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APPLICATION OF OPTIMIZATION MODELING TO WATER RESOURCE PLANNING IN EAST-CENTRAL FLORIDA

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Application of Optimization Modeling to Water Resource Planning in East-Central Florida

by

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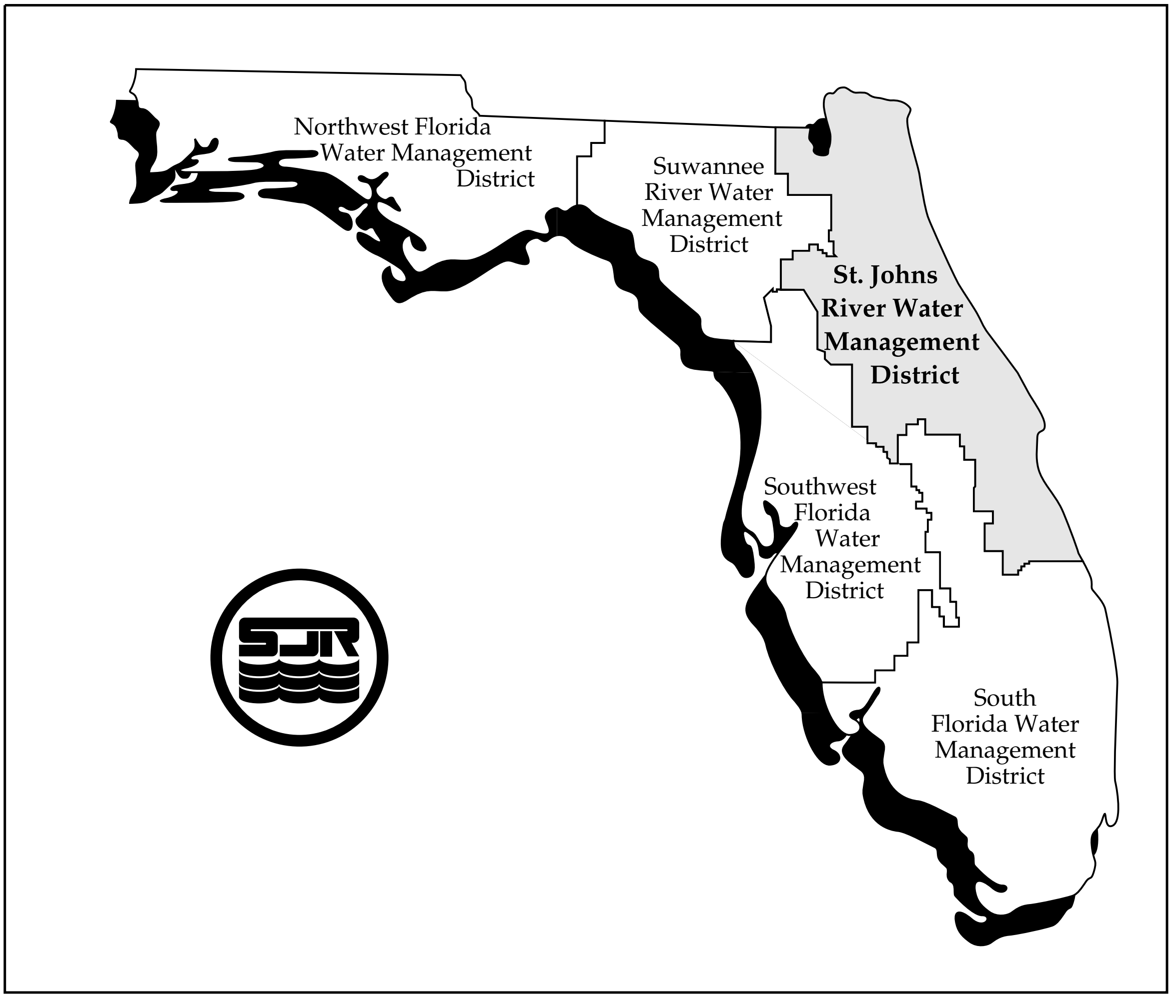

The St. Johns River Water Management District (SJRWMD) was created by the Florida Legislature in 1972 to be one of five water management districts in Florida. It includes all or part of 18 counties in northeast Florida. The mission of SJRWMD is to ensure the sustainable use and protection of water resources for the benefit of the people of the District and the state of Florida. SJRWMD accomplishes its mission through regulation; applied research; assistance to federal, state, and local governments; operation and maintenance of water control works; and land acquisition and management.

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EXECUTIVE SUMMARY

INTRODUCTION

Due to the increasing demands placed on Florida's water resources, the state of Florida adopted legislation in 1989 to direct future water supply planning programs. This legislation requires each water management district to completely evaluate its water needs and sources through the year 2020 and delineate critical areas identified as water resource problems. Once completed, the water management districts are expected to develop possible alternative water supply strategies that will correct or avoid adverse unacceptable impacts associated with the development of water supplies.

In east-central Florida, public supply demands are expected to double between 1995 and 2020. Current plans are to meet these demands with additional groundwater withdrawals from the Floridan aquifer system. This increased demand and additional groundwater withdrawals are expected to result in water resource problems related to native vegetation, spring discharges, and saltwater intrusion. In order to optimally distribute the water resources available today to east-central Florida, the St. Johns River Water Management District (SJRWMD) has designed two optimization models to assist in decision-making. The optimization models are the deficit model and the decision model. The results arising from the application of these two models are the focus of this report.

DEFICIT OPTIMIZATION MODEL

Usually, the decision on projected water use allocation made for a given demand area is based on the area's water use history. Because the allocated quantity is not based on any sound mathematical judgment, SJRWMD has determined the need to check the appropriateness of these allocated quantities through the use of optimization models. The deficit model takes projected water allocations for all utilities in the east-central Florida model region and determines whether these allocations could result in the development of deficits by minimizing the sum of deficits subject to known constraints within the region. For instance, if for a given area j, the model determines a deficit of QD_j due to a projected water demand, DM_j, then the actual projected allocation, AA_j, for this demand area must be $AA_j = DM_j - QD_j$, in order to ensure a zero deficit.

In an attempt to identify the deficits for demand areas, the deficit model is formulated to minimize the cumulative effect of deficits ($QD = DM - AA$) for the entire east-central Florida model region in the given time horizon. Of course, some demand areas may be reported by the deficit model as having zero deficits. For these areas, it is assumed that the projected allocated water use previously made (DM_j) is the actual allocation quantity (AA_j). The constraints incorporated in the deficit model include (a) limits on surficial aquifer drawdown, (b) limits on spring discharge to minimum flows and levels, and (c) limits on well withdrawal rates. The deficit model is based on water quantity projections for public supply wells. The use of groundwater for agricultural purposes (agricultural wells) is not considered as part of the decision variables in the current optimization models.

DECISION OPTIMIZATION MODEL

The decision model proposes alternative water sources to balance the water deficits resulting from the use of groundwater as the only available resource. It is therefore developed as an extension of the deficit model with the incorporation of alternative sources of water. The alternative water sources include (a) new public supply wells (containing both fresh and brackish water), (b) surface water, and (c) public supply interconnects. The decision model minimizes the cost associated with (1) construction of new well facilities, (2) treatment and transport of surface and well water, and (3) water transport between interconnects. The constraints in the decision model include all of the environmental constraints from the deficit model plus limitations on surface water use.

DEFICIT MODEL RESULTS

The results from the deficit model indicate that, given the projected quantity and spatial distribution of groundwater withdrawals, east-central Florida will potentially reach the limit of sustainable withdrawals within the planning horizon. An analysis conducted on a time basis indicates that current demands will exceed the 2020 projected allocation by the year 2010. The results from the deficit model were verified by comparing its predicted heads at the control points and spring locations with corresponding predictions from MODFLOW. The results of the two models compared favorably. MODFLOW is a forward simulation model popularly used within the United States and beyond for predicting hydraulic heads in a designated model region.

To ensure a fair distribution of available water resources, the concept of equity was introduced into the deficit model. The background implication of the concept of equity is to take water from users getting ‘too much’ and transfer it to those getting none or ‘too little.’ This equity concept was introduced into the deficit model by adopting an additional constraint that permits users to pump an extra 40% of their respective 2020 water allocation. The concept was illustrated by first making a run without equity (i.e., ignoring the equity constraints) and subsequently comparing the results with a second run that incorporated the equity constraints. The results from the two runs demonstrated that the incorporation of the additional equity constraint into the optimization model might serve as an appropriate means of introducing a uniform pattern of distribution of the available water resource across the model region. For the different scenarios considered in the deficit runs, the extent of deficits likely to be encountered without the use of alternative sources ranges from 127 million gallons per day to 270 million gallons per day.

DECISION MODEL RESULTS

Based on the concept of minimizing cost, the decision model advises users on whether or not to rely on new wells (fresh or brackish), surface water, or interconnects for their water needs. Generally, the model results indicate that east-central Florida will reach the limit of its sustainable water supply if alternative water sources (other than groundwater) are not considered in the resource allocation procedures. In almost all of the decision model runs, the need to adopt the use of surface water as an alternative to groundwater was emphasized. Expectedly, the need to supplement existing groundwater with new groundwater sources was rarely advised by the decision model due to the environmental impact that excessive withdrawal from groundwater is likely to cause. East-central Florida has readily available surface water sources in the region, and the need to exploit this resource to supplement groundwater use must be encouraged.

RECOMMENDATIONS

Results from the decision model indicate that surface water can be very reliable in the alleviation of water deficit problems in the east-central Florida region. It is suggested that future allocation procedures encourage customers to consider surface water as a suitable alternative to the usual groundwater resource. Since the MODFLOW flow model does not have the capability to simulate coupled fresh- and saltwater flow, water quality effects were not considered in the current study. Since saltwater upconing due to freshwater pumping is known to occur in the east-central Florida model region as a result of the closeness of some portions of its boundary to the coast, it is suggested that water quality issues be considered in future modifications of the model. The time required for the generation of the influence coefficients matrix, an important first step for the formulation of both optimization models, may be improved by distributing the process for multiple processors.

