3	1	S8	R9
R4	2	S5 S4	R5 R7
R5	1	S6	R8
R6	1	S1	free outfall
R7	1	S10	free outfall
R8	uncontrolled	none	ocean
R9	uncontrolled	none	ocean

Furthermore, the names of the farthest maintained downstream canal reach for each branch must be defined; R6, R7, R5 and R3 are used in the hypothetical example. Note that R8 and R9 are uncontrolled on the downstream end and as such, cannot be maintained. The names must be input in the order by which the model will sum the surface water requirements. With this information (Table 3.5.2.3), the model knows where to begin or continue when a canal branches into tributaries. Thus, through user-input, the model can calculate the total volume of water required to maintain any number of canal reaches within a canal network.

Table 3.5.2.3 Branch Information for Example Hypothetical Canal Network

Total Number of Branches in Canal Network	Names of Most Downstream Canal Reaches (to be maintained) for Each Branch
4	R6 R7 R5 R3

Surface Water Requirements for a Single Canal Reach

In order to estimate the surface water requirements for a single canal reach, a simple mass balance approach is used. The volume of water needed to maintain a canal reach at a desired minimum level is:

$$VOL_j = \left(desired\_min\_level_j - cstg\right)\left(area_j\right) - \sum_{i=1}^{ncells_j} seep_i - \sum_{i=1}^{ncells_j} ovlnf_i \qquad (3.5.2.1)$$

where:

j = index of the canal of interest which is the j th canal input in canal definition file;
i = grid cell index where canal j passes through;
$ncells_j$ = number of grid cells where canal reach j passes through;
$cstg_j$ = simulated downstream stage in canal j at the beginning of time step;
$area_j$ = surface area of canal j;
$seep_i$ = canal-groundwater interaction; net seepage inflow into canal j; and
$ovlnf_i$ = canal-surface water interaction at the i th grid cell; net sheetflow into canal j.

The $desired\_min\_level_j$ is defined for the most downstream grid cell of the canal reach, i.e., at the headwater of the downstream structure. At any other grid cell i where the canal reach passes through, the desired minimum level can be calculated as

$$desired\_min\_level_{j,i} = \left[\ (\text{distance of i th grid cell from downstream structure} \div \text{total length of canal reach})(\text{total head drop})\ \right] + desired\_min\_level_j \qquad (3.5.2.2)$$

Desired_minimum_level$_{j,i}$ and the average groundwater level at the i th grid cell are used in the calculation of $seep_{j,i}$. On the other hand, $cstage_{j,i}$ and the average surface water level (land surface elevation + ponding) for the i th grid cell are used in the calculation of $ovlnf_{j,i}$.

VOL_j can be positive or negative. Negative values of VOL_j represent excess water available in canal j which can be used to meet downstream needs.

The total volume of water required for water supply at any structure in a canal network [DQU(j) where j equals the canal number for the first canal directly downstream of the structure] is the sum of:

1. the volume of water required to maintain the canal reach immediately downstream of the structure (VOL_j); and
2. the total volume of water required for all canal reaches downstream of canal j.

If water is available in the canal of interest, i.e., $VOL_j < 0$, and its volume is sufficient to meet downstream needs, i.e., $|VOL_j| \geq DQU(j-1)$, then, no water is required from the structure upstream of canal j. The total water supply requirement for the structure is then set to zero.

Surface Water Requirements for a Canal Network

The methodology applied in determining the total needs for the hypothetical canal network is summarized in Figure 3.5.2.3. The arrows and the numbers in this figure indicate the sequence of calculations. The following discussion pertains to Figure 3.5.2.3.

In order to estimate the surface water requirements for a canal network, an accumulation of needs from the most downstream canal reach to the source structure (S11 in Figure 3.5.2.2) for the network is performed. The most downstream canal reach is the last reach to receive water and the first to drop below its desired minimum level when water is insufficient. Canal reach R6 in the hypothetical canal network is the first in the series of most downstream canal reaches given in Table 3.5.2.3. Starting with this canal reach, the algorithm accumulates the water requirements while moving upstream along the main trunk (R6-R2-R1) until one of the following conditions occurs:

(1) The canal of interest ($canal_j$) branches into at least one tributary whose water level(s) has(have) to be maintained. The number of tributaries equals NBRANCH(j)-1 (column 2 in Table 3.5.2.2); or

(2) The trunk or main branch terminates with the canal of interest. This condition implies that variable IFF(j) equals 0 or a storage area exists upstream of the canal of interest. Variable IFF(j) is the canal reach number immediately upstream of the canal of interest. If the total needs for an entire canal network have been determined without the occurrence of condition **(1)**, the canal network either has no tributaries or none of the canal reaches in the tributaries has to be maintained.

If condition **(1)** occurs, i.e., NBRANCH(j) is greater than 1, the algorithm first determines the water requirements (VOL_j) for the canal reach of interest. In the hypothetical example, condition **(1)** first occurs for R2 which discharges into R4 (a tributary of level 1 relative to R2 which is part of the main branch) through structure S3. Then, beginning with the next most downstream canal reach being maintained (R7 as given in Table 3.5.2.3), the algorithm accumulates the needs upstream along this tributary. Initially, the number of occurrences of condition **(1)** before condition **(2)** corresponds to the number of branches beyond the main branch of the network. When condition **(2)** occurs, the algorithm will add the total needs for the tributary, i.e., DQU(iupsc) where iupsc is the canal number of the most upstream reach, to the volume of water required (VOLj) to maintain canal j immediately upstream of canal iupsc. To determine the individual total needs for each of the remaining tributaries of canal j and then continuing to the remaining canal reaches in the main branch of the canal network, the following procedure is followed:

> Each time condition **(1)** occurs before the next occurrence of condition **(2)** the algorithm will accumulate the needs along the tributary in the next level, beginning with the most downstream canal reach being maintained. This will occur the same number of levels beyond the level at which condition **(2)** last occurred. Condition **(2)** occurs when the needs for all canal reaches in the tributary of interest have been determined.

The hypothetical network of canals shown in Figure 3.5.2.2 has a total of four branches or tributaries - corresponding to the number of occurrences of condition **(2)**. The general flowchart for calculating water supply needs in the model is shown in Appendix F3.

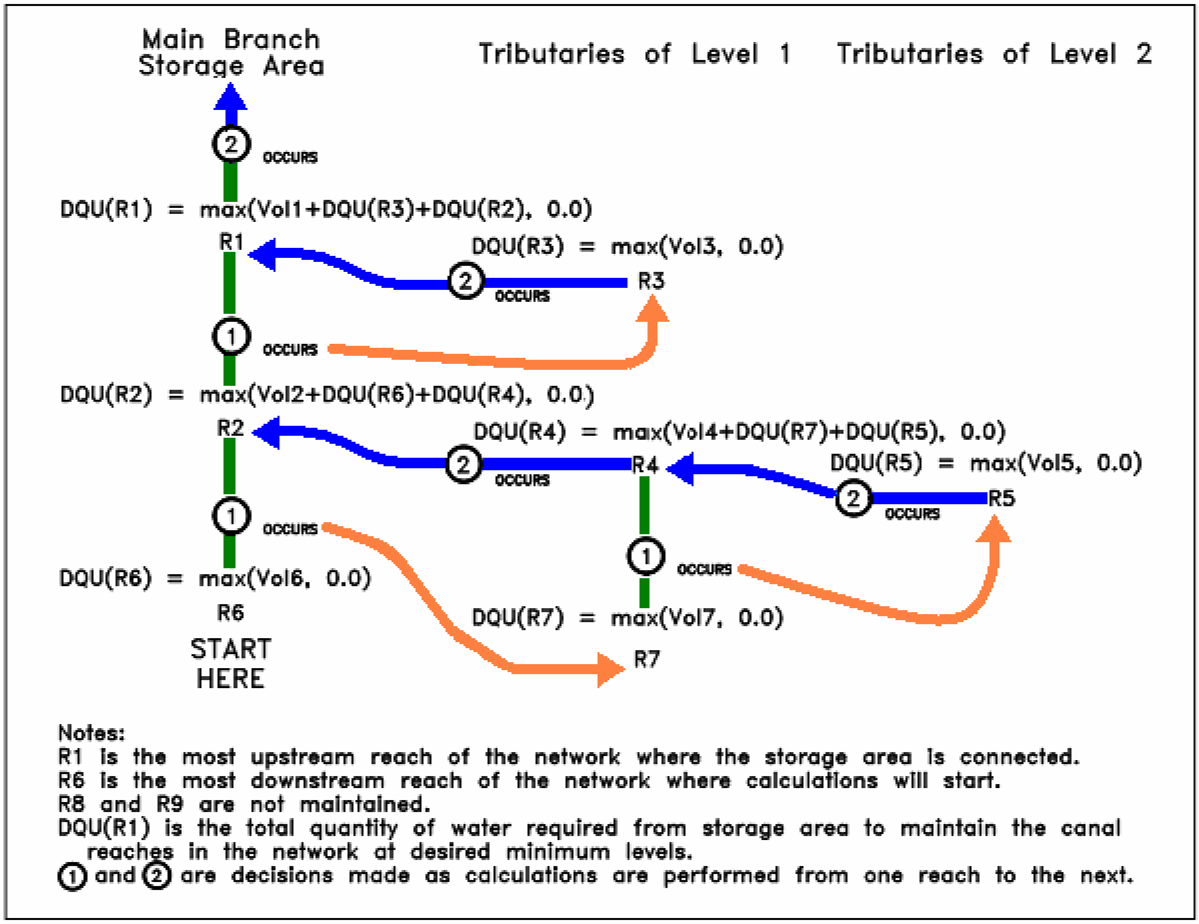


Figure 3.5.2.3 Sequence of Water Supply Needs Calculations for the Hypothetical Canal Network

The availability of surface water from a storage area such as LOK or WCA to meet downstream water requirements depends on a minimum storage level set forth for the particular storage area. Within a WCA, the canal water level below which no outflow will be made to the Lower East Coast without an equivalent amount of inflow from Lake Okeechobee is referred to as the conservation area's floor elevation.

Calculation of Available Supply. After water supply needs from a specific region, e.g., Service Area 2, are calculated, the amount of available water to meet these needs from an upstream source, e.g., WCA-2A, is calculated next. The volume of available water in a WCA is defined by the available storage within the appropriate canal reach, e.g., WCA-2A rim canal. The SFWMM calculates this volume as:

$$\text{AVVOL} = (\text{sim\_canal\_stage} - \text{min\_stage})\ (\text{sim\_surface\_area\_of\_canal})$$

$$+ \sum_{i=1}^{\text{ncells}} \text{ovlnf}_i + \sum_{i=1}^{\text{ncells}} \text{seep}_i + \sum_{i=1}^{\text{ncells}} \text{upstream inflow} \qquad (3.5.2.3)$$

where:

i = grid cell index where canal of interest passes through;
ncells = number of grid cells where canal reach of interest passes through;
sim_canal_stage = simulated canal stage at a given time step;
min_stage = floor elevation or user-defined canal stage with corresponding storage above which can be made available to meet downstream needs;
sim_surface_area_of_canal = length of a canal reach multiplied by its average width (The SFWMM assumes canals with vertical walls such that this value does not vary with canal water level.);
ovlnf_i = canal-surface water interaction at the i th grid cell; net sheetflow into canal of interest;
seep_i = canal-groundwater interaction; net seepage inflow into canal of interest; and
upstream inflow = known net structure inflow to canal of interest.

Surface Water Deliveries from a Storage Area. In situations when a particular storage area, e.g., WCA-2A, has multiple outlet structures, e.g., S-38 and S-34, the model has to decide how to distribute the available storage among the different outlet structures. The amount of water routed through each outflow structure ($QOUT_i$) can be calculated in two ways:

1. If the user chooses the option to pro-rate the available water, i.e., "equal adversity" condition, the outflow through the i th structure will be proportional to the relative water supply demands calculated at that structure. It is calculated as:

$$ratio_i = \frac{AVVOL - \sum_{j=1}^{i-1} QOUT_j}{tot\_water\_required - \sum_{j=1}^{i-1} water\_required\ for\ j^{th}\ structure} \qquad (3.5.2.4)$$

$$Q_i = \min(ratio_i, 1.0)(\text{water required for i}^{\text{th}}\text{ structure}) \qquad (3.5.2.5)$$

$$QOUT_i = \min(Q_i, \text{structure capacity}_i) \qquad (3.5.2.6)$$

2. If the user chooses to deliver the amount of available water in the order by which the downstream structures are simulated, the outflow at the i th structure is calculated as:

$$AV_i = AVVOL - \sum_{j=1}^{i-1} QOUT_j \qquad (3.5.2.7)$$

$$Qout_i = \min(AV_i, \text{structure capacity}_i) \quad (3.5.2.8)$$

Distribution of Water Supply through the Receiving Canal Network. The ability of the system to meet water supply needs calculated above is constrained by the available storage and conveyance capacity of each structure in the receiving canal network. Therefore, conveyance limitations and water availability are checked at every structure throughout the network as water deliveries are being made.

When the supply of water becomes limited for the downstream reaches at a particular canal network, actual water delivery becomes less than the calculated water supply needs. In situations when one canal reach has two or more outflow structures delivering water and available water in the upstream canal is insufficient to meet all the requirements, then the user, similar to the way deliveries are handled from storage areas, is given the following options:

1. pro-rate available water; or
2. deliver available water to meet the requirements at the outflow structures in the order by which they are specified by the user, i.e., in the same order by which the outflow structures are input.

3.5.3 Unsaturated Zone Accounting in the Lower East Coast

As mentioned in Section 2.5, the unsaturated zone is treated as a separate control volume where infiltration, percolation, evapotranspiration and changes in soil moisture content are accounted for. Due to the level of detail required to model crop evapotranspiration rates and irrigation requirements for the major irrigation use types in the LEC (landscape, nursery, golf course and agriculture) an off-line determination, pre-processing, is performed using the ET-Recharge model (Giddings and Restrepo, 1995) whenever necessary. This section explains the accounting procedure used in the SFWMM to integrate pre-processed, cell-based, time series data such as PET, unsaturated zone ET and irrigation schedule, with the other hydrologic components such as infiltration, percolation and runoff.

First, some assumptions used in the model with regards to unsaturated zone accounting are given. To keep the unsaturated zone water budget simple, the model assumes that all soil moisture in the unsaturated zone is readily available for plant use on any given day. This implies that the model does not differentiate between the upper and lower root zones where degrees of moisture extraction vary. The SFWMM performs a moisture accounting on the unsaturated zone whose control volume changes with time. The zone may or may not exist at all at the end of a time step depending on the location of the water table and/or the magnitude of the pre-calculated (by the ET-Recharge model) evapotranspiration amount which the model has to "remove" from the unsaturated zone. In contrast, the AFSIRS component of the ET-Recharge model performs a root zone water budget with time-invariant control volume. The SFWMM also assumes that the inefficient component of irrigation that evaporates does not significantly alter the water budget for the saturated zone.

Finally, the portion of the "inefficient" irrigation that returns to the aquifer does so in the same day it is applied such that it does not affect the solution of the groundwater flow equations. The groundwater flow equations are solved once at the end of the day and processes that can deplete as well as recharge the aquifer within the same time step may add complication to the overall algorithm of the model. Thus, the model accounts only for the net irrigation of the day and includes its contribution to the recharge term prior to the solution of the groundwater flow equations.

Figure 3.5.3.1 is a schematic of the three control volumes (ponding, unsaturated zone, saturated zone) considered in irrigated model grid cells which apply to portions of Palm Beach, Broward and Miami-Dade Counties east of the WCA protective levees (Neidrauer, 1993). It is a simplified form of Figure 2.5.5.2 and shows hydrologic components pertinent to the current discussion. The movement of water among these three control volumes is accounted for in the model on a daily basis. The model distinguishes between evapotranspiration coming from the three distinct control volumes: evaporation from ponding (ETP), evapotranspiration from the unsaturated zone (ETU), and evapotranspiration from the saturated zone (ETS). It further distinguishes between unsaturated zone ET from irrigated portions of a grid cell (ETIU) and non-irrigated portions of the same cell (ETNU). Although both are pre-processed values from the ET-Recharge model, the distinction is necessary in order to implement a water restriction rule. In particular, the model assumes that only the evapotranspiration from the irrigated portions of the model will diminish as a consequence of a water restriction cutback. Net irrigation supply (NIRRSUP) refers to the portion of the pre-processed irrigation requirement that ends up in the unsaturated zone. This quantity becomes less than what is required for the day when a cutback is imposed by the "trigger" module in the model. It varies with irrigation use types. Six predominant irrigation use types were identified in the Water Shortage Plan (SFWMD, 1991): urban landscape, nursery, golf course, low-volume agriculture, overhead agriculture, other agricultural usage. Given acreage information, the ET-Recharge model generates a schedule of irrigation depths per use type per SFWMM grid cell per day. This information is input to the model. The model, in turn, produces restricted (after water restrictions, if any, are implemented) unsaturated zone evapotranspiration for irrigated cells and actual (after water irrigation cutback, if any, are imposed) irrigation supply.

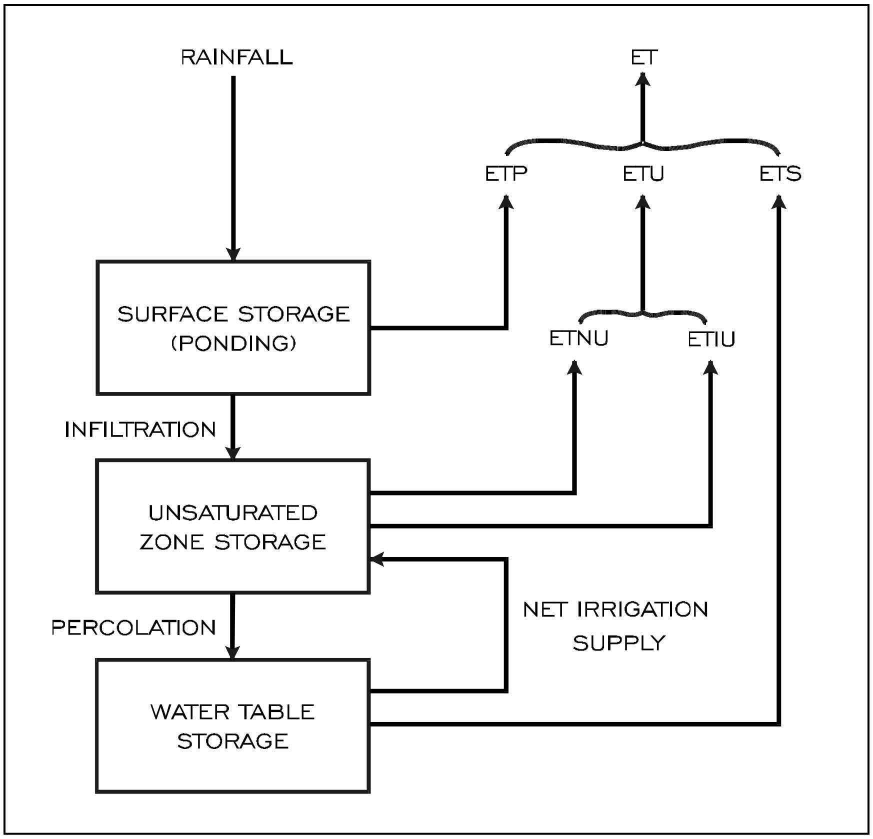


Figure 3.5.3.1 Systems Diagram of Processes Simulated in the SFWMM for Irrigated Cells within the LECSA

3.5.4 Water Shortage Plan for the Lower East Coast

The Water Shortage Plan (SFWMD, 1991) for the Lower East Coast Service Areas is the counterpart of the Supply-Side Management Plan for the Lake Okeechobee Service Area. It is the basis for incorporating a short-term water restriction scheme on the six irrigation use types and public water supply (domestic and industrial consumption). The initial process for declaring water use restriction in the field would be an evaluation of salinity levels at key monitoring points within the area. During droughts, if such levels become abnormally high, a water restriction may be "declared" after consultation among water managers within the District. However, since water quality modeling is not part of the SFWMM, a surrogate measure of water shortage is used: groundwater levels. These levels or heads are monitored within the model at key trigger wells and canals. A "trigger module" was created to incorporate provisions in the Water Shortage Plan in the South Florida Water Management Model. This module is comprised of three major tasks: (1) monitor heads at key gage locations; (2) declare water restriction phase if the monitored heads fall below some threshold values; and (3) cutback water use at appropriate locations: pumpage for public water supply consumption and irrigation, at levels consistent with the restriction phase (Table 3.5.4.1).

Table 3.5.4.1 Proposed Cutbacks for Simulating the Short-Term Water Use Restrictions in the LECSA

Water Usage or Class	Phase I	Phase II	Phase III	Phase IV
Public Water Supply*	.15	.30	.45	.60
Urban Landscape#	20.0	13.3	6.7	3.3
Nursery#	14.5	7.3	4.2	3.0
Agriculture - Overhead#	6.1	6.1	3.6	3.6
Agriculture - Low Volume#	20.0	20.0	20.0	20.0
Agriculture - Other#	20.0	20.0	4.5	3.6
Golf Course#	4.8	3.2	1.4	0.6

NOTE:*For public water supply, cutbacks are expressed in terms of fraction of the total pumpage.
#For irrigation use, cutbacks represent maximum irrigation application rates in inches per month.

First, heads at user-specified grid cells or canals are compared with prescribed limits on a daily basis. If the heads fall below one of the four limits corresponding to the four levels of drought intensity, a counter is updated of such occurrence. This step will inform the model which areas within the model domain are in a drought situation on any given day. These affected areas or "zones" are assumed to be well represented by a proper selection of trigger cells or canals where heads are being monitored. In the current implementation of the trigger module, four zones: North Service Area (with five trigger cells), Service Area 1 (with nine trigger cells), Service Area 2 (with seven trigger cells), and Service Area 3 (with eight trigger cells), are defined (Figure 3.5.4.1). The second task is carried out at the end of each month. A water restriction is declared if the frequency of heads falling below the limits reaches a user-specified maximum number of times. The appropriate water restriction phase, corresponding to the drought intensity is also identified in this task. Finally, based on user-specified levels of cutback, the amount of pumpage for affected public water supply wells is reduced and irrigation application maxima or caps per irrigation use type are imposed for the succeeding months until the end of the dry season. The amount of cutback is lowered from a more severe water restriction phase to a less severe phase within the dry season if heads at the trigger locations assigned to the zones where water usage (public water supply or irrigation use) is being cutback rebound in succeeding month(s) prior to the end of the dry season. A flowchart of the Water Shortage Plan as implemented in the SFWMM is provided in Appendix F4.

Reduced irrigation translates into a decrease in evapotranspiration. Unsaturated zone evapotranspiration rates are calculated on a daily basis by AFSIRS (Smajstrla, 1990) via the ET-Recharge model. They assume unrestricted conditions, i.e., moisture via excess rainfall and irrigation is always available. Restricted unsaturated zone evapotranspiration, on the other hand, is estimated within the SFWMM by means of a regression equation that approximates AFSIRS (refer to Section 3.3.2). The regression equation is of exponential type that treats ET from the unsaturated zone as a function of irrigation, rainfall and potential evapotranspiration.

$$ETIU_{est} = (a)(net\_irrig^{b})(mon\_rf^{c})(mon\_pet^{d}) \quad (3.5.4.1)$$

where:

net_irrig = net monthly irrigation calculated by AFSIRS which, in turn, is called within the ET-Recharge model [in];
mon_rf = total monthly rainfall [in];
mon_pet = total monthly potential evapotranspiration [in]; and
a, b, c, d = regression coefficients which vary as a function of irrigation use type.

Since water use restrictions are imposed on a monthly basis, reductions in ET in months when water use restrictions are imposed can be calculated by subtracting $ETIU_{est}$ from the monthly accumulated unrestricted unsaturated zone ET, $ETIU_{AFSIRS}$. By post-processing this information the actual crop yield reduction can be related to ET reduction using a yield response function (FAO, 1988). The trigger module was designed as a simple procedure for implementing the District's Water Shortage Plan into the South Florida Water Management Model.

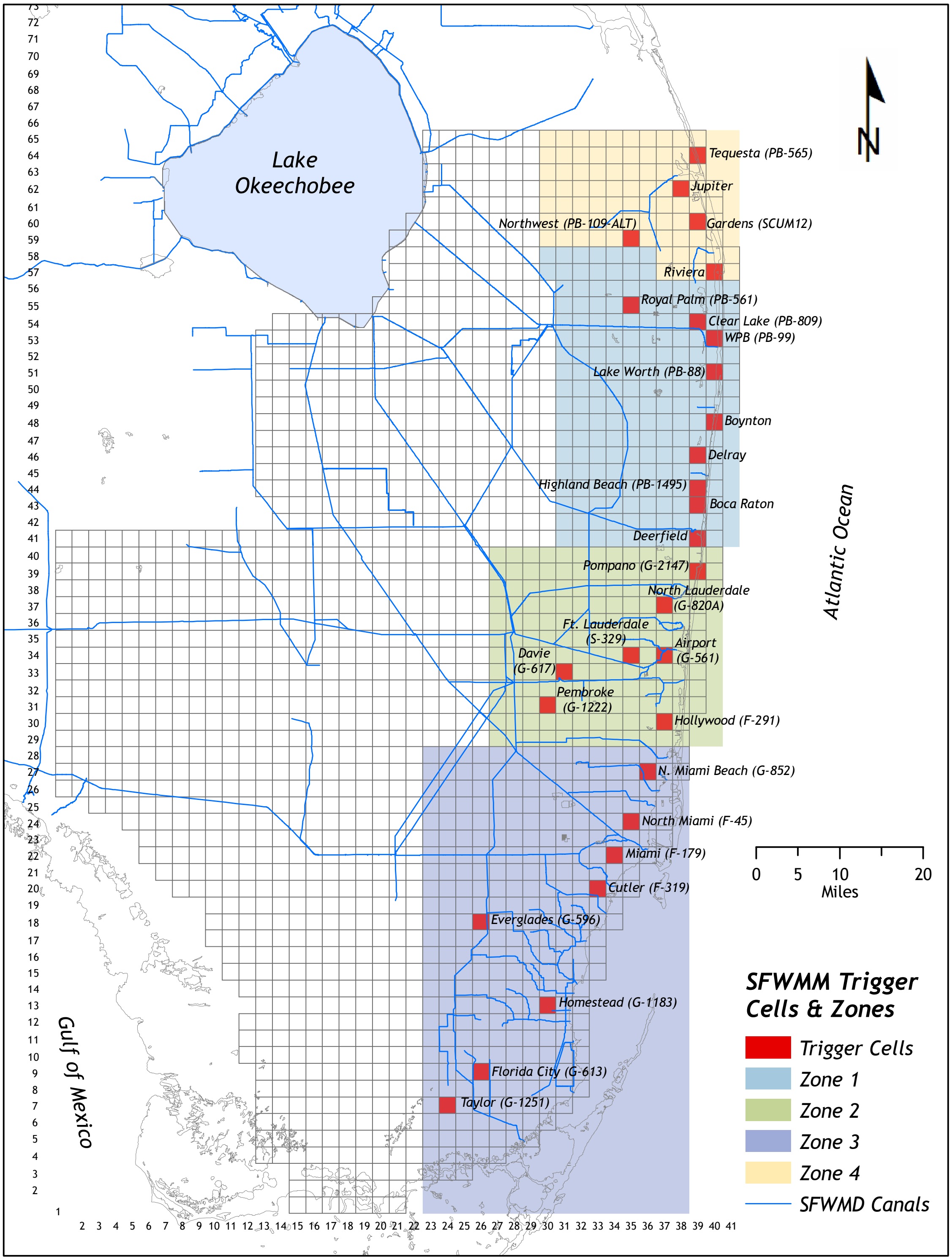


Figure 3.5.4.1 Location of Key Trigger Cells in the SFWMM Used to Trigger Water Restrictions in the LEC Developed Area

3.5.5 Public Water Supply Well Pumpage

The historical well pumpage data file for the SFWMM v5.5 was extended to include the period 1996-2000. Historical pumpage data prior to 1996 was available from earlier model versions (Brion, 1999). The primary source of data was the USGS Water Resource Division, through the publication of historical water use data (1996-2000) for fifteen South Florida counties. The data represents reported monthly pumpages from the different water utilities at different well field locations. Groundwater sources (surficial vs. Floridian aquifers) were also used in the final determination of pumpage input data for the model. Utility-reported pumpage for the last year of simulation, 2000, was obtained from the SFWMD Water Use Regulation Division. Raw total monthly pumpages were used in the final determination of pumpage input data for the model.

A permit is issued by the South Florida Water Management District in order to give water use rights to a public utility or any other entities. Different water use permits are issued to withdraw water from the surface system or from groundwater storage. The permits referred to here apply to groundwater withdrawals. A water use permit specifies the location, and the annual and monthly maximum withdrawal. Public water supply utilities and major irrigation applications require water use permits. These include golf course, nursery and other agricultural operations. Single residential houses are exempt from the permit application process.

Historical pumpage for some water allocation permits were excluded during certain years due to several reasons:

1. the permit might have already expired;
2. the permit was considered significantly small relative to the 2mile-by-2mile resolution of the model;
3. the permit referred to surface water withdrawals which are not explicitly simulated as withdrawal amounts in the SFWMM; or
4. some permits were combined with others as a result of permit re-applications during the 1996-2000 period of record.

A FORTRAN program was used to transform reported pumpages associated with permits to pumpages assigned to SFWMM grid cells. The program has two basic inputs: wellfield pumpage file which shows monthly pumpages sorted by permit number and well distribution file which specifies the SFWMM grid cell assignment for each well that comprises each public water supply permit.

Five wellfield pumpage files were set up corresponding to each of the calendar years 1996 through 2000. Each file contains permit numbers, total pumpage for the year, and 12 monthly distribution factors. The wellfield pumpage files are essentially translations of the raw pumpage data obtained from the public utilities after some initial data screening/refinements as discussed above.

The information provided by a unique combination of a wellfield pumpage file and the well distribution file into the FORTRAN program produces an output file that contains monthly pumpage values assigned to appropriate model grid cells for a particular calendar year. The program was run five times, once for each of the calendar years 1996 through 2000, and the corresponding five output files were concatenated to produce a composite 1996-2000 SFWMM public water supply pumpage input file.

The pumpage data processed for this effort extended the historical data set for SFWMM. A list of the total pumpages for all LEC service areas for the modeling period of record is given in Table 3.5.5.1.

Table 3.5.5.1 Average Daily Withdrawal from Lower East Coast Surficial Aquifer for Public Water Supply in Eastern Palm Beach, Broward and Miami-Dade Counties

Year	Pumpage [MGD]	Year	Pumpage [MGD]	Year	Pumpage [MGD]
1979	607	1987	735	1995	782
1980	607	1988	751	1996	810
1981	624	1989	774	1997	799
1982	614	1990	676	1998	832
1983	604	1991	728	1999	841
1984	639	1992	770	2000	874
1985	674	1993	782		
1986	686	1994	780		

In general, there is a steady increase in public water supply pumpage through the years of calibration/verification (period-of-record 1979-2000). The occasional down trends occur immediately after the dry years, e.g. 1981 and 1989. The 1979-1995 average is 696 MGD while the 1996-2000 average is 831 MGD. On an annual average basis, the distributions of pumpage for all service areas for the 1996-2000 period are shown in Figure 3.5.5.1. More information on the determination of Public Water Supply Well Pumpage Data can be found in Appendix O.

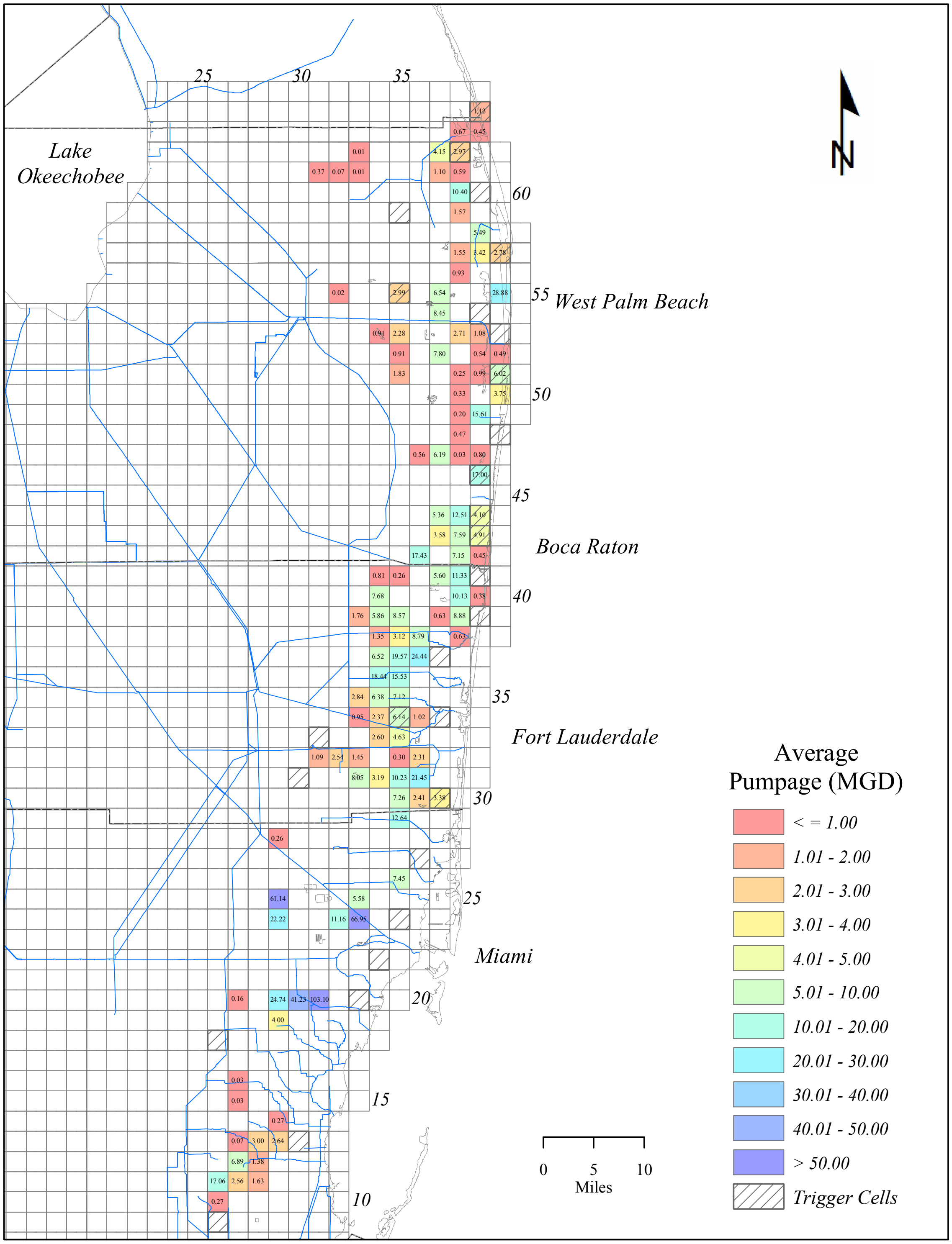


Figure 3.5.5.1 Distribution of Annual Average Public Water Supply Pumpage (1996-2000) in the LEC Based on the SFWMM Grid Network

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3.6 STORAGE AND ADDITIONAL MANAGEMENT OPTIONS

Previous sections of this chapter have addressed many of the capabilities of the SFWMM on a region by region basis. This section will address the generic topic of simulation of storage in the model and will also describe some of the unique system management topics that have not been described to this point. The storage components covered in this section include: Large and Small Reservoirs, and Aquifer Storage and Recovery (ASR). The additional management options covered include: Best Management Practices, Wastewater Reuse and Operational Planning.

3.6.1 Storage Options

The primary types of storage simulated in the SFWMM are reservoirs and ASR. In the SFWMM, reservoirs are water holding systems that capture water either for later use or for the preservation of wetlands within the reservoir system. Aquifer Storage and Recovery is a water management technique in which water is stored underground in a suitable aquifer through a well during times when the water is available and recovered from the same well when needed. In the SFWMM, ASR systems are also considered to be reservoirs.

The SFWMM has the ability to model two types of above-ground reservoirs: small reservoirs that are modeled as separate entities within a grid cell; and large reservoirs that are equal to, or nearly equal to, a grid cell size and are not treated as a separate entity within the cell. For either type, the user has the option to specify basic design parameters, basic hydrologic connections, location, and operations. Reservoirs can be completely contained within a grid cell or across several grid cells. The model assumes all reservoirs to have vertical walls. It accounts for differences in the actual area of the reservoir and the area represented by the grid system, i.e., multiples of four square miles. Since rainfall and evapotranspiration depths are assumed to occur uniformly for each model grid cell, their effect on reservoir stage is transformed using a proportionality factor relating reservoir area and the area of the grid cell(s) where the reservoir is located. For a given reservoir:

$$sfactor = \frac{tot\_reservoirarea}{(no.\ of\ grid\ cells)(gridcellarea)} \tag{3.6.1.1}$$

The change in reservoir stage within time step t is approximated using the following equation:

$$\Delta reservoirstage_t = \frac{RF_t - ET_t + LSEEP_t + GWIN_t}{sfactor} - (RF_t - ET)_t\,(1.0 - sfactor) \tag{3.6.1.2}$$

where:

RF_t = rainfall into grid cell;
ET_t = evapotranspiration out of grid cell;
$LSEEP_t$ = levee seepage into grid cell; and
$GWIN_t$ = net groundwater inflow to grid cell

Reservoir stage is used in determining available storage in the reservoir. It is also the basis for calculating discharges through inlet and outlet structures (pumps and weirs). The primary operations associated with storage features tend to center around rules for transferring between adjacent basins or other storage facilities. Inflow and outflow to storage in the SFWMM can be related to a number of triggering mechanisms including:

- rising or declining adjacent canal stage;
- capture of local basin runoff;
- capture of releases from upstream storage;
- demand in downstream basins including agricultural water supply deficit, environmental water supply, etc. (quantified in a manner similar to that described for structure operations);
- projected long-term or short-term climate conditions (e.g. seasonally varying operations or pre-storm discharges);
- mitigation of high stages in above-ground reservoir (e.g. overflow prevention).

These triggered flows can be subject to a number of constraints including conveyance limitations, maximum storage capacity and coordination with other storage components (e.g. multiple sources associated with one objective). The interaction between storage features and various sub-regions with the SFWMM model domain is one of the unique aspects of the model. As an illustration of this feature, Figure 3.6.1.1 shows an example operational schedule for ASR wells associated with Lake Okeechobee. In this example, if LOK stage is above the Pulse release zone or if Lake Okeechobee stage is forecasted to be above the "ASR Injection" line within three months, Lake Okeechobee water is injected into ASR wells. For recovery during the dry season, water is retrieved from ASR wells if Lake Okeechobee stage is currently below, or is forecasted to be below in six months, the "ASR Recovery" line. During the wet season water is retrieved if LOK stage is below the "ASR Recovery" line and if the climate based inflow forecast is less than 1.5 million acre-ft for the next six months. The reader is reminded that these generalized operational strategies are used as an example of model capability only. Proposed operational rules for these features are continuously evolving with time as they go through brainstorming, field-testing and rule-making processes.

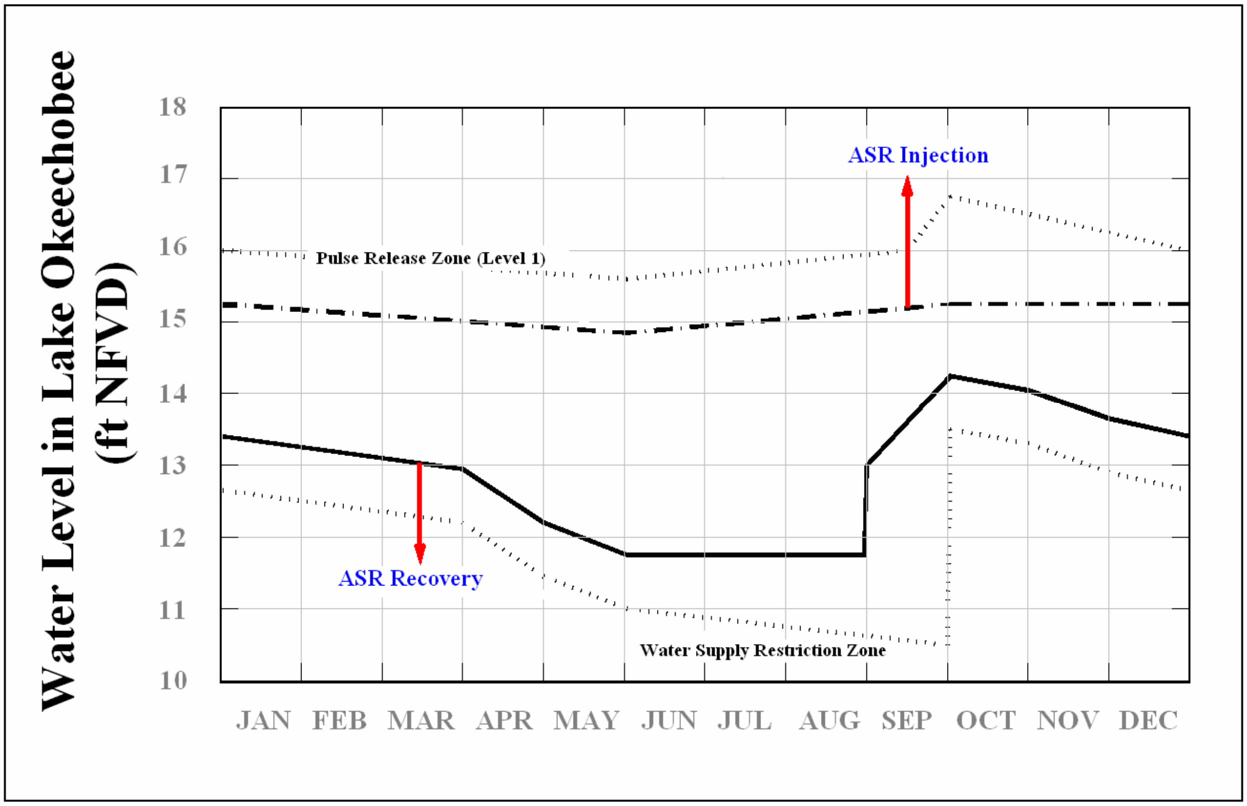

Figure 3.6.1.1 Example Trigger Lines for Proposed Lake Okeechobee Aquifer Storage and Recovery

Large Reservoirs

The simulation of large reservoirs can be categorized as follows:

- Managed Reservoir (e.g. STAs and EAA/LEC Reservoirs)
  - store water for later use;
  - actual area is important in modeling reservoir;
  - localized seepage losses can be simulated as a structural outlet.