CONTENTS

FIGURES

TABLES

SYMBOLS

Subscript	Definition
h	counter for set of control points
i	counter for set of public supply wells
i	decision variable counter
j	counter for set of demand areas
j	decision variable counter
l	counter for set of springs
xx	x-coordinate direction
yy	y-coordinate direction
zz	z-coordinate (or vertical) direction

Parameter	Definition	Unit
$at_{j,q}$	connection map between source j and destination q	—
$dists_{j,q}$	distance from source area j to destination area q	ft
$distsm_{p,j}$	distance from surface water source p to public demand area j	ft
CD_l	conductance of spring at location l	ft^2 / day
D_j	allocation issued to demand area j	mgd
$D\lim_h$	drawdown limit set at control point h	ft
Dm_j	minimum demand at public demand area j	mgd
El_l	elevation head of spring at location l	ft
fll_i	fixed cost for new Lower Floridan public supply well i	\$day / ft^3 / yr
fsu_p	fixed cost for surface water source p	\$day / ft^3 / yr
ftp_i	fixed cost for urban lime softening treatment plant at supply well i	\$day / ft^3 / yr
ful_i	fixed cost for new Upper Floridan public supply well i	\$day / ft^3 / yr
FW_j	fixed cost and treatment cost of new wells in public area j	$
h	hydraulic head	ft
Δh_h	change in hydraulic head at control point h	ft

Parameter	Definition	Unit
Ho_h	estimated hydraulic head for base year 1995 at control point *h*	ft
Hno_h	estimated hydraulic head for projected year 2020 at control point *h*	ft
Hso_l	estimated hydraulic head for base year 1995 at spring *l*	ft
$Hsno_l$	estimated hydraulic head for projected year 2020 at spring *l*	ft
$icept_{j,q}$	fixed cost of linear approximation intercept from source *j* to destination *q*	$\$/\mathrm{yr}$
$icepts_{p,j}$	fixed cost of linear approximation intercept from surface water source *p* to demand area *j*	$\$/\mathrm{yr}$
K_{xx}	hydraulic conductivity in the *xx* direction	$\mathrm{ft/day}$
K_{yy}	hydraulic conductivity in the *yy* direction	$\mathrm{ft/day}$
K_{zz}	hydraulic conductivity in the *zz* (or vertical) direction	$\mathrm{ft/day}$
m	total number of constraints	—
$Mcap_i$	allowable pumping capacity at public supply well *i*	mgd
n	total number of public supply wells	—
ΔQ_i	change in public supply well *Q* at location *i*	mgd
Qno_i	projected 2020 withdrawal rate at public supply well *i*	mgd
r	total number of decision variables	—
$Scap_p$	capacity of surface source *p*	mgd
$Scrit_l$	critical minimum discharge at spring *l*	$\mathrm{ft^3/day}$
$Servm_{i,j}$	service map from well *i* to demand area *j*	—
$Servs_{p,j}$	service map from surface source *p* to demand area *j*	—
$slope_{j,q}$	fixed cost of linear approximation slope per unit flow from source *j* to destination *q*	$\$\mathrm{day/ft^3/yr}$
$slopes_{p,j}$	fixed cost of linear approximation slope from surface water source *p* to demand area *j*	$\$\mathrm{day/ft^3/yr}$
us_p	unit cost of surface water source *p*	$\$\mathrm{day/ft^3/yr}$
uw_i	unit cost of supplies and treatment of public well *i*	$\$\mathrm{day/ft^3/yr}$
W	sink or source term	$\mathrm{day^{-1}}$
$\alpha_{i,h}$	influence of public well *i* on control point *h*	$\mathrm{ft/ft^3/day}$

Parameter	Definition	Unit
$\gamma_{i,l}$	influence of public supply well *i* on spring location *l*	$\text{ft}/\text{ft}^3/\text{day}$

Variable	Definition	Unit
D_h	drawdown at control point *h*	ft
FIC_q	fixed cost of interconnecting to destination area *q*	\$/yr
FNT	total flow rate to all public interconnects	mgd
$FP1_j$	fixed cost of surface water cost for serving public area *j*	\$/yr
$FP2_j$	fixed cost of groundwater transport from a source to destination *j*	\$/yr
Hs_l	hydraulic head at spring location *l*	ft
Ps_l	fraction of spring discharge for simulation year 1995	—
Q_i	optimized well withdrawal rate at public supply well *i*	mgd
$Q1_{i,j}$	consumption rate at demand area *j* from supply well *i*	cfd
$Q2_{j,q}$	interconnect use rate from source *j* to destination *q*	cfd
QD_j	deficit at demand area *j*	mgd
QP_i	total withdrawal rate at public supply well *i*	cfd
QS_p	cumulative use rate of surface source *p*	cfd
$QQS_{p,j}$	use rate of surface source *p* at public demand area *j*	cfd
SD_l	discharge of spring at location *l*	cfd
UI_q	unit cost of interconnect from source area *q*	\$/yr
UP_j	unit cost of supplies serving public area	\$/yr
UST_j	unit cost for surface water transport to demand area *j*	\$/yr
Z	deficit model objective function variable	mgd
Z	decision model objective function variable	\$/yr

INTRODUCTION

In east-central Florida (ECF), public water supply demands are expected to double between 1995 and 2020. Current plans are to meet those demands with additional groundwater withdrawals from the Floridan aquifer system. This increased demand and the additional groundwater withdrawals could result in water resource problems related to native vegetation, spring discharges, and saltwater intrusion, if the resource is not managed wisely. Groundwater flow and optimization models have been developed to support the St. Johns River Water Management District's (SJRWMD) attempt to investigate the technical, environmental, and economic feasibility of water supply strategies which will avoid or minimize adverse impacts.

This report summarizes the theory underlying the development of the ECF optimization model and the extent to which it can be used to support SJRWMD in water resource management. It is important to note that the ECF model results presented in this report are examples that demonstrate how the model may be used for management decision purposes when the need arises. The optimization models developed use a linear programming formulation to identify and quantify potential problems related to water supply and demand. The technique provides a systematic method for determining water resource allocation that meets environmental protection goals at minimal annualized costs. Although the surficial aquifer response to pumping and recharge is known to be nonlinear, available software resources at the time of developing the optimization model could only permit the development of a linear model. Therefore, the surficial aquifer response in the model is assumed to be linear. The models were developed with the widely used three-dimensional groundwater simulation model MODFLOW (McDonald and Harbaugh 1988) and the General Algebraic Modeling System (Brooke et al. 1992). The surficial and Upper Floridan aquifer and spring discharge responses to production wells are simulated using the unit response matrix approach under the assumption that the principle of superposition is applicable to linear systems. The elements of the unit response matrix describe the relationship between stresses (well withdrawal rates) and drawdown at specified locations of interest. These locations of interest are designated as control points. More details about the unit response matrix are presented later. The models were applied to the ECF region of Florida to identify potential water resource problem areas. The two optimization models developed are the deficit model and the decision model. Figure 1-1 shows the discretized ECF study area with the associated boundary conditions.

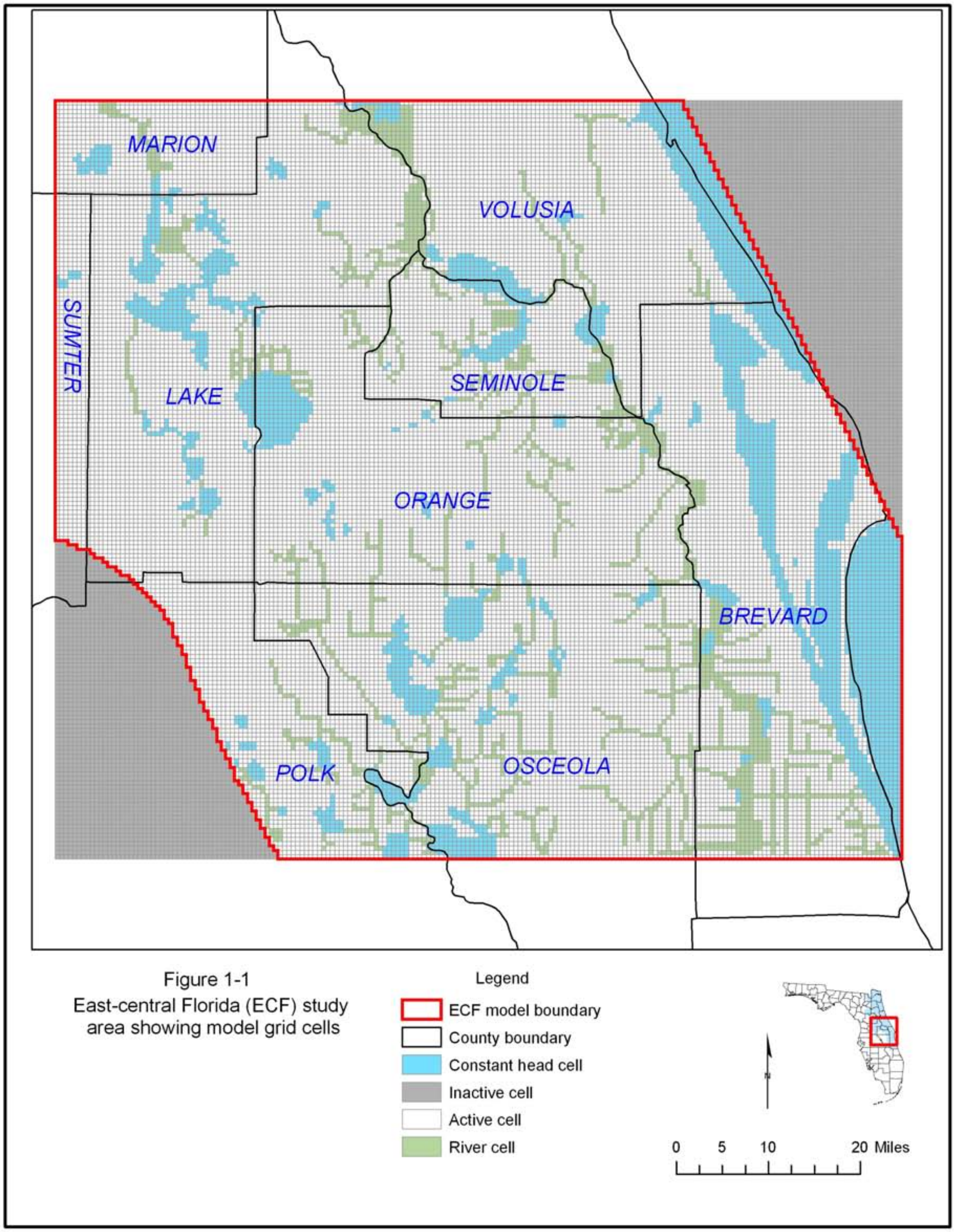

Figure 1-1
East-central Florida (ECF) study area showing model grid cells

PURPOSE AND SCOPE

The purpose of the Water Supply Optimization Modeling Project is to assist SJRWMD in developing a systematic modeling method for determining optimum water supply strategies that satisfy various environmental and hydrological requirements based on existing sources. This optimization modeling method is a combined optimization / simulation technique which incorporates SJRWMD current groundwater flow simulation models for the study area (McGurk and Presley 2002). Using the current available data, the resulting optimization models are run and their results presented. Other pertinent modeling issues such as verification of model results, sensitivity analysis, and timing analysis are also conducted and presented in the models' results.

OPTIMIZATION PROGRAMMING

An optimization model consists of an objective function, or quantity that is to be minimized or maximized, and a number of constraints, or conditions that must be satisfied. In linear programming problems, all of the variables or unknowns in the model are continuous. Linear programming solves the problem of allocating resources among competing users in an optimal manner. Among the various possible ways of allocating given resources, the scheme which will minimize or maximize a specified objective function is chosen. The general linear programming problem may be stated as follows: given a linear set of inequalities or equations, find the values of variables which minimize or maximize a given linear objective function while satisfying some given linear constraints. In other words, a linear program determines the values of the r decision variables x_i (i = 1, 2, ..., r) which will maximize (or minimize) the objective function z, subject to m constraints. Mathematically, this is simply presented as minimize (or maximize)

$$z = \sum_{j=1}^{r} c_j x_j \tag{1-1}$$

subject to the constraints

$$\sum_{i,j} a_{i,j} x_j \ \{\geq, =, \leq\} \ b_i, \ i = 1, ..., m \tag{1-2}$$

where, for each constraint, one of the signs {≤, =, ≥} holds and $a_{i,j}$, b_i, and c_j, are known constants. A set of x_j which satisfies the constraints is called a feasible solution. The feasible solution which yields the optimal value of the objective function is called the optimal solution. The objective is to select that particular solution or set of decision variable values which will optimize the objective function, subject to the specified constraints. The set of elements in x is the decision variables of the problem. In the optimization models, for example, typical decision variables are represented as withdrawal (or pumping) rates at public supply locations in the ECF model region. When c_j is the cost associated with the x_j, then z represents the total cost from operating the system at the activities x_j. Once a problem has been stated in standard linear programming form as in Equations 1-1 and 1-2, the usual algebraic procedure for solving it is the Simplex method (Dantzig 1963). Several software programs are available for solving linear programming by modified and advanced Simplex methods. The primary output of any optimization model is the optimum objective function value and the corresponding values of the decision variables.

WATER RESOURCE OPTIMIZATION

Deficit and decision models were developed to investigate the optimal allocation of groundwater to meet year 2020 demand in east-central Florida. These models were developed to investigate future water allocation strategies that meet or exceed projected water service area demands but do not exceed available water resource supplies. It was assumed that adverse environmental effects could be minimized at specific locations by constraining hydraulic head changes (i.e., drawdown) and spring discharge losses to meet specified environmental goals. The models incorporated control points at which groundwater-level changes were constrained. These points were in areas where native vegetation could be harmed by declines in the surficial aquifer through long periods of water withdrawals.

The models were developed using data generated from numerical simulation models (e.g., information describing aquifer responses to changing stresses such as pumping). The data exist at a spatial discretization scale consistent with the flow simulation model used to develop the pertinent data for the formulations of the optimization models. Information on needs and sources was also included in the formulation of each model. Thus, elemental discharge rates and hydraulic heads given by each model correspond to elemental cumulative discharges (from wells located in a grid cell) and elemental average hydraulic heads in associated cells defined in the flow simulation model. The formulation of the deficit model includes only decision variables from public supply wells. In addition to decision variables from public supply wells, the formulation of the decision model includes additional decision variables from alternative sources such as potential new wells, surface water, and interconnects. Two main objective functions are used in the development of two optimization models: (1) the deficit model simply minimizes the sum of deficits and (2) the decision model minimizes the cost associated with the operation of existing wells, potential new wells, interconnects, and the use of surface water sources.

THE DEFICIT MODEL

The deficit model is used to study the likelihood of the occurrence of deficit during / after water allocations (or requests). SJRWMD relies on the use of historical data to decide on the quantity of water that must be allocated to a user. In view of this, SJRWMD has realized the need to evaluate the appropriateness of the quantity of water allocated to a user. Therefore, the objective for developing the deficit model is to assist in the evaluation of the appropriateness of the quantity of water allocated as a result of a request made by a user. The deficit model simply takes the allocated water quantity for a demand area and determines whether this area could run into water deficit for the given time horizon. If the deficit model estimates a zero deficit for this area, it is understood that the allocation made was appropriate. If, on the other hand, the model estimates a non-zero deficit value, it is concluded that the area is projected to use too much water. To ensure a zero deficit in this case, the allocated value must be reduced by the deficit value of the demand area in question.

For example, if a demand area *j* has a projected water demand allocation of DM_j for a given time horizon, the deficit model uses this projected value to estimate a deficit QD_j. If QD_j > 0, then the actual projected allocation, AA_j, required to ensure zero deficit for demand area *j* may be computed as

$$AA_j = DM_j - QD_j \tag{1-3}$$

If the model estimates a zero deficit, that is, QD_j = 0, then from Equation 1-1, the projected demand allocation DM_j is just enough for demand area *j* for the given time horizon. Therefore, subject to the appropriate constraints, the objective of the deficit model is to minimize potential deficits that could develop due to projected allocations.

THE DECISION MODEL

The decision model offers a means of combining other alternative sources of water available to meet the needs of the user in a cost-effective manner. These alternative water sources include (a) new freshwater wells in other locations of the ECF model region, (b) the use of interconnections from a source area to a destination area, and (c) surface water sources along the St. Johns River. The decision model attempts to minimize the total cost of setting up these sources under hydrological, environmental, and political constraints to identify a suitable combination of the aforementioned sources that can best be suitable to balance the accrued water deficits.

PREVIOUS STUDIES

Several earlier studies using the combined linear programming optimization / simulation model approach are of note. Burger and others (1995) investigated the Volusia subregional model and incorporated water quality constraints using the DSTRAM model (for developing water quality influence coefficients) developed by Geraghty & Miller (1991). Demas and Burger (1995) developed an optimization model for the ECF region. Optimization for the east-central and Volusia regions incorporating mixed-integer programming was performed by Demas and others (1998) in a previous study for SJRWMD, but this study did not incorporate an active nonlinear surficial layer of the east-central region or specific drawdown limits for sensitive wetland areas. The 1998 study served as the basis for present model development.

HYDROGEOLOGIC FRAMEWORK

The ECF model (McGurk and Presley 2002) includes all of Orlando and all or part of Lake, Seminole, Brevard, Volusia, and Osceola counties. The source for nearly all water supply within the study area is the Floridan aquifer system, one of the world's largest and most productive carbonate aquifers, consisting of two distinct producing zones called the Upper Floridan aquifer and the Lower Floridan aquifer. These two aquifers are separated by a layer of dolomitic limestone known as the middle semiconfining unit. Some locations in the Upper Floridan aquifer are artesian, giving rise to 23 springs in the model area. The water quality of the Upper Floridan aquifer is generally better than that of the Lower Floridan aquifer. The Upper Floridan aquifer is the primary source of drinking water in the east-central region.

GROUNDWATER FLOW MODEL

The groundwater flow simulation model used to develop the unit response matrix (influence coefficients) for the optimization models is MODFLOW (McDonald and Harbaugh 1988). This model was selected because it has been used and verified extensively in many groundwater flow applications. The governing equation describing steady-state movement of an incompressible fluid through a porous media is

$$\frac{\partial}{\partial x}(K_{xx}\frac{\partial h}{\partial x}) + \frac{\partial}{\partial y}(K_{yy}\frac{\partial h}{\partial y}) + \frac{\partial}{\partial z}(K_{zz}\frac{\partial h}{\partial z}) + W = 0 \qquad \textbf{(1-4)}$$

where

K_{xx} = hydraulic conductivity along the x-axis [LT^{-1}],
K_{yy} = hydraulic conductivity along the y-axis [LT^{-1}],
K_{zz} = hydraulic conductivity along the z-axis [LT^{-1}],
h = potentiometric head [L], and
W = sink or source term [T^{-1}].

The components of the hydraulic conductivity are assumed to be parallel to the principal axes of the hydraulic conductivity tensor. The W term includes external volumetric flux per unit volume, area recharge, discharge from wells, springs, etc. MODFLOW is a single density fluid model and cannot account for variable density situations (e.g., salt water intruding into freshwater). In view of this, the current optimization models do not directly consider water quality issues.

GROUNDWATER FLOW SIMULATION MODEL

A three-dimensional, finite-difference groundwater simulation model for east-central Florida was developed by McGurk and Presley (2002). Specified head, head-dependent flux, and zero flux boundary conditions were employed. The model simulates spatially variable evapotranspiration, recharge, discharge, head-dependent spring flows, and outflow to agricultural and public supply utility wells. The finite-difference grid consists of 174 rows by 194 columns and 4 layers, resulting in a total of 135,024 cells of which only 99,381 are active, due to boundary condition configurations. The grid cells are 2,500 feet by 2,500 feet throughout the model domain. The model grid and the associated initial and boundary conditions used to estimate hydraulic heads in the ECF model region are shown in Figure 1-1. Figure 1-2 shows the recharge distribution of the Floridan aquifer system that contributes the spring flow and many other desirable hydrologic features of the ECF model region. The estimated hydraulic heads at the wetland control points are used to generate the unit response matrix (or influence coefficients). The unit response matrix is subsequently used in the optimization models to estimate drawdowns (or water level changes) at the wetland control points. With these estimated drawdowns, the drawdown limits set by management at these points could be checked and enforced in the models.

SUMMARY

This chapter has briefly introduced the concept and application of optimization models for management decision purposes. Two optimization models are discussed. The first is a deficit model that has the objective of minimizing the sum of resource allocation deficits. The second is a decision model that has the objective of minimizing costs associated with the construction of new wells, the use of surface water sources, and interconnects as alternative water resource sources. Also presented is the steady-state equation governing the computation of hydraulic heads to be used for calculation of the influence coefficients. These influence coefficients are used by the deficit and decision models to predict drawdowns and hydraulic heads at wetland control points and spring locations. The formulation of the deficit model is presented in chapter 2 and applied to the ECF model region in chapter 3. The decision model is formulated and applied in chapters 4 and 5, respectively.

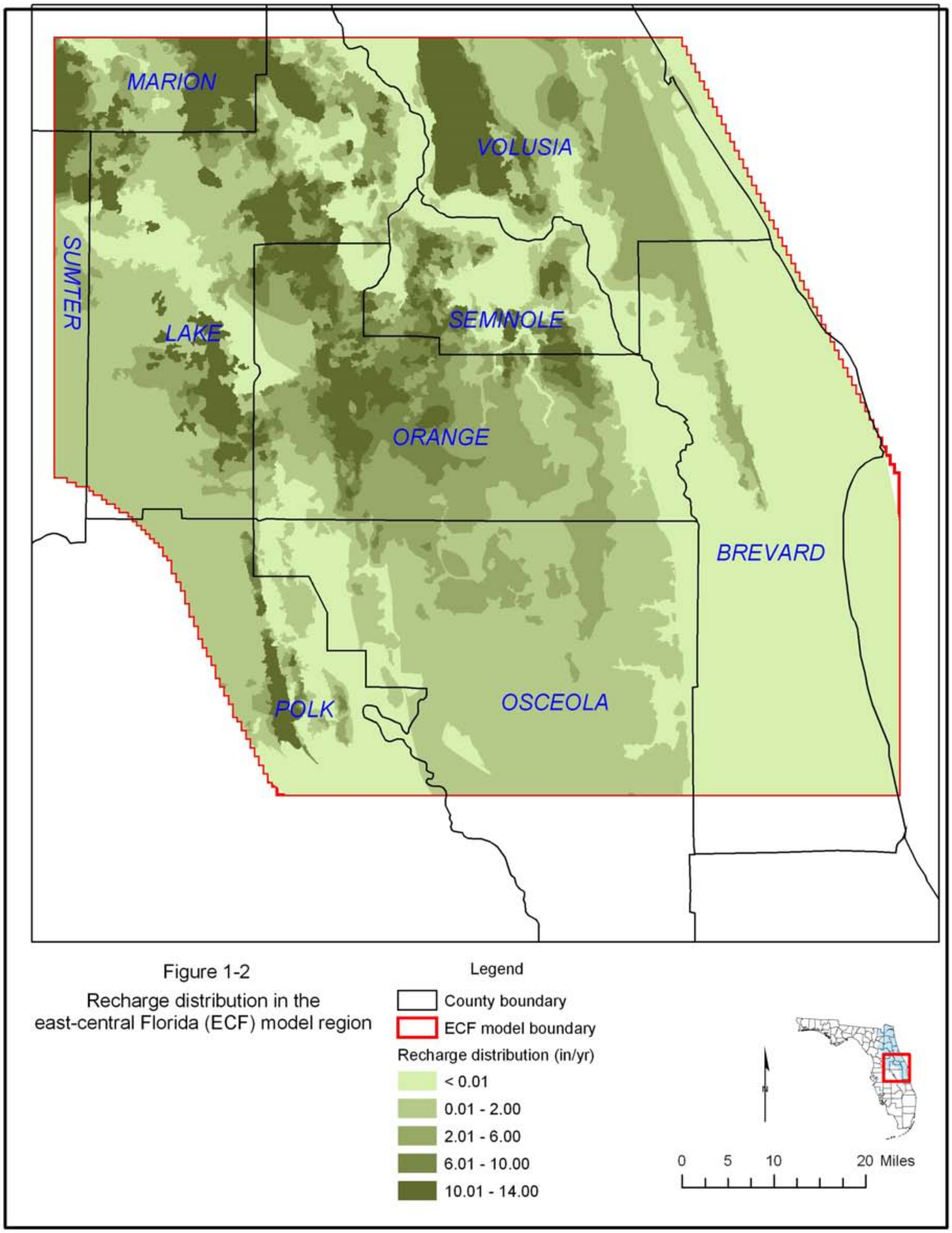

Figure 1-2
Recharge distribution in the east-central Florida (ECF) model region

DEFICIT MODEL FORMULATION

Optimization models were developed to investigate optimum water allocation schemes to meet projected year 2020 demands. The models consist of a collection of algebraic expressions that define an objective function. The objective function is minimized or maximized subject to the behavior of a number of state variables expressed as constraints. The models are built under the assumption that adverse environmental effects could be minimized at specific locations (control points) by constraining hydraulic head changes (drawdowns) and spring discharge losses to meet specified environmental goals.