- Unmanaged Reservoir (e.g. Holey Land)
  - leveed systems which store water that is not intended for later use;
  - approximate area is adequately defined by grid system.

The key modeling elements to be considered when simulating large reservoirs are as follows:

1. the cell(s) in the reservoir are grouped in a separate hydrologic basin;
2. any surface water flow from external cells to the reservoir cells is simulated with passive broad-crested weirs;
3. the reservoir stage equals the grid cell stage when ponding in the cell equals zero;
4. the flow intended for the reservoir is spread over the entire grid cell;
5. the mean stage for the area outside the reservoir (but within the reservoir grid cell) is calculated based upon an accumulated (over time) water budget for the area outside the

reservoir (when ponding in the grid cell is less than zero, the mean stage of the reservoir can not drop below ground level);

6. losses from the reservoir must be adjusted when the accumulated outflows exceed accumulated inflows for the area outside the reservoir; and
7. the reservoir land surface elevation and land use type must be the same as that in the grid cell.

The key modeling limitations of simulating a large reservoir are:

1. there can not be both a large and a small reservoir simulated in the same grid cell;
2. the model is sensitive to the reservoir's location within a grid cell only for levee seepage calculations;
3. the total recommended area of the reservoir must be no greater than 10 percent above the total grid cell area if the topography within the reservoir varies 0.5 feet or greater compared to neighboring cells.

The reservoirs are often designed to operate with a passive weir outflow. In those cases, the calculations will handle hydrologic conditions when the ponding depth in the reservoir is higher than the tailwater condition of the weir, even if the tailwater depth is greater than the height of the weir.

The order of computations, for a daily time step, is as follows:

1. the levee seepage calculations;
2. the reservoir depth adjustments for inflows to the reservoir which may come from the Lake or from a canal;
3. the overland flow, ET, and infiltration calculations;
4. groundwater flow and residual infiltration calculations;
5. the reservoir outflow determination based on the specified operation (weir or target delivery);
6. the reservoir storage calculations; and
7. the daily values written, if desired by user.

Small Reservoirs

Small reservoirs are treated as separate entities (for stage and water budget purposes) from the cell(s) in which the reservoir is placed. The primary function of small reservoirs is to treat and either redistribute or attenuate flows. Examples of small reservoirs would include STA 6 (which treats the inflow) or the C-111 Buffer Strip reservoirs (which redistribute and attenuate inflows). An example of a small reservoir that redistributes flow would be the proposed ACME Basin reservoir. The first check performed by the model to see if the reservoir can be treated as a small, separate entity is the size ratio of reservoir to cell area as input by the model user. If the size ratio is greater than the input value (typically 0.6), the reservoir should be treated as a large reservoir. A reservoir can be considered small even if the reservoir spans two or more cells, but the size ratio in any one cell should be less than the input value.

The key modeling considerations when simulating small reservoirs are:

1. cells containing a reservoir do not have to be grouped in a separate hydrologic basin;
2. overland flow can be simulated through a reservoir cell in a similar manner to that in remaining cells;
3. the reservoir stage is independent of grid cell stage;
4. groundwater interaction with the grid cell is via a seepage rate;
5. inflow destined for the reservoir enters the reservoir directly;
6. the mean stage for the area outside a reservoir (within the cell) is simply groundwater level plus ponding;
7. no area adjustment in losses from the reservoir is necessary;
8. reservoir land surface elevation and land use type can be different from that in the grid cell; and
9. one-dimensional overland and groundwater flow within long narrow reservoirs can be simulated (independent of the grid cell).

There are two principal inflows to small reservoirs: direct structural inflow and direct rainfall. Outflows can be structural or non-structural (e.g. seepage). When a multi-cell, narrow reservoir is modeled, it must have a linear arrangement. In those cases, an equation (similar to Manning's equation, but based on effective roughness) is used for one-dimensional flow, flow is computed in a 6-hour time step and the flow width is assumed to be equal to the reservoir width.

The order of computations, for a daily time step in a small reservoir, is similar to that of the large reservoirs (presented earlier). However, after the calculation for groundwater flow and infiltration, there are two routines specifically written to handle small reservoir overland flow and to handle groundwater flow.

Aquifer Storage and Recovery

Although the use of Aquifer Storage and Recovery (ASR) in South Florida is only in its infancy, ASR is a viable water management feature for the region. The popularity of ASR can be seen in Figure 3.6.1.2 (adapted from ASR Systems LLC, 2004) in a measles map – which, over time, shows a growing trend in red dots (new ASR sites). The ability to model ASR was included in the SFWMM to allow for evaluations of the potential application.

The potential uses of ASR in South Florida include: (1) provide additional regional storage while reducing both evaporation losses and the amount of land removed from current land use (e.g. agriculture) that would normally be associated with construction and operation of above-ground storage reservoirs; (2) increase Lake Okeechobee's water storage capability to better meet regional water supply demands for the Everglades, for agriculture, and for the Lower East Coast urban areas; (3) manage a portion of regulatory releases from the Lake Okeechobee primarily to improve Everglades hydropatterns and to meet supplemental water supply demands of the Lower East Coast; (4) reduce harmful regulatory discharges to the St. Lucie and Caloosahatchee Estuaries; (5) maintain and enhance the existing level of flood protection; and (6) for improvement to Lake Okeechobee water levels.

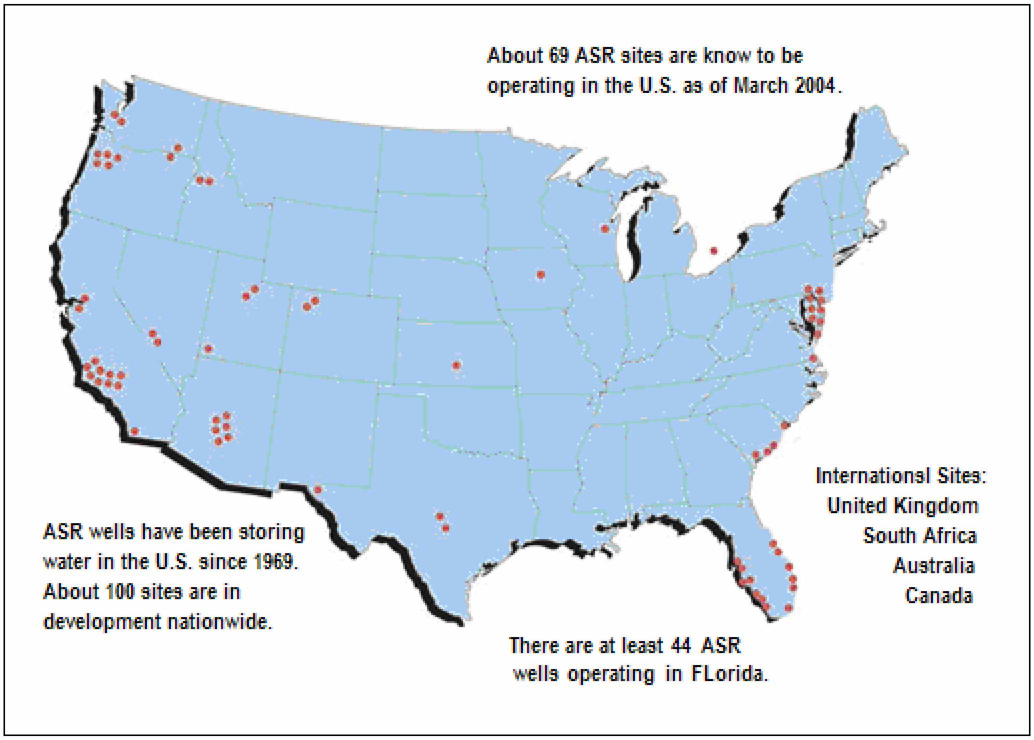

Figure 3.6.1.2 Measles Map Showing Spread of Aquifer Storage and Recovery Wells (Adapted from ASR Systems LLC, 2004).

The SFWMM simulates ASRs by performing a simple water budget on the amount of injected water (assumed to be well below the surficial aquifer) taking into consideration inefficiencies in injection and withdrawal phases of the operation, and basically treating an ASR as a regular reservoir with one obvious difference: ASRs do not lose water via evapotranspiration which is significant in above-ground reservoirs.

In the SFWMM, several forms of ASR are simulated. One form is utility ASR where groundwater is pumped down from the surficial aquifer to the deeper confined aquifer using municipal utilities as the source during the wet season and later retrieved by the municipal utilities to help meet urban needs during the dry season. This is simulated in the SFWMM by simply altering the municipal wellfield data file, which includes increasing pumpage from the surficial aquifer during the wet season and decreasing pumpage during the dry season for the affected wellfields, taking into account the capacity of the utility ASR, and the efficiency in retrieving the water from the utility ASR.

Other forms of ASR simulated are in association with excessive canal flow, local reservoirs, and/or Lake Okeechobee. Pumpage down to ASR is simulated as an additional outlet from the appropriate source. Water recovered from ASR is routed to the appropriate destination. The efficiency of ASR retrieval is controlled by input options, but is typically assumed to be 70%. The net accumulation of excess water injected into the deep aquifer, known as the ASR bubble, is assumed to have no minimum or maximum limit in size unless specified by the user. The ASR bubble size, which is updated on a daily basis, can be a limiting factor in ASR recovery during extended drought periods when there is little or no water left to recover.

ASRs can potentially be placed anywhere within the modeling domain of SFWMM, however several areas have been pre-defined in the model. These areas are: around Lake Okeechobee, the Caloosahatchee Basin with reservoir, along the C-51 canal, several areas (associated with reservoirs) in Palm Beach County; and in the Site 1 reservoir and along the Hillsboro Canal (along the border of Palm Beach County and Broward County). Additionally, a utility ASR well field is located in Miami-Dade County. Other ASR facilities could be added to the model as needed.

3.6.2 Additional Management Options

Best Management Practices

As part of the Everglades Forever Act (Florida Statutes, Chapter 373.4592, 1994) requirements, Best Management Practices (BMPs) have been implemented in the Everglades Agricultural Area (EAA). The objective of BMP implementation in the EAA is to improve water quality in the Everglades Protection Area (EPA) by reducing phosphorous loads.

As a result of BMP implementation, there is an expected runoff reduction from the EAA. The Everglades Forever Act required that the District develop a model to quantify the amount of water to be replaced from Lake Okeechobee to the EPA. District Rule Chapter 40E-63, F.A.C., Part II adopted on October, 1995, established the model for the quantification of runoff reduction during a water year and replacement water to be delivered from the Lake to the EPA from October to February of the next water year. The replacement water was based on data from the 1979 to 1998 base period.

Since BMP replacement water deliveries are a function of rainfall, time series of BMP replacement water spanning the period of simulation (1965-2000) are required. Due to the lack of historical data spanning the 1965-2000 period of simulation, a rainfall-based approach has been used to estimate BMP replacement water time series for the entire period of simulation. The method is based on the strong ($R^2 = 0.97$) logarithmic relationship between EAA average rainfall for water years 1995-2000 and the historical replacement water target for water years 1996-2001 (Figure 3.6.2.1). EAA average rainfall is a weighted-average of rainfall at 9 District monitoring stations defined in Rule Chapter 40E-63 (Table 3.6.2.1).

There are two main reasons for the selection of water years 1995-2000 to assemble the model used here: (1) the 2000 Base simulation should reflect full BMP implementation which was completed around 1995, and (2) water years 2001 and 2002 were excluded due to the extreme drought conditions and water shortages affecting South Florida. For implementing the method, rainfall for nine SFWMM grid cells (Table 3.6.2.1), representing the nine District monitoring stations, was extracted from the SFWMM input rainfall binary file. EAA nine-cell average rainfall was calculated based on the Thiessen weights defined in Rule Chapter 40E-63, which are listed in Table 3.6.2.1. Figure 3.6.2.2 shows that the EAA nine-cell average rainfall for water years 1979-2000 very closely matches the EAA nine-station average rainfall ($R^2 = 0.99$) as expected.

Based on the logarithmic relationship shown in Figure 3.6.2.1 (adapted from Abtew, 2002), the EAA average rainfall obtained from the SFWMM rainfall binary file for water years 1965-2000 was used to estimate target replacement water deliveries for the next water year (Table 3.6.2.2, Figure 3.6.2.2).

Rule Chapter 40E-63 defines fixed monthly percentages of the target replacement water to be delivered to the EPA during October to February of the next water year (Table 3.6.2.3). These monthly factors were applied to the estimated target replacement water deliveries for a water year to obtain monthly target deliveries. January-February, 1965 and October-December, 2000 target deliveries were estimated circularly based on the estimated target replacement water deliveries for a water year made up of the combination of 2000 and 1965 data. For creating the daily time series of replacement water, monthly target deliveries were uniformly distributed throughout the month. Actual replacement water deliveries may be lower than the target given by the Rule due to canal conveyance limitations or the Water Conservation Areas (WCAs) exceeding their regulation schedules. In addition, makeup water deliveries may be suspended when the Lake is under supply-side management.

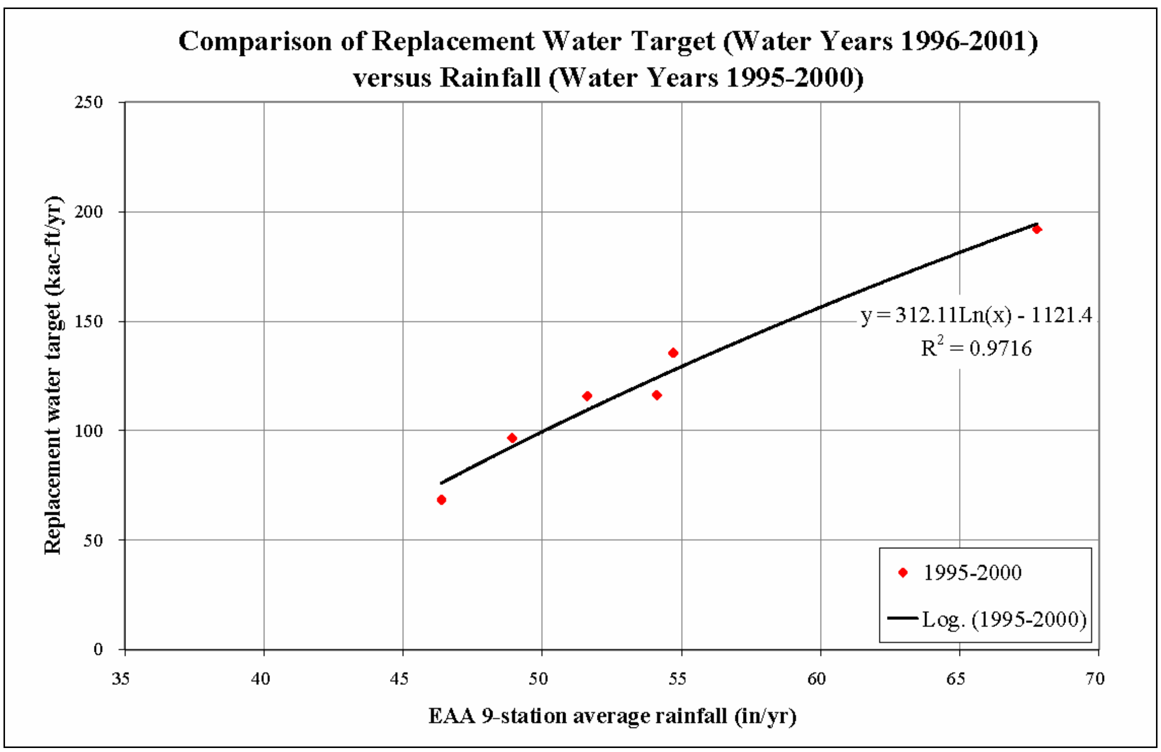

Figure 3.6.2.1 Logarithmic Relationship between EAA Nine-Station Average Rainfall for Water Years 1995-2000 (Adapted from Abtew, 2002).

Table 3.6.2.1 District's Rainfall Monitoring Stations in the EAA used in BMP Replacement Water Calculations

STATION	Thiessen Weight	DBKEY	XCORD	YCORD	SFWMM cell (Row, Col)
ALICO	0.0974	15197	662050.191	792096.255	(48,11)
BELLE-GLADE	0.1617	15200	777082.691	844670.746	(53,22)
MIAMI LOCK_R	0.1076	15198	719458.180	853630.902	(54,17)
PAHOKEE1_R	0.1438	15201	798481.181	901482.752	(58,24)
S5A_R	0.0989	15202	862679.587	855002.715	(54,30)
S6_R	0.0763	15203	837525.021	777745.664	(46,28)
S7_R	0.0592	15204	807896.812	728054.542	(42,25)
S8_R	0.1743	15205	730112.505	726534.776	(42,18)
SOUTH BAY_R	0.0844	15199	753759.327	847537.884	(53,20)

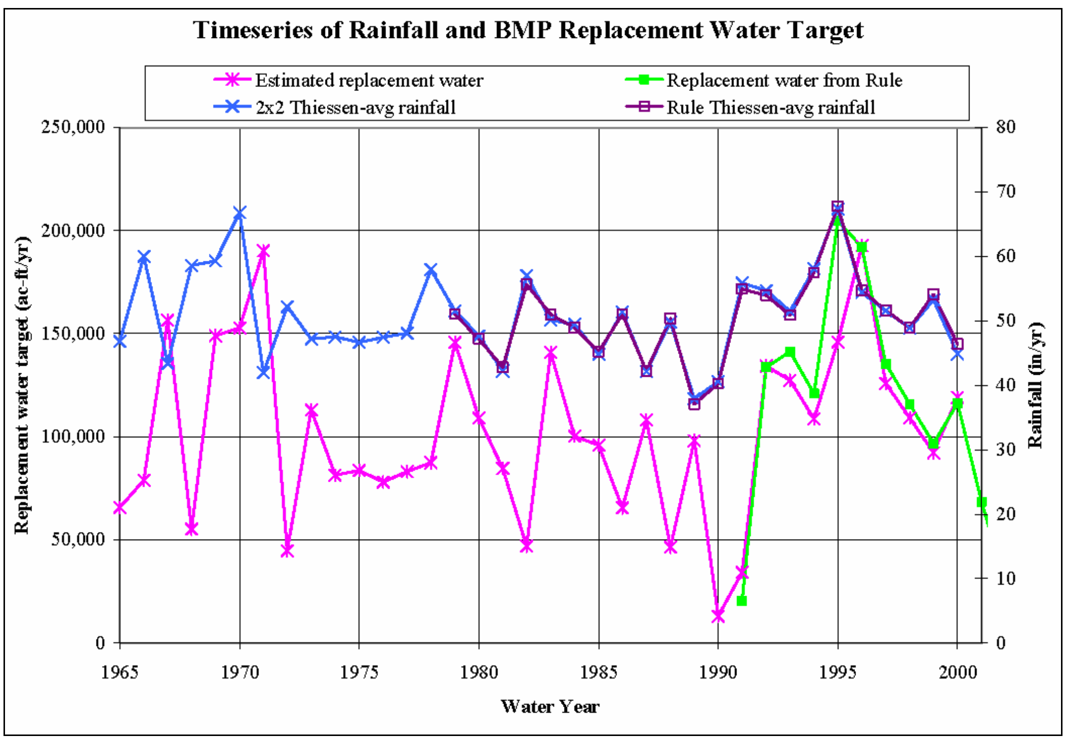

Figure 3.6.2.2 Annual Time Series of EAA Nine-Station Average Rainfall from BMP Rule

Note: Calculations used EAA 9-cell average rainfall from SFWMM rainfall binary, replacement water from BMP Rule and estimated replacement water for SFWMM. There is a one year lag between rainfall and BMP replacement water.

Table 3.6.2.2 Statistics of the Estimated BMP Replacement Water Target Time Series by Water Year

Water Year	Water Year Rainfall (in/yr)	Estimated BMP Replacement Water Target (ac-ft/yr)
1965	46.8	65,747
1966	60.0	78,942
1967	43.4	156,480
1968	58.6	55,167
1969	59.3	149,082
1970	66.8	152,755
1971	41.9	190,077
1972	52.2	44,743
1973	47.2	112,976
1974	47.5	81,451
1975	46.7	83,625
1976	47.4	78,034
1977	48.1	83,043
1978	58.0	87,492
1979	51.5	145,702
1980	47.7	109,023
1981	42.2	84,694
1982	57.1	46,860
1983	50.1	140,917
1984	49.4	100,290
1985	44.8	95,910
1986	51.4	65,409
1987	42.2	108,120
1988	49.8	46,430
1989	*Min: 37.9*	98,161
1990	40.6	*Min: 13,058*
1991	55.9	34,296
1992	54.7	134,301
1993	51.5	127,463
1994	58.0	108,723
1995	*Max: 67.4*	145,978
1996	54.4	*Max: 192,673*
1997	51.5	125,889
1998	48.9	109,099
1999	53.2	92,337
2000	44.9	119,143
Average for base period (water years 1979-1988)	48.6	89,581
Average for 36-yr period of simulation (1965-2000)	50.8	101,780

Note: The base period includes water years 1979-1988 prior to BMP implementation. The BMP Replacement Water Rule was developed based on observations for the base period. Note the one year lag between rainfall and BMP replacement water. For example, rainfall for water year 1981 includes rainfall from October, 1980 to September, 1981. Rainfall for water year 1981 is used to estimate BMP replacement water target for delivery from October to February of the next water year (water year 1982: October, 1981-February, 1982).

Table 3.6.2.3 Monthly Distribution Target Percentages for BMP Makeup Water Deliveries

Month of Water Year	Target Percentage
October	28.7%
November	22.8%
December	26.5%
January	14.9%
February	7.1%

Wastewater Reuse

The wastewater reuse concept is associated with the advanced treatment of wastewater to make it suitable for environmental or groundwater release. Reuse was identified as a possible source of water in the Comprehensive Everglades Restoration Plan (CERP) in several specific areas; however the model has the ability to include reuse in any grid cell. The inflows are specified by the user and can be input on a monthly-average basis. The source of the water is assumed to come from a source currently removed from the system, (e.g. by deep-well injection) but is redirected as a source of new water. The reuse water can be introduced back into the system at either a grid cell or canal location.

Operational Planning

The SFWMM normally runs in a planning tool mode to establish existing or base conditions. The existing or base conditions can be used to determine such metrics as National Environmental Policy Act (NEPA) baseline requirements, existing levels of service or water reservations analysis. However, the model can also be run as an operational planning tool. In the operational planning mode, it can be used to support real-time operational decisions. When the model is run in the operational planning mode it is referred to as Position Analysis (Cadavid, et al. 1999). In South Florida, droughts and floods occur over relatively long periods of time due to the slow-paced hydrology. As a result, the Position Analysis (PA) provided by the SFWMM can be useful in predicting potential results of real-time operations over the next several months based on modeling outputs from past hydro-meteorological events (or predicted historical traces). Because actual historic data does not represent the system as is operated today (or in its current configuration of structures), model predictions are used to create the simulated response of the modeled system to historical climate conditions (Obeysekera, et al. 2000). This ensures that the current (or proposed) operations are accounted for in the analysis when using past hydro-meteorological data. There are two kinds of PA runs: Conditional (which incorporates climate forecasts) and Unconditional (which is based only on the historical climate data).

In PA mode, all the storage areas in the model are initialized to current conditions for each of the 36 years in the 1965-2000 simulation period. Once the initial conditions are set, the model simulates, under different climatological input scenarios and current operational practices, different outcomes (stage and flow) of the system for the ensuing 12-month period. Establishing existing conditions as the initial condition for a model is a daunting task given the difficulties of having to determine several parameters such as storage volumes in reservoirs, stages in canals and lakes, soil moisture levels, groundwater levels, and inflows (including flow predictions for the Kissimmee River). In order to accomplish this condition, collected raw data at gage sites are compared to snapshots (from SFWMM runs) to find a similar condition. Statistical analysis of these snapshots is used in selecting an initial grid condition such as shown in Figure 3.6.2.3.

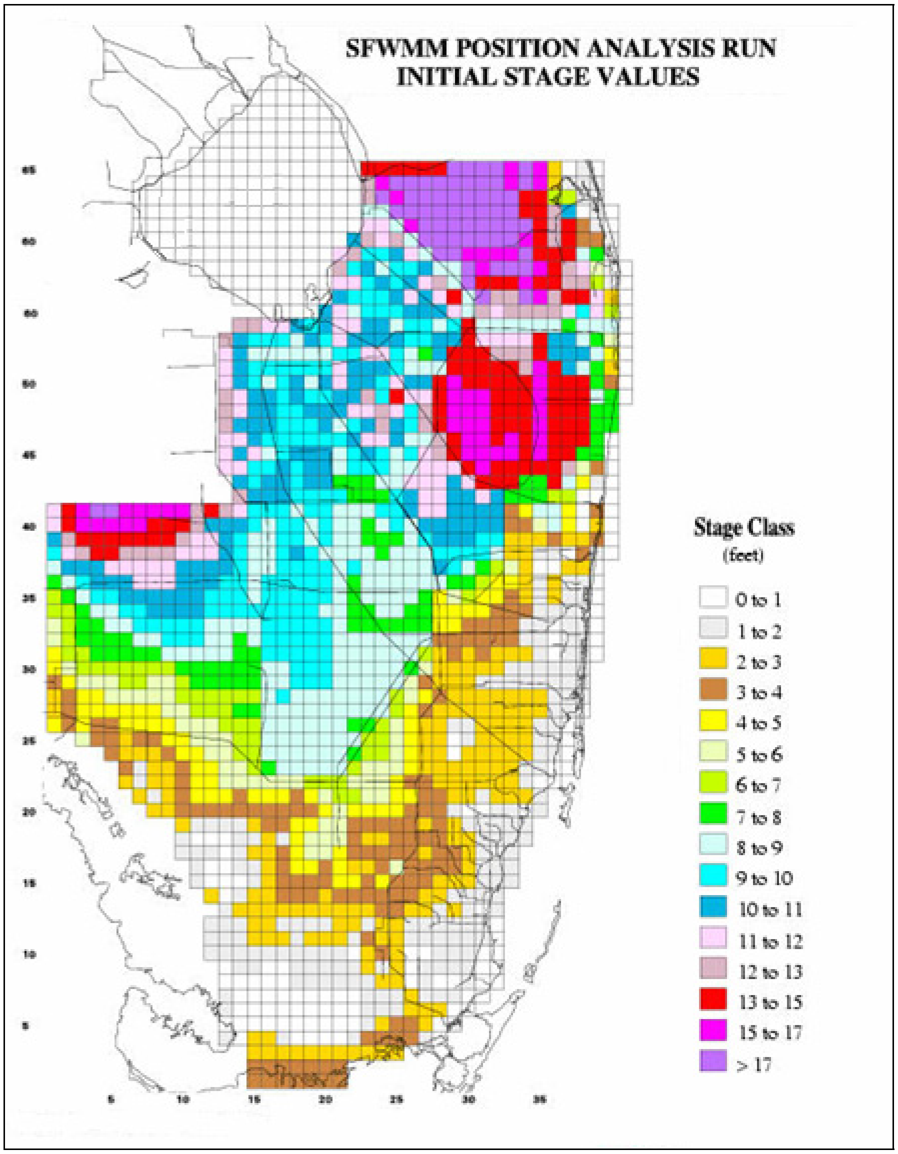

Figure 3.6.2.3 SFWMM Grid Values of Initialized Stage

Once the initial conditions input have been developed, the SFWMM begins a 36-year simulation. When the starting month is reached after each year of simulation, the run is re-initialized to the starting conditions. Without the re-initialization the resulting stage prediction output for Lake Okeechobee appears as shown in Figure 3.6.2.4. With the re-initialization, the stage prediction output appears as shown in Figure 3.6.2.5. By processing the historical traces, probabilities can be developed and associated with each yearly prediction (also shown in Figure 3.6.2.6). Such predictions are a special form of risk analysis. When a new climate forecast is input into the model, a conditional PA run is made. An example of the change that might occur between a conditional and unconditional run is shown in Figure 3.6.2.6.

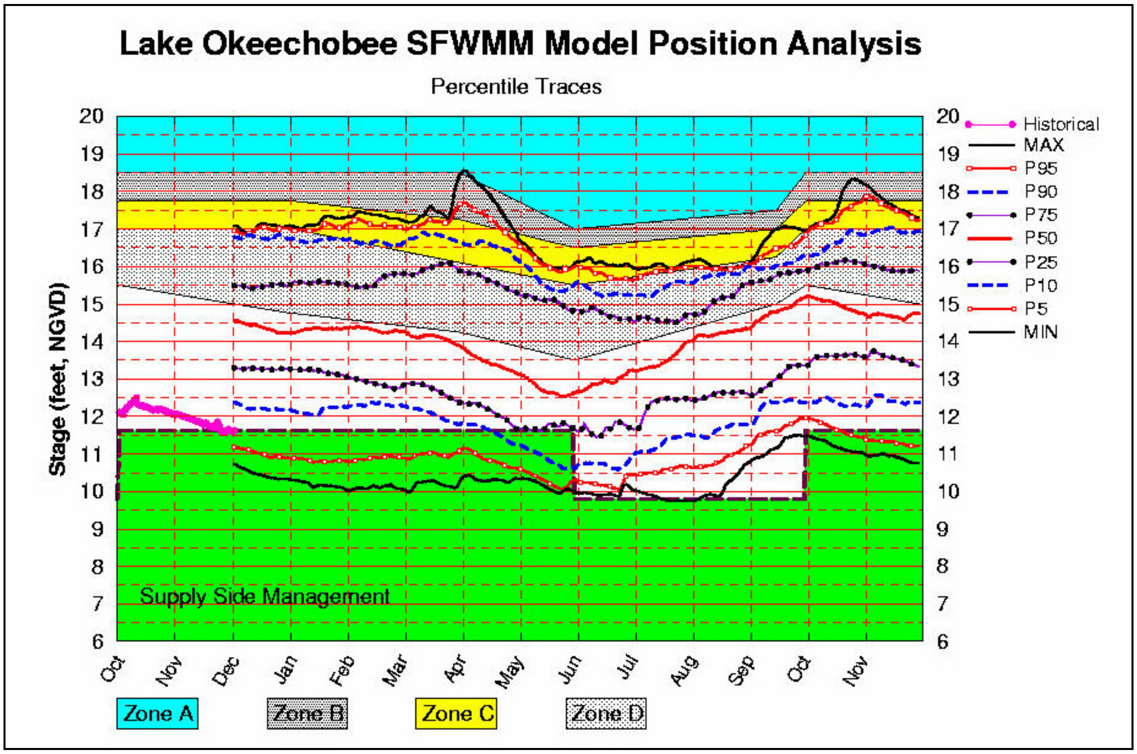

Figure 3.6.2.4 Historical Traces of Stage Predictions in Lake Okeechobee

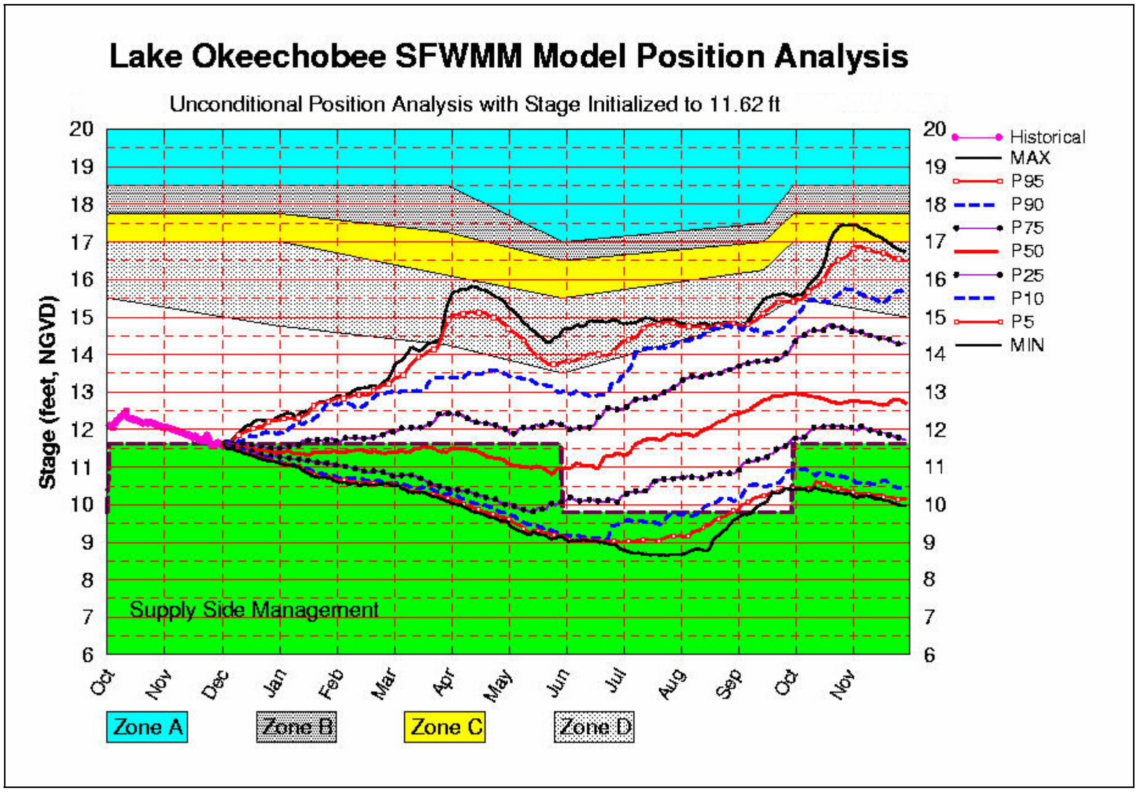

Figure 3.6.2.5 Historical Traces Re-Initialized to Starting Conditions for Lake Okeechobee

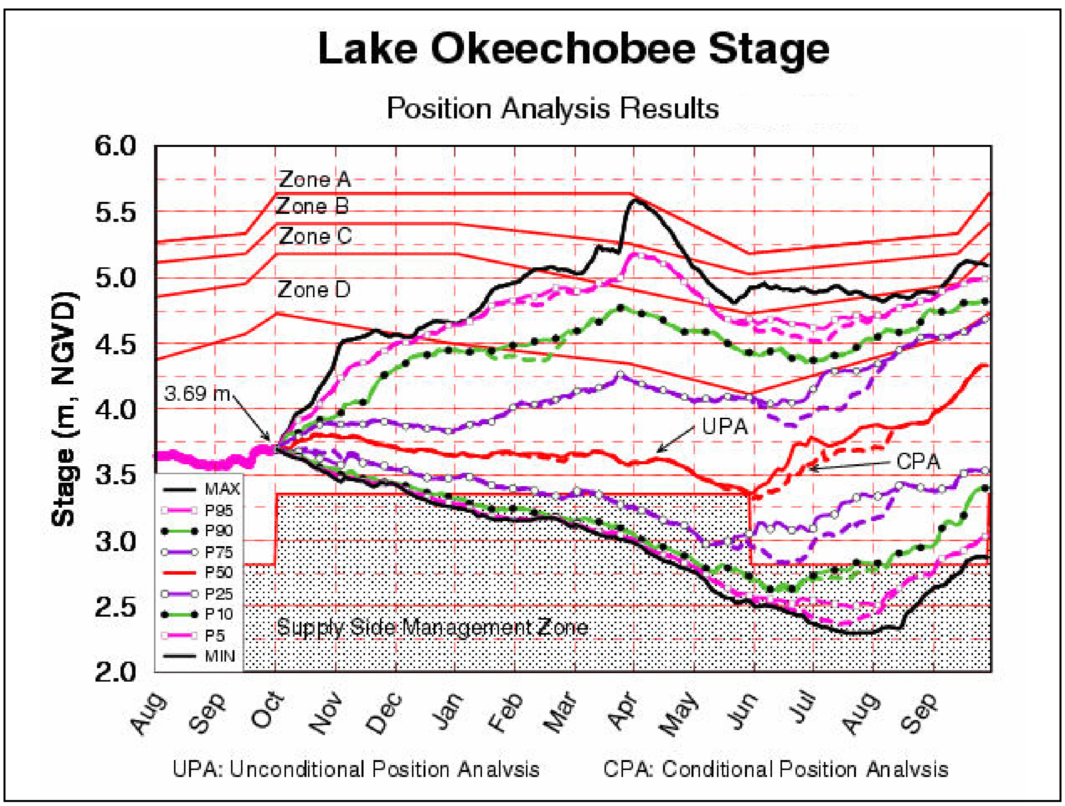

Figure 3.6.2.6 Conditional and Unconditional Position Analysis Stage Predictions for Lake Okeechobee

4 CALIBRATION AND VERIFICATION

In this Chapter, the calibration/verification of three different regions is covered. First, the calibration and verification of the Everglades Agricultural Area (EAA) is presented followed by the calibration and verification of the Everglades and Lower East Coast (LEC). Lastly, the calibration of the lumped Lake Okeechobee Service Area (LOSA) basins is covered. Generally, calibration and verification is conducted on a limited data set of one to three years. However, in the C&SF Project, a 36-year period of record for modeling exists. As a result, lengthy calibration and verification periods can be established. Determining periods when few systems changes occurred and where hydrologic extremes exist are important considerations in addition to the normal concerns for data integrity.

Prior to model calibration and verification, an extensive quality assurance/quality control (QA/QC) check of all stage and flow data was conducted. Personnel from three South Florida Water Management District (SFWMD or District) departments were involved in the review. The QA/QC included statistical analyses, comparative analyses, flagging of data known to be impacted by unusual local events (e.g. drawdown tests), and an update of flow data where stage-flow relationships were improved.

Calibration, as applied to the South Florida Water Management Model (SFWMM), is the process by which model parameters are changed until a reasonable match between model output, primarily stage and discharge, and observed data is achieved. In this context, calibration can be more appropriately called history matching (Konikow and Bredehoeft, 1992). Model calibration relates to the assumption that a well calibrated model enhances its predictive capability. Verification is the process where the calibrated model parameters are used to predict hydrologic responses during periods where comparisons can be made to a different historical data set.

Due to the unique way by which the EAA is simulated in the model (refer to Section 3.2), only simulated runoff and demand volumes were compared with historical values. For the Everglades/LEC region, a set of water level monitoring/observation points and structure headwater stages were selected. Historical water level measurements at these locations and historical discharges through selected outlet structures were compared against stages and discharges simulated by the model, respectively. The calibration of the lumped LOSA basins is presented in the third section with flow comparisons to historical data.

The following guidelines, which apply to hydrologic models in general, were used in calibrating the model:

1. The availability of historical stage and flow data dictated the extent of the calibration period. Rainfall, the primary driving force in South Florida's hydrology, further limits the length of time by which historical and simulated stages and/or flows are to be compared. The period of comparison should include extremely wet and dry conditions.
2. The historical (field-measured) data set should be limited by what can be considered reliable. For example, the quality of historical data on discharges at some coastal structures was considered poor. Flows through these structures, which were normally considered as boundary conditions, were simulated when the model was run in

calibration mode. Therefore, after a field data verification process was conducted, graphical plots of simulated versus historical flow data were created.

3. The period of comparison should be short enough such that no significant changes in operational schemes occur in the middle of the simulation period. This assumption is important since most of the parameters used in the model are time invariant. As contrasted to succession models, a long-term simulation model such as the SFWMM has limited capability in making changes to certain operating parameters in the middle of a simulation run. For example, the policy of holding back more runoff in the EAA due to Best Management Practices (BMPs) has been implemented in the field only in the last few years. This policy also impacts Lake Okeechobee water release rules. Thus, calibration parameters could be markedly different depending on which years (pre-BMP vs. BMP) are emphasized.
4. The frequency by which available historical data was compared should be consistent with regional modeling space and time resolution. For the SFWMM, comparisons are typically done only on a monthly basis: monthly total discharges, end-of-month nodal stages and monthly mean canal levels. The succeeding discussions on calibration results will address space resolution and model discretization issues to some extent.
5. Display calibration results by plotting historical and simulated values on the same graph (e.g. clustered bar graphs for flow comparison, XY or scatter plots for stage comparison) and quantifying goodness-of-fit by using some statistical measures (e.g. r-squared, bias).

The scope of the entire SFWMM calibration process can be divided into three parts:

1. data update which includes time series (rainfall, reference ET, structure flows, stages at monitoring points and canals) and static data (land elevation, land use) updates;
2. computer program update which involves changes to existing subroutines and/or creation of new computer code, e.g., improvements to ET and overland flow algorithms; and
3. actual model calibration which requires accuracy checks on model algorithms, both old and new, and adjustments of model parameters that affect calculated water levels and discharges.

4.1 CALIBRATION OF THE EVERGLADES AGRICULTURAL AREA BASIN

The goal of the Everglades Agricultural Area calibration effort was to match, as closely as possible, supplemental irrigation requirements (demand) and drainage (runoff) in the EAA. As mentioned earlier, the calibration of the EAA was performed in a way that differs from the rest of the model. Simulated flow volumes, both supplemental irrigation requirement and runoff, were compared to historical volumes. Due to the lack of groundwater data throughout the EAA, limited matching of historical water levels, specifically in the Rotenberger area, was performed. This procedure may not be a serious shortcoming because stages in the highly irrigated EAA are maintained within a very narrow range (Abtew and Khanal, 1992).

4.1.1 Methodology

The EAA calibration period was from January 1984 to December 1995 and the verification covered two periods from January 1979 to December 1983 and January 1996 to December 2000. Version 5.5 of the SFWMM reflects the most up-to-date values of the calibration parameters.

Three parameters were adjusted during the EAA calibration: ET calibration coefficients **KCALIB**, and dimensionless local storage parameters **fracdph_min** and **fracdph_max** (refer to Section 3.2). Local storage parameters define the soil moisture level in the soil column at which runoff occurs and the level that triggers supplemental deliveries from other sources. All EAA calibration parameters vary monthly.

Since all parameters being adjusted were defined for each month, comparisons between historical and simulated monthly total long-term (averaged over calibration period) runoff and supplemental irrigation requirements were made. Runoff and supplemental irrigation requirements (which are mutually exclusive equations, i.e. one or the other is used) are defined as follows:

$$\text{Runoff} = \sum \text{structure outflows} - \sum \text{structure inflows} \quad (4.1.1.1)$$

$$\text{Supplemental Irrigation} = \sum \text{structure inflows} - \sum \text{structure outflows} \quad (4.1.1.2)$$

The general rules for adjusting EAA parameters are shown in Table 4.1.1.1.

Table 4.1.1.1 General Rules Used in Adjusting Calibration Parameters for the EAA in the SFWMM

Comparison of Runoff If simulated value is:	**Comparison of Supplemental Irrigation** If simulated value is:	**Action**
> Historical	< Historical	Increase ET calibration coefficient, KCALIB.
< Historical	> Historical	Decrease ET calibration coefficient, KCALIB.
> Historical	> Historical	Increase local storage (decrease soil moisture level triggering supplemental deliveries and/or increase soil moisture level triggering runoff).
< Historical	< Historical	Decrease local storage (increase soil moisture level triggering supplemental deliveries and/or decrease soil moisture level triggering runoff).

As mentioned in Section 3.2, the parameter KCALIB is used as an adjustment factor for a theoretical set of vegetation coefficients [KVEG in Equation (3.2.2.1)] determined from an earlier study (Abtew and Khanal, 1992). The limits on KCALIB (see Table 4.1.1.2) were established based on the desire not to alter the original values of KVEG significantly. The limits on parameters **fracdph_min** and **fracdph_max**, on the other hand, were established based on the assumption that the mean soil moisture level, [(SOLCRNF + SOLCRT) ÷ 2,] does not vary substantially during the year. The final values of **fracdph_min** and **fracdph_max** are given in Table 4.1.1.3. The limits on soil moisture content, SOLCRT and SOLCRNF, can be calculated as the product of the assumed soil column depth (1.5 feet), the storage coefficient, and the limits on ratios fracdph_min and fracdph_max, respectively. SMAX and SMIN (for the Miami River Basin), in Figure 4.1.1.1 represent the limits on soil moisture content, expressed in terms of equivalent depths of water, in the unsaturated zone for a storage coefficient equal to 0.20.

The calibration parameters were adjusted until the mean monthly simulated and historical runoff and supplemental irrigation requirements (over the 1984-1995 time period) matched within about one percent.

Table 4.1.1.2 KCALIB Calibration Coefficients for Unrestricted Evapotranspiration in the EAA

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
0.500	0.615	0.840	0.650	0.840	1.055	0.630	0.675	0.575	0.500	0.570	0.505

Table 4.1.1.3 Final Values of Calibration Parameters (fracdph max and min) used for the EAA in the SFWMM v5.5

Month	Miami River Basin		North New River And Hillsboro Basins		West Palm Beach Basin	
	Max	Min	Max	Min	Max	Min
January	.2175	.0457	.1975	.0457	.1875	.0457
February	.1700	.0854	.1600	.0854	.1400	.0854
March	.2275	.0704	.2275	.0704	.2175	.0704
April	.2250	.0404	.2150	.0404	.2050	.0404
May	.5550	.0000	.5500	.0000	.5350	.0000
June	.2400	.0287	.2400	.0287	.2200	.0287
July	.2020	.0367	.1920	.0367	.1820	.0367
August	.2440	.0167	.2340	.0167	.2240	.0167
September	.1505	.0000	.1505	.0000	.1405	.0000
October	.1750	.0400	.1750	.0400	.1750	.0400
November	.1530	.0400	.1480	.0400	.1460	.0400
December	.1600	.0267	.1500	.0267	.1450	.0267

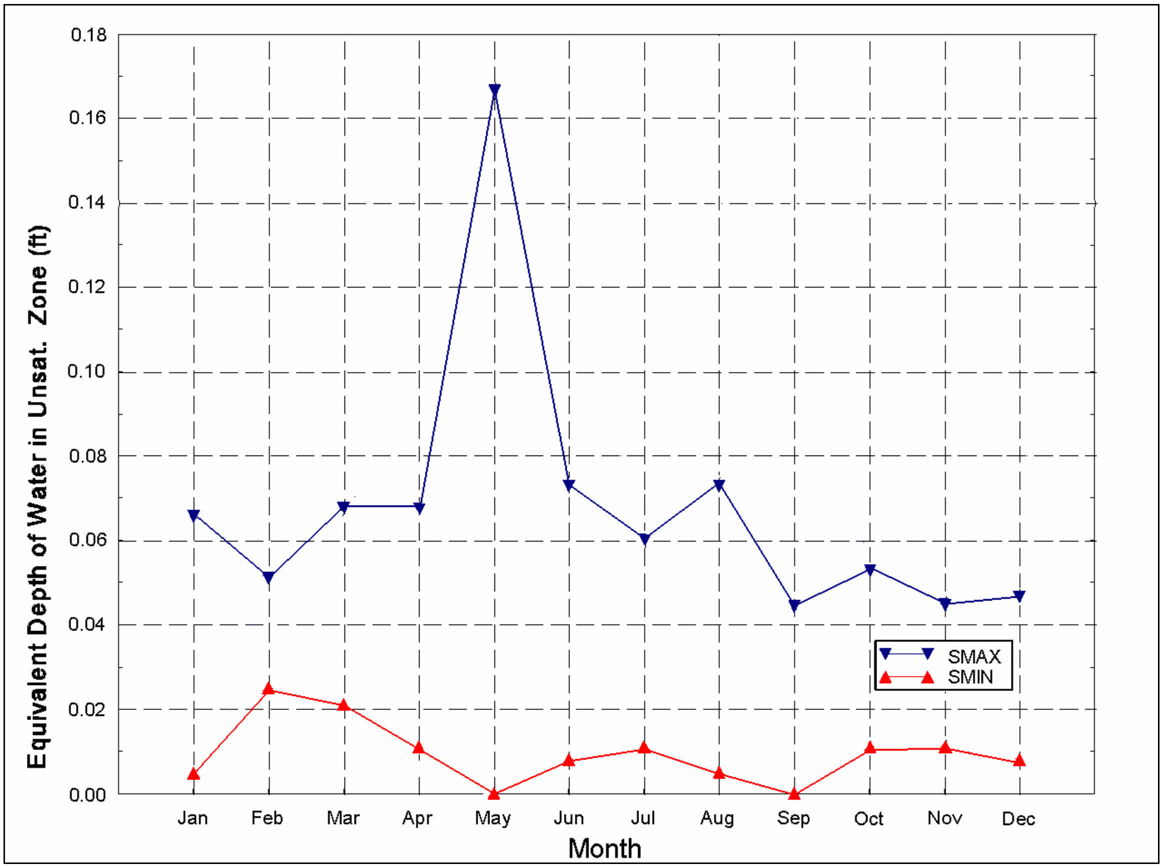

Figure 4.1.1.1 Miami River Basin Unsaturated Zone Storage Triggers for Runoff and Supplemental Flow as Implemented in the SFWMM

4.1.2 Everglades Agricultural Area Calibration and Verification Results

Time series plots comparing simulated and historical flow volumes for the entire EAA, and for each of the three sub-basins simulated by the model, were prepared. Both the calibration period and the verification period are presented. Annual volumes (Figures 4.1.2.1 and 4.1.2.2), daily flows (Figures 4.1.2.3 through 4.1.12), and monthly volumes and flows (Figures 4.1.2.13 through 4.1.2.18) were compared for the entire EAA. By plotting simulated versus historical values on the y- and x- axes, respectively, the goodness-of-fit for daily/monthly runoff and daily/monthly irrigation requirements can be evaluated (Figures 4.1.2.5 through 4.1.2.8 and Figures 4.1.2.17 through 4.1.2.20). A good fit is denoted by a regression line with a slope of unity and y-intercept at the origin.

Overall, differences between simulated and historical flow volumes can be attributed to a number of factors. They include:

1. errors in input data (static data, structure discharge, rainfall, etc.);
2. model inaccuracies due to model resolution (4-mile^2 grid cells, limited number of rainfall stations); and
3. oversimplified algorithm used to describe actual field-scale management of water by the farmers.