The process of optimization design involves a sequence of decisions between alternatives. Alternatives are established so that they meet the technical objectives of the project. In most projects, it is necessary to meet these objectives within a set of technical, legal, political, or economic constraints. Optimization alternatives are differentiated from one another on the basis of their technical components. The variables that can be used to define and differentiate alternatives are known as the decision variables. Decision variables may take on discrete values, giving rise to discrete alternatives, or they may be continuous functions, giving rise to a continuous range of alternatives. A typical example of decision variables used in the model is the well pumping rates.

In general, management optimization problems consist of an objective function and an associated set of constraints. As with any model, such a management tool is only an approximation. It does not capture every element of a real world system in its simple formulation, but rather it represents the most important features of the problem. Frequently, after one has formally gone through the exercise of problem formulation, the solution may be apparent, or it may become obvious that there are severe conflicts that call for reformulation and redesign until a reasonable solution is achieved.

In any optimization problem, one begins a formulation process by defining the following:

Objective function—the goal that is minimized or maximized (e.g., total cost, total deficit, risk).

Decision variables—unknown quantities that can be managed or controlled (e.g., pumping rates).

Constraints—restrictions that must be obeyed in the final design (e.g., local drawdowns or drawdowns must not go beyond a certain limit).

State variables—quantities that define the state of the system (e.g., head). State variables are usually functions of decision variables. In other words, the computation of state variables is directly dependent on the values of the decision variables.

LINEAR SYSTEMS APPROACH

The simultaneous management and simulation of groundwater for linear systems is established using two different methods: (1) embedding method and (2) response matrix approach. In both methods, a groundwater simulator is included as part of the constraint sets of the linear optimization problem. The difference between the two stems from the manner in which the flow simulator is represented.

In the embedding approach, a groundwater flow model is called each time the optimization algorithm needs to evaluate the cause / effect relationships between a given set of decision variables and state variables. That is, the flow model is 'embedded' in the optimization problem along with other constraints that restrict local hydraulic heads, gradients, and pumping or recharge rates. The solution to such a problem gives the optimal pumping and injection rates plus the simulated hydraulic heads at every finite-difference or finite element cell or node. Thus, the embedding method is not efficient if one is only interested in the hydraulic heads at a few locations within the simulation region or when a large number of forward simulations is required by the optimization algorithm.

In the response matrix approach, the groundwater flow model is not included in the constraint set, but rather, unit solutions to the flow equations are developed and linearly superposed (added together) to simulate any configuration of pumping or recharge. Each unit solution describes the response in terms of the change in head at selected observation points due to a pulse of pumping at a particular location. The responses are assembled into a response matrix. The elements of the unit response matrix are called the influence coefficients. Hence, the unit response matrix is sometimes referred to as the influence coefficients matrix. Since the responses are only recorded at certain critical locations of interest in managing the system, the response matrix is used to predict the state variables only for these locations. In view of this property, the response matrix approach emerges as the better of the two in situations where a solution of hydraulic heads for all nodes or cells within the discretized region is not required.

In the ECF region, one of the major concerns is to ensure that the drawdowns within the wetland control points do not exceed a certain specified limit, indicating that only the heads at the wetland control points are required to be estimated. In view of this, the unit response matrix approach is used in developing both the deficit and the decision optimization models.

BRIEF THEORY OF THE UNIT RESPONSE MATRIX

Apart from the primary output of the optimum objective function and its decision variables (optimum well flow rates), the deficit model also computes the drawdown corresponding to current flow rates at each iterative step until an optimum solution is found. Computation of drawdowns is required to check against the drawdown constraints imposed at the wetland control points.

In reality, the practical way of computing drawdown is to pass the current flow rates evaluated by the optimizer to a forward simulation model (which is MODFLOW, in our case) to compute heads and subsequent drawdowns at the wetland control points. If the optimization model has to go through 5,000 iterative steps to get to the optimum solution, then the forward simulation model will have to be executed 5,000 times. Considering the fact that MODFLOW takes about 3 minutes to run the steady-state version of the ECF model, one will have to wait for about 3 weeks for the results of the deficit model. This is highly impractical and inefficient, especially if a number of other scenarios are waiting to be analyzed with the same deficit model. The most expedient solution to this dilemma is through the use of the unit response matrix.

The coefficients in the unit response matrix are simply the drawdowns corresponding to a unit change in stress applied to the simulation region. Consider the points $v = 1,2,\ldots,p$ as locations of interest in a simulation region. Also assume that the initial hydraulic heads at each of these points is h_v. If a single stress of magnitude 1 unit is applied at a given point in the same simulation region to result in a final head f_v, then the unit response at each point v, r_v is $r_v = f_v - h_v$. Knowing this response, if a stress of Q units is applied at the same location as the previous unit stress, then the expected response at a point *v*, rq_v, may be evaluated as $rq_v = Q*(f_v - h_v)$. It is important to note here that this response is evaluated based on the history of its unit response. It does not call for the rerun of the forward simulation model for the new stress being experienced at the same location.

Now let *n* stresses, denoted by $w = 1,2,\ldots,n$, be acting simultaneously at some locations in the same simulation region. Then each of these stresses will exhibit a unit response at each of the points of interest $r_{v,w}$. The value $r_{v,w}$ may be interpreted as the unit response at location *v*, owing to unit stress *w* acting at some point within the simulation region. For the entire number of unit stresses acting simultaneously, there will be *p* x *n* number of unit response values in the unit response matrix. If this two-dimensional unit response matrix is represented by *U*, then each unit response coefficient, $r_{v,w}$, can conveniently be represented in *U* as

$$U = \begin{bmatrix} r_{1,1}, r_{1,2}, \cdots, r_{1,n} \\ r_{2,1}, r_{2,2}, \cdots, r_{2,n} \\ \vdots \\ r_{p,1}, r_{p,2}, \cdots, r_{p,n} \end{bmatrix} \qquad \textbf{(2-1)}$$

If the unit stresses are replaced by set a of stresses $q_1, q_2, \cdots, q_n$, then these stresses may be represented by the vector *Qv*, the contents of which may be expressed as

$$Qv = \begin{bmatrix} q_1 \\ q_2 \\ \vdots \\ q_n \end{bmatrix} \qquad \textbf{(2-2)}$$

If the total response as a result of the elements of *Qv* acting together at the *p* locations is represented by $t_1, t_2, \cdots, t_p$ in a vector *T*, then by the principle of linear superposition, the contents of *T* may be evaluated as

$$T = \begin{bmatrix} t_1 \\ t_2 \\ \vdots \\ t_p \end{bmatrix} = UQv = \begin{bmatrix} r_{1,1}, r_{1,2}, \cdots, r_{1,n} \\ r_{2,1}, r_{2,2}, \cdots, r_{2,n} \\ \vdots \\ r_{p,1}, r_{p,2}, \cdots, r_{p,n} \end{bmatrix} \begin{bmatrix} q_1 \\ q_2 \\ \vdots \\ q_n \end{bmatrix} \tag{2-3}$$

Using the concept of linear superposition, the total response at, say, $p = 1$, is obtained from Equation 2-3 as

$$t_1 = q_1 r_{1,1} + q_2 r_{1,2} + \cdots + q_n r_{1,n} \tag{2-4}$$

The concept of the linear superposition is relatively simple: if one adds up the component influences on hydraulic heads due to all hydraulic stresses (including initial and boundary conditions), then the behavior of the entire system is reproduced. In a similar reasoning, the total response at any given point p at the location of interest can be expressed from Equation 2-3:

$$t_p = q_1 r_{p,1} + q_2 r_{p,2} + \cdots + q_n r_{p,n} \tag{2-5}$$

The basic concept to grasp here is that the unit response matrix U (containing the influence coefficients) is all that is needed to make predictions for different values of stresses. However, it must be understood that when the locations of the stresses and / or the location of the points of interest change, the entire unit response matrix will have to be regenerated to accommodate this change. Thus, the unit response matrix is evaluated only for the computation of responses at some specified locations as a result of the action of some stresses at other specified positions in the simulation region. The application of the principle of linear superposition assumes that the response of the aquifer due to stresses is linear. In view of the limitation of the principle of linear superposition to handle only linear situations, the known nonlinear response of the surficial aquifer system due to external stress (such as recharge and pumping) is assumed to be linear in this development.

RESPONSE MATRIX FOR THE DEFICIT MODEL

There are 224 control points in the ECF model region where drawdown limits are strictly controlled. There are 620 public supply wells, the unit responses of which need to be evaluated at each of the 224 control points. Therefore, the size of the two-dimensional unit response matrix for the computation of heads and drawdowns at the control points is 224 x 620. The unit response matrix is evaluated only once outside the optimization model using the steady-state ECF flow model. Generally, the unit response at location *h* due to a unit stress at location *i* is defined mathematically as

$$\alpha_{i,h} = \frac{hi_{i,h} - hf_{i,h}}{(Q95_i + \Delta Q_i) - Q95_i} = \frac{\Delta h_{i,h}}{\Delta Q_i} \qquad \textbf{(2-6)}$$

where

$hi_{i,h}$ = the initial hydraulic head predicted at control point *h* due to 1995 flow rate $Q95_i$ at public supply well *i*,
$hf_{i,h}$ = the final hydraulic head as result of the perturbation of $Q95_i$ by an amount ΔQ_i, and
$\Delta h_{i,h}$ = the change in hydraulic head at location *h* as a result of a perturbation in pumping rate at well *i* by an amount ΔQ_i.

The algorithm for the generation of the unit response matrix can be summarized as follows:

1. Using 1995 flow rates at the public supply wells, compute the hydraulic heads for all of the 224 control points.

2. Consider public supply well *i* and perturb its 1995 flow rate $Q95_i$ by an amount ΔQ_i.

3. Compute the heads (using MODFLOW) at all control points due to this new flow rate $(Q95_i + \Delta Q_i)$ while keeping the others the same as their original values.