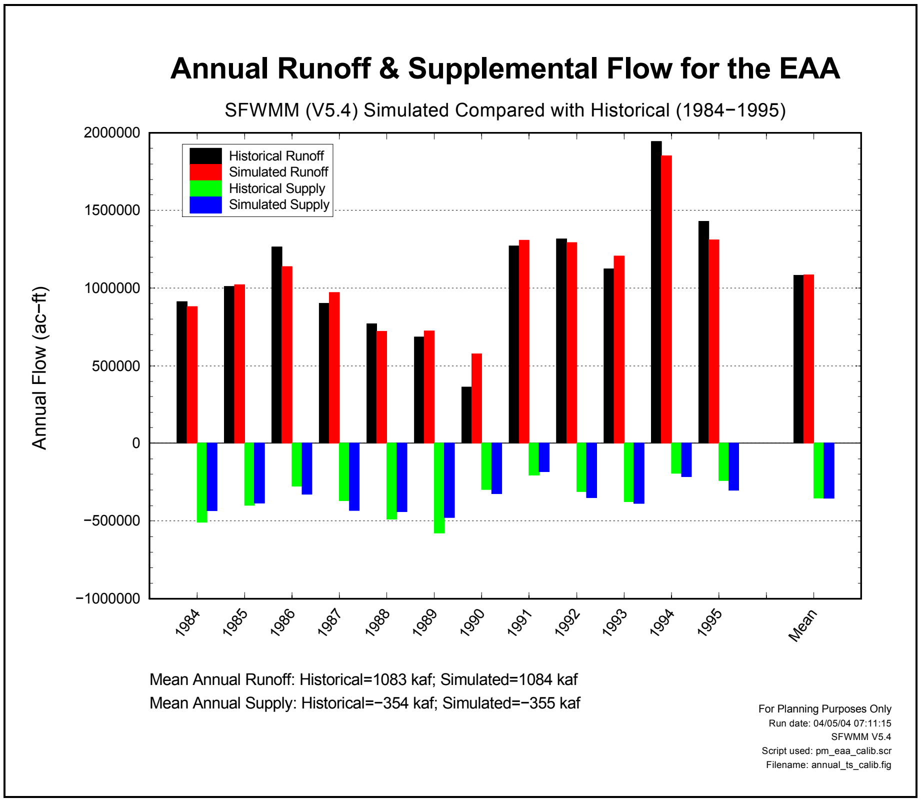

Figure 4.1.2.1 Calibrated Annual Runoff and Supplemental Flow for the EAA

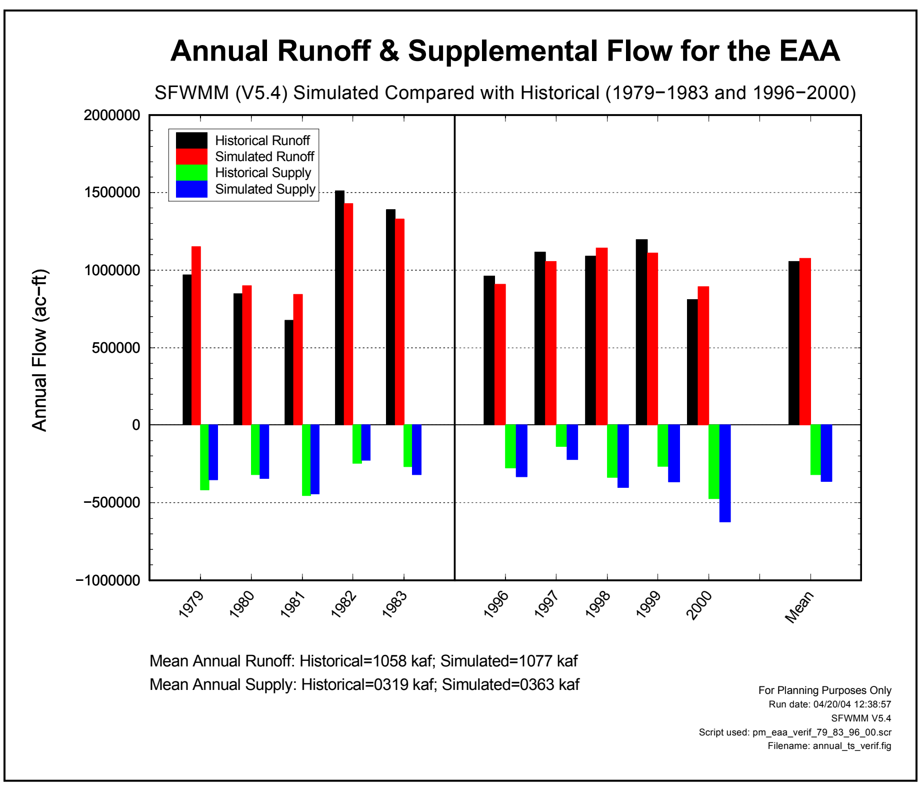

Figure 4.1.2.2 Verified Annual Runoff and Supplemental Flow for the EAA

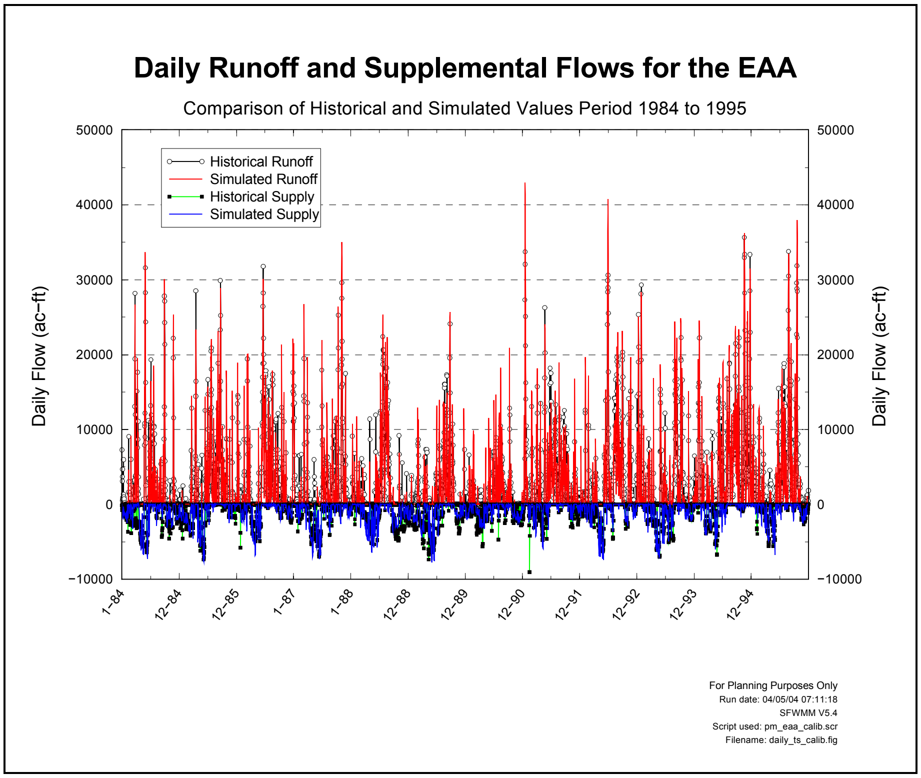

Figure 4.1.2.3 Calibrated Daily Runoff and Supplemental Flows for the EAA

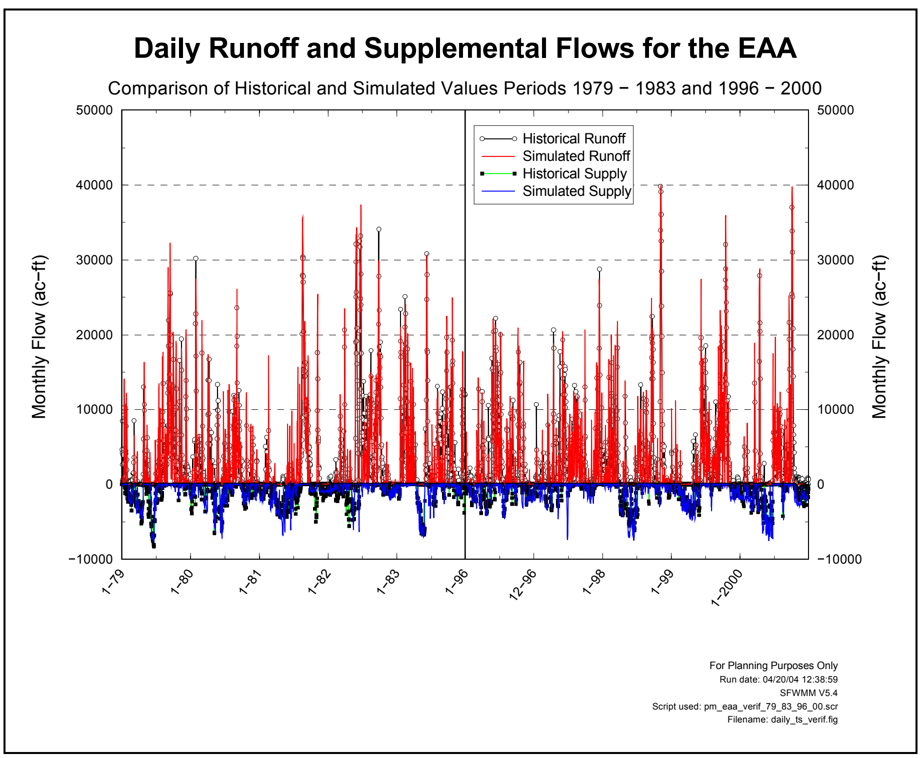

Figure 4.1.2.4 Verified Daily Runoff and Supplemental Flows for the EAA

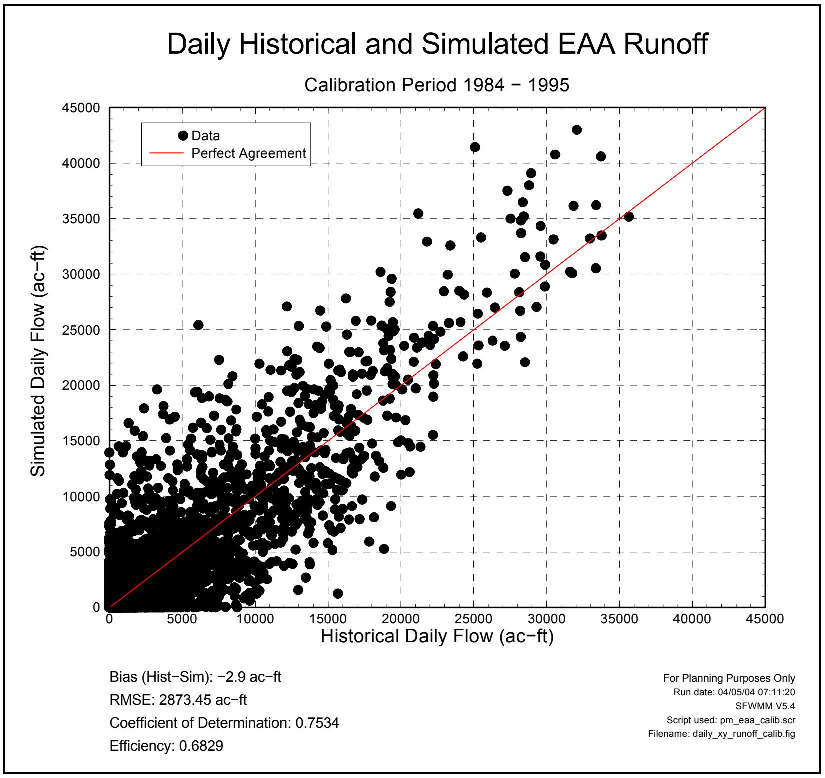

Figure 4.1.2.5 Calibrated Daily Historical and Simulated EAA Runoff

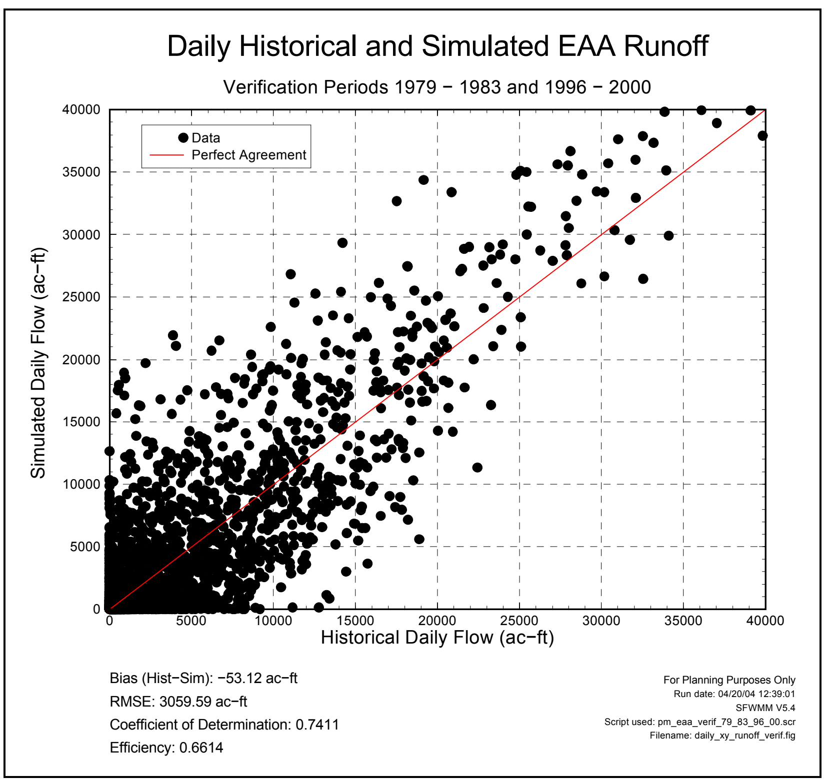

Figure 4.1.2.6 Verified Daily Historical and Simulated EAA Runoff

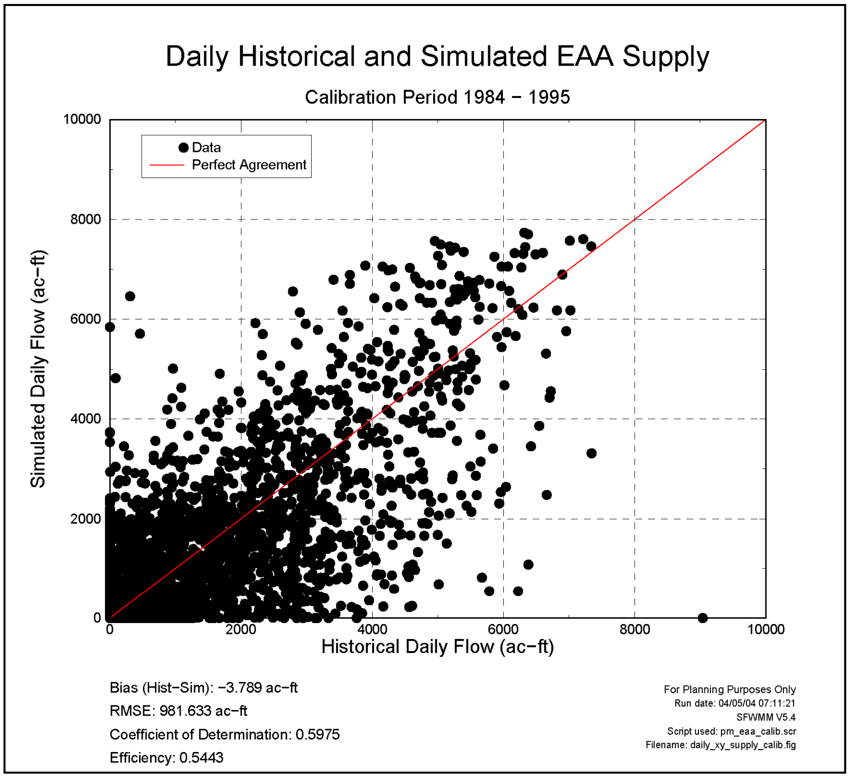

Figure 4.1.2.7 Calibrated Daily Historical and Simulated EAA Supply

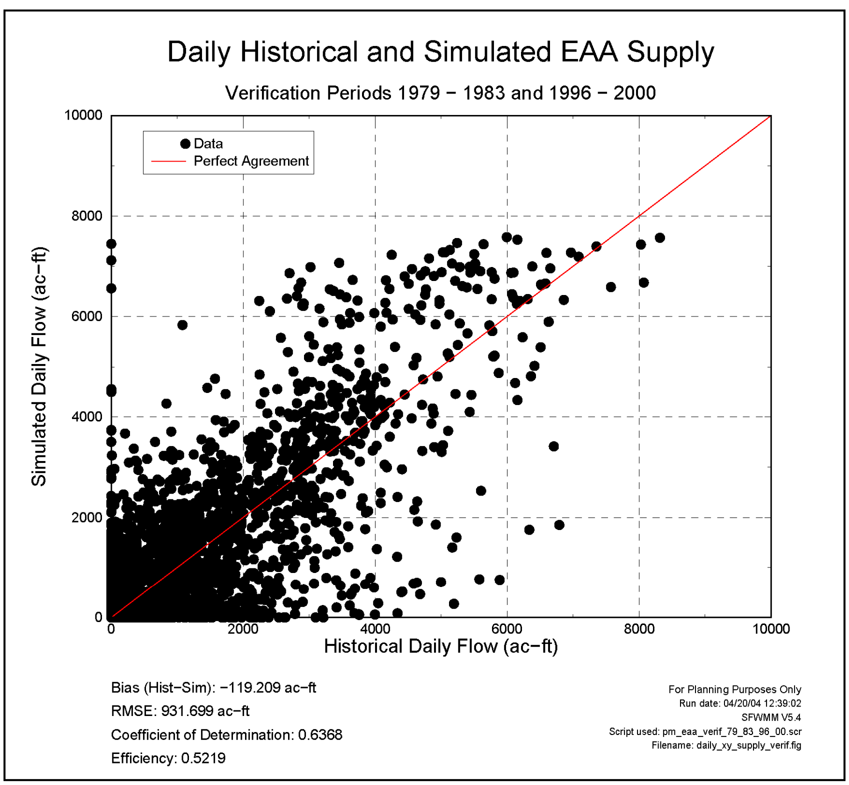

Figure 4.1.2.8 Verified Daily Historical and Simulated EAA Supply

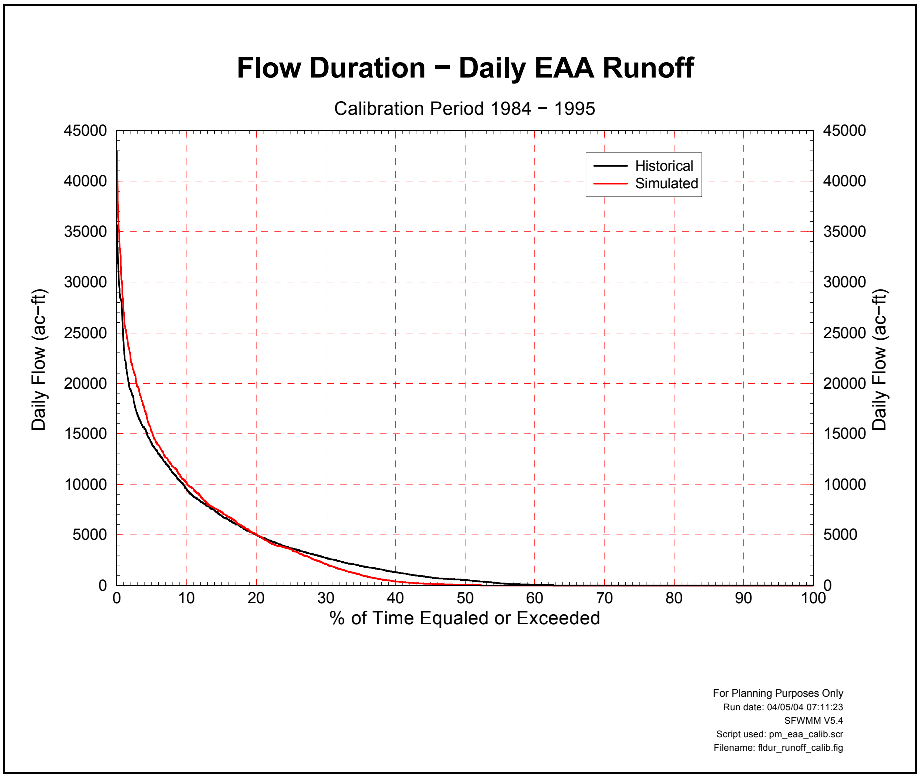

Figure 4.1.2.9 Calibrated Flow Duration – Daily EAA Runoff

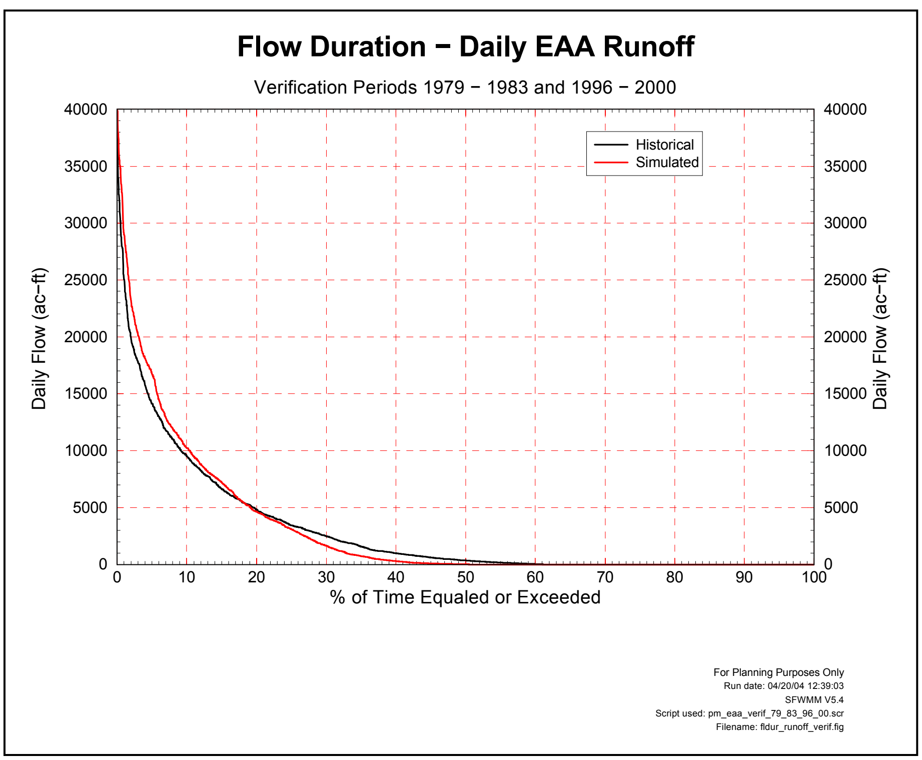

Figure 4.1.2.10 Verified Flow Duration – Daily EAA Runoff

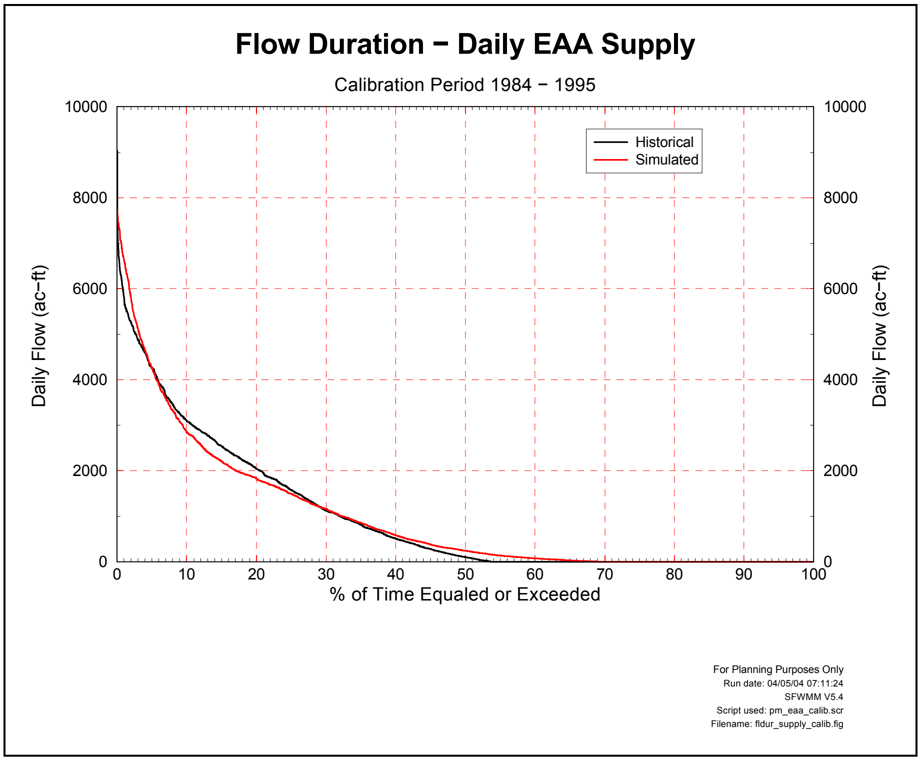

Figure 4.1.2.11 Calibrated Flow Duration – Daily EAA Supply

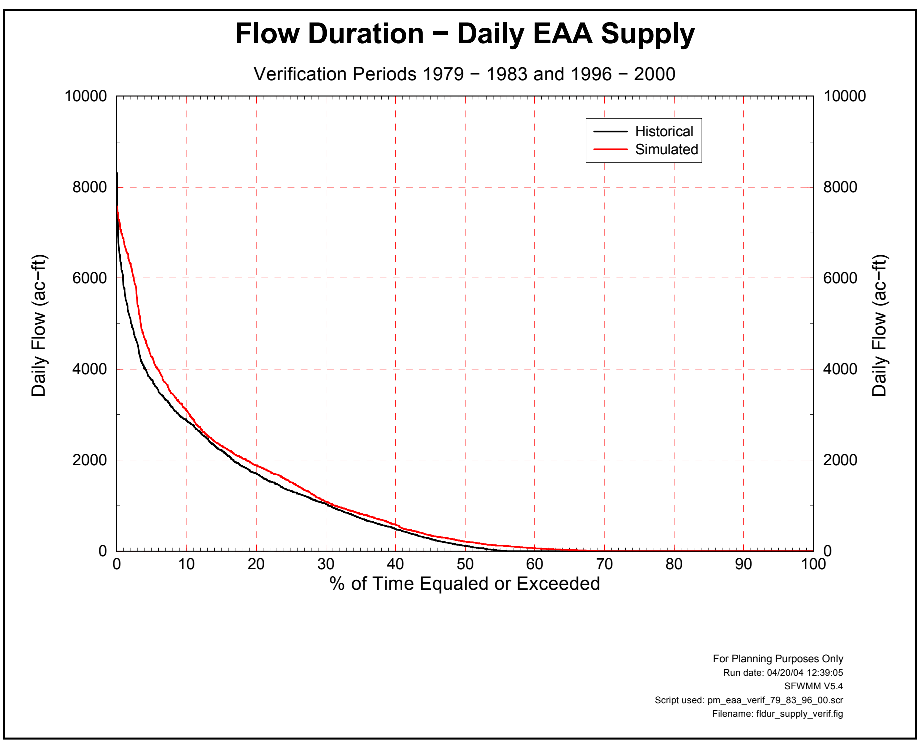

Figure 4.1.2.12 Verified Flow Duration – Daily EAA Supply

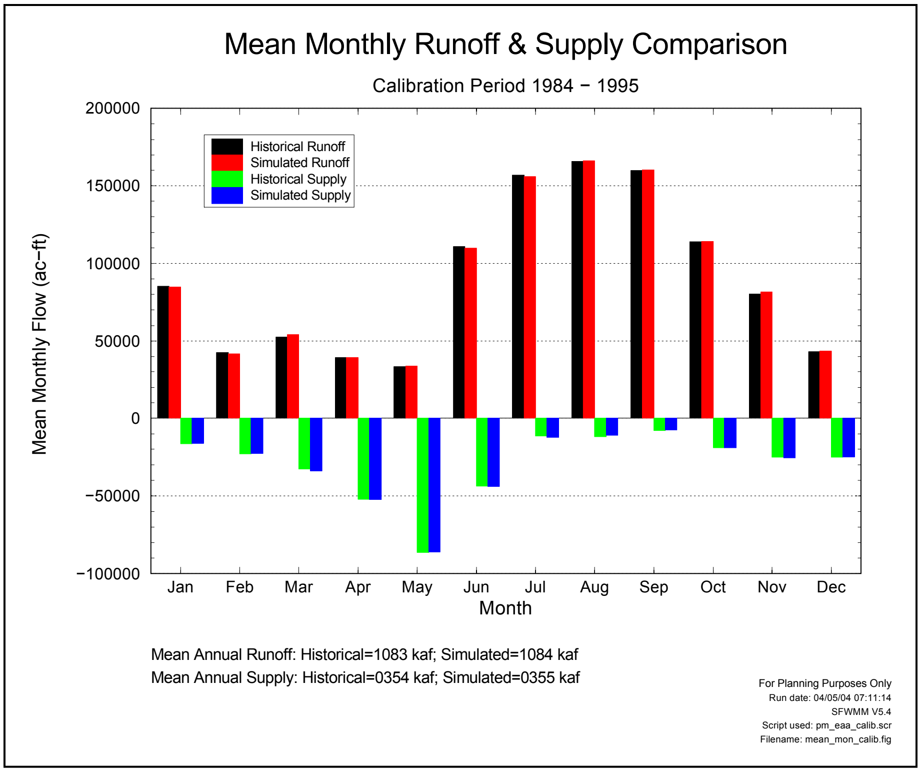

Figure 4.1.2.13 Calibrated Mean Monthly Runoff and Supply Comparison

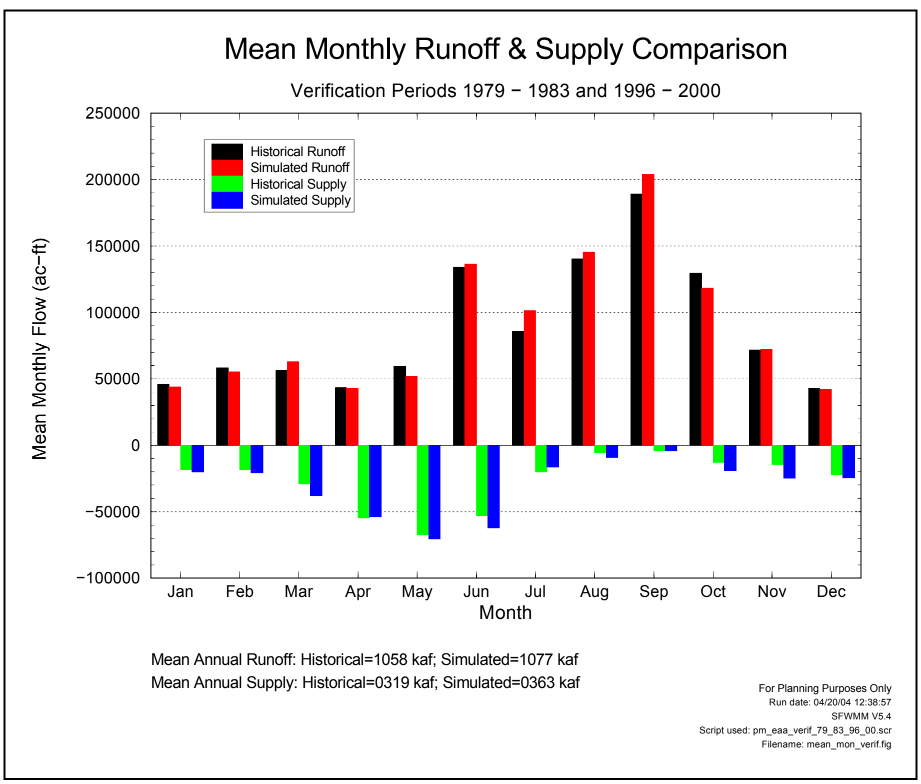

Figure 4.1.2.14 Verified Mean Monthly Runoff and Supply Comparison

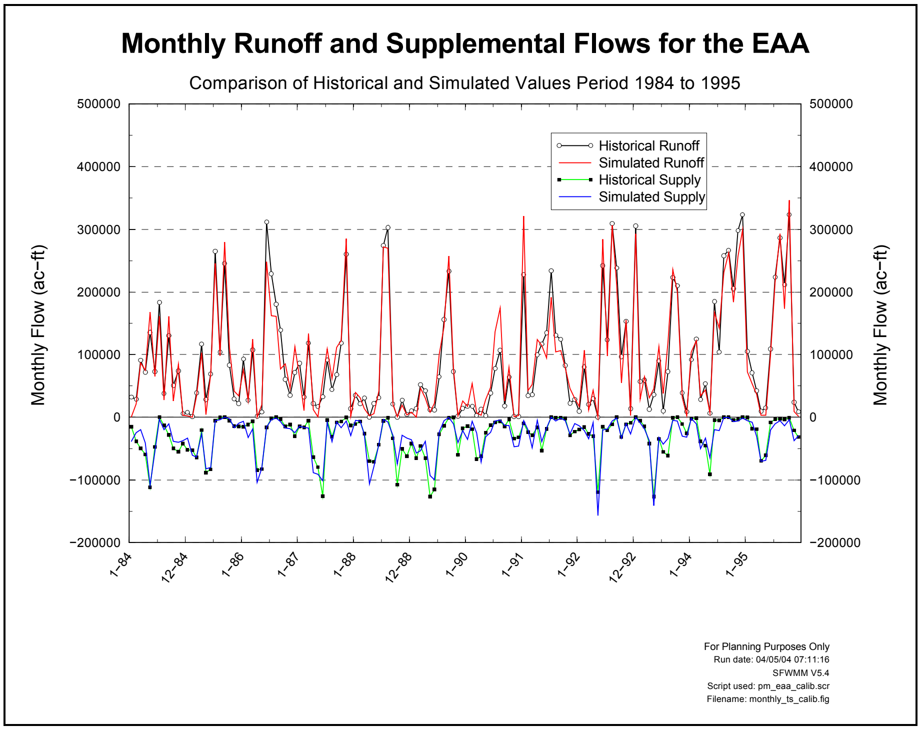

Figure 4.1.2.15 Calibrated Monthly Runoff and Supplemental Flows for the EAA

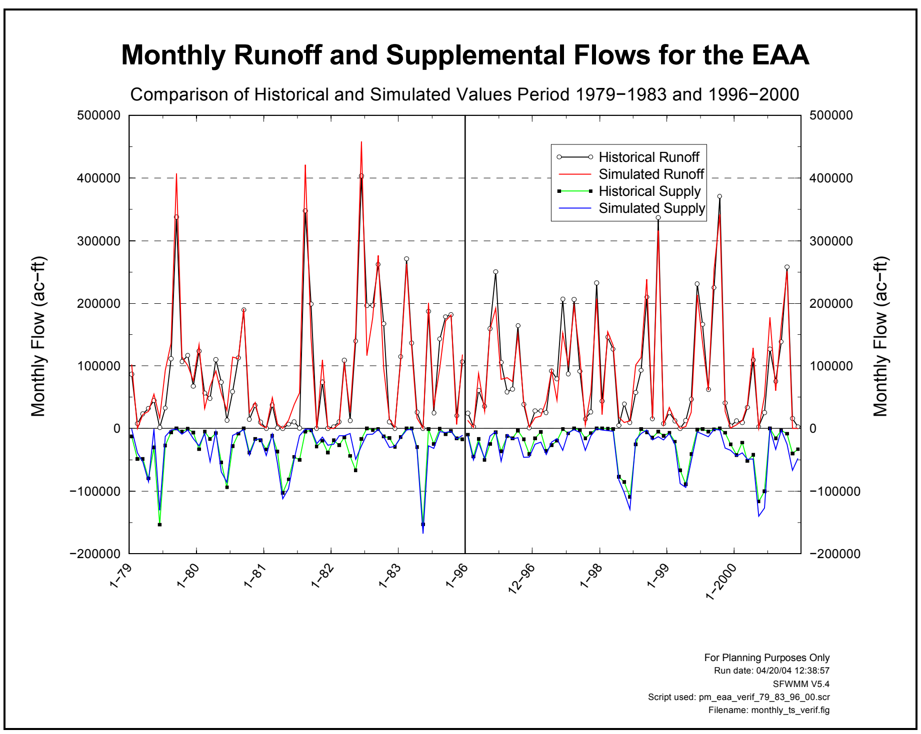

Figure 4.1.2.16 Verified Monthly Runoff and Supplemental Flows for the EAA

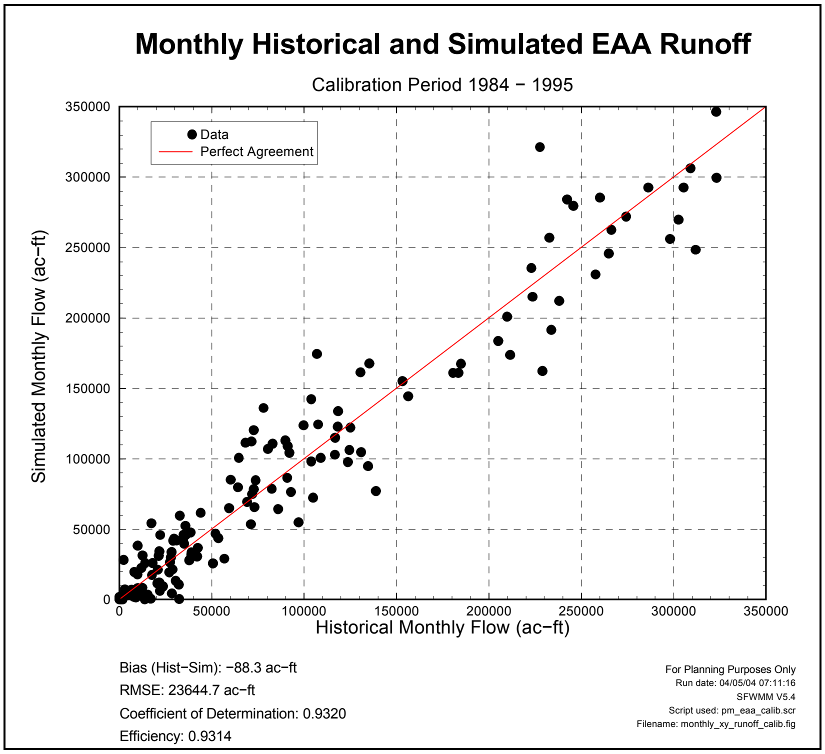

Figure 4.1.2.17 Calibrated Monthly Historical and Simulated EAA Runoff

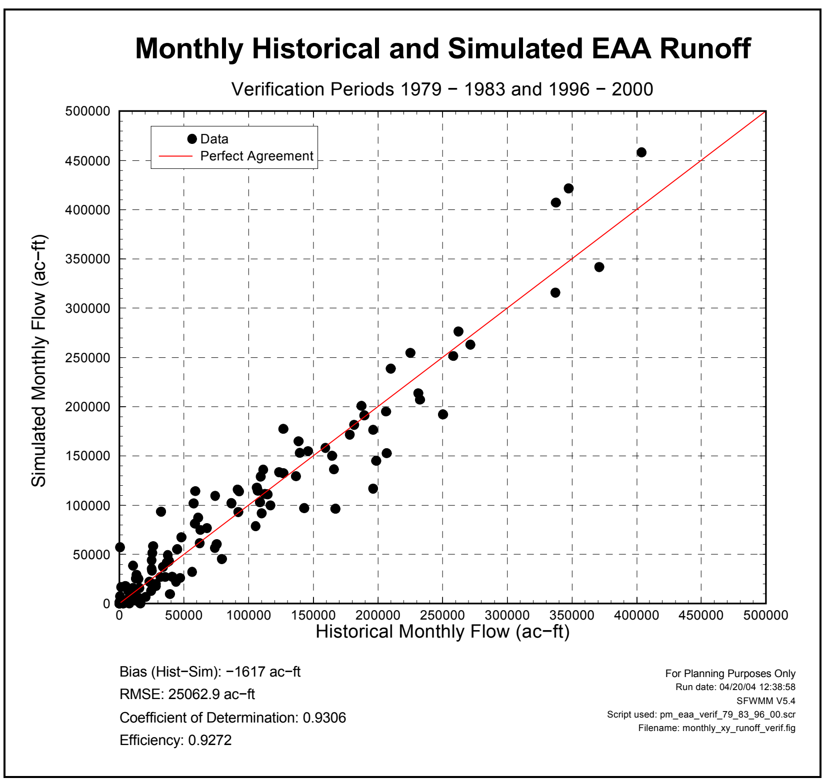

Figure 4.1.2.18 Verified Monthly Historical and Simulated EAA Runoff

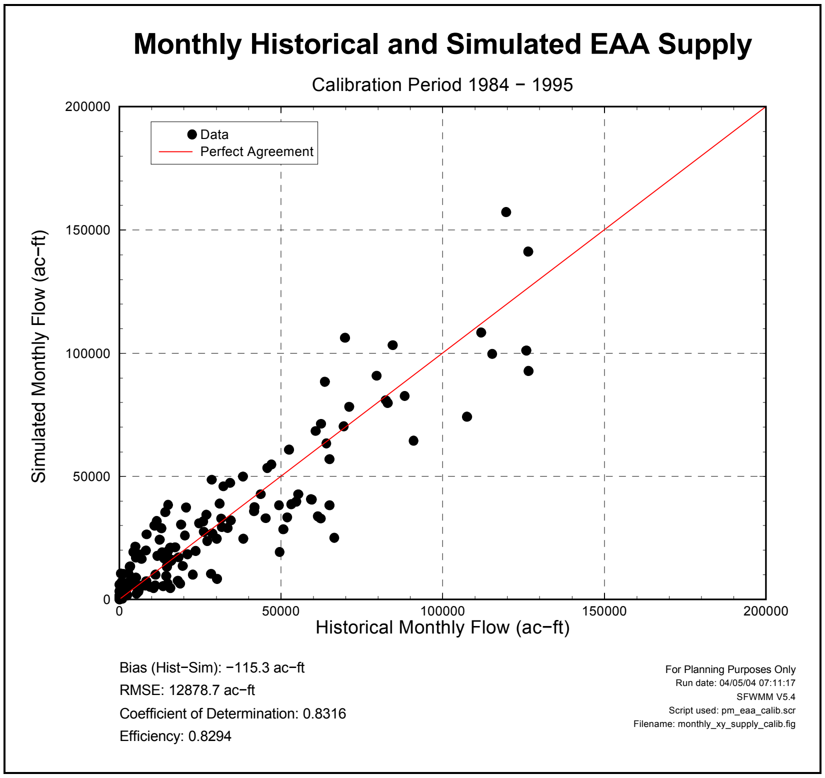

Figure 4.1.2.19 Calibrated Monthly Historical and Simulated EAA Supply

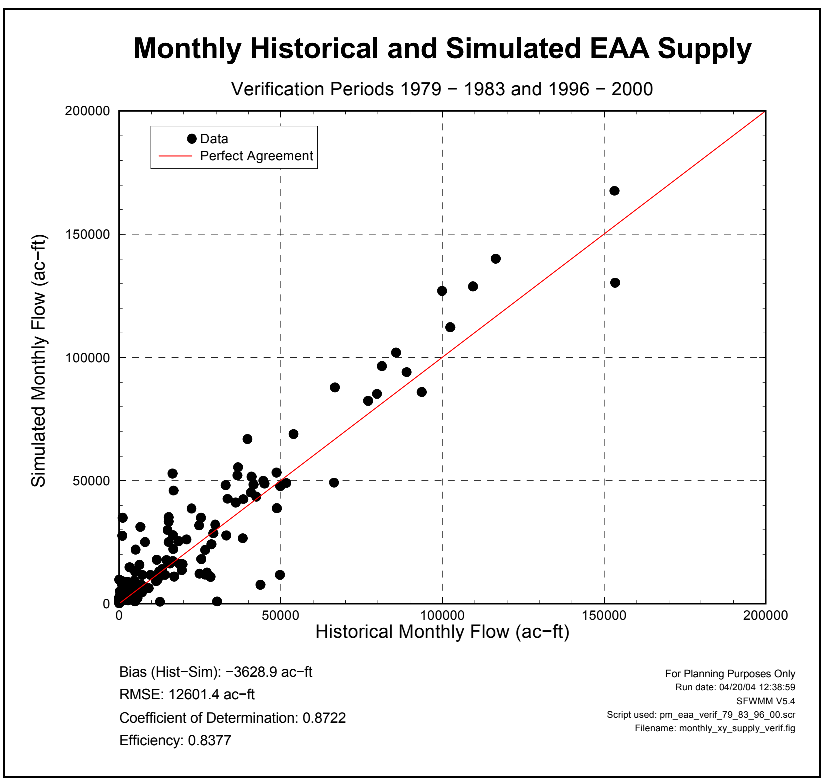

Figure 4.1.2.20 Verified Monthly Historical and Simulated EAA Supply

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4.2 CALIBRATION AND VERIFICATION OF THE EVERGLADES AND THE LOWER EAST COAST

This section presents the results of the calibration and verification for the gridded model domain outside of the Everglades Agricultural Area (EAA) including: the Water Conservation Areas (WCAs), the Big Cypress National Preserve (BCNP), Everglades National Park (ENP), the Holey Land and Rotenberger Wildlife Management Areas, and the Lower East Coast Service Areas (LECSAs).

4.2.1 Methodology

The primary goal of the SFWMM calibration and verification procedure for the majority of the model domain (LEC, WCAs, ENP, and BCNP) was to determine appropriate values for the many physically based parameters used by the model in order to ensure that the tool can reproduce the historically observed response of the South Florida system. In order to achieve this goal, historical water level observations from a network of ground and surface water monitoring locations maintained by the South Florida Water Management District (SFWMD or District) and the U.S. Geological Survey (USGS) were used to make stage comparisons during calibration (Figure 4.2.1.1). Simulation was performed on a daily basis and simulated water levels were compared with historical data on a daily basis for marsh or groundwater gage locations and on an average weekly basis for canal locations. Since the primary goal of calibration was to determine physical, not operational parameters, matching to structural flow was not considered in the determination of calibration parameters. A breakdown of the most significant parameters refined or determined by the calibration procedure is given below.

1. Lower East Coast
   a. Canal parameters
      i. Channel - aquifer hydraulic conductivity coefficient [CHHC in Equation (2.5.2.1)]
      ii. Surface water - channel interaction [N in Section 2.6]
      iii. Coefficients for operation of outlet structures
   b. Detention depths (refer to Section 2.4)
   c. ET coefficients (KVEG, DSRZ, DDRZ in Section 2.3)
2. Everglades (WCAs, ENP and BCNP)
   a. ET coefficients (KVEG, DSRZ, DDRZ in Section 2.3)
   b. Effective roughness N ($N = Ah^b$ for overland flow; mainly A is adjusted)
   c. Levee seepage rate coefficients [β_0, β_1, β_2 in Equation (2.5.3.1)]
   d. Detention depths (refer to Section 2.4)
   e. Canal parameters (refer to Sections 2.5 and 2.6)

Because the period of record available for modeling spans 36 years, the record could be divided into periods for both calibration and verification. The period used for calibration was from January 1, 1984, to December 31, 1995. Due to operational and structural changes in the Central and South Florida Flood Control (C&SF) Project around 1990, the calibration period was further broken into two sub periods: 1984 to 1990 (using operations for the 1980's) and 1991 to 1995 (with operations for the 1990's). The verification record spanned two time periods: January 1, 1981, to December 31, 1983; and January 1, 1996, to December 31, 2000. Determining periods when few system changes occurred and where hydrologic variability was well represented were important considerations in addition to the normal concerns for data integrity. In the earlier years of the calibration/verification period, the operations of water control structures may have involved some field-level decision-making. During the later years, in contrast, decision-making was fully centralized, which in turn followed operating manuals more closely.

To help account for variation in operation practices, as a general rule, available time series of historical structure flows were input to the model as internal boundary conditions between different hydrologic basins. The use of historical flows as internal boundary conditions at structures (instead of simulated flow through those structures) allowed physically based processes to be calibrated without being affected by possible changes to operating practices over time. In general, the flow records at many of the structures throughout the system were complete with high quality data. In some cases, particularly in some Lower East canals, internal structures were simulated rather than imposed during the calibration and verification periods. This practice was applied only where historical data was sparse and/or not available, where the quality of the data was poor or where the model representation of the contributing runoff basin was significantly different than what was in the field due to issues of scale. For many of these flow locations, as shown in Figure 4.2.1.2, reasonability checks are made on monthly, seasonal and annual bases to verify simulated flows against available historical data. These checks were not used in helping to determine calibrated parameters, but rather led to changes in the structural operational assumptions used for the calibration and verification runs.

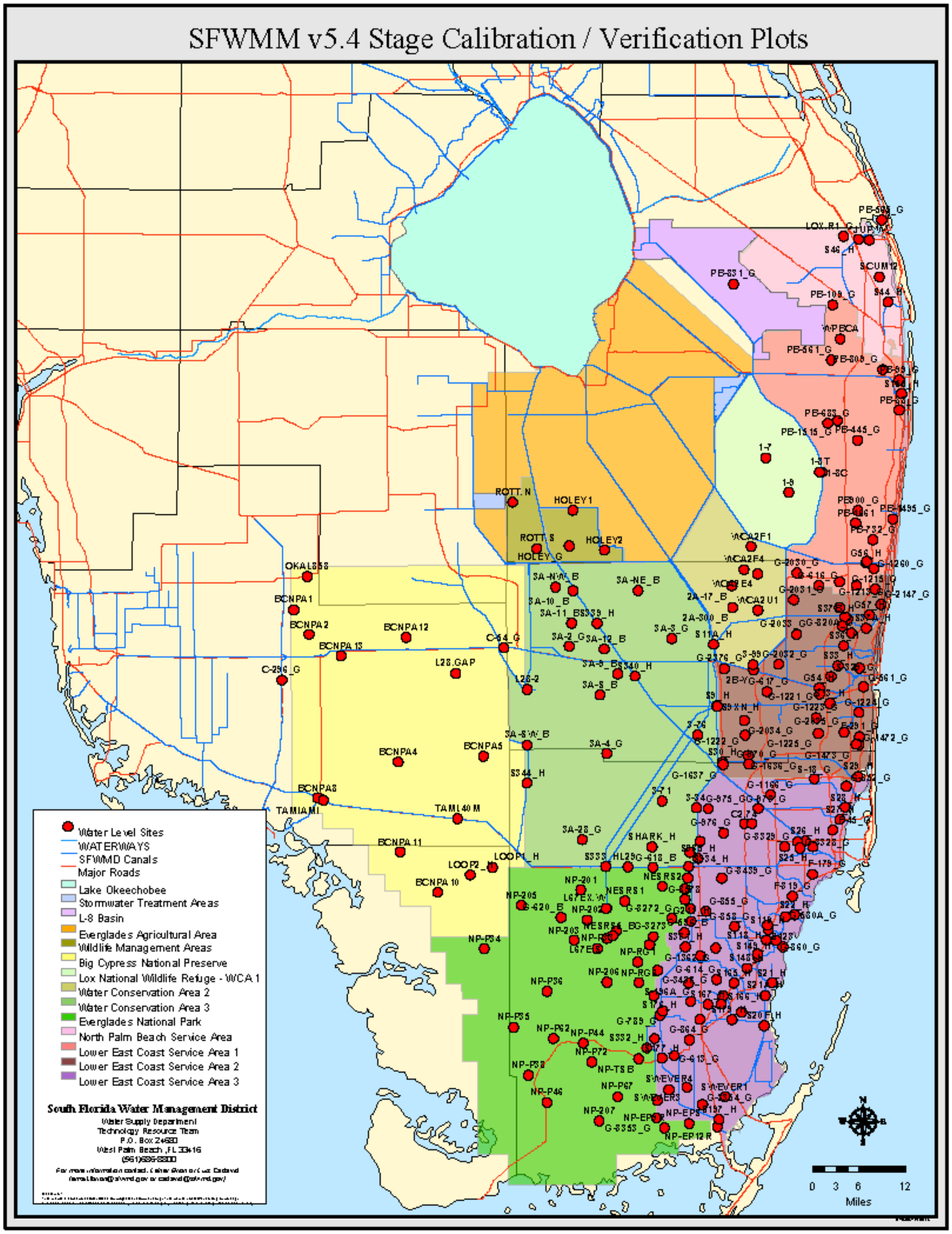

Figure 4.2.1.1 Location of Stage Calibration and Verification Sites

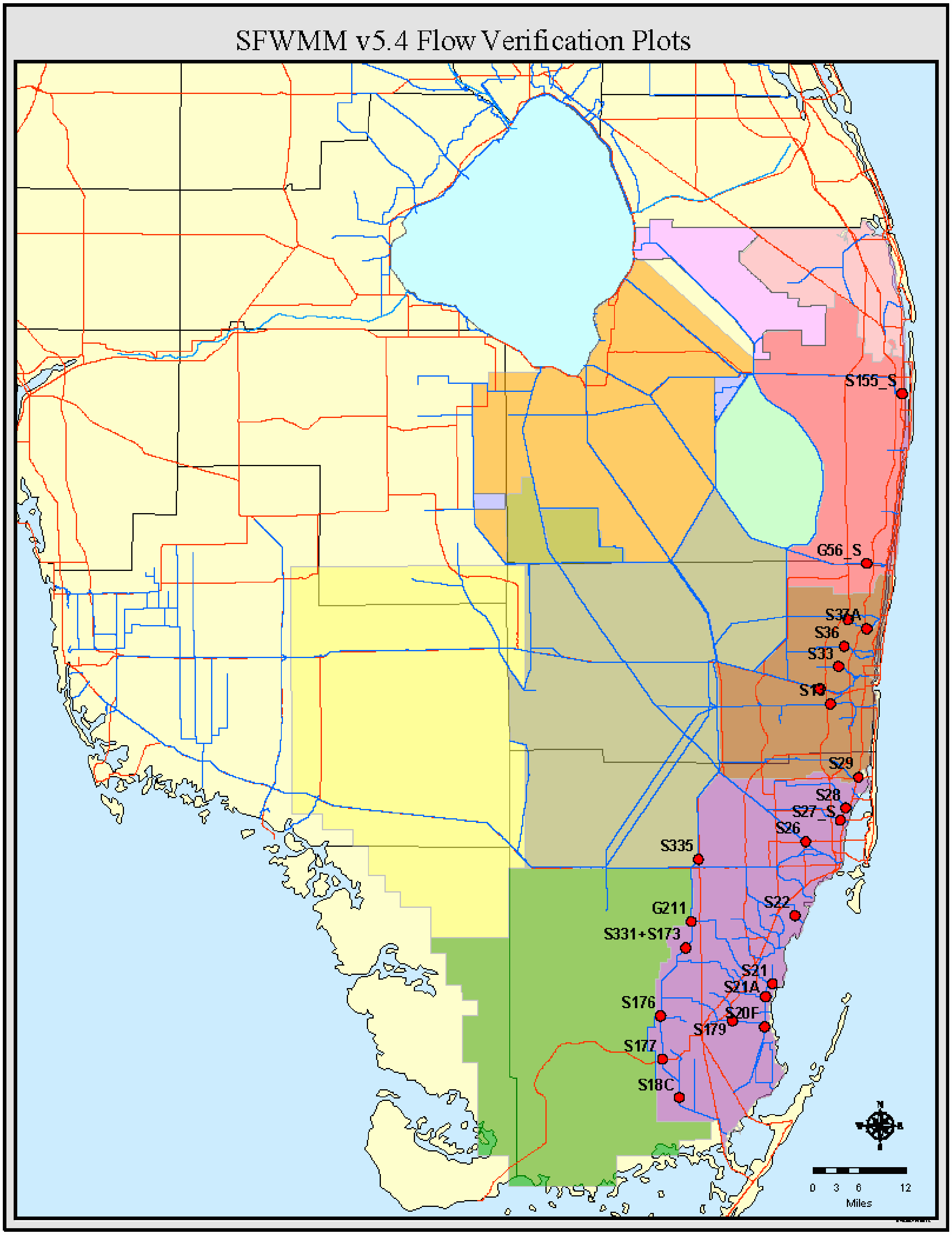

Figure 4.2.1.2 Locations of Flow Validation Sites

Calibration Procedure

Calibration was performed in an iterative fashion: (1) simulated stages were compared with historical stages at selected monitoring points and simulated flows were compared with historical flows at selected control structures; (2) appropriate calibration parameters were modified in order to make simulated values match historical values more closely; (3) the model was rerun with the revised parameters; and (4) steps 1 through 3 were repeated until an acceptable match between simulated and historical values was obtained.

The general guidelines used in calibrating the model were discussed in Section 4.1. Additional guidelines specific to the Everglades/LEC region are listed below.

1. The calibration period covered historical data consistent with a relatively static network of canals and water control structures, and constant structure operating rules.
2. Local parameters such as canal properties and cell-based data were adjusted before regional parameters were adjusted. Regional parameters such as land use type have influence over a greater area. This procedure was followed to minimize the undesirable effect of the calibration getting better in some areas but negatively affecting other areas in the model domain.
3. The ET-Recharge model was re-run for several snapshots of land use. The 1988 FLUCCS land use coverage was used as input to the ET-Recharge model for the 1984-1995 calibration period and the 1981-1983 verification period. The 2000 FLUCCS coverage was used for the 1995-2000 verification period.
4. It was shown (Trimble, 1995a) that canals heavily influence groundwater levels within their immediate proximity. The monitoring point closest to the canal, assuming that several observation points exist within the cell where the canal is located, is given priority for the stage matching. This allows for a better representation of the canal-groundwater interaction.

In order to determine the "acceptability" of a calibration run, many statistical measures and individual time series plots were used to help assess model performance. These will be shown in more detail in Section 4.2.2. In addition to comparing seasonal and annual sums and means, the following statistical measures and their corresponding ranges were used to evaluate the status of the calibration after each parameter change.

Coefficient of determination or correlation coefficient, R^2:

$$R^2 = \left[\frac{\sum_{i=1}^{n} (x_i - x_m)(\hat{x}_i - \hat{x}_m)}{\sqrt{\sum_{i=1}^{n} (x_i - x_m)^2 \sum_{i=1}^{n} (\hat{x}_i - \hat{x}_m)^2}} \right]^2 \qquad 0 \le R^2 \le {}^{+}1 \qquad (4.2.1.1)$$

Root mean square error, rmse:

$$rmse = \sqrt{\frac{\sum_{i=1}^{n}\left(\hat{x}_i - x_i\right)^2}{n-1}} \qquad 0 \le rmse \le {}^{+}\infty \qquad (4.2.1.2)$$

Bias:

$$bias = \frac{\sum_{i=1}^{n}\left(\hat{x}_i - x_i\right)}{n} \qquad \text{-}\infty \le bias \le {}^{+}\infty \qquad (4.2.1.3)$$

Nash-Sutcliffe Efficiency:

$$Eff = 1 - \frac{\sum_{i=1}^{n}\left(x_i - \hat{x}_i\right)^2}{\sum_{i=1}^{n}\left(x_i - x_m\right)^2} \qquad (4.2.1.4)$$

where:

n = number of data points
x_i = observed data point
x_m = mean of observed data points
$\hat{x}_i$ = simulated data point
$\hat{x}_m$ = mean of simulated data points

The Nash-Sutcliffe Efficiency can also be expressed as:

$$Eff = 2\,R\,\frac{S_{\hat{x}}}{S_x} - \frac{bias^2}{S_x{}^2} - \frac{S_{\hat{x}}{}^2}{S_x{}^2} \qquad (4.2.1.5)$$

where the standard deviation for the historical (S_x) and estimated ($S_{\hat{x}}$) data are:

$$S_x = \sqrt{\frac{\sum_{i=1}^{n}(x_i - x_m)^2}{n-1}} \qquad (4.2.1.6)$$

$$S_{\hat{x}} = \sqrt{\frac{\sum_{i=1}^{n}(\hat{x}_i - \hat{x}_m)^2}{n-1}} \qquad (4.2.1.7)$$

All comparisons using the above statistical measures were performed by limiting the number of data points by the size of available historical data. In other words, simulated data with no corresponding historical data were not considered in the statistical calculations. As a result, statistics generated from different sample sizes (varying from less than 100 to over 4000) were considered.

When comparing historical data with simulated values, several factors beyond the statistical matches were also considered. As a general rule, good engineering judgment must be used to supplement the information provided by the calibration statistics and plots. These included the following:

1. Exact matching of historical data may not be desirable in some cells during the calibration process. The simulated stage represents the average water level computed for a 4 square mile area. Comparing historical stage, a point measurement, against simulated stage, an estimated areal average, is a source of discrepancy in itself. As an example, if there is significant well pumpage in close proximity to the gage, the observed data can be strongly influenced; whereas the average effect of the well pumpage (over 4 square miles) can be fairly minimal. Similarly, a gage located next to a canal would show more variability in measured values than an average stage from a 4-square-mile cell, although in other cases it may be more desirable to use such a gage to better represent the canal-groundwater interaction.
2. The spatial resolution of the model, 2-miles by 2-miles, is too coarse for modeling local phenomena such as wellfield drawdowns and levee seepage.
3. The time resolution of the model, 1 day, may not always satisfy certain assumptions in the model. For example, in the overland flow subroutine, in order to maintain stability in the solution procedure, volume constraints during some simulation days may override the assumption that overland flow is a diffusion type process.
4. The scale of the model must also be considered in making stage comparisons in canals. The mean simulated stage over a two mile (or longer) reach may not be directly comparable to a point measurement on the canal just upstream of a water control structure.
5. When interpreting how well the model is matching the observed data, considerations must be given for the accuracy of the observed data. In some cases, observed data are known to reflect deviations from normal operating policy, such as pre-storm drawdowns, and would therefore not match the predicted values by the model. The model has time-varying rules of operation only for outlet structures of reaches with daily variation in simulated canal slope (dynamic canal slope option), where the criteria vary from normal condition to flood condition depending on antecedent rain. In some cases, the observed data was considered to be generally reliable, but suspect for a specific time period (based on comparisons with neighboring gages and hydro-meteorological responses).

As previously stated, the iterative calibration procedure was followed with consideration for the many statistical, graphical and anecdotal metrics refining model parameters for the local to regional scale. Once little or no improvement in history matching was observed with additional changes in parameters, the calibration effort was deemed complete. The next section discusses the results of the SFWMM v5.4 calibration and verification. With minor changes, v5.4 will become v5.5.

4.2.2 Calibration and Verification Results

Table 4.2.2.1 shows the calibration and verification statistics for the WCAs, the ENP, BCNP, Holey Land and Rotenberger WMAs, and the LECSAs. Because the full set of maps and figures showing the time series data at individual sites is so large, the maps and figures are provided in Appendix C. Examples of time series graphics are illustrated in Figures 4.2.2.1 and 4.2.2.2. When interpreting how well the model is matching the observed data, considerations must be given for the many issues of scale and data accuracy as outlined in the previous section. From Table 4.2.2.1, the following observations can be made:

1. WCA-1. One canal site and three marsh sites were available for comparisons with observed data; the sampling size for both calibration and verification was good. The R^2 values ranged from 0.7 - 0.8. The bias was about 0.1 ft or less, except at one site where the bias was 0.2+ ft.
2. WCA-2A. There were six marsh stations and one canal station used for comparisons. The calibration sampling size ranged from 400 to 4,000 values. The verification sampling size ranged from about 1,500 to 2,900 values. The R^2 values for calibration were generally in the 0.7 to 0.9 range with the verification R^2 values were about 0.6 and ranging from 0.3 - 0.7. The calibration and verification bias were generally less than 0.2 ft.
3. WCA-2B. There were two marsh stations used for comparisons. The calibration and verification records were good. The R^2 values range from 0.7 - 0.8. Calibration bias averaged about 0.1 ft and the verification bias was about 0.3 ft.
4. WCA-3A. There were fifteen marsh stations and five canal stations used for comparisons. In both cases the sampling records were good. In the marsh stations, calibration and verification R^2 values range from 0.8 - 0.9 generally. In the marsh calibration, the bias range from less than 0.1 to one station being high at about 0.7 ft. The verification bias ranges from 0.1 - 0.2 ft, again with the same one station having a high bias of 0.9 ft. For the canal gages, the R^2 values range from 0.8 - 0.9 and the bias range from less than 0.1 - 0.2 ft, generally speaking.
5. WCA-3B. There were five marsh stations used for comparisons. Sampling period was good at all but one station. For the calibration, R^2 values range from 0.4 - 0.8, and the verification R^2 values range from 0.6 - 0.8. Calibration bias was generally less than 0.1 feet with one station being 0.3 ft. The verification bias ranged from 0.1 - 0.3 ft.
6. ENP. There were 34 marsh stations, 4 well stations and 1 canal site used for comparisons. They were generally good sampling sizes at all but five stations. Generally, the R^2 values range from 0.8 - 0.9 with the lowest being about 0.4. The bias stations were generally in the range of 0.1 - 0.3 ft.
7. BCNP. Seventeen marsh stations were used for comparisons. Five had good sample sizes, two were poor and the rest had fair sampling sizes. The R^2 values for calibration ranged from 0.4 - 0.9; verification R^2 values, being a little less, ranged from 0.4 - 0.8. The bias generally ranged from less than 0.2 ft up to 0.7 ft; only one station was high.
8. NPBSA. There were five well sites and two canal sites used for comparisons; four had good records and three had poor records for sampling size. Calibration R^2 values range from about 0.3 - 0.6, and the R^2 values for verification range from 0.5 - 0.7. Only one canal station had very poor R^2 readings. The bias generally ranged from 0.1 up to 1.0 ft.
9. LEC-SA1. There were two marsh stations, fourteen well stations and three canal stations

used for comparisons. Twelve sampling records were good. The R^2 values generally ranged from 0.4 - 0.7 and twelve stations had generally less than 0.2 ft bias with one site up to 0.6 ft for the calibration period. For the verification period, nine stations had less than 0.2 ft with a range up to 1.0 ft.

10. LEC-SA2. There were 29 well sites and 11 canal sites used for comparisons. The period of record was generally good with very few exceptions. For well sites, the R^2 values range from about 0.0 - 0.8. For canal sites, the calibration R^2 values ranged from 0.0 - 0.6; verification R^2 ranged from 0.2 - 0.7. The bias in all cases was generally less than 0.2 ft.

11. LEC-SA3. There were 7 marsh stations, 35 well stations and 20 canal stations used for comparisons. There was a good sampling size at all sites. For the well and marsh stations, the R^2 values generally varied from 0.6 - 0.8 both in calibration and verification. For the canal sites, the R^2 values generally ranged from 0.2 - 0.8 for calibration and from 0.1 - 0.8 for verification. In all cases, the bias was generally less than 0.2 ft with many stations being less than 0.1 ft.

General Observations

Figure 4.2.2.3 displays the calibration correlation values for the stage locations. Figure 4.2.2.4 displays the verification correlation values for the stage locations. Green symbols denote a good correlation (0.61 - 1.00). Figure 4.2.2.5 displays the calibration bias for the stage locations. Figure 4.2.2.6 displays the verification bias for the stage locations. The darker green symbols denote an acceptable bias (within $\pm$0.5 feet of observed). Sign convention (positive or negative) of the bias value is also denoted inside the symbols in gage locations shown in the maps. The following general observations can be made from Figures 4.2.2.3 through 4.2.2.6:

1. The marsh areas tend to have higher R^2 values, generally in the 0.8 - 0.9 range, while the groundwater well sites in developed areas had lower R^2 values, generally ranging from 0.4 - 0.7.
2. With some exceptions, the bias was relatively small (generally less than 0.2 ft), with many values being less than 0.1 ft. The small bias occurred in marsh areas, both in the natural areas (undeveloped) and developed areas.
3. In the developed areas, the canals generally had poor R^2 values compared to well sites or marsh sites.
4. The R^2 values for the marsh sites in the developed areas (0.5 - 0.8 range) were not as good as the marsh areas in the natural areas.

The following comments are based on a review of the figures presented in Appendix C:

1. With few exceptions, the natural marsh areas have predicted hydrographs that correlate well with observed hydropatterns.
2. The observed data for the LEC canals have greater variability than the predicted patterns. The lower stages may be due to pre-storm drawdowns, while the greater overall variability may be due to the highly managed operations.
3. The observed data in the LEC marsh and well sites correlated well with predicted hydropatterns.
4. Although flow comparisons were not used to refine model calibration parameters, the monthly flow predictions at structures did match observed data reasonably well.

Table 4.2.2.1 Calibration and Verification Statistics for the WCAs, ENP, BCNP, Holey Land and Rotenberger Water Management Areas, and the LECSAs

SFWMM v5.4 Calibration (1984-1995) and Verification (1981-1993,1996-2000) Statistics for Stage Locations

Basin/Region	Station	Gage Type (1)	Land Use Type (2)		SFWMM		R^2		RMSE (ft.)		BIAS (ft.)		Efficiency		Sample Size	
			Calib.	Verif.	Row	Col	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.
WCA-1	1-7	Marsh	RS3	RS3	48	31	0.745	0.781	0.404	0.353	-0.106	-0.072	0.570	0.549	4124	2922
	1-8C	Canal	CNL	CNL			0.736	0.791	0.728	0.556	0.046	0.090	0.694	0.781	4383	2922
	1-8T	Marsh	RS3	RS3	47	34	0.751	0.783	0.510	0.444	0.208	0.214	0.533	0.607	4028	2841
	1-9	Marsh	RS3	RS3	46	33	0.813	0.820	0.380	0.346	0.127	0.163	0.574	0.686	3939	2922
WCA-2A	2A-17	Marsh	RS3	RS3	40	29	0.901	0.663	0.332	0.529	-0.088	-0.115	0.876	0.545	4383	2922
	2A-300	Marsh	RS3	RS3	39	29	0.835	0.591	0.470	0.653	-0.150	-0.154	0.807	0.521	4112	2733
	S11AHW	Canal	CNL	CNL			0.699	0.278	0.767	1.174	-0.194	-0.283	0.285	-0.951	2902	1818
	WCA2E4	Marsh	RS5	RS5	41	31	0.885	0.644	0.349	0.578	0.033	-0.185	0.870	0.406	433	1584
	WCA2F1	Marsh	MIX	MIX	43	30	0.776	0.717	0.536	0.501	-0.237	-0.297	0.717	0.545	432	1762
	WCA2F4	Marsh	RS5	RS5	41	30	0.870	0.566	0.373	0.514	0.032	-0.155	0.841	0.357	432	1509
	WCA2U1	Marsh	RS3	RS3	39	31	0.865	0.537	0.426	0.671	0.174	0.053	0.826	0.457	433	1717
WCA-2B	2B-Y	Marsh	RS4	RS4	35	30	0.740	0.828	1.227	0.418	0.073	0.300	0.716	0.607	3688	1663
	3-99	Marsh	RS4	RS4	35	30	0.830	0.791	0.594	0.438	0.146	-0.262	0.784	0.674	1589	1749
WCA-3A	3A-10	Marsh	MIX	MIX	40	19	0.852	0.802	0.278	0.300	0.095	-0.037	0.832	0.797	3797	2648
	3A-11	Marsh	RS4	RS4	38	19	0.900	0.823	0.723	0.904	-0.689	-0.864	-0.070	-1.113	3785	2673
	3A-12	Marsh	RS4	RS4	36	21	0.594	0.783	0.538	0.353	0.021	-0.046	0.572	0.775	3755	2760
	3A-2	Marsh	RS4	RS4	36	18	0.908	0.887	0.363	0.439	-0.093	-0.265	0.876	0.755	4308	2837
	3A-28	Marsh	RS2	RS2	24	19	0.880	0.888	0.487	0.410	0.242	0.285	0.780	0.784	4383	2912
	3A-3	Marsh	RS5	RS5	37	25	0.869	0.879	0.536	0.406	0.118	-0.095	0.847	0.861	4383	2922
	3A-4	Marsh	RS2	RS2	29	21	0.916	0.931	0.352	0.278	-0.095	-0.151	0.891	0.903	4383	2922
	3A-9	Marsh	RS4	RS4	35	21	0.918	0.908	0.339	0.462	-0.161	-0.366	0.885	0.684	4383	2586
	3A-NE	Marsh	SAW	SAW	40	23	0.631	0.917	0.823	0.455	-0.052	-0.237	0.618	0.810	4150	2663
	3A-NW	Marsh	RS5	RS5	40	18	0.847	0.852	0.339	0.409	-0.024	-0.100	0.839	0.787	3860	2771
	3A-S	Marsh	RS2	RS2	33	20	0.919	0.857	0.273	0.403	-0.095	-0.261	0.905	0.741	4285	2586
	3A-SW	Marsh	RS2	RS2	30	16	0.890	0.908	0.347	0.238	-0.008	-0.081	0.797	0.870	4131	2510
	G618	Marsh	RS4	RS4	22	23	0.853	0.887	0.313	0.297	0.094	-0.101	0.837	0.838	4255	2869
	L28-2	Marsh	CAT	CAT	33	16	0.903	0.823	0.366	0.540	-0.275	-0.466	0.765	0.276	2194	1813
	L29	Marsh	RS4	RS4	22	22	0.868	0.845	0.314	0.437	-0.168	-0.296	0.814	0.638	4383	2922
	S333HW	Canal	CNL	CNL			0.815	0.871	0.616	0.410	0.039	0.214	0.681	0.818	4383	2922
	S334HW	Canal	CNL	CNL			0.856	0.877	0.370	0.325	-0.236	-0.225	0.745	0.763	4383	2922
	S339HW	Canal	CNL	CNL			0.859	0.836	0.482	0.558	0.035	-0.085	0.851	0.806	4177	2922
	S340HW	Canal	CNL	CNL			0.861	0.818	0.491	0.561	-0.215	-0.271	0.822	0.748	4366	2901
	S344HW	Canal	CNL	CNL			0.961	0.894	0.527	0.316	0.500	-0.115	0.586	0.718	354	1827
WCA-3B	3B-2	Marsh	RS4	RS4	26	24	0.435	0.757	0.450	0.426	0.119	-0.348	0.391	0.214	1604	1808
	3B-29	Marsh	RS4	RS4	26	26	0.583	0.839	0.480	0.215	0.066	0.031	0.514	0.821	992	641
	3B-3	Marsh	RS4	RS4	30	27	0.638	0.699	0.332	0.270	0.086	-0.122	0.611	0.604	1571	1819
	3B-SE	Marsh	RS4	RS4	23	26	0.840	0.606	0.685	0.567	0.310	0.350	0.715	0.321	3003	1624
	SHARK	Marsh	RS4	RS4	23	24	0.857	0.708	0.360	0.316	0.006	-0.194	0.843	0.526	4228	2301
ENP	EP12R	Marsh	MAN	MAN	5	28	0.694	0.730	0.240	0.169	-0.155	-0.087	0.441	0.631	2495	333
	EP9R	Marsh	MAN	MAN	5	25	0.817	0.809	0.216	0.213	0.066	0.007	0.761	0.444	2223	365
	EPSW	Marsh	MAN	MAN	5	26	0.766	0.745	0.340	0.411	-0.232	-0.307	0.120	-0.874	3497	1743
	G1502	Marsh	MLP	MLP	17	24	0.878	0.832	0.482	0.474	0.256	-0.215	0.830	0.782	4352	2822
	G3272	Well	MLP	MLP	19	25	0.805	0.644	0.669	0.443	0.493	0.234	0.572	0.503	488	2029