4. Consider a control point *h* and evaluate Equation 2-6. Store the results in matrix *U* at location $U_{i,h}$.

5. Increase *h* to *h + 1* and go to step 4 if $h \neq 224$.

6. Reset $(Q95_i + \Delta Q_i)$ to $Q95_i$.

7. Increase *i* to *i+1* and go to step 2 if $i \neq 620$.

It must be mentioned that the unit response matrix approach is an approximate technique. However, this approximation is typically more accurate whenever small perturbations in pumping rate values are required. In view of this, a formal evaluation of the accuracy of prediction of hydraulic heads at sensitive locations is presented in this report.

DRAWDOWN COMPUTATION WITH RESPONSE MATRIX

Computation of drawdown is executed in the optimization models by the direct application of Equation 2-7:

$$D_h = \sum_{i=1}^{n} \alpha_{i,h}(Q_i - Qno_i) = \alpha_{1,h}(Q_1 - Qno_1) + \alpha_{2,h}(Q_2 - Qno_2) + ... + \alpha_{n,h}(Q_n - Qno_n) \quad \textbf{(2-7)}$$

where

D_h = drawdown at control point *h*,
Q_i = optimized well withdrawal rate at well *i*,
Qno_i = non-optimized well withdrawal rate at well *i*,
$\alpha_{i,h}$ = unit drawdown response value from well *i* acting at control point *h*, and
n = number of public supply wells.

GENERAL ALGEBRAIC MODELING SYSTEM

The general algebraic modeling system (GAMS) (Brooke et al. 1992) is an optimization coding environment within which optimization models can be compiled and executed. The coding process for GAMS can be accomplished in any text editor. After successful compilation, GAMS may call a solver of the user's choice to solve the optimization problem that he or she has formulated. Solvers designed to solve optimization problems, in general, require the formulated objective function and the constraints to solve the problem. If the solver finds a feasible solution, its output will generally consist of the optimum objective function value and the values of the decision variables that yield the optimum solution.

DEFICIT MODEL INDICES

The deficit model incorporates indices to develop arrays and matrices for the variables used in developing the optimization code. The current indices include *h*, the set of control points where drawdowns are constrained; *i*, the set of public supply utility wells or well grid cells; *j*, the set of public supply utility demand areas; and *l*, the set of springs.

DEFICIT MODEL PARAMETERS

The deficit model also uses parameters that include the hydrogeologic information for aquifer responses, well information, and service maps for sources serving demand areas. In GAMS, parameters are recognized as either known data or data that will be computed somewhere within the code before the optimization iteration begins.

DEFICIT MODEL VARIABLES

An optimization model requires the use of variables to define equations describing significant hydrogeologic processes. The general variables include hydrogeologic and water management relationships that are controlled through changes in the decision variables. In GAMS, variables are recognized as data, the value of which at any iterative step is determined by the optimization algorithm. The decision variables of every optimization system are always declared as variables in GAMS.

OBJECTIVE FUNCTION OF THE DEFICIT MODEL

As already explained, the deficit model is used to evaluate the appropriateness of the amount of water resource that has been allocated. For example, if an allocation of DM_j is issued to a demand area *j*, and the deficit model comes out with a non-zero deficit of QD_j for this area, then the actual allocation to be made in order to ensure a zero deficit for this demand area will depend on the value of QD_j. If $QD_j = 0$, then the original amount allocated is accepted as the appropriate true allocation for the demand area *j*. On the other hand, if $QD_j > 0$, then the actual amount that should be allocated to ensure a zero deficit must be $DM_j - QD_j$. The deficit model minimizes the sum of water supply resource deficits while meeting environmental impact constraints. Its objective function, *Z*, is expressed as minimize

$$Z = \sum_j QD_j \tag{2-8}$$

QD_j is a non-negative variable in the optimization model. The constraints binding the minimization on Equation 2-8 are outlined in the following sections.

DEFICIT MODEL CONSTRAINTS

The deficit model is developed to allow for the maximum use of available groundwater for public supply purposes while meeting specified constraints. These constraints are specific restrictions on environmental impacts or limits on water supply components. The constraints incorporated into the deficit model include limits on surficial aquifer drawdown, limits on spring discharge reduction, and limits on well withdrawal rates.

DRAWDOWN CONSTRAINTS

The purpose of the drawdown constraints is to ensure protection of lakes and wetland communities within the surficial aquifer that can be impacted by withdrawals from the Floridan aquifer. Studies have been conducted to provide estimates of allowable drawdown that would limit, avoid, or reduce the extent of the projected future impacts on these systems (CH2M HILL 1998a). Drawdown constraints at the specified control points are implemented using the unit response matrix that describes hydraulic head changes at each control point due to groundwater pumping at each well grid cell. The drawdown at a control point, *h*, is computed as

$$D_h = Ho_h - Hno_h + \sum_i \alpha_{i,h} [QP_i - Qno_i] \tag{2-9}$$

where

Ho_h = estimated hydraulic head for the base year 1995 at control point *h*,

Hno_h = estimated hydraulic head for projected year 2020 at control point *h*, and

QP_i = total withdrawal rate at public supply well *i*.

The total withdrawal rate, QP_i, from a public well, *i*, consists of the sum of withdrawal rates serving all public demand areas,

$$QP_i = \sum_j Q1_{i,j}$$

where

$Q1_{i,j}$ = pump rate at public well cell *i* supplying demand area *j*.

For the ECF region, a drawdown limit is specified at each control point *h*. Figure 2-1 shows the wetlands and locations of the control points where constraints are applied. Currently, the drawdown limits imposed at the control points range between 0.4 foot and 0.9 foot. The drawdown constraint at these control points is expressed in the model as

$$D_h \le D\lim_h \tag{2-10}$$

where

$D\lim_h$ = drawdown limit imposed at control point *h*.

SPRING CONSTRAINTS

Similar to the wetland drawdown constraint, the spring constraints incorporate influence coefficient $\gamma_{i,l}$ that describes hydraulic head changes at spring locations *l* due to groundwater pumping at each well grid cell *i*. Twenty-three springs exist and are simulated in the deficit model. Eight of these springs have established minimum flows and levels (MFLs). The MFLs are best defined as a set of minimum hydrologic criteria or statistics that will limit the shift in the existing hydrologic regime to maintain a minimum hydrologic regime that serves to prevent adverse and significant impacts to the water resource (CH2M HILL 1998b). To incorporate these mandated MFLs, spring constraints have been developed in the deficit model to limit the change in hydraulic heads at the spring location to a level not less than the established MFL. If no MFL exists, the spring head change is constrained to a 15% allowable loss. Table 2-1 summarizes the spring flow data used in the deficit model. The spring locations of these springs are shown in Figure 2-2.