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Table 4.2.2.1 (*cont.*) Calibration and Verification Statistics for the WCAs, ENP, BCNP, Holey Land and Rotenberger Water Management Areas, and the LECSAs

Basin/Region	Station	Gage Type (1)	Land Use Type (2)		SFWMM		R^2		RMSE (ft.)		BIAS (ft.)		Efficiency		Sample Size	
			Calib.	Verif.	Row	Col	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.
ENP	G3273	Marsh	MLP	MLP	17	24	0.887	0.857	0.480	0.418	0.281	-0.250	0.827	0.563	4310	1827
	G3437	Well	MLP	MLP	15	24	0.835	0.828	0.524	0.539	-0.153	-0.441	0.782	0.476	3288	1796
	G3576	Well	RS4	RS4	21	26	0.871	0.816	0.152	0.163	0.063	0.032	0.809	0.808	297	1763
	G3578	Well	RS4	RS4	20	26	0.911	0.754	0.245	0.273	0.190	0.144	0.630	0.654	259	1775
	G620	Marsh	MLP	MLP	19	18	0.840	0.920	0.431	0.158	0.079	0.044	0.719	0.907	3899	2212
	L67ES	Marsh	RS4		17	21	0.903		0.246		-0.008		0.902		1887	
	L67EXE	Marsh	RS4	RS4	19	22	0.752	0.786	0.341	0.315	0.120	-0.263	0.704	0.280	4097	1760
	L67EXW	Marsh	RS4	RS4	19	21	0.913	0.928	0.347	0.339	0.060	-0.266	0.864	0.759	4194	1833
	NESRS1	Marsh	RS4	RS4	20	22	0.812	0.700	0.299	0.217	0.127	-0.126	0.771	0.465	4205	2331
	NESRS2	Marsh	RS4	RS4	21	25	0.843	0.863	0.306	0.161	0.145	-0.059	0.764	0.823	3872	2356
	NESRS3	Marsh	RS4	RS4	21	26	0.828	0.816	0.387	0.277	0.095	-0.225	0.801	0.459	3660	1827
	NESRS4	Marsh	RS4	RS4	18	21	0.891	0.831	0.340	0.259	0.270	0.155	0.370	0.539	1054	1696
	NESRS5	Marsh	RS4	RS4	18	22	0.752	0.818	0.391	0.167	0.311	0.031	0.320	0.811	3118	1817
	NP-201	Marsh	MLP	MLP	21	19	0.869	0.861	0.351	0.425	-0.097	-0.323	0.852	0.646	3831	1875
	NP-202	Marsh	RS1	RS1	19	20	0.894	0.910	0.274	0.238	0.057	-0.138	0.887	0.861	4002	2722
	NP-203	Marsh	RS1	RS1	17	19	0.893	0.901	0.253	0.253	0.073	-0.167	0.880	0.811	3780	2354
	NP-205	Marsh	MLP	MLP	20	15	0.803	0.630	0.504	0.539	0.091	0.121	0.786	0.546	4275	2874
	NP-206	Marsh	MLP	MLP	15	21	0.839	0.838	0.630	0.468	0.367	0.188	0.756	0.807	3737	2884
	NP-207	Marsh	MLP	MLP	6	20	0.792	0.820	0.449	0.319	-0.293	-0.170	0.512	0.543	4286	2472
	NP-33	Marsh	RS1	RS1	17	20	0.881	0.854	0.361	0.219	0.264	0.010	0.734	0.801	4190	2793
	NP-34	Marsh	MLP	MLP	17	13	0.831	0.837	0.431	0.371	0.066	-0.060	0.773	0.784	4109	2864
	NP-35	Marsh	RS1	RS1	12	15	0.643	0.712	0.421	0.348	0.098	0.164	0.536	0.617	4252	2511
	NP-36	Marsh	RS1	RS1	14	17	0.800	0.889	0.340	0.181	0.118	-0.049	0.755	0.849	4138	2814
	NP-38	Marsh	RS1	RS1	9	16	0.871	0.848	0.264	0.217	-0.042	0.049	0.839	0.828	4092	2797
	NP-44	Marsh	MLP	MLP	11	19	0.801	0.800	0.675	0.560	0.088	0.140	0.785	0.785	4116	2289
	NP-46	Marsh	MLP	MLP	7	17	0.630	0.675	0.610	0.340	-0.408	-0.135	-0.010	0.465	3717	2718
	NP-62	Marsh	RS1	RS1	11	17	0.823	0.872	0.467	0.301	0.060	-0.079	0.798	0.838	3488	2636
	NP-67	Marsh	RS1	RS1	7	22	0.801	0.851	0.406	0.333	-0.238	-0.223	0.670	0.707	3964	2408
	NP-72	Marsh	MLP	MLP	9	20	0.845	0.798	0.534	0.523	-0.007	0.011	0.839	0.798	3812	2763
	NP-RG1	Marsh		MLP	16	23		0.912		0.397		0.255		0.764		1269
	NP-RG2	Marsh		MLP	15	23		0.894		0.402		0.215		0.774		1492
	NP-TSB	Marsh	MLP	MLP	9	23	0.814	0.799	0.690	0.775	-0.372	-0.548	0.612	0.465	4383	2916
	RUTZKE	Marsh	MLP	MLP	14	24	0.854	0.841	0.440	0.385	0.363	0.248	0.529	0.721	542	1827
	S332HW	Canal	CNL	CNL			0.434	0.615	0.636	0.709	0.083	0.063	-0.065	0.289	4383	2922
BCNP	BCNP10	Marsh	FWT	FWT	20	10	0.357	0.345	0.250	0.249	-0.104	-0.063	0.023	-0.310	1327	1149
	BCNP12	Marsh	FWT	FWT	37	8	0.437	0.443	0.486	0.672	-0.189	-0.115	0.329	0.364	1461	1827
	BCNP13	Marsh	FWT	FWT	36	4	0.906	0.442	0.158	0.632	-0.120	0.124	0.682	0.369	96	1827
	BCNPA2	Marsh	SAW	SAW	37	2	0.626	0.615	0.745	0.852	-0.530	-0.434	0.113	0.428	1817	1827
	BCNPA5	Marsh	FWT	FWT	29	13	0.755	0.802	0.406	0.391	-0.069	-0.156	0.535	0.732	1773	1784
	BCNPA8	Marsh	SAW	SAW	26	2	0.646	0.449	1.260	1.471	1.118	1.242	-1.776	-1.330	1798	1827
	BEARI	Marsh	FWT	FWT	39	1	0.540	0.467	0.848	0.886	-0.534	-0.393	-0.179	-0.028	1823	1827
	C296	Well	SAW	SAW	34	1	0.782	0.682	0.548	1.146	-0.310	-0.680	0.642	0.484	310	2439
	C54	Well	FWT	FWT	36	14	0.505	0.532	0.683	0.764	0.114	0.332	0.447	0.340	4378	2647
	L28.GA	Marsh	FWT	FWT	34	11	0.703	0.478	0.487	0.564	0.238	0.227	0.587	0.282	4323	2070
	LOOP1	Marsh	FWT	FWT	22	14	0.709	0.719	0.566	0.226	-0.322	-0.046	0.398	0.658	3782	1776
	LOOP2	Marsh	FWT	FWT	22	12	0.776	0.664	0.522	0.362	-0.155	0.167	0.719	0.573	3857	1828

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Table 4.2.2.1 (*cont.*) Calibration and Verification Statistics for the WCAs, ENP, BCNP, Holey Land and Rotenberger Water Management Areas, and the LECSAs. The yellow highlights indicate LEC Cutback Trigger Locations.

Basin/Region	Station	Gage Type (1)	Land Use Type (2)		SFWMM		R^2		RMSE (ft.)		BIAS (ft.)		Efficiency		Sample Size	
			Calib.	Verif.	Row	Col	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.
BCNP	MONRD	Marsh	FWT	FWT	29	7	0.695	0.718	0.841	0.606	-0.710	-0.276	-1.156	0.525	1774	1827
	OKA858	Marsh	CIT	ROW	41	2	0.539	0.412	0.786	1.132	-0.165	0.479	0.308	0.117	1503	1761
	ROBLK	Marsh	FWT	FWT	23	8	0.711	0.672	0.405	0.669	-0.170	0.079	0.597	0.593	1830	1719
	TAMI40	Marsh	FWT	FWT	25	11	0.760	0.886	0.536	0.409	-0.201	-0.191	0.720	0.830	4373	2908
	TAMIAM	Marsh	FWT	FWT	26	3	0.678	0.609	0.837	1.071	0.524	0.727	0.464	0.268	4325	2556
HoleyLand	HOLEY1	Marsh	MIX	MIX	45	19	0.532	0.448	0.476	0.611	0.204	0.515	0.390	-0.925	1795	1646
	HOLEY2	Marsh	RS5	RS5	42	21	0.341	0.292	0.620	0.548	-0.162	0.100	0.142	0.116	1746	1750
	HOLEYG	Marsh	RS5	RS5	43	18	0.536	0.369	0.587	0.507	-0.068	0.366	0.037	-0.325	2939	1658
Rotenberger	ROTT.N	Marsh	SAW	SAW	46	15	0.218	0.539	0.730	0.652	-0.103	0.147	-0.008	0.263	2674	1482
	ROTT.S	Marsh	MIX	MIX	43	16	0.624	0.693	0.611	0.465	-0.428	-0.230	-0.332	0.585	2806	1687
NPBSA	JUP.W	Well	FUP	MDU	62	38	0.666	0.514	0.721	0.905	-0.012	-0.214	0.660	0.483	163	34
	LOXR1	Well	FUP		63	36	0.603		0.374		0.031		0.567		1435	
	PB109	Well	FWT	FWT	58	36	0.495	0.759	0.892	0.495	0.426	-0.002	0.287	0.723	2775	1095
	PB565	Well	LDU	MDU	64	39	0.263	0.545	1.737	1.274	0.997	0.417	-0.391	0.275	4283	2912
	S44HW	Canal	CNL	CNL			0.079	0.008	0.205	0.267	-0.159	-0.018	-1.886	-0.073	4349	2866
	S46HW	Canal	CNL	CNL			0.628	0.467	0.733	0.606	0.395	0.246	0.301	0.362	4363	2922
	SCUM	Well	FUP	LDU	60	39	0.566	0.545	0.937	1.347	-0.106	-1.075	0.543	-0.579	93	33
LEC-SA1	E3HW	Canal	CNL	CNL			0.223	0.077	0.285	0.246	-0.158	0.005	-0.147	0.074	4229	2468
	G1213	Well	LDU	MDU	40	36	0.755	0.626	0.528	0.644	-0.019	0.110	0.752	0.614	4378	2816
	G1260	Well	HDU	HDU	41	38	0.847	0.733	0.755	0.932	0.117	-0.305	0.822	0.697	4380	2919
	G1315	Well	MDU	MDU	40	37	0.705	0.702	0.751	0.744	0.060	-0.062	0.673	0.488	4303	2816
	G2030	Well	CIT	MDU	41	33	0.445	0.504	0.571	0.750	0.203	0.042	0.278	0.237	2036	1095
	G56HW	Canal	CNL	CNL			0.047	0.033	0.946	1.240	0.029	0.002	-0.358	-0.197	4383	2922
	PB1495	Well	MDU		44	39	0.676		0.511		-0.136		0.636		2933	
	PB1515	Well	LDU		51	36	0.705		0.521		-0.180		0.517		611	
	PB1661	Marsh	LDU	MDU	44	37	0.770	0.785	0.530	0.438	-0.444	-0.329	0.229	0.499	2179	1746
	PB445	Well	ROW	MDU	49	37	0.276	0.356	0.458	0.562	0.014	0.325	-0.055	-0.439	4340	2823
	PB561	Well	LDU	MDU	55	35	0.658	0.587	0.932	0.902	0.128	0.062	0.642	0.499	4326	2830
	PB683	Well	LDU	LDU	51	35	0.592	0.625	0.933	0.954	-0.658	-0.726	0.096	0.059	4279	2856
	PB732	Well	MDU	MDU	43	38	0.804	0.656	0.454	0.670	-0.020	0.011	0.764	0.592	4253	2681
	PB809	Well	HDU	HDU	54	39	0.697	0.543	0.743	1.334	0.473	1.002	0.221	-0.134	4305	2887
	PB88	Well	HDU	HDU	51	40	0.376	0.703	1.098	1.265	0.385	0.085	0.184	0.686	3146	809
	PB900	Well	ROW	MDU	45	37	0.460	0.437	0.422	0.425	0.179	0.139	0.130	0.039	4271	1422
	PB99	Well	MDU	MDU	53	40	0.701	0.792	0.658	0.610	-0.092	-0.085	0.547	0.661	4320	2779
	S155HW	Canal	CNL	CNL			0.154	0.163	0.339	0.407	-0.217	-0.281	-1.276	-1.019	4225	2308
	WPBCA	Marsh	MAR	MAR	56	36	0.511	0.719	0.592	0.991	-0.037	-0.828	0.489	-2.697	564	1227
LEC-SA2	F291	Well	MDU	MDU	30	37	0.794	0.804	0.367	0.395	-0.036	-0.008	0.730	0.781	4267	2826
	G1215	Well	MDU	MDU	40	38	0.766	0.649	1.228	1.749	-0.113	-0.635	0.709	0.565	4219	2381
	G1220	Well	MDU	MDU	35	37	0.792	0.824	0.372	0.328	-0.191	-0.091	0.707	0.803	4336	2838
	G1221	Well	MDU	MDU	33	35	0.529	0.609	0.485	0.405	0.032	0.071	0.494	0.591	4308	2231
	G1222	Well	LDU	MDU	31	30	0.518	0.713	0.507	0.558	-0.105	-0.335	0.485	0.550	2678	1095
	G1223	Well	MDU	MDU	31	34	0.709	0.716	0.411	0.459	0.029	0.059	0.544	0.506	4349	2634
	G1224	Well	MDU	MDU	32	37	0.838	0.856	0.389	0.391	0.100	0.125	0.647	0.747	4296	2854
	G1225	Well	MDU	MDU	31	34	0.862	0.862	0.316	0.358	0.048	0.016	0.858	0.861	4322	2721
	G1316	Well	HDU	HDU	39	36	0.455	0.713	0.556	0.364	-0.072	-0.148	0.260	0.523	4191	1760
	G1472	Well	MDU	MDU	30	37	0.762	0.810	0.384	0.399	-0.118	0.005	0.712	0.806	3529	1095

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Table 4.2.2.1 (*cont.*) Calibration and Verification Statistics for the WCAs, ENP, BCNP, Holey Land and Rotenberger Water Management Areas, and the LECSAs. The yellow highlights indicate LEC Cutback Trigger Locations.

Basin/Region	Station	Gage Type (1)	Land Use Type (2)		SFWMM		R^2		RMSE (ft.)		BIAS (ft.)		Efficiency		Sample Size	
			Calib.	Verif.	Row	Col	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.
LEC-SA2	G1473	Well	MDU	MDU	30	37	0.805	0.796	0.378	0.409	0.090	-0.010	0.711	0.752	4361	2903
	G1636	Well	LDU	LDU	29	30	0.625	0.637	0.450	0.574	-0.302	-0.379	0.244	-0.067	4293	2890
	G1637	Well	RS4	RS4	29	28	0.692	0.521	0.485	0.508	0.312	0.244	0.464	0.316	4219	2764
	G2031	Well	MDU	MDU	39	33	0.576	0.396	0.438	0.493	-0.134	0.010	0.462	0.074	4350	2922
	G2032	Well	LDU	LDU	35	32	0.429	0.367	0.536	0.604	0.218	0.231	0.208	-0.195	4320	2839
	G2033	Well	HDU	HDU	37	33	0.462	0.518	0.496	0.430	0.102	0.078	0.344	0.303	4306	2835
	G2034	Well	LDU	MDU	31	30	0.538	0.610	0.465	0.429	-0.163	0.121	0.409	0.541	4291	2813
	G2035	Well	MDU	MDU	31	36	0.801	0.786	0.627	0.636	-0.454	-0.429	0.104	0.352	4308	2922
	G2147	Well	MDU	MDU	39	39	0.607	0.510	0.896	1.155	0.275	0.062	0.562	0.508	4296	2858
	G2275	Well	MDU	MDU	37	37	0.725	0.852	0.602	0.566	0.079	0.358	0.713	0.740	1040	859
	G2376	Well	RS5	RS5	35	28	0.700	0.574	0.378	0.312	-0.082	0.099	0.682	0.522	4092	494
	G2443	Well	MDU		38	36	0.645		0.519		0.322		0.419		2895	
	G2444	Well	MDU		37	36	0.732		0.686		0.233		0.560		2789	
	G54HW	Canal	CNL	CNL			0.054	0.012	0.549	0.701	0.229	0.052	-0.527	-0.289	4346	2922
	G561	Well	HDU	HDU	34	37	0.780	0.789	0.357	0.349	-0.100	0.026	0.715	0.756	4291	2850
	G57HW	Canal	CNL	CNL			0.120	0.000	0.185	0.348	-0.021	0.066	-1.053	-0.680	1835	1826
	G616	Well	MDU	MDU	40	34	0.536	0.653	0.752	1.366	-0.001	-0.619	0.511	0.541	3618	681
	G617	Well	LDU	LDU	33	31	0.437	0.552	0.445	0.443	0.041	0.050	0.219	0.477	4383	2834
	G820A	Well	MDU		37	37	0.853		0.574		-0.321		0.785		4085	
	G970	Well	LDU	LDU	29	30	0.572	0.518	0.347	0.514	-0.074	-0.154	0.294	-0.102	4161	2738
	S13HW	Canal	CNL	CNL			0.086	0.209	0.300	0.277	-0.112	-0.072	-0.740	-0.271	4383	2922
	S29HW	Canal	CNL	CNL			0.035	0.001	0.284	0.324	0.030	0.016	-0.240	-0.330	4283	2922
	S30HW	Canal	CNL	CNL			0.587	0.205	0.613	0.502	0.167	0.118	0.515	0.136	3136	1827
	S329	Well	MDU	MDU	34	35	0.719	0.731	1.576	1.194	1.426	1.019	-0.575	0.003	4266	2900
	S33HW	Canal	CNL	CNL			0.449	0.129	0.276	0.344	0.011	-0.031	0.331	-0.505	4383	2922
	S36HW	Canal	CNL	CNL			0.049	0.035	0.305	0.322	-0.083	-0.010	-0.258	-0.253	4383	2889
	S37AHW	Canal	CNL	CNL			0.000	0.155	0.250	0.245	0.010	0.089	-0.322	-0.007	4383	2922
	S37BHW	Canal	CNL	CNL			0.021	0.016	0.279	0.353	0.052	-0.070	-0.473	-0.425	4373	2922
	S9HW	Canal	CNL	CNL			0.659	0.650	0.567	0.697	-0.289	-0.400	0.172	-0.068	4380	2891
	S9XNHW	Canal		CNL				0.384		0.330		-0.111		0.239		1214
LEC-SA3	C2.74	Canal	CNL	CNL			0.884	0.511	0.492	0.415	0.139	-0.155	0.848	0.416	4122	1715
	EVER1	Marsh	MLP	MLP	7	29	0.590	0.515	0.477	0.520	-0.189	-0.092	-1.384	-2.298	3441	1687
	EVER2B	Marsh	MLP	MLP	7	27	0.728	0.724	0.369	0.328	-0.142	-0.052	0.430	0.492	3647	1755
	EVER3	Marsh	MLP	MLP	8	26	0.794	0.841	0.221	0.158	0.076	-0.012	0.766	0.833	3347	1772
	EVER4	Marsh	MLP	MLP	8	25	0.846	0.909	0.251	0.206	0.096	0.032	0.752	0.750	3213	1793
	F179	Well	HDU	HDU	22	34	0.763	0.781	0.353	0.365	-0.206	-0.169	0.626	0.697	4383	2907
	F319	Well	MDU	MDU	20	33	0.698	0.567	0.386	0.437	0.149	0.123	0.139	0.099	4263	2857
	F358	Well	MDU	MDU	12	27	0.817	0.805	0.408	0.444	-0.041	0.021	0.653	0.684	4383	2844
	F45	Well	HDU	HDU	24	35	0.823	0.855	0.296	0.307	-0.039	-0.022	0.816	0.848	4354	2909
	FROGP	Well	ROW	ROW	11	24	0.716	0.638	0.353	0.436	0.050	-0.067	0.696	0.611	4120	1827
	G1166	Well	LDU	LDU	27	31	0.588	0.553	0.231	0.262	-0.015	0.021	0.586	0.549	4339	2597
	G1183	Well	HDU	HDU	13	30	0.582	0.580	0.384	0.402	-0.059	0.006	0.373	0.361	4201	2791
	G1251	Well	MLP	MLP	7	24	0.788	0.806	0.414	0.385	-0.140	-0.131	0.486	0.412	4383	2577
	G1362	Well	ROW	ROW	17	28	0.762	0.792	0.431	0.427	0.179	0.127	0.681	0.749	4246	2768
	G1363	Well	CIT	CIT	15	26	0.829	0.846	0.457	0.457	0.249	0.209	0.714	0.767	4346	2909
	G1486	Well	MDU	MDU	13	28	0.819	0.785	0.388	0.437	0.062	0.011	0.603	0.566	4322	2894

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Table 4.2.2.1 (*cont.*) Calibration and Verification Statistics for the WCAs, ENP, BCNP, Holey Land and Rotenberger Water Management Areas, and the LECSAs. The yellow highlights indicate LEC Cutback Trigger Locations.

Basin/Region	Station	Gage Type (1)	Land Use Type (2)		SFWMM		R^2		RMSE (ft.)		BIAS (ft.)		Efficiency		Sample Size	
			Calib.	Verif.	Row	Col	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.
LEC-SA3	G1487	Well	ROW	ROW	19	27	0.701	0.527	0.573	0.476	-0.314	-0.222	0.566	0.385	4296	2020
	G1488	Well	RS5	RS5	24	27	0.765	0.864	0.544	0.441	-0.109	-0.150	0.663	0.701	4220	2863
	G211HW	Canal	CNL	CNL			0.731	0.117	0.430	0.340	0.115	-0.113	0.690	-0.288	1822	1827
	G3264A	Well	MEL	MEL	25	30	0.854	0.655	0.395	0.450	-0.084	0.025	0.845	0.647	4136	1710
	G3327	Well	HDU	HDU	23	34	0.501	0.687	0.348	0.302	0.023	0.146	0.495	0.590	4263	1692
	G3328	Well	HDU	HDU	23	34	0.538	0.691	0.276	0.248	-0.062	-0.061	0.425	0.573	4225	1785
	G3329	Well	MDU	MDU	23	32	0.566	0.616	0.442	0.512	0.120	0.281	0.158	0.081	4314	1773
	G3353	Well	MLP	MLP	6	24	0.779	0.749	0.292	0.287	0.026	-0.028	0.627	0.493	3663	1794
	G3354	Well	MLP	MLP	7	26	0.838	0.850	0.240	0.228	-0.138	-0.155	0.752	0.719	3305	1704
	G3439	Well	MEL	MDU	21	28	0.808	0.734	0.437	0.525	0.156	0.360	0.780	0.453	3017	1558
	G553	Well	MDU	MDU	18	31	0.842	0.741	0.542	0.604	-0.303	-0.248	0.496	0.322	3935	2762
	G580A	Well	LDU	MDU	19	32	0.733	0.657	0.392	0.442	0.047	0.078	0.510	0.442	4334	2905
	G596	Marsh	ROW	ROW	18	26	0.693	0.626	0.548	0.508	0.298	-0.032	0.558	0.624	4273	2922
	G613	Marsh	ROW	ROW	10	26	0.662	0.669	0.369	0.392	0.167	0.082	0.483	0.352	4314	2886
	G614	Well	CIT	CIT	15	28	0.807	0.819	0.494	0.471	0.335	0.140	0.624	0.768	4247	2818
	G757A	Well	ROW	ROW	16	27	0.773	0.812	0.410	0.421	0.127	0.043	0.720	0.773	4282	2860
	G789	Well	CIT	ROW	12	25	0.701	0.755	0.374	0.420	0.018	-0.101	0.696	0.739	4162	2894
	G852	Well	MDU	MDU	27	36	0.638	0.648	0.375	0.486	-0.033	-0.217	0.572	0.499	4178	2916
	G855	Well	MDU	HDU	19	28	0.719	0.727	0.511	0.508	0.038	0.007	0.591	0.599	4162	2876
	G858	Well	HDU	HDU	18	29	0.675	0.806	0.524	0.568	-0.112	0.033	0.551	0.484	3415	1095
	G860	Well	LDU	MDU	17	32	0.648	0.517	0.406	0.455	-0.124	-0.094	0.356	0.216	4383	2918
	G864	Well	CIT	ROW	11	26	0.757	0.751	0.388	0.450	0.000	-0.084	0.739	0.721	4380	2922
	G973	Well	MDU	HDU	26	31	0.654	0.556	0.379	0.370	0.178	0.127	0.557	0.477	4300	2883
	G975	Well	RS5	RS5	26	27	0.658	0.744	0.918	0.786	0.647	0.591	0.180	0.100	4142	2908
	G976	Well	MEL	MEL	24	28	0.797	0.528	0.862	0.713	0.363	-0.281	0.623	0.390	4213	2898
	S118HW	Canal	CNL	CNL			0.825	0.695	0.321	0.406	-0.048	-0.098	0.754	0.522	4376	2922
	S119HW	Canal	CNL	CNL			0.844	0.706	0.480	0.632	-0.212	-0.293	0.513	-0.075	4345	2922
	S123HW	Canal	CNL	CNL			0.563	0.201	0.410	0.472	-0.087	-0.004	0.231	-0.393	3437	1818
	S148HW	Canal	CNL	CNL			0.269	0.310	0.724	0.738	0.062	0.110	0.145	0.235	4221	2875
	S149HW	Canal	CNL	CNL			0.507	0.416	0.406	0.407	0.068	0.067	0.461	0.271	4334	1789
	S165HW	Canal	CNL	CNL			0.507	0.653	0.459	0.385	0.068	0.008	0.491	0.652	4327	2915
	S166HW	Canal	CNL	CNL			0.793	0.802	0.461	0.468	0.288	0.252	0.419	0.430	4383	2922
	S167HW	Canal	CNL	CNL			0.623	0.619	0.421	0.484	0.083	0.109	0.577	0.584	4383	2922
	S176HW	Canal	CNL	CNL			0.717	0.593	0.335	0.425	0.077	-0.060	0.679	0.583	4383	2922
	S177HW	Canal	CNL	CNL			0.518	0.383	0.368	0.428	0.077	0.047	0.460	0.349	4383	2698
	S179HW	Canal	CNL	CNL			0.725	0.679	0.340	0.394	0.022	-0.018	0.520	0.352	4378	2922
	S18	Marsh	MDU	MDU	28	34	0.698	0.737	0.249	0.257	0.101	0.043	0.630	0.720	4328	2740
	S182	Well	MDU	MDU	16	31	0.633	0.667	0.377	0.419	-0.190	-0.190	0.322	0.496	4332	2796
	S18CHW	Canal	CNL	CNL			0.646	0.577	0.242	0.316	0.029	-0.098	0.640	0.200	4380	2922
	S196A	Well	CIT	ROW	13	26	0.836	0.838	0.367	0.377	0.201	0.142	0.754	0.809	4337	2900
	S197HW	Canal	CNL	CNL			0.789	0.645	0.333	0.350	-0.235	-0.214	0.532	0.299	4233	2733
	S20FHW	Canal	CNL	CNL			0.203	0.126	0.378	0.497	-0.102	-0.150	-0.771	-1.908	3750	2206
	S21AHW	Canal	CNL	CNL			0.283	0.175	0.268	0.314	0.055	0.053	0.097	-0.337	4375	2922
	S21HW	Canal	CNL	CNL			0.112	0.040	0.283	0.317	-0.090	-0.001	-0.546	-1.066	4383	2922
	S22HW	Canal	CNL	CNL			0.421	0.230	0.478	0.491	-0.148	-0.193	-0.035	-0.489	4366	2922
	S25HW	Canal	CNL	CNL			0.191	0.010	0.243	0.264	-0.087	-0.095	-0.460	-1.645	3910	2346

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Table 4.2.2.1 (*cont.*) Calibration and Verification Statistics for the WCAs, ENP, BCNP, Holey Land and Rotenberger Water Management Areas, and the LECSAs. The yellow highlights indicate LEC Cutback Trigger Locations.

Basin/Region	Station	Gage Type (1)	Land Use Type (2)		SFWMM		R^2		RMSE (ft.)		BIAS (ft.)		Efficiency		Sample Size	
			Calib.	Verif.	Row	Col	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.	Calib.	Verif.
LEC-SA3	S26HW	Canal	CNL	CNL			0.051	0.027	0.345	0.379	0.106	0.108	-0.174	-0.115	3755	1827
	S27HW	Canal	CNL	CNL			0.073	0.173	0.237	0.246	-0.025	-0.093	-0.196	-0.029	4299	2922
	S28HW	Canal	CNL	CNL			0.059	0.017	0.215	0.281	-0.063	-0.138	-0.183	-0.776	4383	2922
	S335HW	Canal	MDU	HDU			0.639	0.610	0.681	0.486	-0.014	0.282	0.416	0.383	4383	1868
	S331HW	Canal	CNL	CNL			0.352	0.088	0.510	0.552	-0.048	-0.199	0.263	-0.356	4369	2231
L8	PB831	Well	FUP	FUP	60	29	0.683	0.759	0.637	0.687	0.044	-0.237	0.556	0.500	4219	2864

Notes: **(1) Statistics for canal stages are derived from a smoothed trace (7-day moving average)**

(3) Denotes LEC Cutback Trigger Location

(2) Land Use Legend

Code	Description
LDU	Low Density Urban
CIT	Citrus
MAR	Freshwater Marsh
SAW	Sawgrass
WET	Wet Prairie
SHR	Shrubland (includes Rangeland)
ROW	Row Crops
SUG	Sugar Cane
IRR	Irrigated Pasture
STA	Stormwater Treatment Area (with dense vegetation)
HDU	High Density Urban
FWT	Forested Wetland
MAN	Mangroves
MEL	Melaleuca
CAT	Cattail
FUP	Forested Uplands
RS1	Ridge & Slough 1
MLP	Marl Prairie
MIX	Mixed Cattail-Sawgrass
WAT	Open Water
RS2	Ridge & Slough 2
RS3	Ridge & Slough 3
RS4	Ridge & Slough 4
RS5	Ridge & Slough 5
MDU	Medium Density Urban
CNL	Canal

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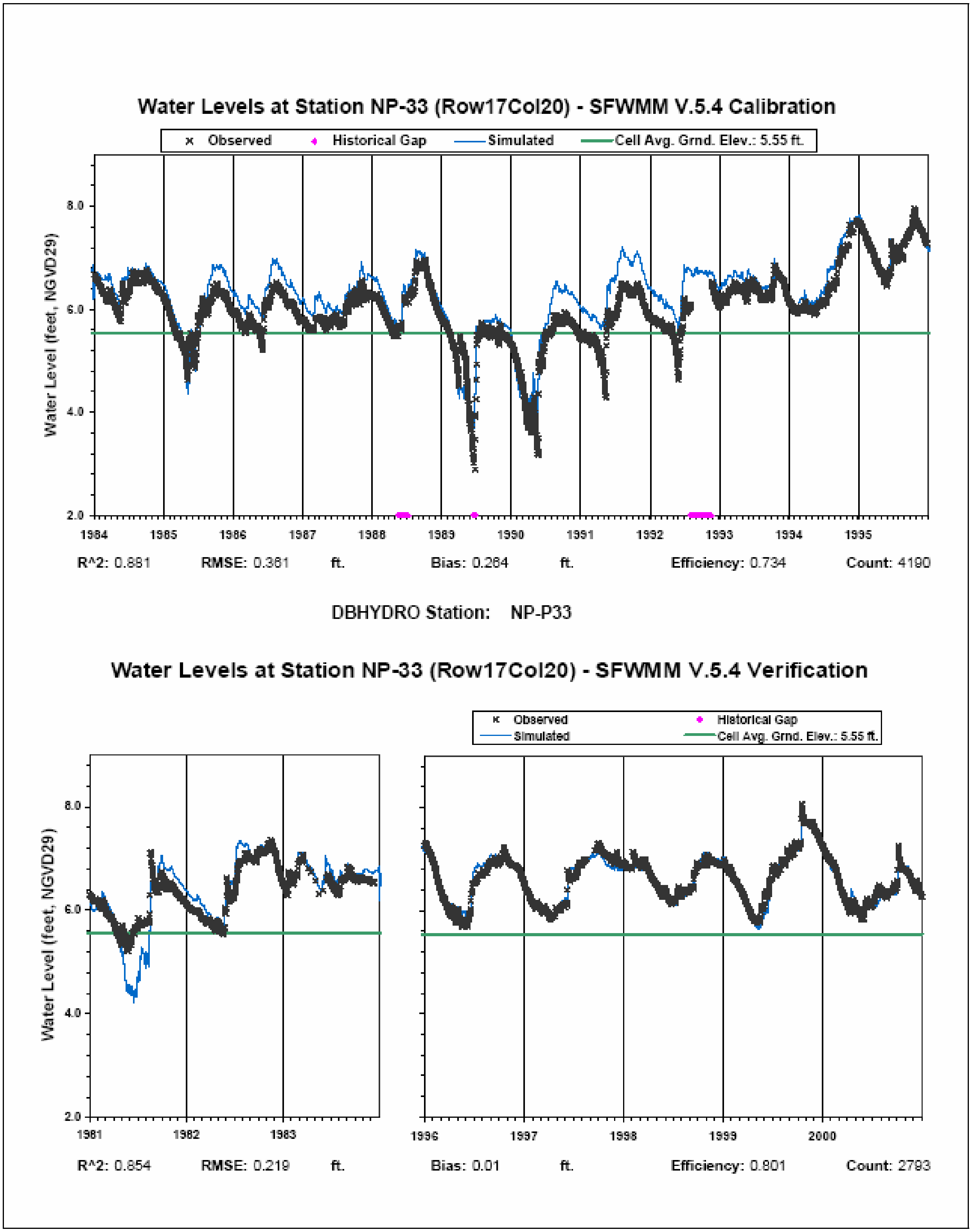

Figure 4.2.2.1 Example Time Series Figures for Water Level Calibration and Verification

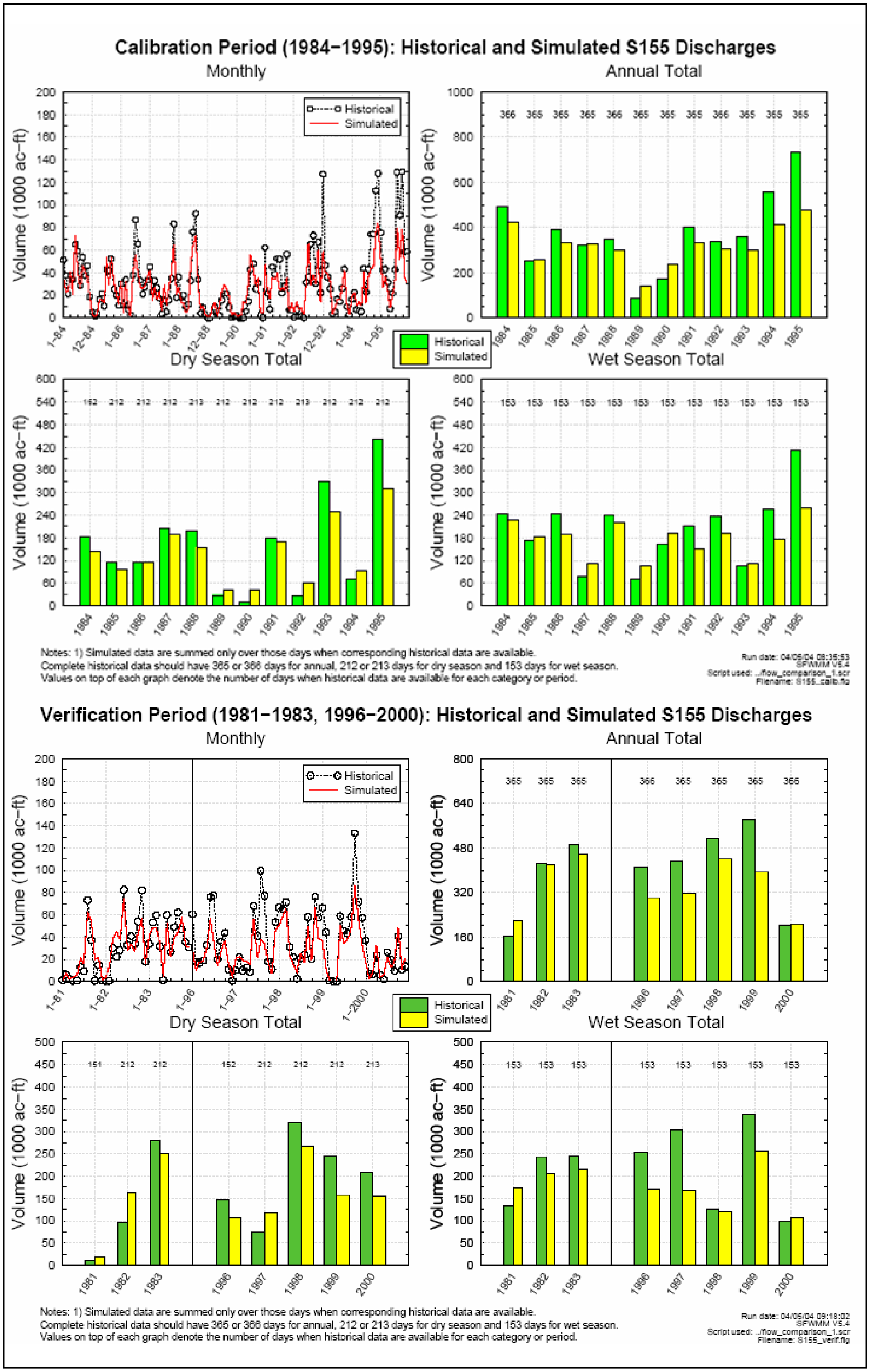

Figure 4.2.2.2 Example Time Series Figures for Flow Validation
(Used Only as a Reasonability Check)

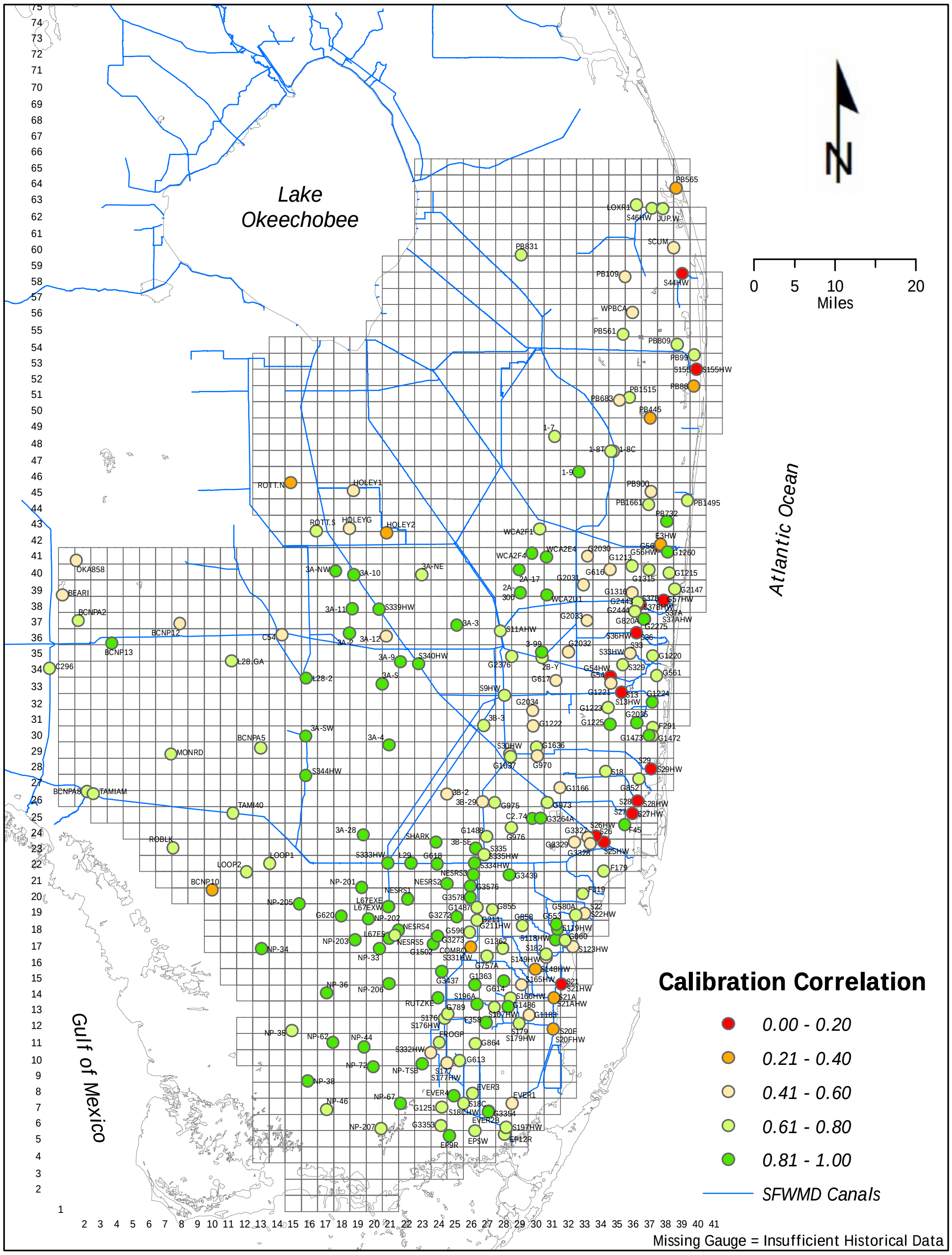

Figure 4.2.2.3 Calibration Correlation

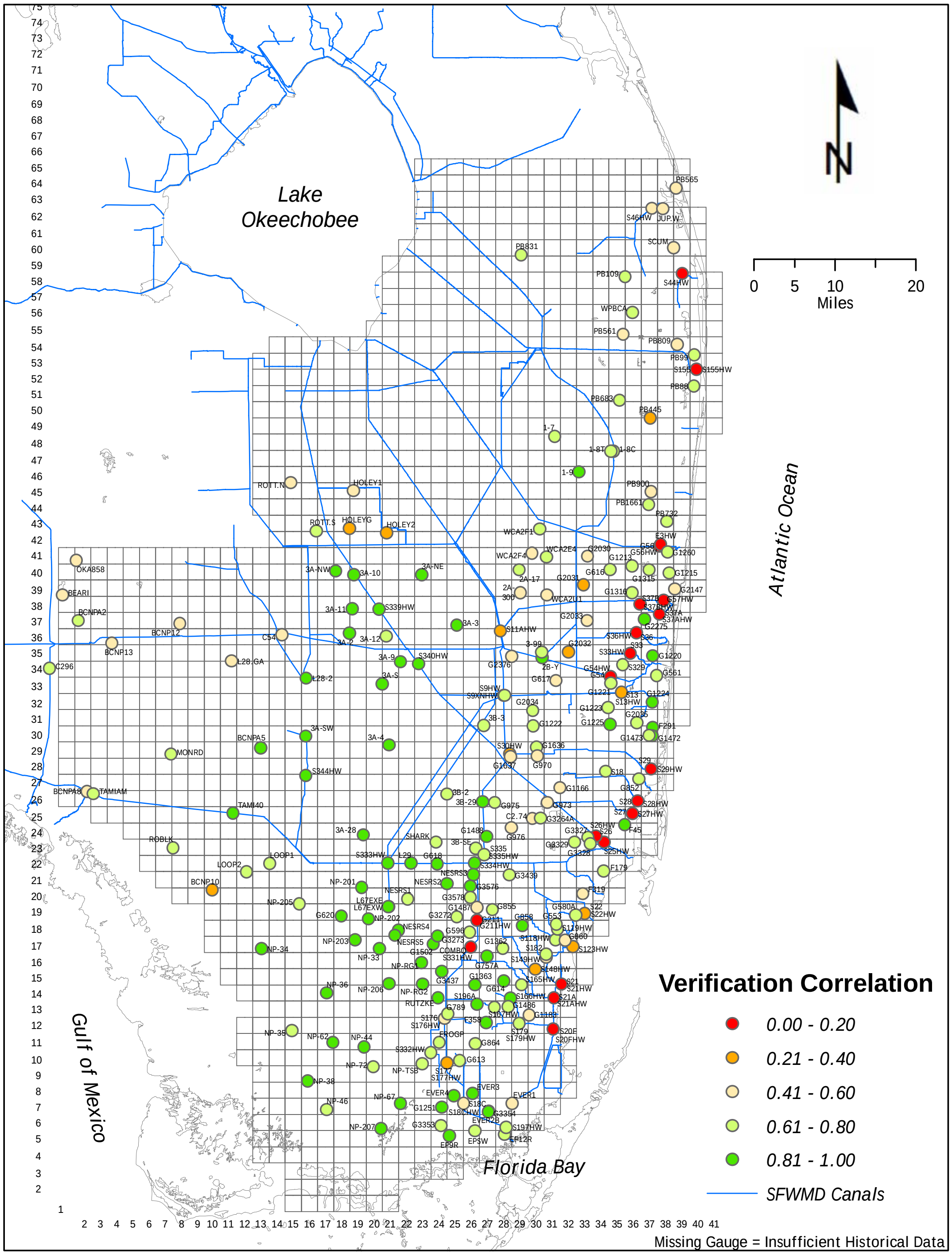

Figure 4.2.2.4 Verification Correlation

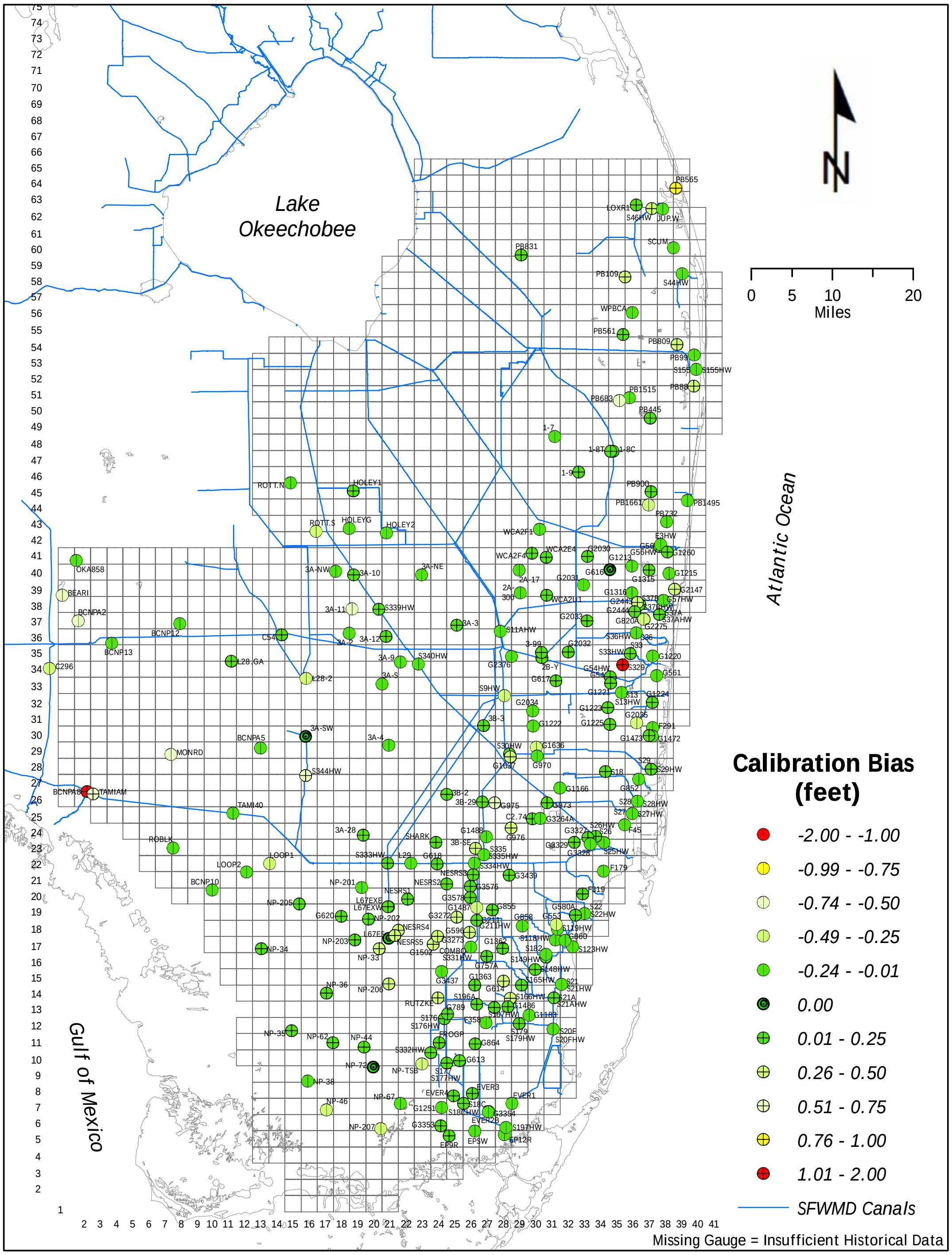

Figure 4.2.2.5 Calibration Bias

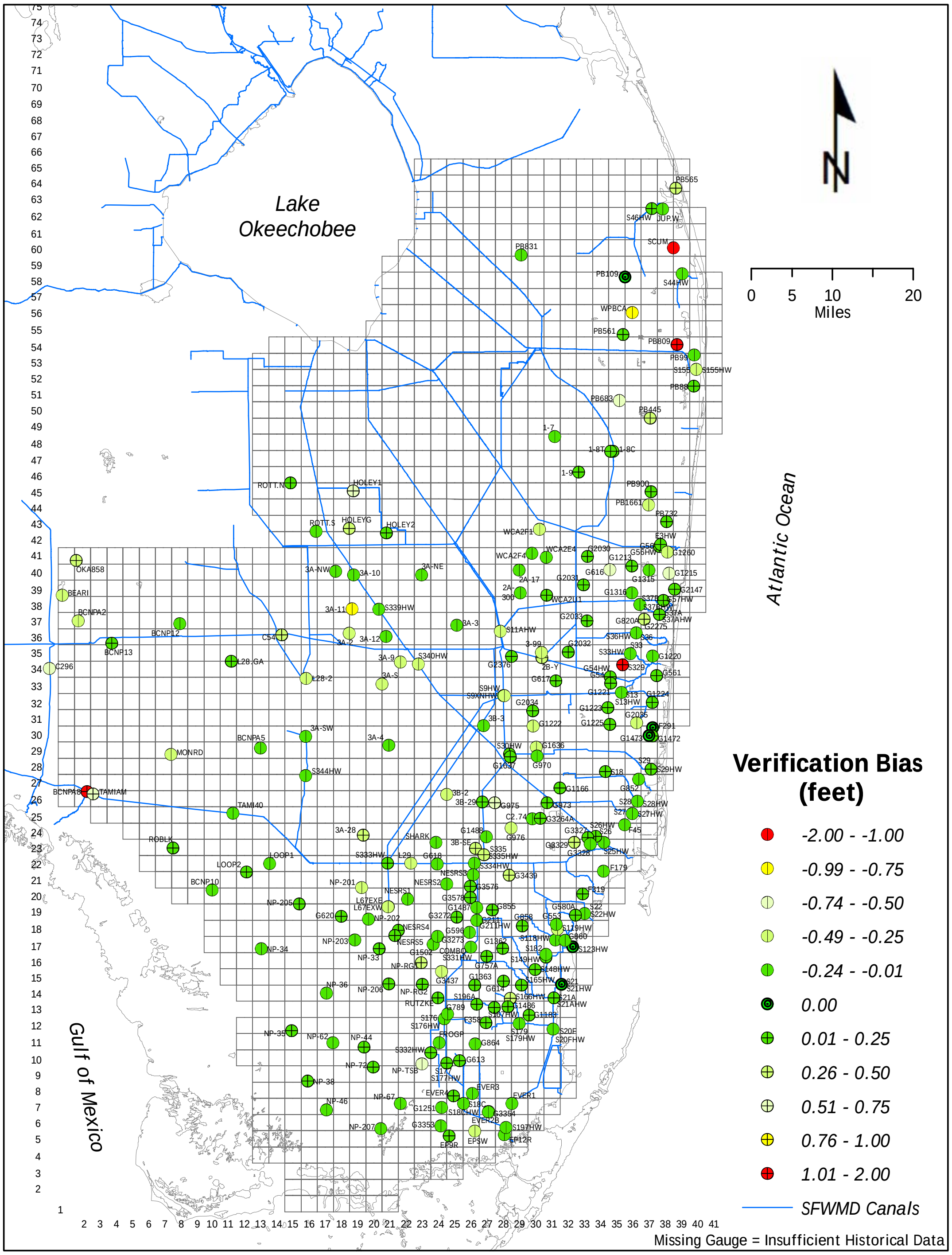

Figure 4.2.2.6 Verification Bias

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4.3 CALIBRATION OVERVIEW OF LUMPED LAKE OKEECHOBEE SERVICE AREA BASINS

The South Florida Water Management Model (SFWMM) requires demand/runoff time series input for (among others) the Caloosahatchee (C-43), St. Lucie (C-44), S4, Lower Istokpoga, and North/Northeast Lake Shore Basins. These basins are geographically close to each other, falling within Lake Okeechobee Service Area (LOSA). Additionally, they share common land use types (predominantly agriculture or natural systems) and land management practices. A review of available data for these basins indicated that the Caloosahatchee Basin has the most reliable and up-to-date flow information. The decision was therefore made to calibrate an implementation of the AFSISRS/WATBAL model for the Caloosahatchee Basin (as conceptually outlined in Section 3.2) for the period 1991-2000. Parameters derived from the Caloosahatchee Basin calibration are then used in modeling the other LOSA basins for regional modeling purposes. Additional calibration efforts were also performed for geographic areas that included the Seminole Brighton and Seminole Big Cypress Reservation lands. This was due to the need to ensure that demand estimations as modeled were consistent with water rights compact entitlement volumes protected by Florida state law. The following sections will describe these independent lumped-system calibration efforts.

4.3.1 Calibration of the Caloosahatchee Basin

Data for use in the Caloosahatchee AFSIRS/WATBAL model comes from a wide variety of sources. As previously stated, a calibration period of 1991-2000 was used. Climate data was taken from available rainfall (Section 2.2) and potential evapotranspiration (PET) (Section 2.3) data sets created for the SFWMM. Historical flow data for boundary structures (S-77, S235 and S-79) were obtained from the SFWMD's DBHYDRO database. There was a substantial increase in irrigated lands within the Caloosahatchee Basin over the calibration period. AFSIRS/WATBAL modeling is able to simulate the changes in irrigation demands and runoff that result from changing land uses. For calibration, historic land use-over-time tables were developed for each irrigation basin. District land use coverages were used to establish 1988 (SFWMD 1994), 1995 and 2000 (SFWMD 2002) land use. Land use for intermediate years was interpolated based on historic countywide crop land use data published by Florida Agricultural Statistics Service (FASS).

The process for calibration of the AFSIRS/WATBAL model is iterative and consists of several steps. Parameters for calibration of the model include two global irrigation parameters, five parameters each for three types of nonirrigated lands and monthly Kc parameters for evapotranspiration estimation for each land use type. The calibration strategy is to select reasonable values for each parameter, run the model, and evaluate the results using several goodness-of-fit (GOF) measures. The GOFs were used to compare the simulated demand and runoff to the measured flows over the calibration period of 1991-2000. Model parameters were adjusted after each run for a subsequent attempt to obtain the best GOFs. An additional check is required after each iteration to ensure that in addition to appropriate basin-scale results, the individual land use performances were also realistic (e.g. no crop had 70 inches of ET demand, rangeland did not flood to 5 feet, etc.).

The final results of the iterative process yielded calibrated parameters as shown in Tables 4.3.1.1, 4.3.1.2 and 4.3.1.3. Calibration summaries and GOF analysis of agricultural demands are presented in Table 4.3.1.4 and Figures 4.3.1.1 to 4.3.1.2. Results of calibration and GOF analysis of watershed runoff are presented in Table 4.3.1.5 and Figures 4.3.1.4 to 4.3.1.6. Table 4.3.1.6 relates the individual water budget summaries for each of the calibrated land use types for a representative sub-basin (ECAL-D).

In general, the results of the calibration are extremely good, especially considering the amount of uncertainty associated with climate, flow, and land use data estimation. Correlations of modeled to measured data are high for both demand and runoff estimation. In addition, the model calibration shows very little bias and is able to reproduce the seasonal variability observed in the measured data. Additionally, the performances of the individual land use types, as presented in Table 4.3.1.6, are within the expected ranges of behavior. Additional, more specific, comments related to the calibration results are presented in bullet form below.

- The value for EFF1of less than 100% in Table 4.3.1.1 indicates that there exists water use within the basin not directly related to crop irrigation requirements. This extra demand (resulting from transmission losses, incidental irrigation, etc.) ends up in the atmosphere but the processes are not modeled.
- The local storage term (STOR1) presented in Table 4.3.1.1 is approximately 0.10 inches which represents a small (approximately 6 inch) water table variation.
- Kc values as derived in Table 4.3.1.3 are intended to be used in conjunction with wet marsh PET estimations by the simplified temperature-based method as used in the SFWMM (Irizarry, 2003b). These Kc values were capped at a maximum value of 1.10 for open water as is consistent with the assumption in the SFWMM.
- AFSIRS/WATBAL is a hydrologic, not a hydraulic model and should not be used to estimate peak runoff rates. However, it can predict total storm runoff. GOF measures for runoff are calculated on five-day moving average daily values.
- Since evaluation of demand estimates is tied to regulatory (1-in-10 months) or more long term time steps, GOF measures for demand are presented on a monthly basis.
- The cumulative demand and runoff traces in Figures 4.3.1.3 and 4.3.1.6 indicate that modeled demand and runoff follow the same pattern as measured data over the period of record. While the model tends to slightly under-predict demand in earlier years and then over-predict in later years, this is most likely due to inaccurate growth estimate in the land use data.

Based on the results and the success of the Caloosahatchee Basin calibration exercise, it is appropriate to apply the AFSIRS/WATBAL V3.0 model with the Caloosahatchee Basin calibrated parameters to all LOSA basins in regional modeling efforts.

Table 4.3.1.1 Caloosahatchee Calibrated Values for AFSIRS Water Budget Model Parameters

Irrigation efficiency1 (consumptive use by plant /amount lost to air) [EFF1]	87%
Local Storage Depth (inches) [STOR1]	0.1
Drainage capacity (inches/day) [CAP1]	7.0
Storage coefficient (day) [COEF1]	7

Table 4.3.1.2 Caloosahatchee Calibrated Values for WATBAL Model Parameters

	Rangeland	Upland Forest	Wetlands
Plant available water (PAW) capacity [inches]	0.8	1.6	2.2
Drainable storage capacity (CAP1) [inches]	7.0	7.0	1.0
Storage coefficient (COEF1) [days]	7	7	8
Total groundwater storage [inches]	7.0	7.0	5.0
Root zone depth [inches]	11.4	22.9	5.5

Table 4.3.1.3 Caloosahatchee Calibrated Values for Monthly Potential Evapotranspiration Correction Factors (Kc)

Month	citrus	Cane	Veg	pasture	up forest	wetlands
1	0.71	0.61	0.28	0.54	0.58	0.67
2	0.66	0.57	0.25	0.55	0.59	0.63
3	0.61	0.51	0.87	0.55	0.59	0.57
4	0.64	0.59	0.58	0.75	0.68	0.65
5	0.87	0.88	0.87	0.89	0.89	0.93
6	0.98	0.98	0.96	0.99	1.04	1.04
7	1.02	1.07	1.00	1.03	1.08	1.10
8	0.83	0.90	0.89	0.88	0.93	0.96
9	0.93	1.00	0.29	0.91	0.96	1.06
10	0.99	1.00	0.32	0.83	0.82	1.06
11	0.84	0.80	0.99	0.60	0.70	0.85
12	0.82	0.72	0.63	0.53	0.57	0.77

Table 4.3.1.4 Caloosahatchee Measures of Goodness of Fit for Calibration of AFSIRS Water Budget Model

Average Annual Demand	
Demand – Modeled	86,407 ac-ft/yr
Demand – Measured	84,367 ac-ft/yr
Goodness of Fit	
Model - Measured Error	2,040 ac-ft/yr
Demand (Model) - Demand (Measured) / Demand (Model)	2.36%
Slope of Modeled - Measured Demand	0.962
Regression Coefficient of Modeled - Measured Demand	0.813
Pearson Correlation Coefficient	0.902
Modeled Bias	-170 ac-ft
Root Mean Squared Error	4,007 ac-ft

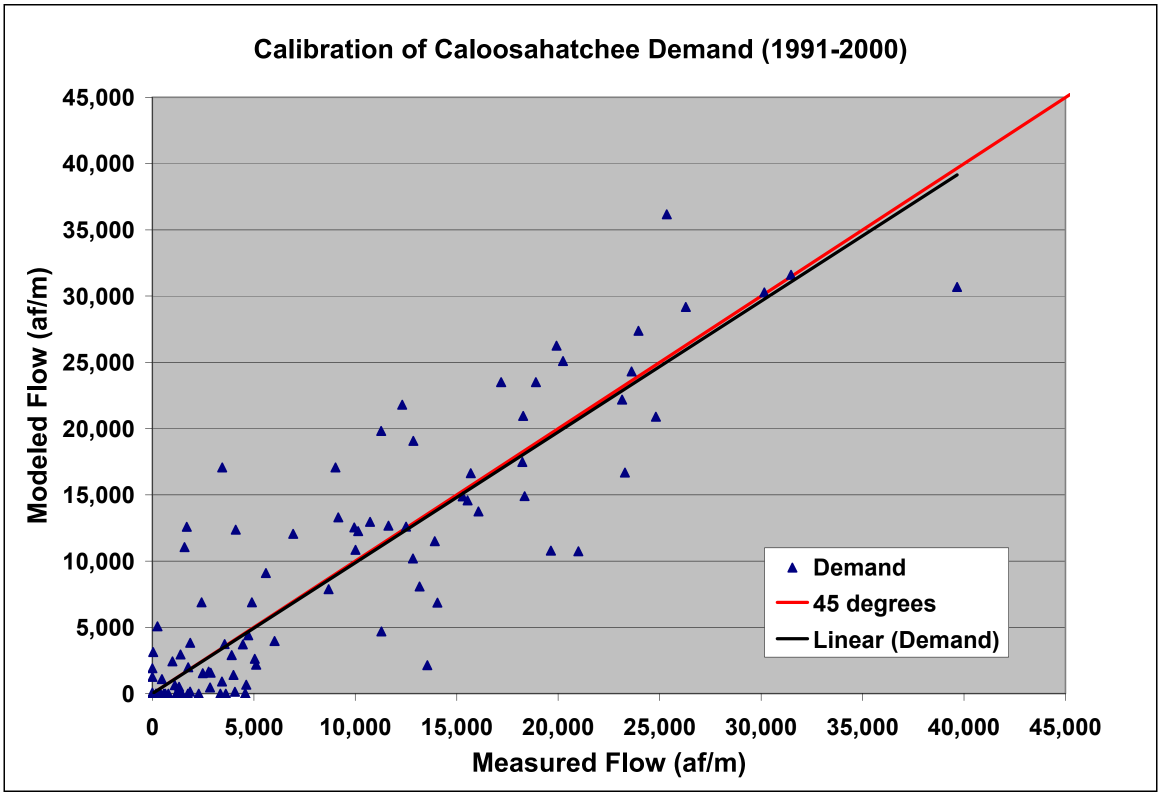

Figure 4.3.1.1 Measured vs. Modeled Caloosahatchee Demand

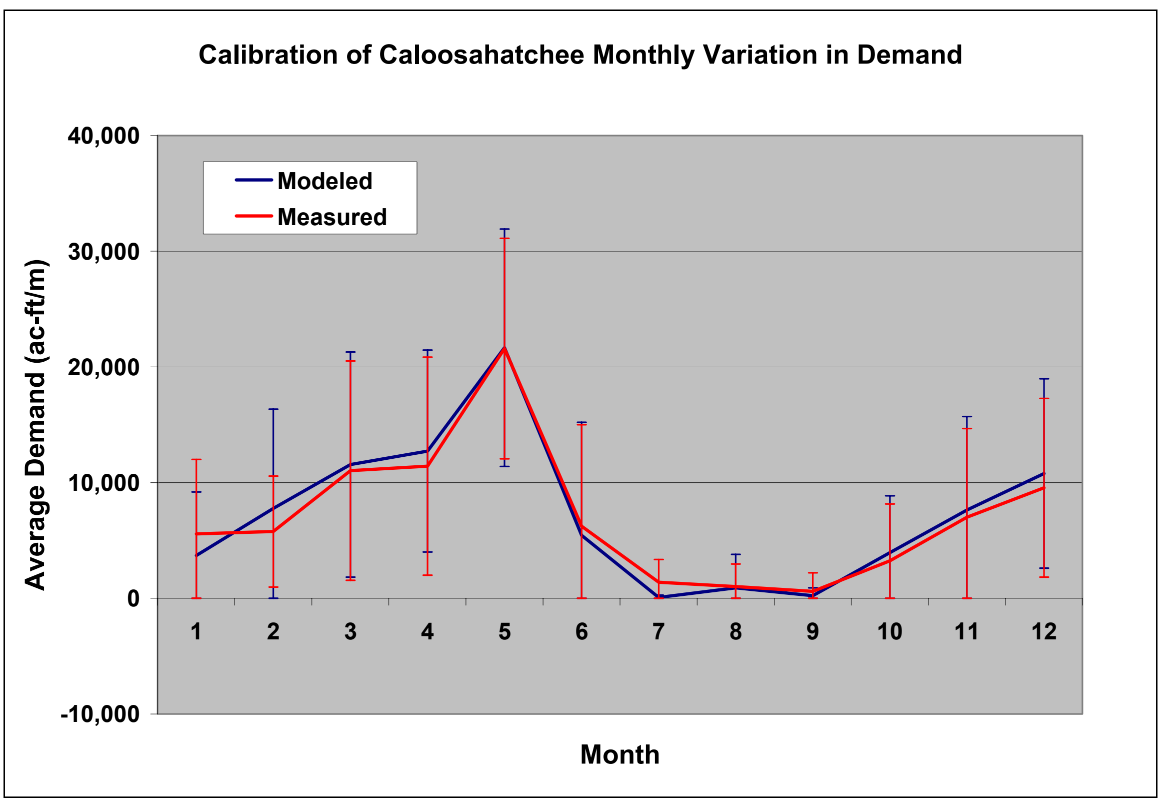

Figure 4.3.1.2 Seasonal Variability in Caloosahatchee Demand

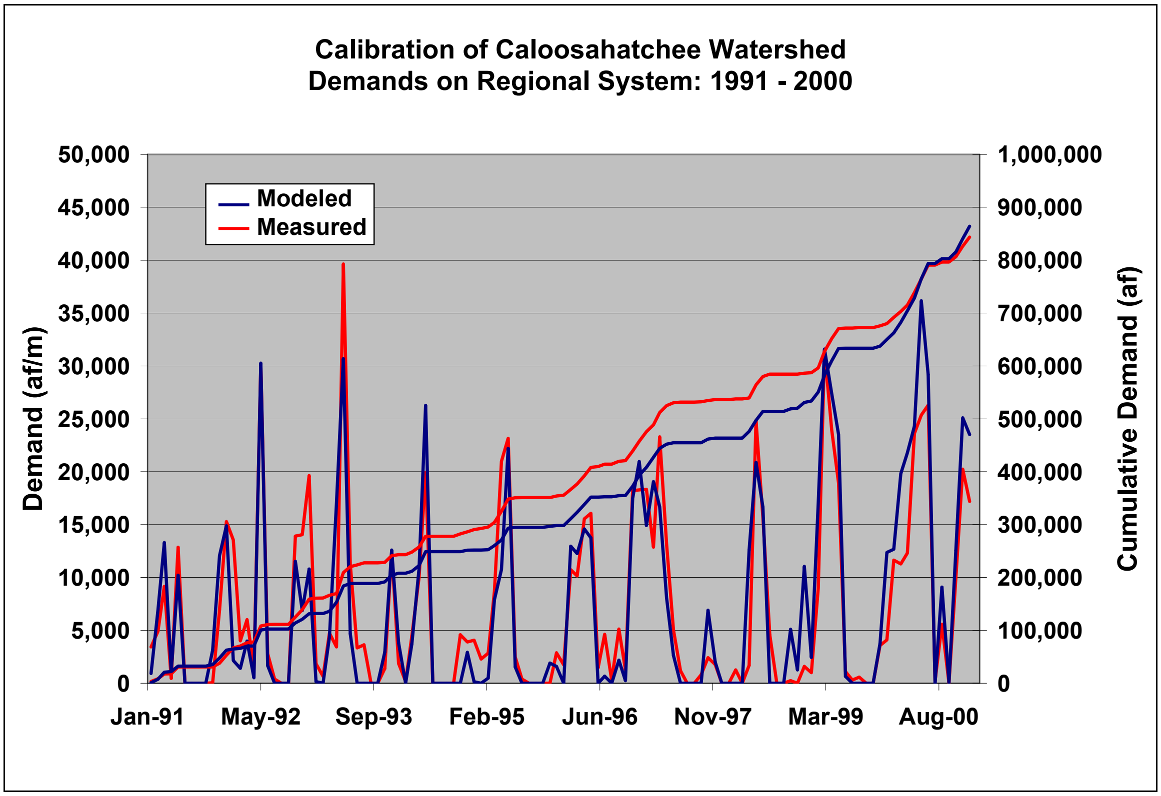

Figure 4.3.1.3 Time Series of Monthly Caloosahatchee Demand and Accumulation

Table 4.3.1.5 Caloosahatchee Measures of Goodness of Fit for Calibration of WATBAL Model

Average Annual Runoff	
Runoff – Modeled	803,863 ac-ft/yr
Runoff – Measured	799,598 ac-ft/yr
Goodness of Fit	
Model - Measured Error	4,265 ac-ft/yr
Runoff (Model) - Runoff (Measured) / Runoff (Model)	0.53%
Slope of Modeled - Measured Runoff	0.973
Regression Coefficient of Modeled - Measured Runoff	0.825
Pearson Correlation Coefficient	0.908
Modeled Bias	12 ac-ft
Root Mean Squared Error	1,477 ac-ft

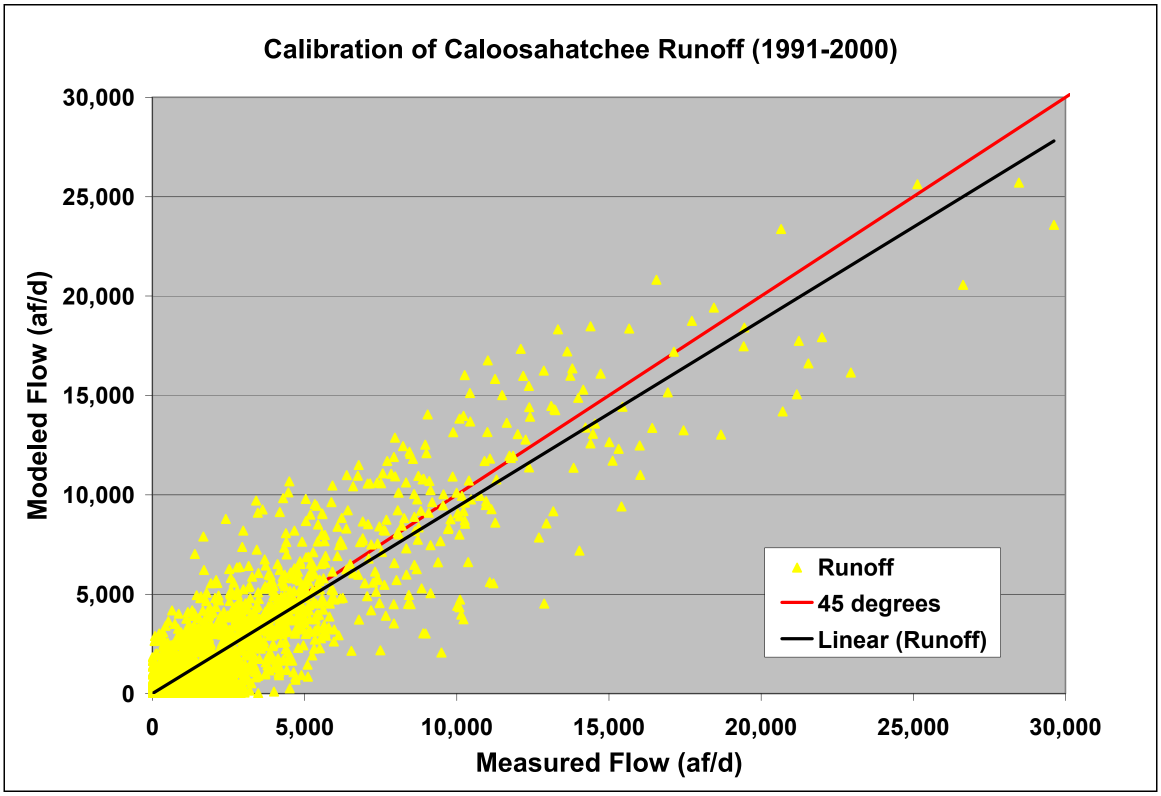

Figure 4.3.1.4 Measured vs. Modeled Caloosahatchee Runoff

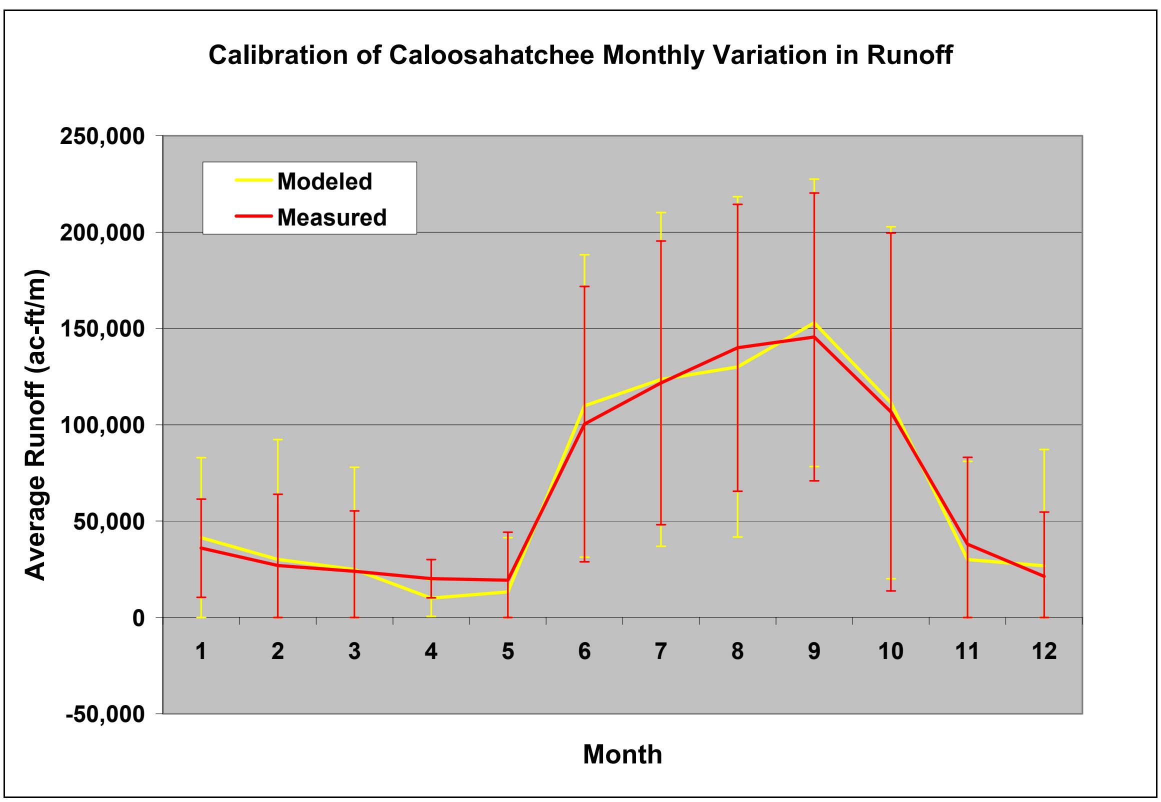

Figure 4.3.1.5 Seasonal Variability in Caloosahatchee Runoff

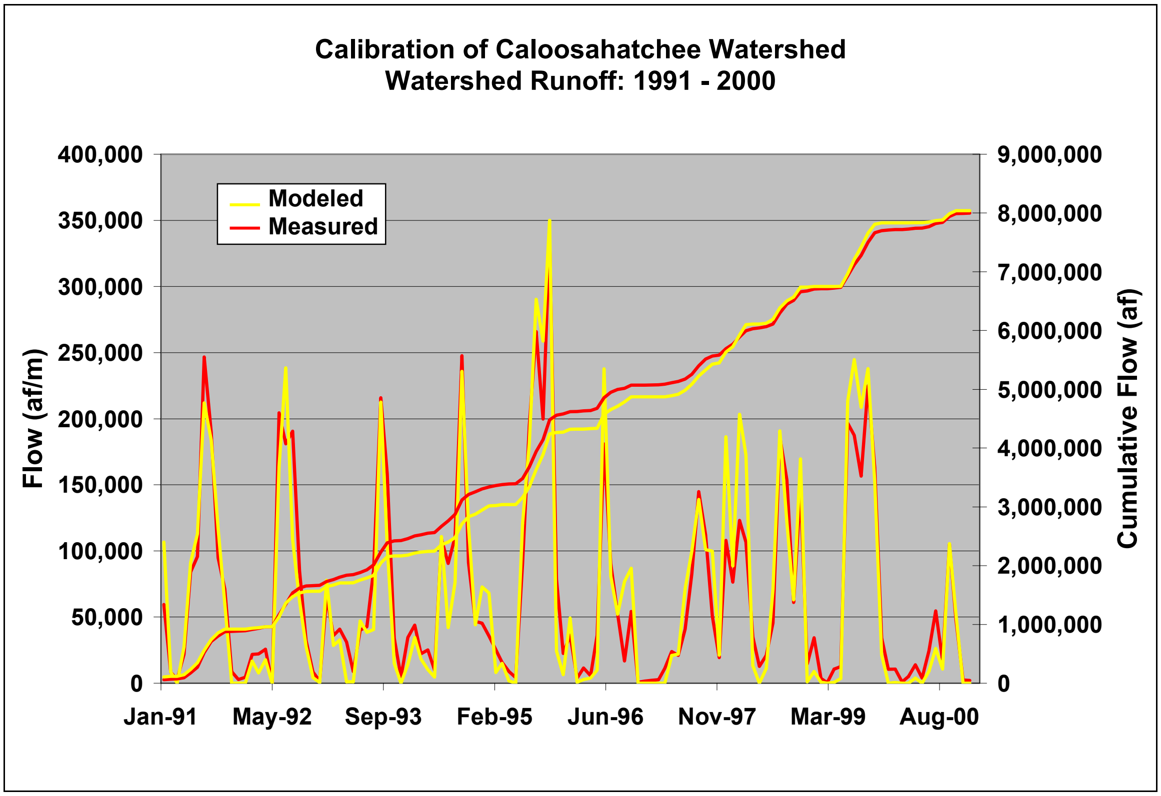

Figure 4.3.1.6 Time Series of Monthly Caloosahatchee Runoff and Accumulation

Table 4.3.1.6 Caloosahatchee Water Budget Summaries for Calibrated Land Use Types (ECAL-D sub-basin)

	Land Use						
	Citrus - crownflood irrigated	**Citrus - microjet irrigated**	**Sugar cane-subseepage irrigated**	**Tomatoes - microspray irrigated**	**Range-land**	**Upland Forest**	**Wetland**
Rain [in/yr]	50.6	50.6	50.6	50.6	50.6	50.6	50.6
Actual Evapo-transpiration [in/yr]	48.8	48.7	47.9	45.2	34.2	37.5	40.6
AFSIRS Irrigation [in/yr]	15.9	6.8	16.9	10.4	-	-	-
AFSIRS Runoff [in/yr]	17.7	8.7	19.6	15.8	-	-	-
Drainage and Recharge [in/yr]	-	-	-	-	16.4	13.1	10.0
Maximum Flooding Depth [in]	-	-	-	-	0.0	2.4	9.9

4.3.2 Calibration of the Brighton Seminole Reservation and Lower Istokpoga Basin

The Brighton/Istokpoga calibration implementation of the AFSIRS/WATBAL model is conceptualized as a single basin model (as conceptually outlined in Section 3.2) covering the lands between S-70/S-75 and S-71/S-72 that influence the regional system. This area includes the Seminole Brighton Reservation as well as additional irrigated and non-irrigated lands. In general, reliable flow and land use data in the defined basin is limited. While flow data exists for the last several decades, it contains large periods of missing data and a water budget analysis created by utilizing these flows shows several months of unrealistically high or low demand conditions. Land use data for the basin is also in short supply, especially before the 1995 FLUCCS land use coverage. Due to these data limitations, a calibration period of 1995-2000 was selected. While this is a relatively short period of simulation, it should prove sufficient for parameter estimation, especially since the model will be applied with land use assumptions consistent with circa-2000 conditions. Once the calibration period was selected, historical flow data for boundary structures (S-70, S-71, S-72, S-75, G207 and G208) was obtained from the SFWMD's DBHYDRO database. Additionally, a historic land use table was developed based on a combination of District land use coverage for 1995 and 2000 permitted agricultural land use as used in Supply Side Management implementation (SFWMD, 2002).

Once data had been collected, an iterative calibration process was attempted in a manner similar to that undertaken for the Caloosahatchee Basin AFSIRS/WATBAL model (Wilcox 2003a, presented in Section 4.3.1). The goal of the Brighton/Istokpoga calibration was to be able to match a measured demand condition as closely as possible. Due to this consideration and also taking into account the uncertainty in measured data for the Brighton/Istokpoga model, many of the Caloosahatchee Basin calibrated model parameters were incorporated without modification. In fact, only two of the AFSIRS/WATBAL model parameters were modified during calibration. These demand related calibration terms were the irrigation efficiency [EFF1] and the Local Storage Depth [STOR1]. The final results of the iterative process yielded calibrated parameters as shown in Tables 4.3.2.1 and 4.3.2.2 (with rangeland Kc factors from Caloosahatchee Basin being applied to pasture/sod in Brighton/Istokpoga). Calibration summaries and Goodness of Fit (GOF) analysis of agricultural demands are presented in Table 4.3.2.3 and Figures 4.3.2.1 to 4.3.2.3.