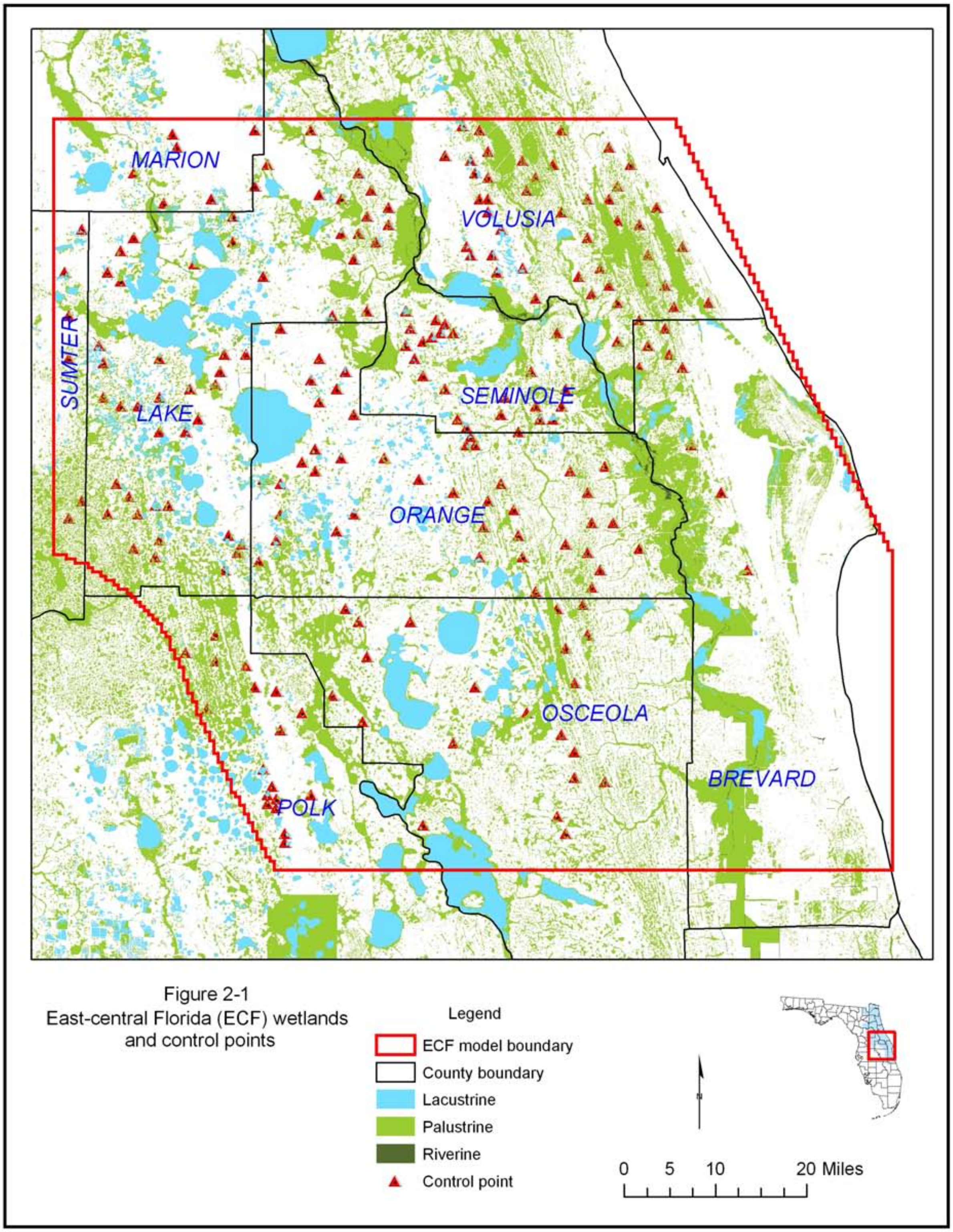

Figure 2-1
East-central Florida (ECF) wetlands and control points

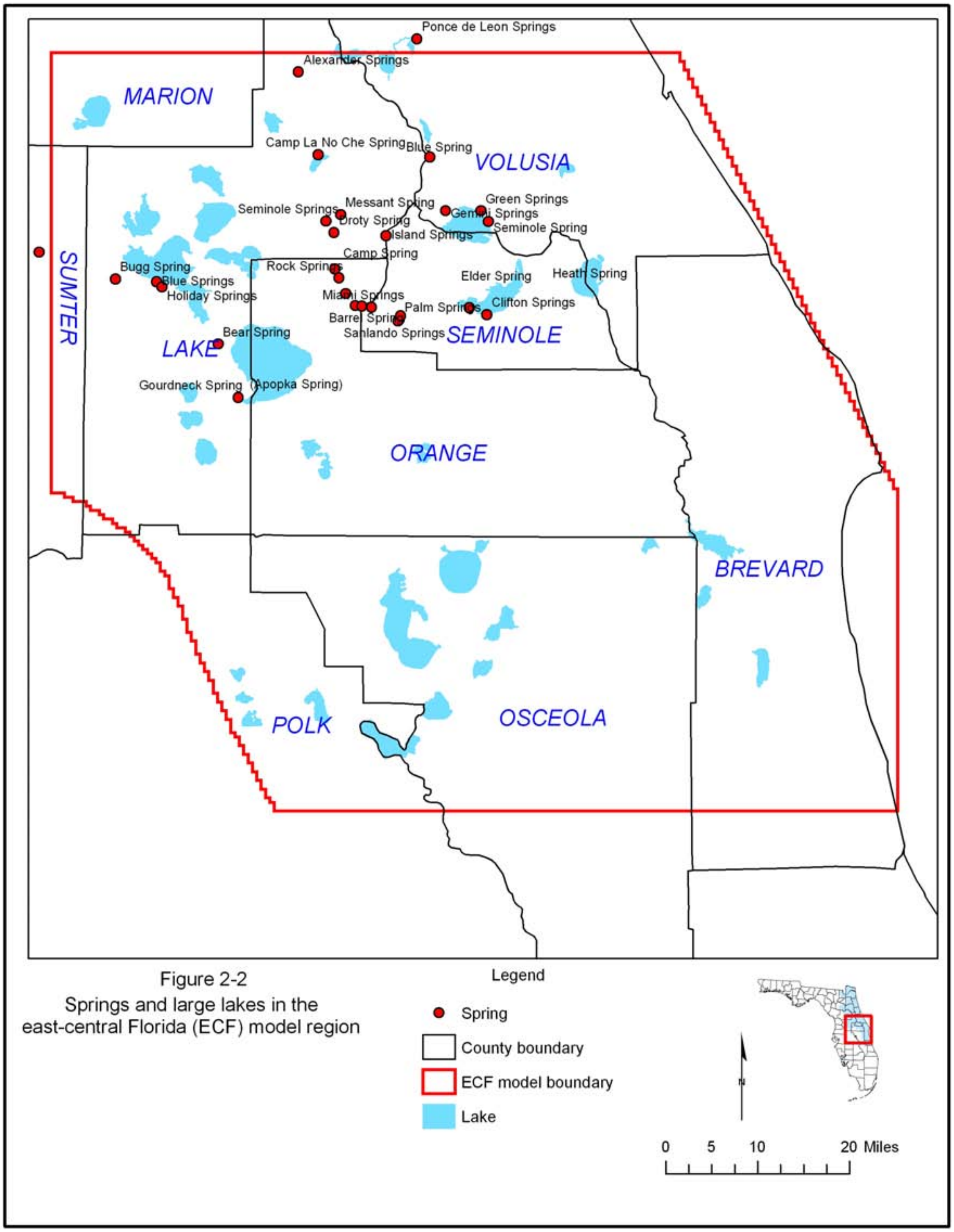

Figure 2-2
Springs and large lakes in the
east-central Florida (ECF) model region

Table 2-1. Spring flows and constraint levels for modeled springs

Spring	Estimated 1995 Flow (base case) [cfs]	Predicted Average 2020 Flow (base case) [cfs]
Blue (Volusia County)	150.4	137.8
Alexander	102.4	101.8
Wekiva	73.0	65.5
Rock	61.4	52.5
Seminole	38.9	34.0
Alexander Creek	33.0	32.8
Apopka	32.2	24.7
Palm and Sanlando	28.2	20.7
Messant	16.4	15.1
Starbuck	14.9	11.6
Bugg	10.7	10.2
Gemini	8.1	7.3
Island	6.4	5.8
Miami	6.2	5.4
Blue (Lake County)	3.2	1.9
Holiday	3.1	1.1
Witherington	2.2	1.9
Green	1.9	1.7
Clifton	1.7	1.5
Camp La No Che	1.0	0.9
Lake Jesup	1.0	0.9
Sulphur	0.8	0.7
Droty	0.7	0.7

Hydraulic head at a given spring location l, Hs_l, is computed by evaluating the expression

$$Hs_l = Hsno_l - \sum_i \gamma_{i,l} [QP_i - Qno_i] \qquad \textbf{(2-11)}$$

where

$Hsno_l$ = estimated hydraulic head for projected year 2020 at spring l, and

$\gamma_{i,l}$ = response at spring l due to a unit stress at public supply well i.

The spring's fraction of simulation year 1995 discharge, Ps_l at spring location l, is given by the expression

$$Ps_l = \frac{Hs_l - El_l}{Hso_l - El_l} \quad \textbf{(2-12)}$$

where

El_l = elevation head of spring *l*, and
Hso_l = estimated hydraulic head for base year 1995 at spring *l*.

If no MFL has been specified as the spring discharge rate, it must exceed 85% of the 1995 discharge rate. This constraint is expressed in the model as

$$Ps_l \geq 0.85 \quad \textbf{(2-13)}$$

The spring discharge, SD_l at spring location *l*, may be defined as

$$SD_l = Cd_l(Hs_l - El_l) \text{ if } Hs_l > El_l \quad \textbf{(2-14)}$$
$$SD_l = 0 \text{ if } Hs_l \leq El_l$$

where

Cd_l = spring conductance at location *l*.

If the MFL discharge for a spring is available, spring discharge must exceed this critical minimum. This constraint is expressed as

$$SD_l \geq Scrit_l \quad \textbf{(2-15)}$$

where

$Scrit_l$ = the critical minimum discharge at spring *l*.

WITHDRAWAL RATE CONSTRAINTS

Existing and proposed wells are constrained to withdrawal rates that reflect realistic pumpage from these systems. This constraint places a maximum pumping rate (capacity) for all existing wells at 125% of the projected 2020 rates. The public supply well capacity limits were obtained through direct correspondence with water supply utilities in the ECF region and from the literature.

The withdrawal rate, QP_i, at well *i* must not exceed its capacity, $Mcap_i$. This requirement is guided by the following constraint:

$$QP_i \leq Mcap_i \tag{2-16}$$

The demand constraint for public supply areas requires that the supply from the wellfields satisfy the requirements of the demand area *j* in question. A well from wellfield source *i* may be mapped to a demand area *j* by a service location map $Servm_{i,j}$. $Servm_{i,j}$ is 1 if well *i* is connected to demand area *j*. The demand constraint is then expressed in the deficit model as

$$\begin{aligned} &\sum_i Q1_{i,j} Servm_{i,j} + QD_j \geq DM_j \\ &QD_j \geq 0 \end{aligned} \tag{2-17}$$

where

DM_j = minimum demand at public demand area *j*.

The value of DM_j for a demand area *j* is obtained as a summation of projected allocations of all wells mapped to this demand area. As much as practicable, an attempt is made to ensure that wells mapped to a given demand area belong to a common owner. This measure is adopted to avoid confusion in potential water use litigation issues.

EQUITY CONSTRAINTS

Generally, equity may be defined as fairness in the patterns of distribution of costs and benefits. The ECF deficit model has shown that small utilities get penalized in their projected water demands due to huge demands by large utilities in the region. To ensure fairness in projected water allocations for all utilities, both small and large, additional water withdrawal limit constraints is incorporated in the optimization models. These constraints are termed the equity constraints. The equity constraints permit additional percentage, *x*, of water to be pumped by all utilities provided the constraint in Equation 2-18 is not violated. The value, *x%*, is a calibrated quantity whose final value for a given datum is obtained just before the solution becomes infeasible as a result of perturbations in x. For the 2020 water use data, the value of x was determined to be 40%. This equation is expressed as

$$\sum_{i=1}^{620} Q1_{i,j} * Servm_{i,j} \geq \frac{x}{100} * DM_j \qquad \textbf{(2-18)}$$

where

$Q1_{i,j}$ = current withdrawal at well i by demand area j, and
DM_j = 2020 demand at demand area j.

DEFICIT MODEL FORMULATION SUMMARY

The objective of the deficit model is to minimize

$$Z = \sum_j QD_j \qquad \textbf{(2-19)}$$

subject to the following constraints:

- Drawdown constraints

$$D_h = Ho_h - Hno_h + \sum_i \alpha_{i,h}[QP_i - Qno_i] \qquad \textbf{(2-20)}$$

$$D_h \leq D\lim_h \qquad \textbf{(2-21)}$$

- Spring constraints

$$Hs_l = Hsno_l - \sum_i \gamma_{i,l}[QP_i - Qno_i] \qquad \textbf{(2-22)}$$

$$Hs_l \geq Ps_l(Hso_l - El_l) + El_l \qquad \textbf{(2-23)}$$

$$Ps_l = \frac{Hs_l - El_l}{Hso_l - El_l} \qquad \textbf{(2-24)}$$

$$SD_l = Cd_l(Hs_l - El_l) \qquad \textbf{(2-25)}$$

$$SD_l \geq Scrit_l \qquad \textbf{(2-26)}$$

- Public water management constraints (withdrawal rate)

$$QP_i = \sum_j Q1_{i,j} \qquad \textbf{(2-27)}$$

$$QP_i \leq Mcap_i \tag{2-28}$$

$$\sum_i Q1_{i,j} Servm_{i,j} + QD_j \geq DM_j \tag{2-29}$$

- Equity constraints

$$\sum_{i=1}^{620} Q1_{i,j} * Servm_{i,j} \geq \frac{x}{100} * DM_j \tag{2-30}$$

DEFICIT MODELING PROCEDURE

The method used to determine optimum water allocation scenarios is an iterative process and can be divided into the following five steps:

1. Solve the deficit model with first estimates of influence coefficients. The optimization model represents a system of linear equations solved during the first step, using GAMS.

2. Compare optimization model results to MODFLOW simulation predictions. This comparison is achieved by evaluating hydraulic heads at the control points in MODFLOW using the optimum well rates predicted by the deficit model and checking the results against those predicted by the deficit model.

3. If the hydraulic head results are below allowable tolerances, calculate revised set of influence coefficients using different perturbations.

4. Solve the deficit model to obtain the optimized pumping rates with revised influence coefficients.

5. Repeat steps 2–4 until an acceptable level of agreement is reached between the deficit and MODFLOW simulation results.

SUMMARY

The formulation of the deficit model developed for the ECF region for the 2020 water projected year has been presented. The objective of the deficit model is to minimize the sum of deficits subject to a number of constraints. These constraints have been explained and presented. Application of the model to problem-solving is presented in the following chapter.

Deficit Model Runs

This chapter presents the verification, sensitivity, and timing analysis performed with the deficit model. The use of equity constraints as a means of introducing fairness in the allocation of water resources is demonstrated through two deficit runs. The first run is conducted without any equity; the second introduces 40% equity constraints.

Initial Model Input

The most basic data available for generating initial input data applied to the deficit model are the 1995 pumping rate and the projected allocated 2020 pumping rate (*Qno* and *QP*). Initial hydraulic heads (required by the deficit model) at the wetland control points (*Hno* and *Ho*) and spring areas (*Hso* and *Hsno*) are estimated by running MODFLOW using the base 1995 pumping rate and projected 2020 well flow data. In a field-scale modeling scenario, the contents of *Qno* are usually the permitted average daily quantity allocated to (or requested by) the user of the water resource in the ECF model region, and the decision is based on utility and municipality growth studies conducted by SJRWMD. Other factors such as environmental impact are also considered in making up the pumping rate numbers that populate the array parameter *Qno*.

Limitations of the Deficit Model

The three major limitations of the deficit model are as follows:

1. The deficit model is limited to steady-state conditions only, because the forward model used to develop the influence coefficients is of the steady-state type. Proposals have been made and plans are currently under way to consider dynamic conditions of the Floridan aquifer system in future studies.

2. The model is incapable of handling the nonlinear response of the surficial aquifer system due to pumping and recharge. The application of the principle of superposition used to predict heads (and drawdowns) implicitly shows that the model assumes such responses to be linear.

3. The model does not account for water quality issues of the Floridan aquifer system (e.g., saltwater intrusion into freshwater). Preparations are under way to consider water quality aspects in future models.

VERIFICATION OF THE DEFICIT MODEL

Verification of the deficit model is accomplished by comparing its predictive accuracy with that from a standard model. In the current verification work, MODFLOW was used as the standard model since its performance has been documented extensively in the literature. The objective here is to investigate if the hydraulic heads predicted by the deficit optimization model are within some acceptable level of numerical accuracy.

In a model verification, one attempts to verify the performance of the predictive capability of the model in question by comparing its output to that of another model or observed data. The results of the MODFLOW flow model are being used for this purpose because there are currently no observed data for the selected control points and spring locations. Verification with MODFLOW was accomplished by substituting the optimum public supply flow rates predicted by the deficit model into a well file designed to be used in MODFLOW. The results of the heads predicted by MODFLOW (using this well file) at the predefined wetland control points and spring locations were printed out and compared with heads obtained from the deficit model’s prediction. These results are shown in Figures 3-1 and 3-2 for the spring locations and wetland control points, respectively. Figures 3-3 and 3-4 show plots of the residuals between the results of the two models at the spring locations and wetland control points, respectively.

It is observed in the plots that the heads predicted by the deficit model at the wetland control points agree favorably with the corresponding values from MODFLOW. The R^2 values for both regressions are evaluated to be 1, which shows a good correlation between the output of the deficit model and MODFLOW. The residual plots indicate, however, that errors in the order of 10^{-1} and 10^{-2} can be observed for the heads at the wetland control points and spring locations, respectively. The large residuals at the wetland control points are probably due to nonlinear responses to withdrawal rates/recharge in the surficial aquifer. Due to the manner in which the state variables (hydraulic heads) are predicted in the deficit model, nonlinear effects in the surficial

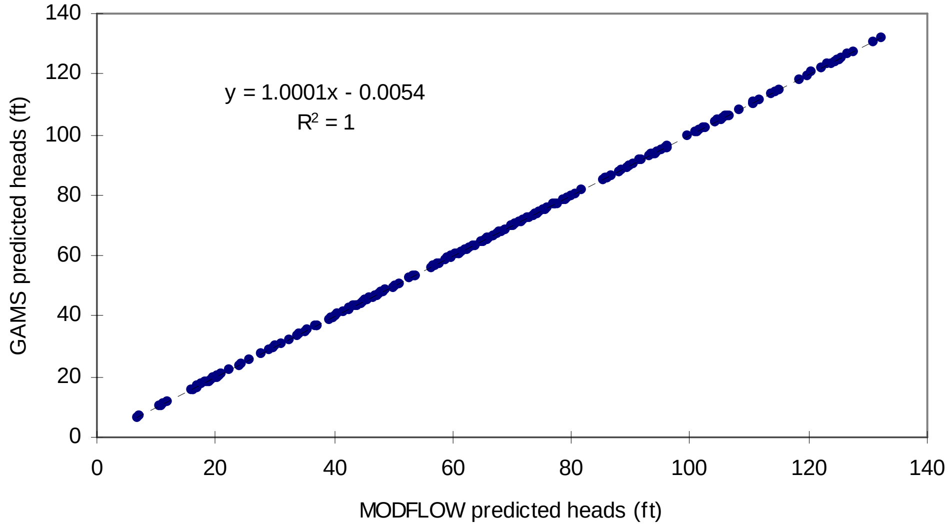

Figure 3.1 Regression of GAMS-predicted heads on MODFLOW-predicted heads at spring locations

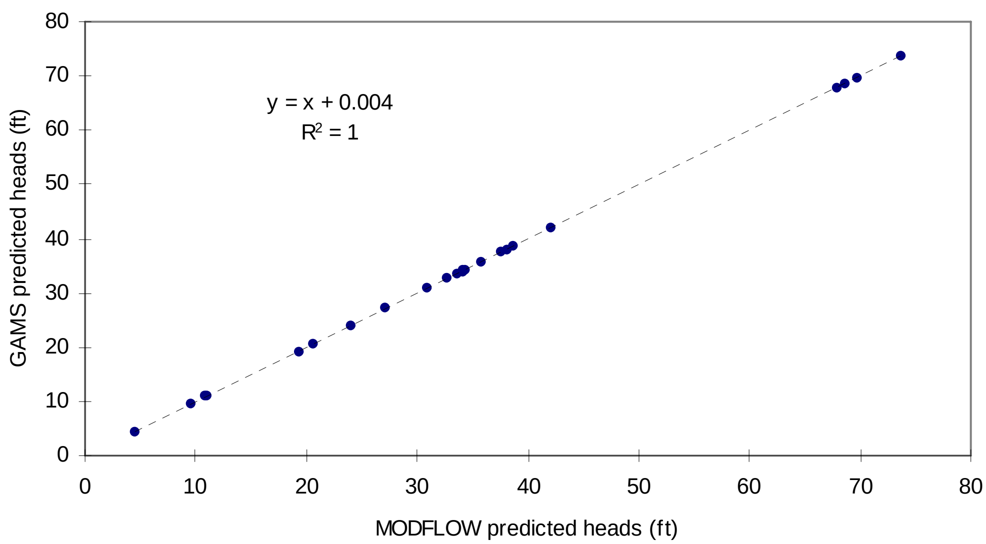

Figure 3.2 Regression of GAMS-predicted heads on MODFLOW-predicted heads at wetland control points

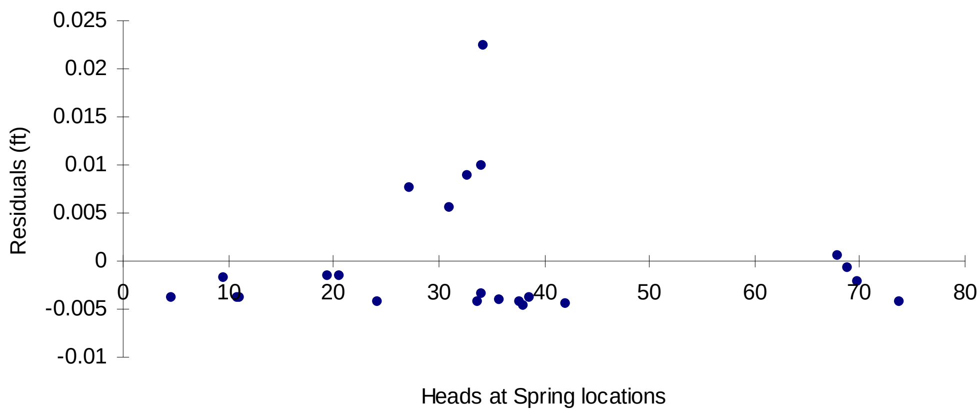

Figure 3.3 Residuals between GAMS- and MODFLOW-predicted heads at spring locations

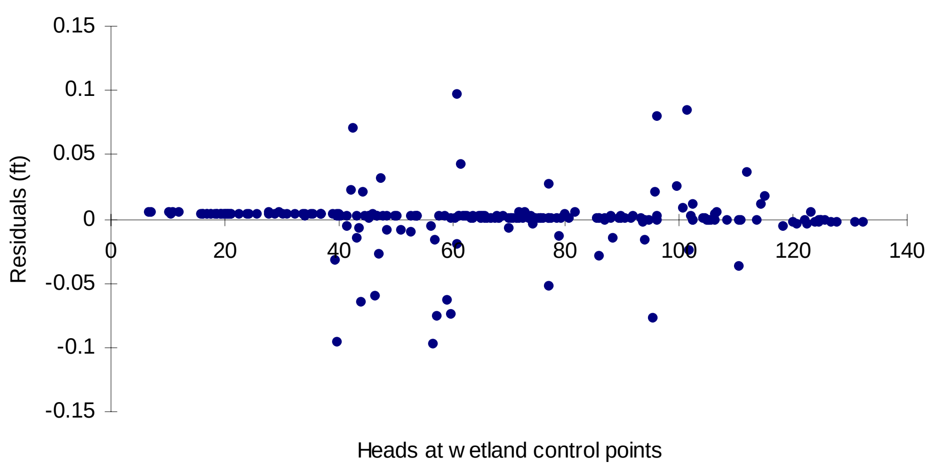

Figure 3.4 Residuals between GAMS- and MODFLOW-predicted heads at wetland control points

aquifer responses are difficult to account for. Another factor that may contribute to these large residuals is the truncation errors associated with the computation of the influence coefficients.

EQUITY RUN

The concept of equity is shown in Table 3-1 through two model runs. A baseline run (without any equity) is first performed, and followed by another run with 40% equity. Table 3-1 gives a brief summary of the results for a selected number of users.

Table 3-1. Summary results of equity in resource allocation

Owner	Owner ID	P2020 (mgd)	DFWE (mgd)	DFAE (mgd)	AAWE (mgd)	AAAE (mgd)
Floribra USA	17	0.55	0.55	0.33	0.00	0.22
Water Oaks Estates	64	0.57	0.57	0.34	0.00	0.23
Legends	32	0.75	0.75	0.45	0.00	0.30
McNamara, John	38	0.83	0.83	0.50	0.00	0.33
Eatonville, town of	15	1.77	1.77	1.06	0.00	0.71
Ocoee, city of	44	6.66	6.66	4.00	0.00	2.66
Winter Garden, city of	65	8.31	8.31	4.99	0.00	3.32
Winter Park, city of	66	16.50	16.50	9.90	0.00	6.60
Apopka, city of	2	26.66	20.04	16.12	6.82	10.74
Seminole County PWD	55	30.22	17.66	14.90	12.56	15.32
Orange County Utilities	45	82.64	30.25	21.26	52.39	61.38
Casselberry, city of	5	7.02	6.04	4.21	0.98	2.81
Southlake Utilities Inc.	56	4.55	0.00	2.73	4.55	1.82
Lake Utility Services Inc.	29	5.53	0.76	3.32	4.77	2.21
Altamonte Springs, city of	1	10.00	2.86	5.95	7.14	4.05
Sanlando Utilities Corp.	54	11.12	0.00	4.72	11.12	6.40
Reedy Creek Utilities Co.	52	34.00	5.50	20.40	28.50	13.60
Orlando Utilities Commission	47	124.71	30.74	70.32	93.97	54.39

Note: AAAE = allowable allocation after 40% equity
AAWE = allowable allocation without equity
DFAE = deficits after 40% equity
DFWE = deficits without equity constraints
mgd = million gallons per day
P2020 = projected year 2020 allocation
PWD = Public Works Department

The efficiency of the use of equity constraints as a means of introducing equitable distribution of water is clearly obvious in columns 6 and 7 of Table 3-1. The first eight rows in Table 3-1 indicate that before the invocation of 40% equity, Floribra USA, Water Oaks Estates, Legends, McNamara, the town of Eatonville, and the cities of Ocoee, Winter Garden, and Winter Park were not permitted to pump. After the invocation of the equity constraints, these utilities were given some allocation of water. Significant among them are the cities of Ocoee, Winter Garden, and Winter Park, the allowable projected allocations of which were increased to 2.66, 3.32, and 6.60 million gallons per day (mgd), respectively, from 0.0 mgd. Also, the city of Apopka, the Seminole County Public Works Department, Orange County Utilities, and the city of Casselberry experienced an increase in usage from 6.82, 12.56, 52.39, and 0.98 mgd to 10.74, 15.32, 61.38, and 2.81 mgd, respectively. Of course, these increases are the result of a reduction in other locations, as shown in rows 13 to 18. Significant among the utilities with reduced projected allocations are Sanlando Utilities Corp., Reedy Creek Utilities Co., and Orlando Utilities Commission, whose water requirements have been reduced from 11.12, 28.50, and 93.97 mgd to 6.40, 13.60, and 54.39 mgd, respectively, through the application of equity constraints.