Table 4.3.2.1 Brighton/Istokpoga Calibrated Values for AFSIRS Water Budget Model Parameters

Irrigation efficiency1 (consumptive use by plant / amount lost to air) (EFF1)	60%
Local Storage Depth (STOR1) [inches]	0.2
Drainage capacity (CAP1) [inches/day]	7.0
Storage coefficient (COEF1) [day]	7

Table 4.3.2.2 Brighton/Istokpoga Values for Monthly Potential Evapotranspiration Correction Factors (Kc) as Calibrated in Caloosahatchee Basin

Month	citrus	sugarcane	pasture/sod
1	0.71	0.61	0.54
2	0.66	0.57	0.55
3	0.61	0.51	0.55
4	0.64	0.59	0.75
5	0.87	0.88	0.89
6	0.98	0.98	0.99
7	1.02	1.07	1.03
8	0.83	0.90	0.88
9	0.93	1.00	0.91
10	0.99	1.00	0.83
11	0.84	0.80	0.60
12	0.82	0.72	0.53

Table 4.3.2.3 Brighton/Istokpoga Measures of Goodness of Fit for Calibration of AFSIRS Water Budget Model

Average Annual Demand	
Demand – Modeled	49,723 ac-ft/yr
Demand – Measured	49,514 ac-ft/yr
Goodness of Fit	
Model - Measured Error	209 ac-ft/yr
Demand (Model) - Demand (Measured) / Demand (Model)	2.36%
Slope of Modeled - Measured Demand	0.933
Regression Coefficient of Modeled - Measured Demand	0.507
Pearson Correlation Coefficient	0.712
Modeled Bias	-17 ac-ft
Root Mean Squared Error	3,032 ac-ft

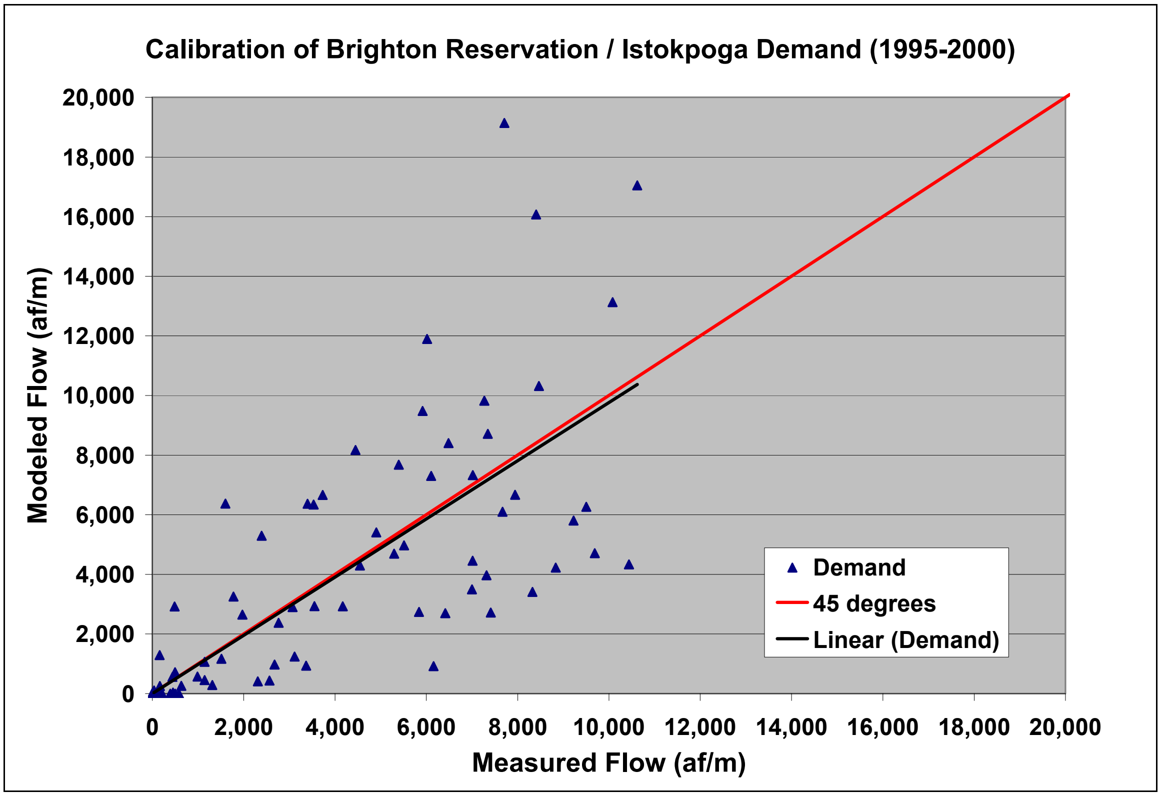

Figure 4.3.2.1 Measured vs. Modeled Brighton/Istokpoga Demand

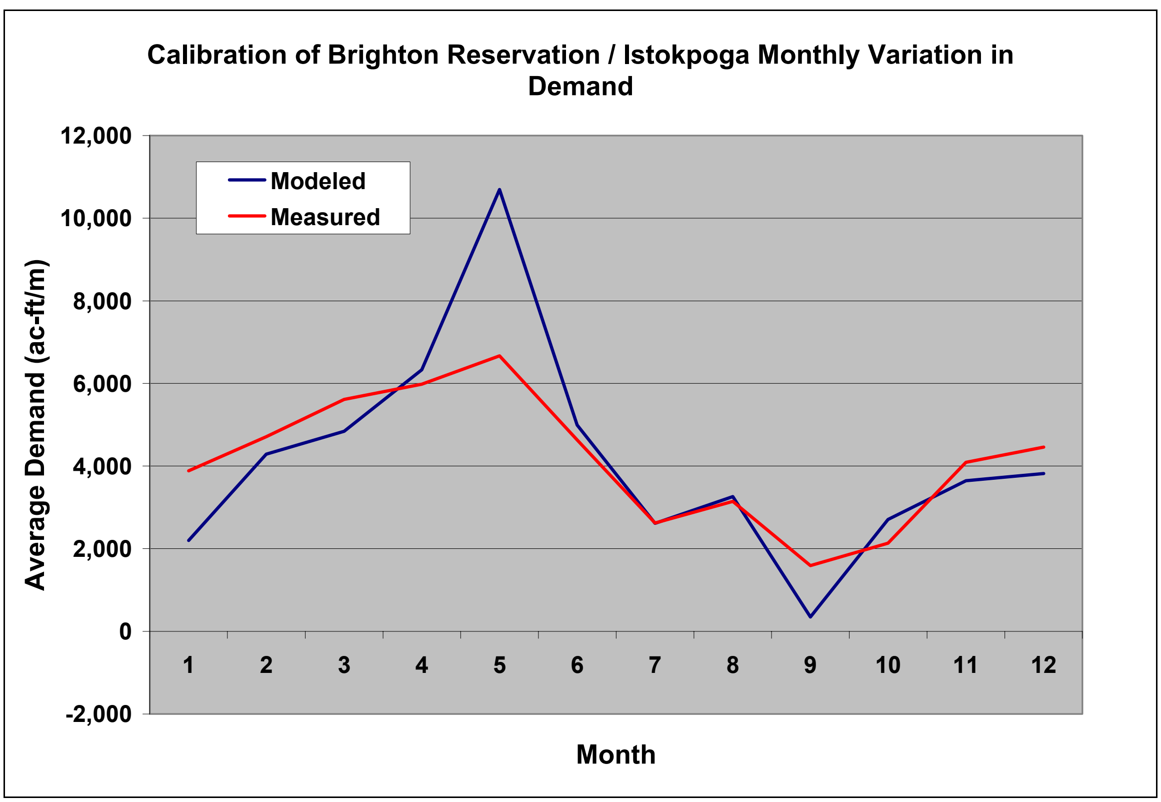

Figure 4.3.2.2 Seasonal Variability in Brighton/Istokpoga Demand

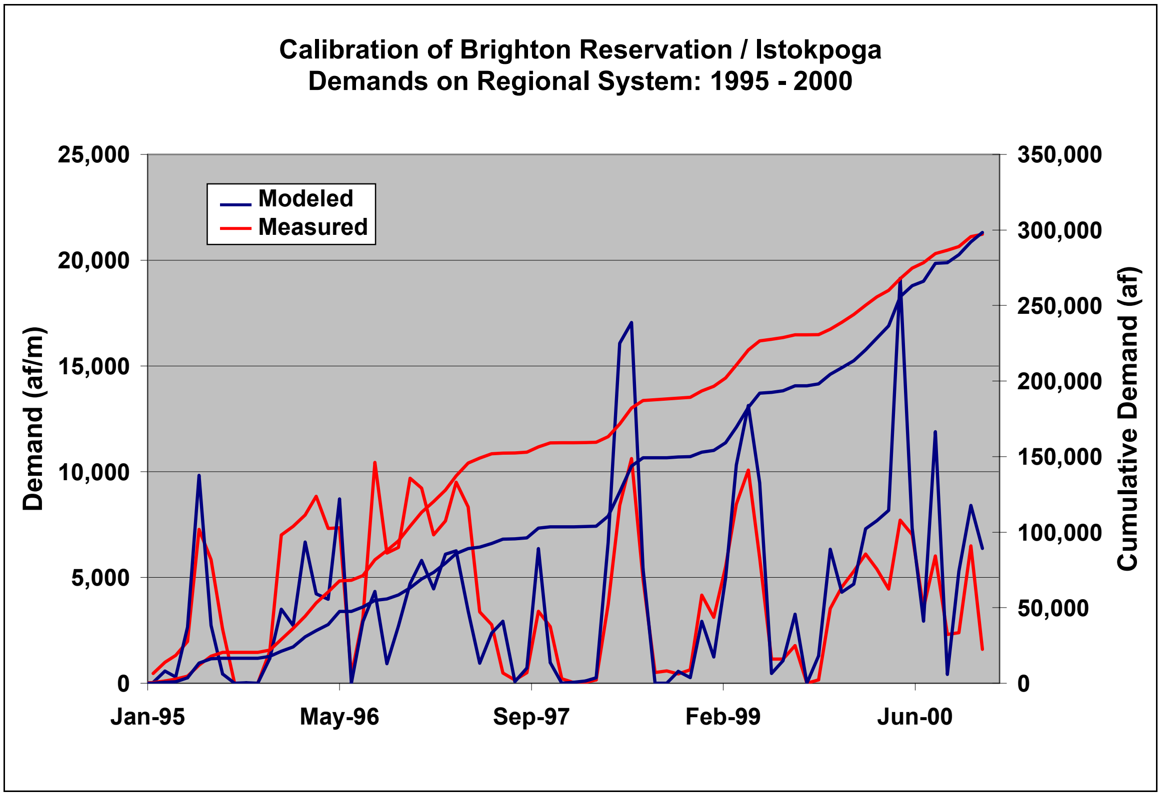

Figure 4.3.2.3 Time Series of Monthly Brighton/Istokpoga Demand and Accumulation

In general, results of this calibration exercise are acceptable, although not as good as those observed in the Caloosahatchee Basin for AFSIRS/WATBAL V3.0. The main strength of the calibrated model is its ability to predict the timing of when periods of demand occur. Calibration of the Brighton/Istokpoga Basin to both timing and magnitude of demand was significantly more difficult than for the C-43 due to the previously outlined data issues in conjunction with the relatively small magnitude of demand in the basin. Additional, more specific, comments related to the calibration results are presented in bullet form below.

- The calibrated EFF1 term was lowered to 60% (from 87% in the Caloosahatchee Basin) indicating an increase in un-captured loss terms. However, this term still falls well within the range of reasonability and is on the same order as previous modeling exercises for the Caloosahatchee Basin with AFSIRS/WATBAL V2.0 for the CWMP plan (58%).
- The change in STOR1 from 0.1 inches to 0.2 inches represents increased uncertainty in water table fluctuation.
- The correlation of measured to modeled demand is good overall, with the exception of a few outlier points - May 2000 in particular. In this month, the modeled demand is over double the magnitude of the measured basin demand. This inconsistency is clearly evident in both the scatter plot (Figure 4.3.2.1) and the seasonal variability (Figure 4.3.2.2), which shows a marked bias in "overestimation" of May demand. It is strange that measured demand is not higher given that May 2000 was one of the driest months in history and this observation may point to problems with the measured data.
- The model tends to slightly under-predict demand in earlier years and then over-predict in later years - this is most likely due to inaccurate estimate in the land use data which

was assumed to be constant during the calibration period due to the lack of reliable data related to land use growth.

Based on the results of the Brighton/Istokpoga calibration exercise, it seems appropriate to apply the AFSIRS/WATBAL V3.0 model in regional modeling efforts associated with demand estimation for the Brighton Reservation.

4.3.3 Calibration of the Big Cypress Seminole Reservation and Feeder Canal Basin

Due to the interrelationship between the Big Cypress Seminole Reservation and the Feeder Canal Basin (as described in Section 2.6.3), two separate AFSIRS-WATBAL models were implemented during a joined calibration effort: 1) a model of the Big Cypress Reservation (BCR) lands, and 2) a model of the Feeder Canal Basin. Consistent parameters between the two areas were derived during calibration and were applied in both model implementations.

Big Cypress Seminole Reservation Basin-Scale Demands

The AFSIRS-WATBAL model was used to estimate basin-scale net irrigation demands for the Big Cypress Reservation. The AFSIRS portion of the model was used to estimate field-scale irrigation requirements for four major land uses (Table 4.3.3.3) as specified in the Work Plan Authorization. The WATBAL portion of the model transforms the field-scale net irrigation demands into basin-scale demands by accounting for local basin storage and basin efficiency, which includes losses to air and water conveyance losses. Non-irrigated lands in the BCR were not incorporated into the model and so did not contribute toward meeting needs in the irrigated lands.

Due to the lack of historical water use data, the modeled BCR demand had to be compared with the permitted demands from the Work Plan Authorization. Using an iterative process, the 4 basin parameters shown in Table 4.3.3.1 were modified until the 2-in-10 monthly demand matched the permitted demands (Table 4.3.3.2). Of notable interest, the efficiency term had to be lowered to 50% to be able to match the permitted demands. In addition, land use-specific performance was checked for reasonableness (Table 4.3.3.3).

Table 4.3.3.1 Big Cypress Reservation Calibrated Values for AFSIRS Water Budget Model Parameters

Irrigation efficiency1 (consumptive use by plant / amount lost to air) (EFF1)	50%
Local Storage Depth (STOR1) [inches]	0.05
Drainage capacity (CAP1) [inches/day]	7.0
Storage coefficient (COEF1) [day]	6

Table 4.3.3.2 Big Cypress Reservation Comparison of Modeled Demands to Work Plan Entitlement for the period 1965-2000

Average Annual Demand – Modeled	28,509 ac-ft/yr
Modeled 1-in-5 monthly demand	8,157 ac-ft/mo (2,659 mgm)
1-in-5 monthly demand from Work Plan Authorization	7,994 ac-ft/mo (2,606 mgm)

Table 4.3.3.3 Big Cypress Reservation Water Budget Summaries for Calibrated Land Use Types (1991-2000 calibration period)

	Land Use			
	Citrus - crownflood irrigated	**Citrus - microjet irrigated**	**Tomatoes – microspray irrigated**	**Irrigated Pasture**
Acreage (acres)	1,730	494	1,151	10,441
Rain (in/yr)	53.6	53.6	53.6	53.6
Actual Evapo-transpiration (in/yr)	51.9	52.1	44.3	46.8
AFSIRS Irrigation (in/yr)	21.1	6.5	10.2	12.8
AFSIRS Runoff (in/yr)	22.9	8.0	19.5	19.6

Feeder Canal Basin Runoff

The AFSIRS-WATBAL model for the Feeder Canal Basin was calibrated to monthly runoff totals as measured at S-190. To get rid of the effect of structure operations in the measured discharges at S-190, it was necessary to calibrate to monthly runoff totals. The calibration period was selected as 1991-2000, however only a single land use snapshot (circa 2000) was used in the calibration. As shown in Table 4.3.3.6, both irrigated and non-irrigated lands were included in the model. The global irrigation parameters shown in Table 4.3.3.1 were used in the Feeder Canal Basin model. Using an iterative procedure, five parameters were calibrated for each of the three non-irrigated land uses (Table 4.3.3.4).

Table 4.3.3.5 and Figures 4.3.3.1 and 4.3.3.2 show the model performance for the 1991-2000 calibration period. From these figures, it can be observed that the model captures the monthly and interannual variability in runoff reasonably well. The correlation of measured to modeled monthly runoff is also reasonably good ($R = 0.82$). However, it is evident that the model underestimates runoff by approximately 6%. One particular event (December 1994) is responsible for about two-thirds of the cumulative runoff error over the calibration period. The year 1994 was an unusually wet year with higher than normal rainfall occurring during the typically dry months of November and December. Hurricane Gordon dropped more than 5 inches of rainfall over the area in November 1994. November of 1994 had a total of 6.2 inches of rainfall versus averages of 2.3 and 2.8 inches observed for the 1965-2000 and 1991-2000 periods, respectively. Average rainfall for December of 1994 was 9.9 inches compared to averages of 1.7 and 2.1 inches for the 1965-2000 and 1991-2000 periods, respectively. Measured runoff for December of 1994 was 67,722 ac-ft/mo while the model simulated 32,940 ac-ft/mo of runoff. This unusually wet event was identified from the beginning of the calibration; however, additional efforts to reduce the gap between modeled and observed runoff for this event were unsuccessful.

Table 4.3.3.6 summarizes the land use-specific performance, which was also checked for reasonableness. Figure 4.3.3.3 shows the seasonal variation in modeled demand for the Feeder Canal Basin. Due to lack of historical data, the modeled demand time series could not be verified. However, the time series of demand for the Feeder Canal Basin is not used by the SFWMM in any form.

Table 4.3.3.4 Feeder Canal Basin Calibrated Values for WATBAL Model Parameters

	Rangeland	Upland Forest	Wetlands
Plant available water (PAW) capacity [inches]	0.3	1.0	2.0
Drainable storage capacity (CAP1) [inches]	7.0	7.0	1.0
Storage coefficient (COEF1) [days]	6	8	7
Total groundwater storage [inches]	7.0	7.0	5.0
Root zone depth [inches]	4.3	14.3	5.0

Table 4.3.3.5 Feeder Canal Basin Measures of Goodness of Fit for Calibration of WATBAL Model (1991-2000 period)

Average Annual Runoff	
Runoff – Modeled	84,863 ac-ft/yr
Runoff – Measured	90,113 ac-ft/yr
Goodness of Fit	
Model - Measured Error	-5,250 ac-ft/yr
Runoff (Model) - Runoff (Measured) / Runoff (Model)	-5.83%
Slope of Modeled - Measured Runoff	0.68
Regression Coefficient of Modeled - Measured Runoff	0.68
Pearson Correlation Coefficient	0.82
Modeled Bias	-437.6 ac-ft/mo
Root Mean Squared Error	5,900 ac-ft/mo

Table 4.3.3.6 Feeder Canal Basin Water Budget Summaries for Calibrated Land Use Types (1991-2000 calibration period)

	Land Use					
	Citrus - crownflood irrigated	**Citrus - microjet irrigated**	**Tomatoes – microspray irrigated**	**Range-land**	**Upland Forest**	**Wetland**
Acreage (acres)	1,608	3,752	9,000	24,419	7,468	18,107
Rain (in/yr)	53.6	53.6	53.6	53.6	53.6	53.6
Actual Evapo-transpiration (in/yr)	51.9	52.1	44.3	32.4	37.1	41.4
AFSIRS Irrigation (in/yr)	21.1	6.5	10.2	-	-	-
AFSIRS Runoff (in/yr)	22.9	8.0	19.5	-	-	-
Drainage and Recharge (in/yr)	-	-	-	21.3	16.5	12.2
Maximum Flooding Depth (in)	-	-	-	0.0	3.1	3.8

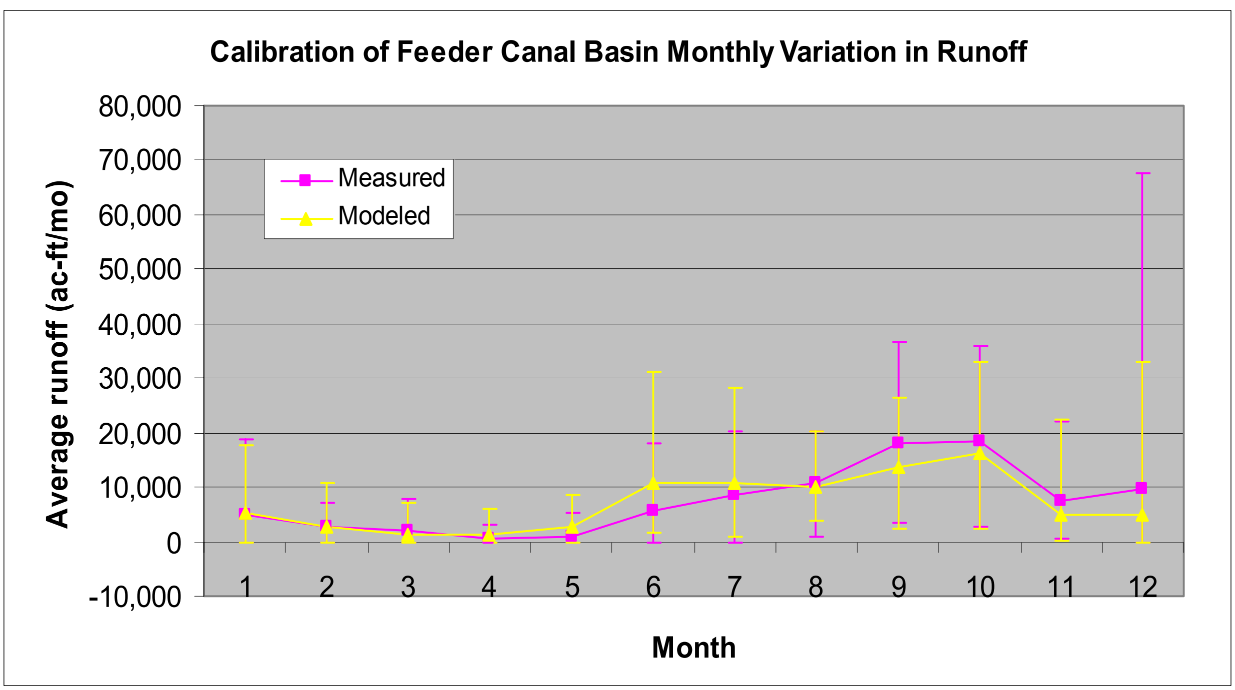

Figure 4.3.3.1 Seasonal Variability in Feeder Canal Basin Runoff

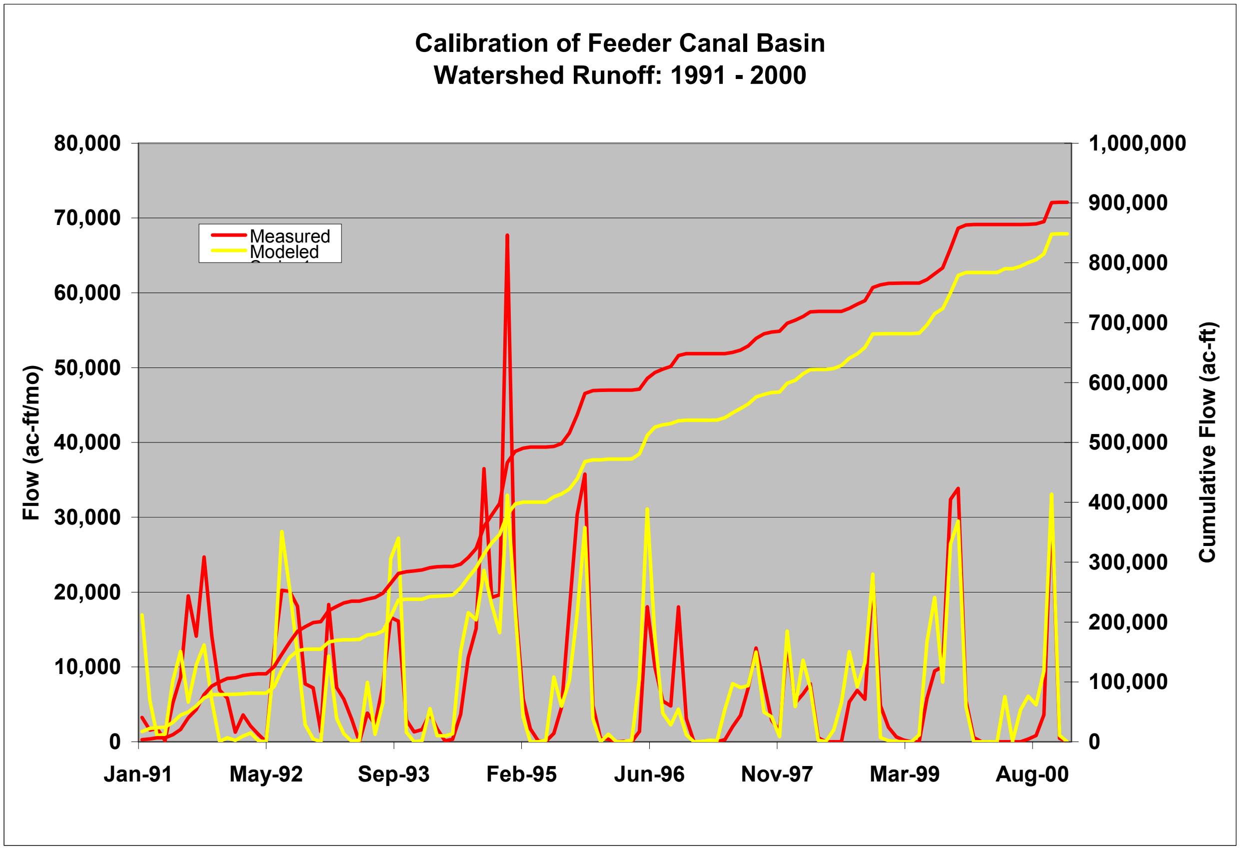

Figure 4.3.3.2 Time Series of Monthly Feeder Canal Basin Runoff and Accumulation

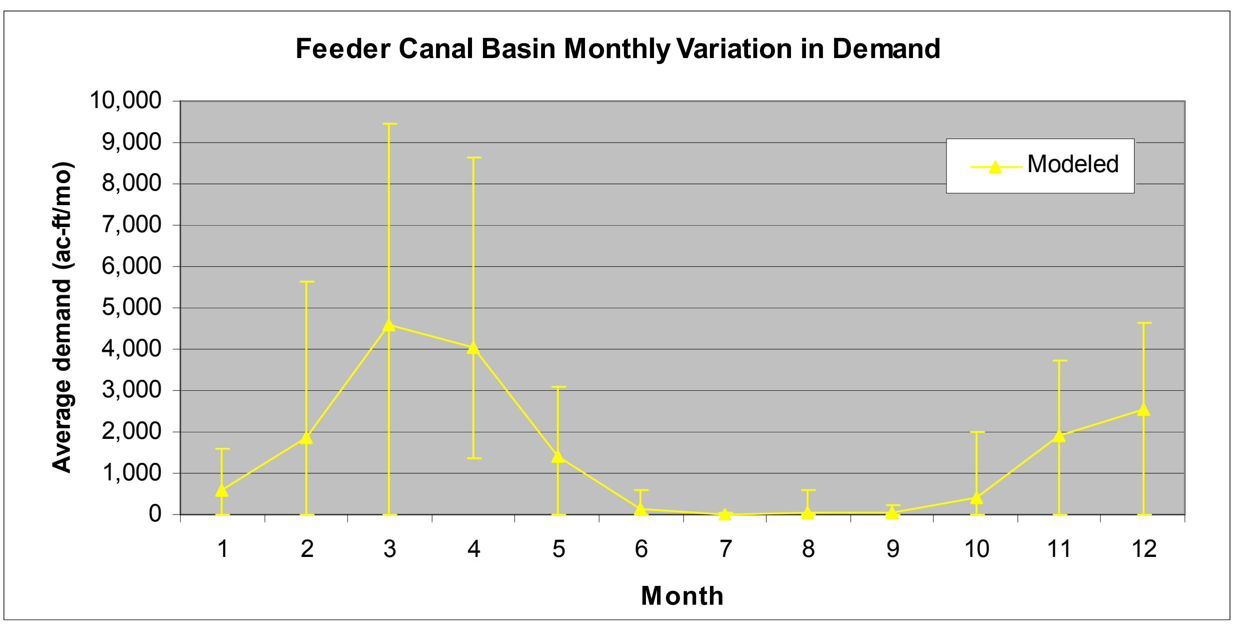

Figure 4.3.3.3 Seasonal Variability in Feeder Canal Basin Demand

5 SENSITIVITY ANALYSIS

> *Note to Readers:*
>
> *This chapter describes the results of the work conducted by District staff to evaluate the sensitivity of stage to changes in model parameters for the SFWMM v5.5. The 2005 Peer Review Panel made significant recommendations regarding improvement of both sensitivity and uncertainty analyses. The District intends to continue its efforts to improve the analyses. This documentation will be updated accordingly. However, at this time, the documentation presented hereafter does not fully incorporate all the recommendations from the Peer Review Panel.*

Sensitivity analysis is the process of varying model input parameters and evaluating how model output changes with such variations. The significance of model sensitivity analysis is two-fold:

1. It provides information on the behavior of model output to input parameters which, in turn, can be used in model calibration.
2. It gives insight in establishing priorities related to future data collection efforts.

The sensitivity analysis is distinguished from that of the uncertainty analysis. The sensitivity analysis is a measure of the relative importance that each input parameter has on the range of simulated outputs. Whereas, an uncertainty analysis quantifies the confidence one can have with particular output variables. While the sensitivity analysis often is limited to parameter sensitivity, the uncertainty may be generated by a number of factors including: 1) parameter uncertainty, 2) model spatial and temporal resolution, 3) availability and quality of data, and 4) model algorithm. This chapter deals with sensitivity analysis as applied to version v5.5 of the South Florida Water Management Model (SFWMM).

5.1 METHODOLOGY

The sensitivity of the output variables to variations in input parameters is estimated by the traditional approach of varying one parameter at a time. A sensitivity matrix is set up that summarizes the response at model cells where corresponding gages are located, to changes in individual parameters. Model response is expressed in terms of simulated nodal stages within the model domain. The following input parameters are systematically varied universally over the whole model domain in order to analyze model output sensitivity:

1. **Effective Roughness Coefficient for overland flow (ERC)**
   In the model, the Effective Roughness Coefficient is simulated as an exponential function of ponding depth: $N = A(POND)^b$ **.**
2. **Reference ET for Wetland (WPET)**
   The calculation of ET in the model is based on reference crop ET which is adjusted according to crop type, available soil moisture content, and location of the water table. In non-irrigated areas such as the Water Conservation Areas (WCAs), Everglades National Park (ENP) and part of the Big Cypress National Park (BCNP), three assumptions are made: (1) moisture content between land surface and water table does not change; (2) ET comes only from the saturated zone and/or ponding; and (3) infiltration equals percolation. Total ET is calculated as the sum of open water evaporation and saturated

zone (water table) ET. This part of ET variation is represented by the parameter WPET in the Sensitivity Analysis.

3. **Potential ET for Coastal areas (CPET)**
   In irrigated areas within the Lower East Coast (LEC), a simple accounting procedure is used to calculate unsaturated zone ET while saturated and open water ET are calculated based on the reference crop ET. The SFWMM model simulation of this part of ET gets input from running the Agricultural Field Scale Irrigation (AFSIRS) model. This part of ET is represented by the parameter CPET in the Sensitivity Analysis.
4. **Groundwater Hydraulic Conductivity (GWHC)**
   This parameter describes the groundwater flow rate through different types of land.
5. **Seepage Coefficient (SEEP)**
   The SFWMM model grid size is too coarse for modeling local groundwater phenomenon such as levee seepage. In the model, an empirical levee seepage equation is used to solve for levee seepage.
6. **Detention Parameter (DET)**
   This parameter represents the ponding depth below which no overland flow is allowed to occur for different land use types.
7. **Canal-groundwater Hydraulic Conductivity (CHHC)**
   This parameter describes the hydraulic connectivity between the canal and aquifer.
8. **Storage Coefficient (STOC)**
   The Storage Coefficient is the volume of water that an aquifer releases from storage per unit surface area of the aquifer per unit change in head.

Since the ranges of acceptable parameter values to be used for sensitivity analysis are not available in the literature, parameters were varied over a range for which the model calibration was assumed to remain valid (Loucks and Stedinger, 1994). Acceptable ranges of variation for input parameters were decided based on the model output response to the change of parameters.

A sensitivity or influence matrix is set up that summarizes the response at model cells where corresponding gages are located, to changes in individual parameters. Each element of this matrix can be represented by the following relationship (Trimble, 1995a):

$$\alpha_{ij} = \frac{\partial y_j}{\partial x_i} \approx \frac{y_j^c - y_j^o}{\Delta x_i} \qquad \forall \quad i = 1, \ldots n; j = 1, \ldots m \tag{5.1.1}$$

where:

α_{ij} = sensitivity of the jth simulated output/performance to the ith parameter;
y_j = jth model simulated output/performance;
x_i = ith parameter being tested;
n = number of parameters being studied;
m = number of model cells where gages are located;
^{o} = simulated output/performance corresponding to the original calibrated parameter;
^{c} = simulated output/performance corresponding to the parameter which is changed by an incremental Δx_i.

A matrix factorization technique – single value decomposition (SVD) is applied to the sensitivity matrix in order to understand the relationships between the parameters, and isolate groups of parameters that are dependent on one another (Lal, 1995).

5.2 RESULTS OF SENSITIVITY ANALYSIS

Sensitivity of model output by varying key input parameters is quantified by calculating the bias and root mean square error (rmse) of the simulated water levels versus observed water levels at selected model nodal locations. For each parameter, series of model runs were completed to determine a range of acceptable values such that each parameter value within the range can be used without significantly affecting the calibration. The results are grouped by magnitude of errors (expressed in terms of bias and rmse) in each region. By using this method of analysis, one is able to determine whether the variation of a parameter affects all monitoring gages or just a subset of monitoring gages.

Figures 5.2.1 and 5.2.2 show the response of the output stages in terms of bias and rmse at all gages in the WCAs at the key percentile points (5^{th} percentile; lower quartile; median; upper quartiles and 95^{th} percentile) for each parameter change. It can be seen that increasing or decreasing the parameter WPET slightly would not increase the bias immensely. If the change of WPET is within ±20%, the calibration of the model won't be affected significantly. To keep the modeling output valid, the recommended change of WPET is ±20%. This is assumed to be the upper and lower limit of the recommended parameter value for WPET.

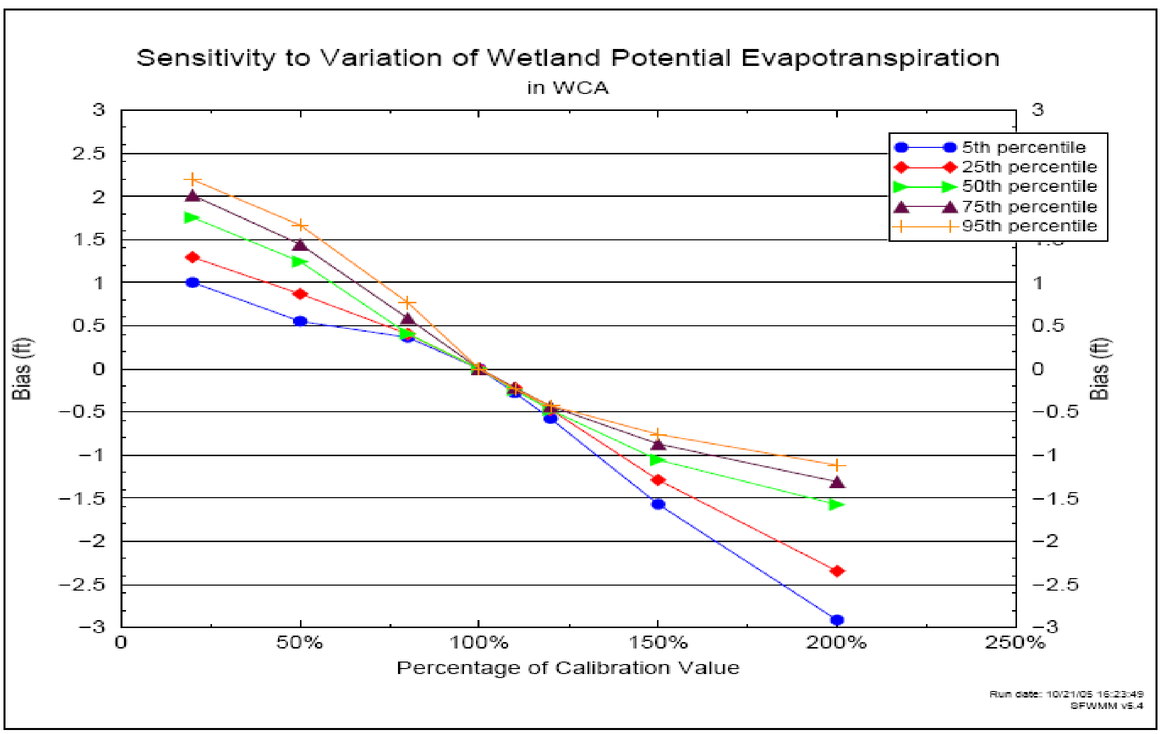

Figure 5.2.1 Sensitivity Percentile in terms of Bias to Variation of Wetland Potential Evapotranspiration in WCAs

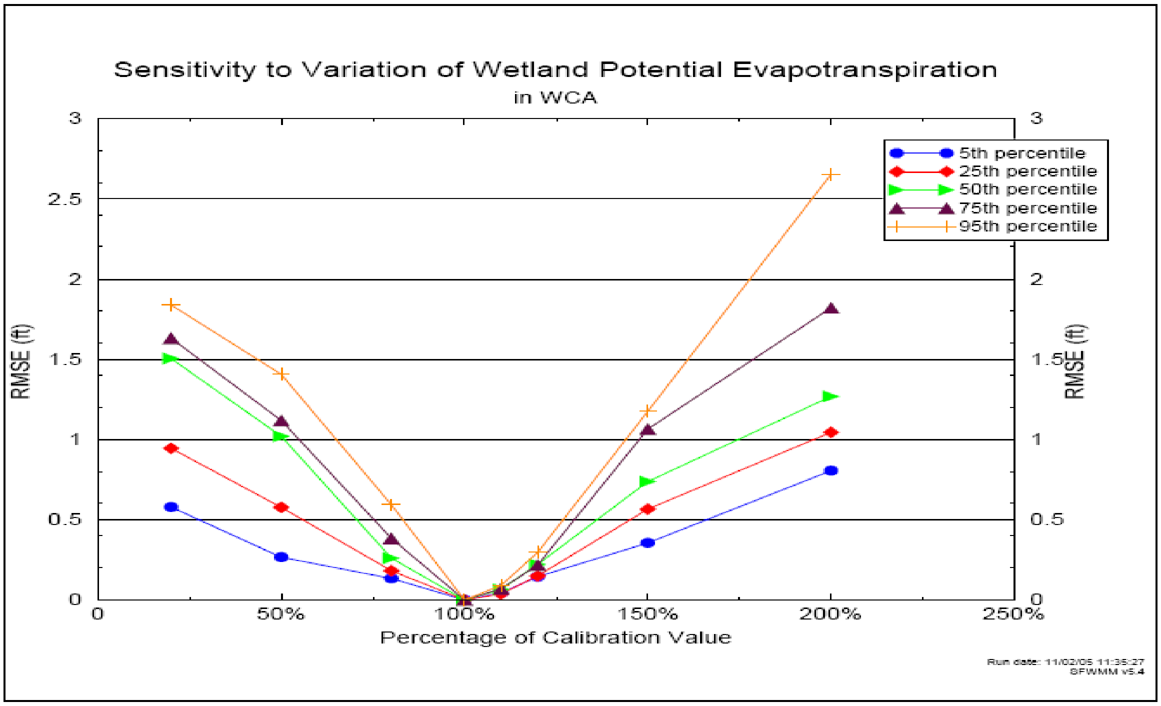

Figure 5.2.2 Sensitivity Percentile in terms of Root Mean Square Error to Variation of Wetland Potential Evapotranspiration in WCAs

Based on the response of the model output, a ± 50% variation of the calibration value is recommended (Trimble, 1995a) for all parameters except the coastal and wetland ET. For Coastal PET, a ± 30% change from the calibrated value is recommended to represent the upper and lower limit of parameter value; while for Wetland PET, as shown in the plots, a ± 20% change from the calibrated value is recommended. The recommended parameter variation is summarized in Table 5.2.1.

Table 5.2.1 Recommended Parameter Variation limit

Parameter	**Recommended parameter variation limit**
WPET	± 20%
GWHC	± 50%
CHHC	± 50%
DET	± 50%
SEEP	± 50%
ERC	± 50%
CPET	± 30%
STOC	± 50%

Figures 5.2.3 to 5.2.9 show the components of the sensitivity matrix for stages at different monitoring gages for different regions within the model domain including: BCNP, ENP, Lower East Coast Service Areas (LECSAs) 1-3, WCAs and Canals.

Equation 5.1.1 was modified as follows:

$$\alpha_{ij} = \frac{\partial y_j}{\partial x_i} \approx \frac{O_{upper} - O_{calibrated}}{P_{upper} - P_{calibrated}} \qquad \forall \;\; i = 1, \ldots n; j = 1, \ldots m \tag{5.2.1}$$

where:

O_{upper} = output variable value (stage) when input parameter is set at upper limit;
$O_{calibrated}$ = output variable value when input parameter is set at the calibrated value;
P_{upper} = parameter value at the recommended upper limit;
$P_{calibrated}$ = parameter at the calibrated value.

The response of the output variables was normalized to be the response per 100% change of each parameter value. For example: for WPET, the parameter value at the recommended upper limit is assumed to be a 20% increase from the calibrated value. Equation (5.2.1) is modified as follows:

$$\alpha_{ij} = \frac{\partial y_j}{\partial x_i} \approx \frac{O_{upper} - O_{calibrated}}{P_{upper} - P_{calibrated}} = \frac{Stg_{120\%} - Stg_{100\%}}{P_{WPET120\%} - P_{WPET100\%}} = \frac{Stg_{120\%} - Stg_{200\%}}{0.2} \bullet \frac{1}{P_{WPET100\%}} \tag{5.2.2}$$

where $\frac{Stg_{120\%} - Stg_{100\%}}{0.2}$ is the component of the sensitivity matrix used in the Sensitivity Analysis.

The sensitivity matrices for all regions are shown in Figures 5.2.3 to 5.2.9.

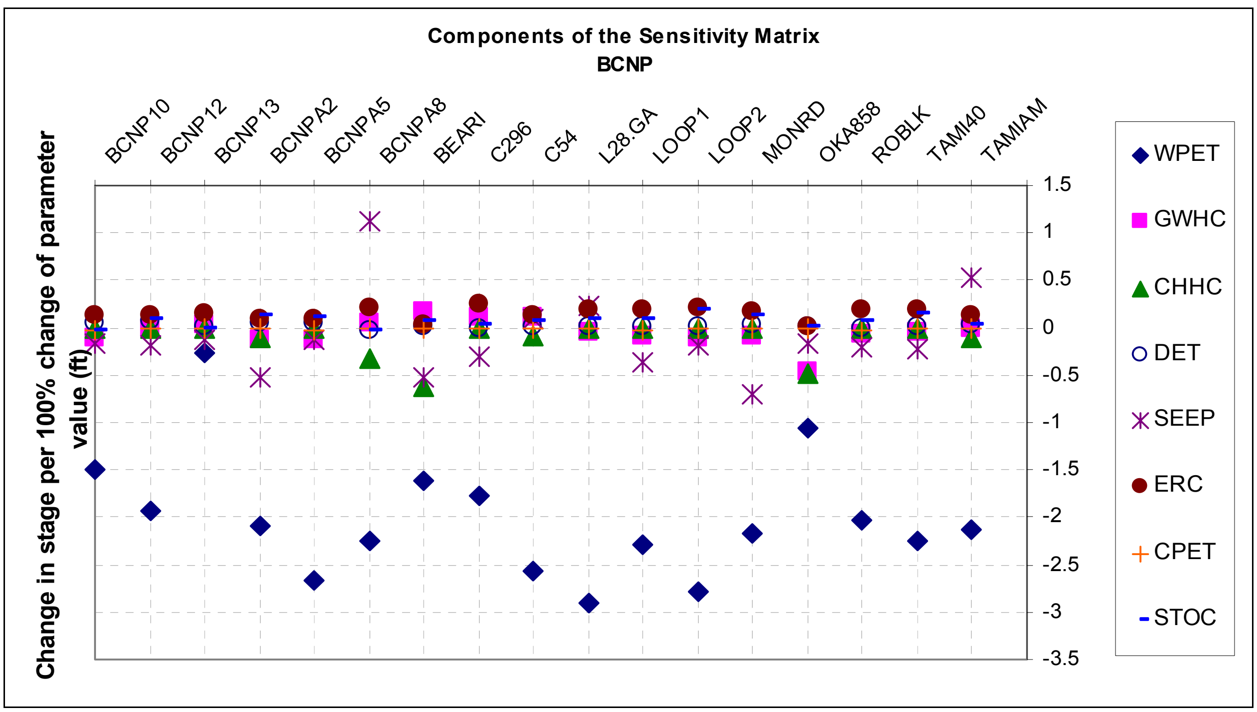

Figure 5.2.3 Components of the Sensitivity Matrix for BCNP

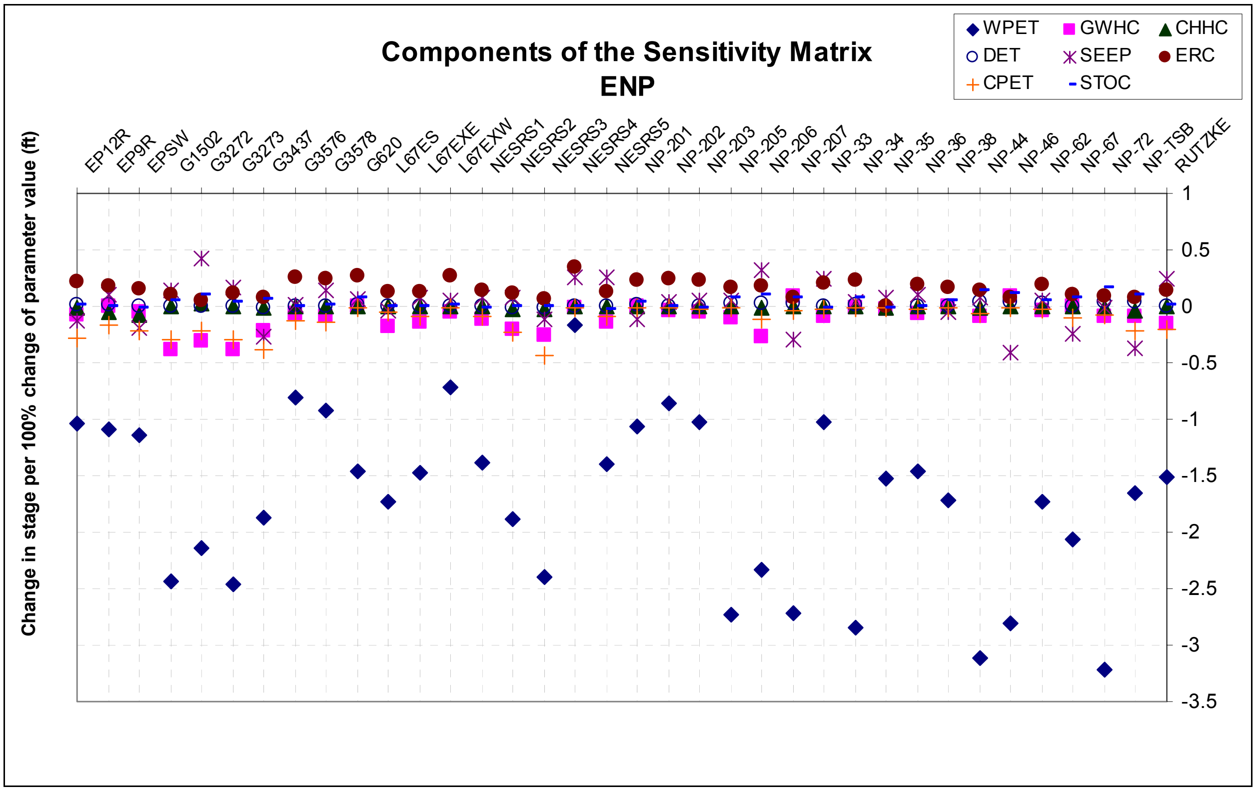

Figure 5.2.4 Components of the Sensitivity Matrix for ENP

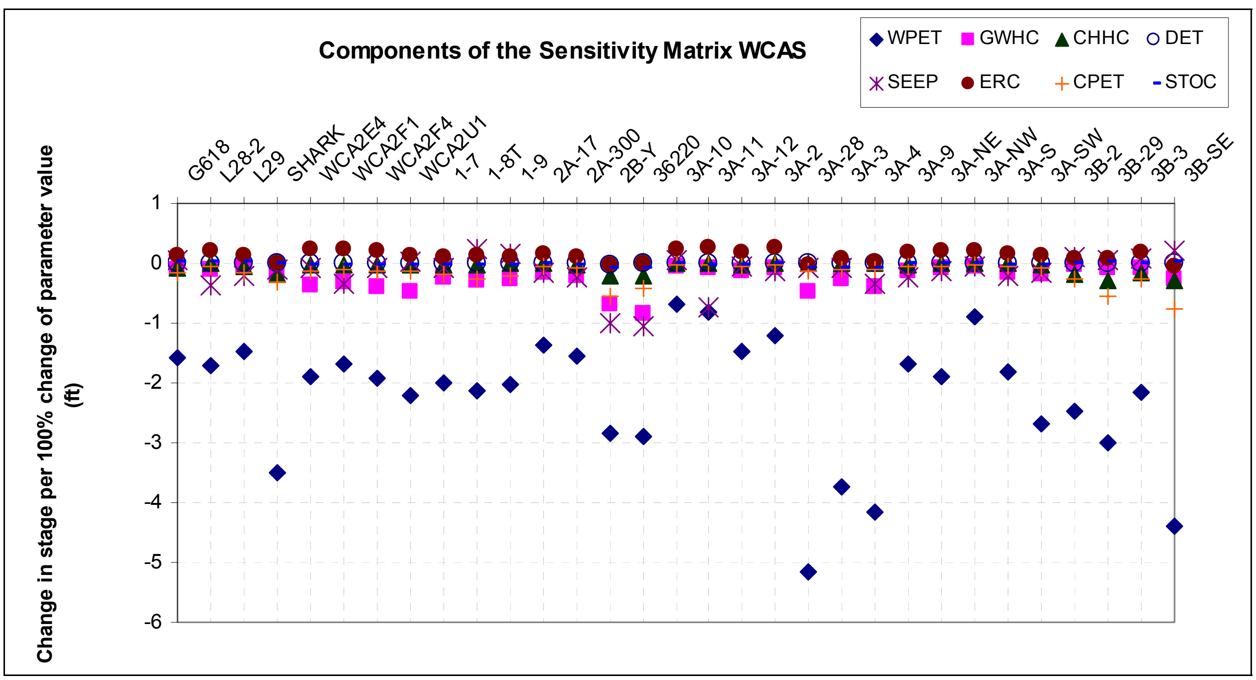

Figure 5.2.5 Components of the Sensitivity Matrix for WCAs

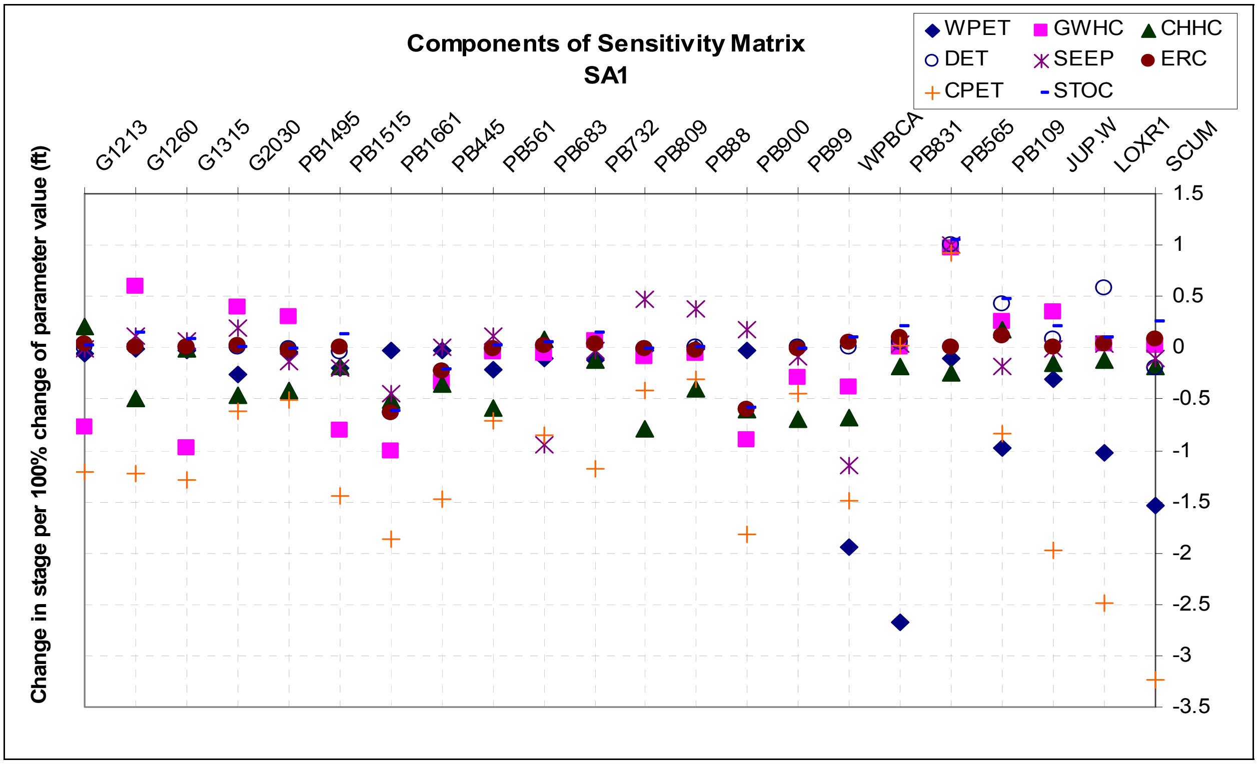

Figure 5.2.6 Components of the Sensitivity Matrix for LECSA1

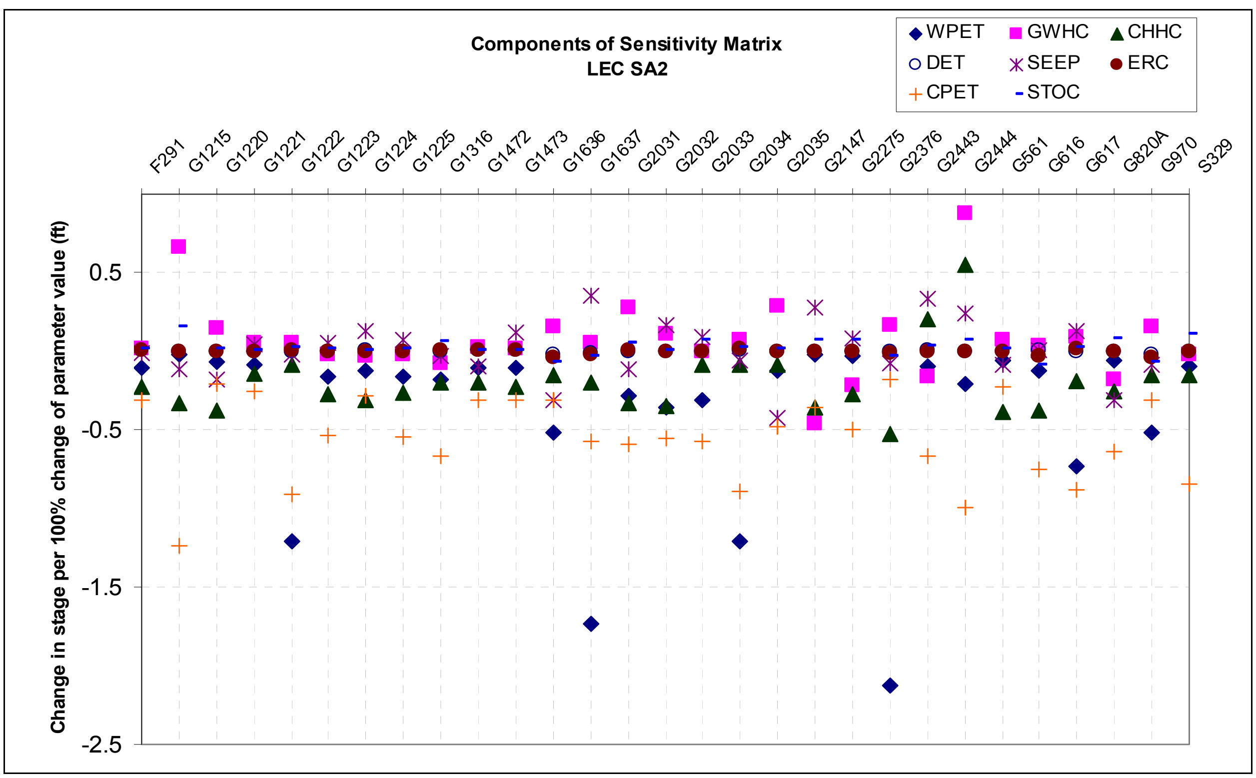

Figure 5.2.7 Components of the Sensitivity Matrix for LECSA2

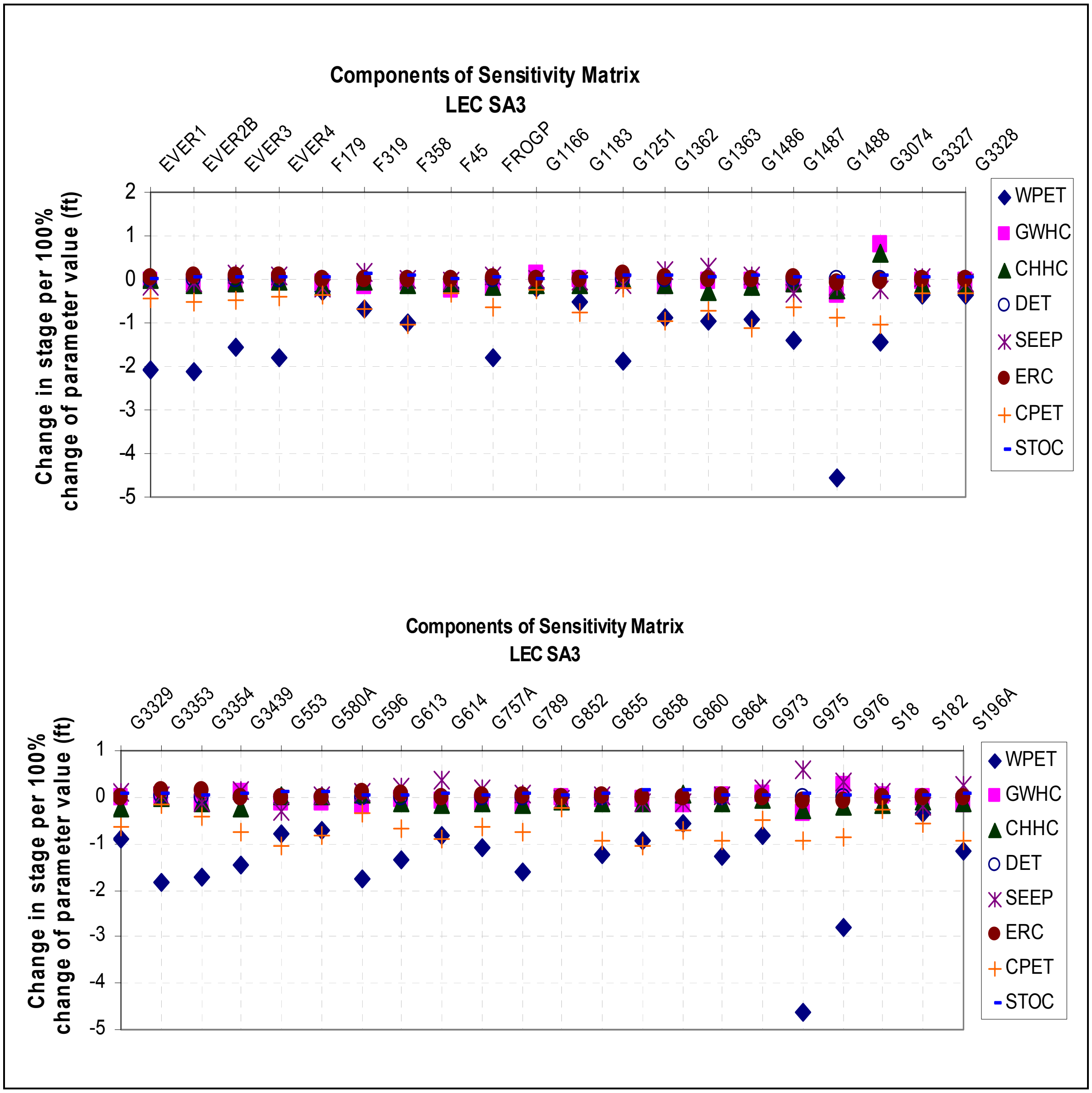

Figure 5.2.8 Components of the Sensitivity Matrix for LECSA3

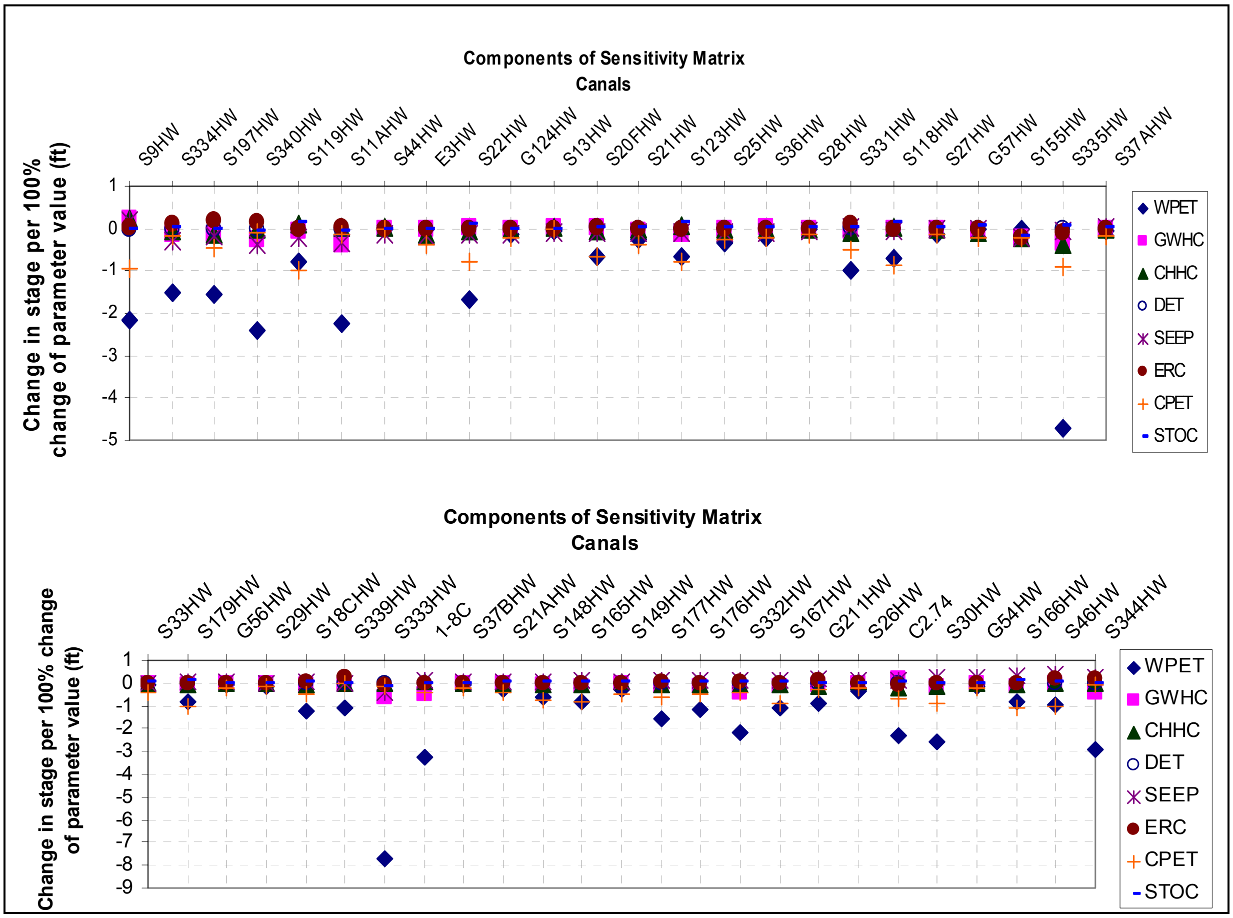

Figure 5.2.9 Components of the Sensitivity Matrix for Canals

The following observations can be made regarding Figures 5.2.3 – 5.2.9:

1. All regions are most sensitive to Wetland PET (WPET), especially BCNP, ENP, WCAs.
2. Coastal PET (CPET) has strong influence upon LEC areas.
3. Canal Groundwater Hydraulic Conductivity (CHHC) has strong influence on LECSA1 and LECSA2. The other regions, ENP, WCAs, LECSA3 and Canals are not sensitive to CHHC.
4. Effective Roughness Coefficient (ERC) has relative stronger influence in ENP. Canals and LECSAs only have slight impact from the variation of ERC value.
5. All regions are quite sensitive to the variation of Levee Seepage (SEEP).
6. Ground Water Hydraulic Conductivity (GWHC) variation affects the ENP, WCAs, LECSA1 and LECSA2 the most. All the other regions are just slightly influenced.
7. Detention Parameter (DET) has very slight influence upon all regions.
8. Storage Coefficient (STOC) has impact on BCNP, LECSA1 and LECSA2. All the other areas are affected very slightly.

A product of the SVD method is the parameter resolution matrix, as shown in Table 5.2.2, which is a measure of the independence of parameters used in a model. For the SFWMM, the resolution matrix is well resolved, all the elements are in the order of 10^{-8}, which means that each parameter is uniquely determined and should be treated separately as far as its influence in determining model output sensitivity.

Table 5.2.2 Parameter Resolution Matrix

	WPET	GWHC	CHHC	DET	SEEP	ERC	CPET	STOC
WPET	1.00	10^{-8}	10^{-8}	10^{-10}	10^{-8}	10^{-8}	10^{-8}	10^{-8}
GWHC	10^{-8}	1.00	10^{-8}	10^{-7}	10^{-9}	10^{-8}	10^{-8}	10^{-8}
CHHC	10^{-8}	10^{-8}	1.00	10^{-7}	10^{-7}	10^{-8}	10^{-8}	10^{-9}
DET	10^{-10}	10^{-7}	10^{-7}	1.00	10^{-7}	10^{-8}	10^{-7}	10^{-9}
SEEP	10^{-8}	10^{-9}	10^{-7}	10^{-7}	1.00	10^{-7}	10^{-8}	10^{-7}
ERC	10^{-8}	10^{-8}	10^{-8}	10^{-8}	10^{-7}	1.00	10^{-8}	10^{-8}
CPET	10^{-8}	10^{-8}	10^{-8}	10^{-7}	10^{-8}	10^{-8}	1.00	10^{-8}
STOC	10^{-8}	10^{-8}	10^{-9}	10^{-9}	10^{-7}	10^{-8}	10^{-8}	1.00

Additional useful information that can be derived from SVD method is the correlation matrix, as shown in Table 5.2.3. This matrix shows that there is only modest correlation between model input parameters. The range of values does not indicate positive or negative correlation. They range from 0.0 for no correlation and 1.0 for perfect correlation. Wetland PET and Effective Roughness coefficient show a relatively stronger correlation (0.22).

Table 5.2.3 Parameter Correlation Matrix

	WPET	GWHC	CHHC	DET	SEEP	ERC	CPET	STOC
WPET	1.00	0.10	0.03	0.00	0.02	0.22	0.10	0.00
GWHC	0.10	1.00	0.00	0.00	0.10	0.00	0.00	0.01
CHHC	0.03	0.00	1.00	0.01	0.01	0.01	0.00	0.01
DET	0.00	0.00	0.01	1.00	0.00	0.00	0.01	0.00
SEEP	0.02	0.10	0.01	0.00	1.00	0.00	0.00	0.00
ERC	0.22	0.00	0.01	0.00	0.00	1.00	0.02	0.02
CPET	0.10	0.00	0.00	0.01	0.00	0.02	1.00	0.05
STOC	0.00	0.01	0.01	0.00	0.00	0.02	0.05	1.00

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GLOSSARY

C-XXX. The letter C followed by a number, designates a Central and Southern Florida Flood Control Project Canal. For example, C-111 reads as "Canal 111". Some canals also have proper names. For example, C-31 reads as "canal 31", also known as the St. Cloud Canal. C-32G reads as "Canal 32G", in which G represents a specific section of the Canal 32 connecting Alligator Lake to Lake Lizzie.

Culvert#XXX. The word culvert followed by a number designates a Central and Southern Florida Project culvert through one of the levees on the perimeter of Lake Okeechobee. Each culvert connects the lake to an adjacent basin. All are under the operation of the USACE.

G-XXX. The letter G followed by a number, designates a Central and Southern Florida Flood Control Project structure. For example, G-72 reads as "Control Structure 72". G structures were built by the District.

HGS-X. The letters HGS followed by a number refer to the Hurricane Gate Structure. These structures were in the levee around Lake Okeechobee and connected the lake to various canals and basins. All of the structures have been replaced by gated spillways.

L-XXX. The letter L followed by a number, designates a Central and Southern Florida Flood Control Project levee. For example, L-38E reads as "Levee 38 east".

L-DX. The letter L followed by the letter D and a number refers to a Central and Southern Florida Project levee on the perimeter of Lake Okeechobee. For example, L-D9 refers to Levee 9 on the perimeter of the lake.

S-XXX. The letter S followed by a number, designates a Central and Southern Florida Flood Control Project structure. For example, S-26 reads as "Control Structure 26". S structures were built by the U.S. Army Corps of Engineers.

1995 base case. The 1995 base case represents the South Florida Water Management Model's estimation of the hydrology of the model area as it would appear if the 1995 facilities and operational policies had been in place for the entire simulation period. The 1995 base case uses 1989 wellfield pumpages, the 1990 District water shortage policy, and 1988 land use and associated demands. Details are defined in USACE and SFWMD (1998).

2000 base case. A South Florida Water Management Model simulation of conditions and operations that approximately represents the year 2000.

2050 base case. The 2050 (future) base case represents the South Florida Water Management Model's estimation of the hydrology of the model area as it would appear if the current facilities and operational policies had been in place under 1965 through 2000 rainfall conditions (35-year simulation period). The year 2050 base case uses 2015 estimated wellfield pumpages at existing locations, the current (1990) District water shortage policy, and 2050 land use and associated demands based on local Comprehensive Plan projections. It also includes environmental enhancement projects that are expected to be implemented by 2050. Details are defined in USACE and SFWMD (1998).

acre-foot. Unit of volume (generally water) with a base area of one acre and a height of one foot; 43,560 cubic feet; 325,872 gallons.

aquifer. A geologic formation, group of formations, or part of a formation that contains sufficient saturated permeable material to yield useful quantities of ground water to wells, springs or surface water.

backpumping. The practice of pumping water that is leaving an area as runoff back into a surface water reservoir or recharge area.

borrow canal. In most cases, the material for construction of a levee is obtained by excavation immediately adjacent to the levee. The excavation is termed a "borrow". When the borrow paralleling the levee is continuous and allows for conveyance of water, it is referred to as a "borrow canal". For example, the canal adjacent to L-8 levee is called the L-8 borrow canal. Many borrow canals, such as the L-8 borrow canal, are important features of the Project.

CERP. Comprehensive Everglades Restoration Plan as approved by the Water Resources Development Act of 2000.

control structures. Man-made structures designed to regulate the level and/or flow of water in a canal or from a lake or reservoir (e.g. weirs, dams).

crest elevation. The crest elevation of a structure is the level below which water cannot pass the structure. Where the crest elevation of a structure is used to control water flow, the crest elevation is set to maintain the desired upstream water level.

culvert. A culvert is a closed conduit for conveyance of water. Within the District, culverts may be made of corrugated metal pipe or reinforced concrete. The concrete culvert may be either circular or rectangular in cross section. When it is rectangular, the culvert is usually referred to as a box culvert. The cross-sectional area and length of the culvert determine, and in some cases limit, the amount of flow possible through the culvert for given head water and tailwater conditions. Further control of flow through the culvert may be affected by placing a gate or a riser and stoplogs at the headwater end.

Corps. Generally refers to the U.S. Army, Corps of Engineers, but often specifically refers to the Corps of Engineers District, Jacksonville, Florida.

demand. The quantity of water needed to be withdrawn to fulfill a human, environmental or agricultural need.

District. This refers to the South Florida Water Management District (formerly the Central and

South Florida Flood Control District), the agency which operates and maintains the Project.

drainage. Drainage is the amount of removal of groundwater from a basin to maintain optimum groundwater levels. Overdrainage is the lowering of groundwater levels below desired levels. See **water control**.

drainage basin. A drainage basin can be defined as a certain area that due to its topographic characteristics is able to convey the runoff produced by rainfall on it to a final location, commonly known as the outlet of the basin. If rain falls over a large area, some of the runoff from that storm will likely enter one stream, and some of it will enter other streams. It is said that those streams "drain different basins" or that they are in "different drainage basins." Thus, a drainage basin of a stream is all the land that contributes runoff to the stream or its tributaries. The boundary between drainage basins is represented by the lines of highest elevation or "divide" in a topographic map, from which water is able to establish two or more flow patterns. Usually a large drainage basin or watershed is divided into basins. This creates more accurate calculations because different factors affecting each basin can be taken into consideration. Also, by subdividing a large area (watershed or basin) into basins, hydrologic results can be obtained at intermediate points of the entire basin, which, in this case, are represented at each subbasin. See **runoff and drainage**.

excess water. Excess water in a basin is water that must be removed from the basin for flood protection or to maintain optimum water levels for agriculture. The excess water may come from rainfall, seepage through levees, or from surface water inflows from adjacent basins.

flood control. Flood control is the removal of surface water from a basin to prevent or minimize flood damages.

gated spillway or culvert. A spillway or culvert is "gated" when water flow through the structure is controlled by a gate. Within the C&SF Project, almost all gates open upward to allow flow beneath the gate.

General Design Memorandum (GDM). This is a document prepared by the U.S. Army Corps of Engineers that reports all work done prior to preparation of the final design of a project. In the GDM for the Central and Southern Florida Flood Control Project four important aspects of the Project are developed: (1) each of the surface water management basins is delineated, (2) a set of design storms is specified for each basin and the resulting basin discharges are estimated, (3) the flood protection to be afforded each basin is specified, and (4) the size, number and general location of canals and structures needed to achieve the desired level of flood protection are determined. The final design of the canals and structures is given in the **Detail Design Memorandum (DDM)**.

ground water or groundwater. All water found beneath the surface of the earth in the voids, fractures, and pores or other openings of soil and rock material.

irrigation. The application of water to crops by artificial means. The reasons for irrigating may include, but are not limited to, supplying evapotranspiration needs, leaching of salts, and environmental control.

levee. An embankment to prevent flooding, or a continuous dike or ridge for confining areas of land for irrigation by surface flooding.

Natural System Model (NSM). A two dimensional, integrated surface and ground water model used to estimate the hydrology of the Everglades prior to the influence of man. The NSM performs, on a daily basis, a continuous simulation for 36 years (1965-2000) of historic meteorologic data.

NGVD. National Geodetic Vertical Datum; reference sea level 1929, from which elevations are measured.

Pre-CERP Baseline. From the CERP Programmatic Regulations, this is defined as "…the hydrologic conditions in the South Florida ecosystem on the date of enactment of WRDA 2000, as modeled by using a multi-year period of record based on assumptions such as land use, population, water demand, water quality, and assumed operations of the Central and Southern Florida Project."

Project. This refers to the Central and Southern Florida Project for Flood Control and Other Purposes. The Project was responsible for the construction of most of the major canals and structures in South Florida.

regulation schedule. A regulation schedule specifies the outlet operational strategy for a reservoir (e.g., Lake Okeechobee) as a function of the water level in the reservoir and the time of year. In general, a regulation schedule optimizes the reservoir's ability to receive excess water in the wet season and to provide water supply in the dry season.

regulatory release. A regulatory release is water discharged from a reservoir to lower the water level in the reservoir in accordance with its regulation schedule.

reservoir. A man-made or natural lake where water is stored.

riser and stoplogs. Riser and stoplogs refer to a means of regulating the water level upstream of a culvert or weir. Stoplogs are individual beams, of fixed dimension, set one upon the other to form a bulkhead supported by channels or grooves (i.e., the riser) at either end of the span. The stoplogs slide in or out of the riser; the number of stoplogs determines the crest elevation of the bulkhead. The structure may be effectively closed by addition of enough stoplogs. The riser is located at the headwater end of the culvert or on top of the weir.

runoff and drainage. All water moving in the landlocked portion of the hydrological cycle is derived either directly or indirectly from precipitation, also known as rainfall. Several things happen to rain after it falls to earth. At the beginning of a rainfall event, part of it forms surface retention. Surface retention consists mainly of two hydrologic processes: interception and depression storage. Interception is that portion of rainfall that is captured by vegetative cover. Rainfall not intercepted continues its downward movement and fills up surface puddles to form depression storage. These components are commonly referred to as initial abstractions. After this, most of the water reaching the ground surface will infiltrate through the soil. As the soil becomes saturated, infiltration rate will decrease, and at the same time, evapotranspiration begins. The process of evapotranspiration (ET) consists of evaporation and transpiration. Evaporation is defined, in this case, as the process by which water is changed into a gaseous state and returned to the atmosphere. Transpiration is the process by which water vapor escapes from a living plant, principally the leaves, and enters the atmosphere. In field conditions, it is practically impossible to differentiate between evaporation and transpiration if the ground surface is covered by vegetation. The two processes are commonly linked together and referred to as evapotranspiration.

Once infiltrating water has passed through the surface layers, it percolates downward under the influence of gravity until it reaches the saturation zone at the phreatic surface or "water table". This zone is also known as groundwater. Many soils in South Florida are sandy and underlying rock strata. Flow of water is easily accomplished through these permeable soils. When the water table level is higher than the local surface water levels, water will enter the surface water from groundwater. When the water table is lower than the local surface water level, flow is from surface water to groundwater. Usually groundwater supplements stream flow during periods of low rainfall, and surface water recharges groundwater storage during periods of high rainfall.

In general, part of the storm rainfall retained on or above the ground surface is surface retention, which, with the infiltration and evapotranspiration losses, is subtracted from input rainfall resulting in the rainfall excess. This "effective" part of the original rainfall is the one capable of yielding surface runoff after routing to the basin outlet.

The term "drainage" is used to refer to the total surface and subsurface flows entering a lake and/or canal, or a creek from their drainage basin. It is important to keep in mind that during a rain event (especially one severe enough to cause flooding), it is surface runoff that is the important contributor to this flow, and, at times, between rain events, subsurface flow from groundwater to surface water is most important.

Runoff from a drainage area is a function of several factors: how much rain has fallen and how often it has occurred, the depth to the water table, and how the land in the drainage area is utilized. The amount of recent rain and the depth to the water table impose how much water there is in the soil. The degree to which the soil is saturated, in turn, determines how much of the falling rain may infiltrate the soil, and correspondingly, how much of the rain will runoff to local streams.

Land use has a large influence on the amount of surface runoff entering local streams, which will convey the water to the lakes, canals or creeks. Much of the surface area in an urban development (i.e., roofs, roads, and parking lots) is considered impervious to water. Almost all the rain falling on impervious areas become surface runoff. Some water may be detained and will evaporate, but the percentage of rainfall that enters local stream by surface runoff in an urban development is usually high. As a result, urban developments are subject to high stream flows during rain events, and consequently they need to be provided with drainage systems to avoid or minimize flooding damage.

A vegetated area can intercept and retain a significant part of the rainfall and, consequently, surface runoff will diminish. This intercepted water has an additional opportunity to evaporate or seep into the ground. Commonly, a small percentage of the rain falling on a vegetated area will enter local streams, and consequently will produce runoff. For this reason, stream flows in vegetated areas are moderated compared to urban developments.

saltwater intrusion. In coastal areas of South Florida, fresh and salt groundwaters meet. The fresh groundwater is less dense than the salt groundwater. It floats on, but does not mix with the saltwater. It is necessary to maintain the water table in coastal areas high enough to prevent saltwater from entering the local groundwater and contaminating any nearby wellfields.

spillway. A spillway is a means of passing water from one location to another (e.g., from a lake to a canal or from one part of a canal to another). The purpose of the spillway is to control the flow of water. Control may be affected by gates or by the crest elevation of the spillway or both. Control by gate operation allows variable control of water flow and may control either amount of flow or the upstream water level. Control by the crest elevation is usually not variable and controls only the upstream water level. When water control is strictly by the crest elevation of the spillway, the spillway is usually referred to as a weir.

stage. The elevation of water surface in a water body with respect to a specified datum, usually the National Geodetic Vertical Datum (NGVD) of 1929.

surface water. Water upon the surface of the earth, whether contained in an area created naturally or artificially or diffused.

water control. Water control is the regulation of groundwater levels (i.e., by the regulation of canal water levels) at all seasons and the conservation of water during the dry season. During wet periods, water must be removed from basins to maintain desired groundwater levels. This is sometimes referred to as drainage and is differentiated from flood control which generally refers to removal of surface water from a basin. During dry periods, outflows from the basin are restricted to retain water in the basins to prevent "overdrainage" (i.e. lowering of groundwater levels). In agricultural areas, overdrainage can lead to crop yield reduction or failure, and in coastal areas, to saltwater intrusion to ground water. In some cases, water must be supplied to the basin to maintain groundwater levels.

water control structures. Water control structures are devices, e.g., weirs, spillways, and culverts, placed in or between canals to regulate water levels (stage divide), amount of flow, or direction of flow (flow divide) in the canals. A structure may have more than one function. A divide structure is usually located at or near a basin boundary. When it is closed, it prevents water in one basin from entering the other basin. A water supply structure is usually located near a basin boundary. It is used to pass water from one canal to another, i.e., from one basin to another. A divide structure also often serves as a water supply structure.

Water Conservation Areas (WCAs). That part of the original Everglades ecosystem that is now diked and hydrologically controlled by man for flood control and water supply purposes. These are located in the western portions of Dade, Broward and Palm Beach counties, and contain a total of 1,337 square miles, or about 50 percent of the original Everglades.

Water Supply Plans. Regional water resource and demand analyses generated by the District to provide a detailed evaluation of available water supply and projected demands through the year 2010.

water surface elevation. A water surface elevation in a canal or a lake is the vertical distance from the surface of the water to some reference elevation or "datum." The GDM reports from the USACE use the elevations relative to the mean sea level (MSL). In the District, elevations are relative to the National Geodetic Vertical Datum (NGVD) of 1929. For practical purposes MSL coincides with NGVD. Water surface elevations may also be referred to as "stages."

Important water surface elevations for a control structure are the headwater (upstream) stage, and the tailwater (downstream) stage (see **water control structures**). The difference between these stages will affect the flow through or over the structure. In general, flow increases as the difference in elevation increases.

Water surface elevations elsewhere in the canal reach are also important. Obviously, if the stage exceeds the top elevation of the canal, flooding will occur. Not as obvious is the fact that the stage in the canal can heavily influence the water table elevations of local groundwater (see **runoff and drainage**). The stages in the lower reaches (near the ocean) of some coastal canals are maintained at levels high enough to prevent intrusions of saltwater into the local groundwater. In other areas, stages are maintained that keep water table elevations low enough to prevent drainage problems in low lying areas.

The headwater side of a gravity flow structure, e.g., ungated spillway, is the side on which the stage is usually higher. It is possible at some structures for the tailwater to occasionally be higher than the headwater stage. The headwater stage at a pumping station, on the other hand, is usually defined as the side from which water is pumped and usually refers to the side with the lower stage. This convention allows the direction of water flow to be consistently defined as from headwater to tailwater side of any structure.

Water elevations or stages in a reservoir, such as Lake Okeechobee, are of crucial importance. These stages are regulated by means of control structures strategically located at the outlets of reservoir. On any given day of the year, if they exceed the value prescribed by the regulation schedule, releases (regulatory or flood control discharges) are made from the reservoir to bring the stages down below the schedule.

weir. See **spillway**.