Once again it must be emphasized that this simulation is just an example based on the 2020 water use projections and not reflective of the methods and procedures that SJRWMD adopts in water permit allocations.

TIMING ANALYSIS

Although many optimization alternatives have been performed to confirm the water resource problems that are expected to occur as a result of the projected 2020 demand, no attention has been given to the period around which such problems may occur. A timing analysis may be conducted to quantify when water resource problems might occur.

To perform a timing analysis, one must have an idea about the water demand characteristics (for the selected periods) of the area being served by the wells. Once this information is obtained, the timing analysis procedure becomes one of performing a series of runs for the periods in question and observing the periods in which the optimum deficit begins to exceed zero. Because the water demands for the years in question (other than the base year [1995] and the projected year [2020]) are not known in advance, an interpolation scheme is used to obtain a fair approximation.

The current timing analysis seeks to obtain a time information using the timing years 1995, 2000, 2005, 2010, 2015, and 2020. In order to obtain the demands for the remaining years (2000, 2005, 2010, 2015), other than the two extremes (1995 and 2020), a linear interpolation scheme with a time step of 5 years is used. Using the approximated water demand characteristics for the timing years in question, a series of deficit runs are performed to obtain knowledge about the period at which deficits will start showing up before the 2020 projected year. A summary of the timing analysis for the ECF region is shown in Table 3-2. This table shows that, as a result of the projected 2020 demand, water resource problems may begin to show up by the year 2010 (with an optimum deficit of approximately 12.07 mgd).

Table 3-2. Summary of time analysis for projected year 2020

	1995	2000	2005	2010	2015	2020
Total demand (mgd)	290.69	346.08	401.46	456.86	512.24	567.63
Total deficit (mgd)	0.00	0.00	0.00	12.07	55.00	118.67

Note: mgd = million gallons per day

SENSITIVITY ANALYSIS

Sensitivity analysis is one aspect of the optimization modeling procedure where the analyst attempts to study the influence of some input parameters on the objective function. It shows the extent to which the objective function varies when certain parameters of the optimization model are perturbed by some given percentage. For some parameters, infeasible solutions may result if the perturbation goes beyond reasonable limits, giving the impression that the use of the parameter in question at the set value does not make any sense if one is attempting to optimize the objective function. At the conclusion of a sensitivity analysis, the analyst is able to identify those parameters that influence the objective of the optimization model and to what extent. Information about such influential parameters may assist decision makers in the practical implementation of certain decisions regarding resource allocation. A given optimization model may consist of many parameters that may be eligible for consideration in a sensitivity analysis. However, in resource allocation projects, decision makers are particularly interested in the extent to which the constraints they impose may tend to affect the results of the simulation model. Such parameters are the focus of the sensitivity presented in this section.

It is appropriate to note that sensitivity analysis in linear programming studies implies more than just studying the behavior of the objective function with respect to parameter perturbations. In particular, the optimum parameters (i.e., the coefficients of the objective function) must still continue to be the optimum solution when the parameters considered in a sensitivity analysis are perturbed. This implies monitoring such marginal values as shadow prices and reduced costs in a detailed sensitivity analysis work. These details have been ignored in this basic sensitivity work not only to keep the report simple, but also to make it more understandable to readers without any knowledge of linear programming.

The parameters considered in the sensitivity analysis are mostly constraint parameters. These parameters are of interest because they constitute values that decision makers can manipulate in a decision-making process. These include limitations imposed on (1) drawdown at wetland control points (Case 1), (2) heads at spring locations (Case 2), (3) demand at public demand areas (Case 3), and (4) wells capacity (Case 4). Each parameter is perturbed by the factors shown in Table 3-3 according to the expression

$$P_i = P_{original} * PF_i \quad i = 1,2,\cdots,10 \tag{3-1}$$

where

P_i = the new parameter used in sensitivity run *I*, and

$P_{original}$ = the original parameter value used (or assumed) in the model.

In all of the sensitivity runs, the value of the objective function is used as the measure of response of the model to the various perturbations.

Case 1—Perturbation of Drawdown Limits at Control Points

This parameter imposes limitations on the drawdown simulated at the wetland control points. The response of the objective function to the perturbation in drawdown limits at the wetland control points is shown in Table 3-3 along column 2. These results suggest that reduction in drawdown limit values (while keeping the other model parameters intact) is not practicable beyond certain limits. Increasing the imposed drawdown limits, however, poses no such limitation. This implies that the actual limits to drawdowns imposed by management at the control points may be increased to improve upon the optimum deficit as shown in the table. Figure 3-5 shows the trend of the sensitivity results.

Table 3-3. Summary of sensitivity analysis for the east-central Florida deficit model

Perturbation Factor (PF_i)	Optimum Deficit (mgd)			
	Case 1	Case 2	Case 3	Case 4
0.5	Infeasible	Infeasible	0.00	Infeasible
0.6	Infeasible	Infeasible	0.51	Infeasible
0.7	173.08	Infeasible	2.29	Infeasible
0.8	152.13	Infeasible	28.59	Infeasible
0.9	135.08	Infeasible	74.20	259.04
1.0 (original)	123.14	123.14	123.14	123.14
1.1	113.37	110.33	174.61	52.30
1.2	107.02	110.33	227.14	9.22
1.3	101.33	110.33	280.60	1.03
1.4	96.19	110.33	334.82	0.15
1.5	91.60	110.33	389.31	0.00

Note: mgd = million gallons per day

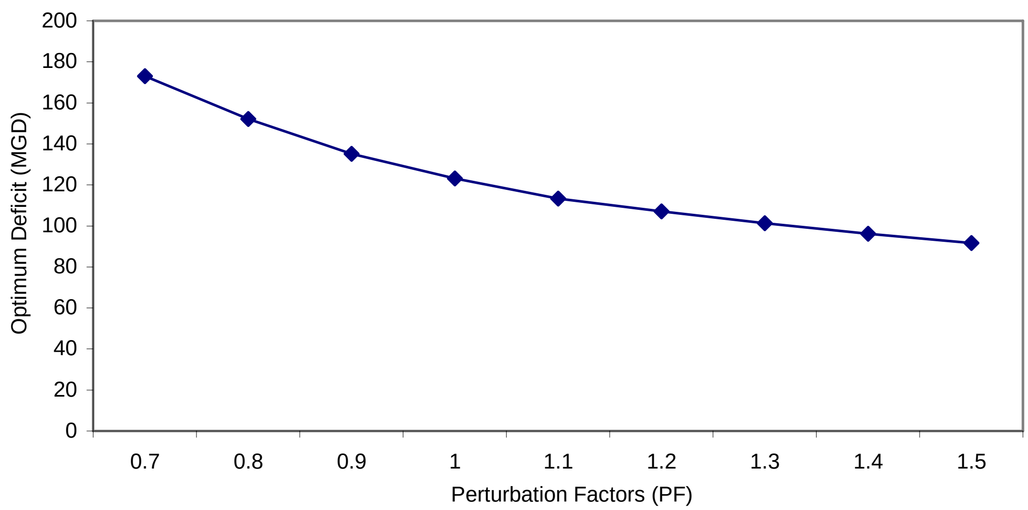

Figure 3.5 Sensitivity of drawdown limits set at wetland control points

Case 2—Perturbation of Spring Heads at Spring Locations

The response of the objective function to the perturbations of the hydraulic heads at spring locations are shown in column 3 of Table 3-3. These results indicate that decreasing the values of the parameter in question (while keeping the other parameters and constraints intact) is not practicable. Increasing the parameter, however, is permitted, but that does not result in any further improvement of the optimum deficit after the first perturbation with $PF_i = 1.1$. This shows that the constraint parameter is insensitive and hence may not be reliable in a management decision-making process. It is not clear why the objective function will not respond to further increments in the parameter value. The only possible explanation here is that the first increment may have triggered some other parameter(s) (or constraints in the model) to render the former insensitive. The trend of the sensitivity of this parameter is shown in Figure 3-6.

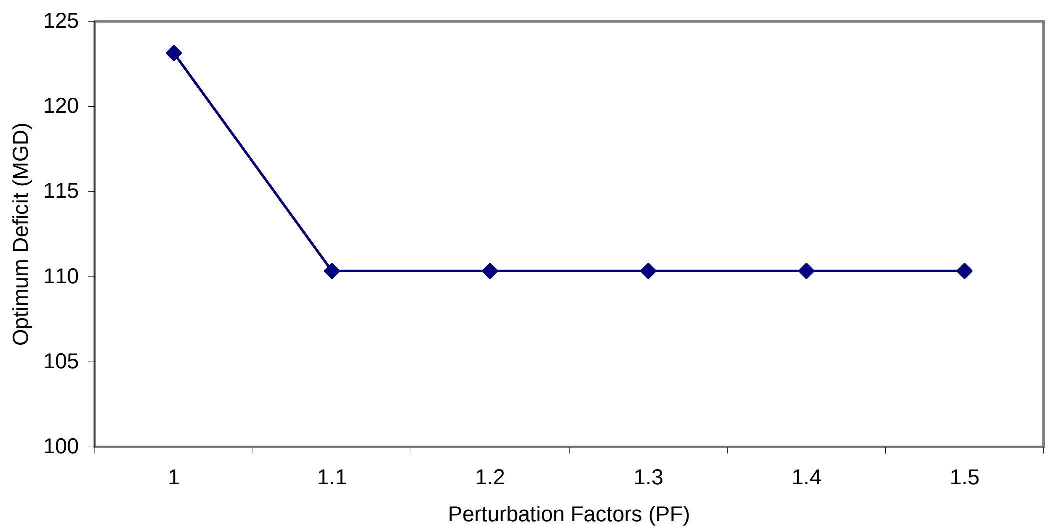

Figure 3.6 Sensitivity of hydraulic heads at spring locations

Case 3—Perturbation of Demand at Public Demand Areas

The response of the objective function to the perturbations in demand is shown in Table 3-3 along column 4. The wide variations in the objective function values for the various perturbations indicate the extreme sensitivity of this parameter to the objective function. In particular, the total deficit tends to approach zero when the parameter is reduced in value. This observation indicates that the total deficit may be reduced by imposing some restrictions on the quantity of demand from needy areas. The trend of the sensitivity output is shown in Figure 3-7.

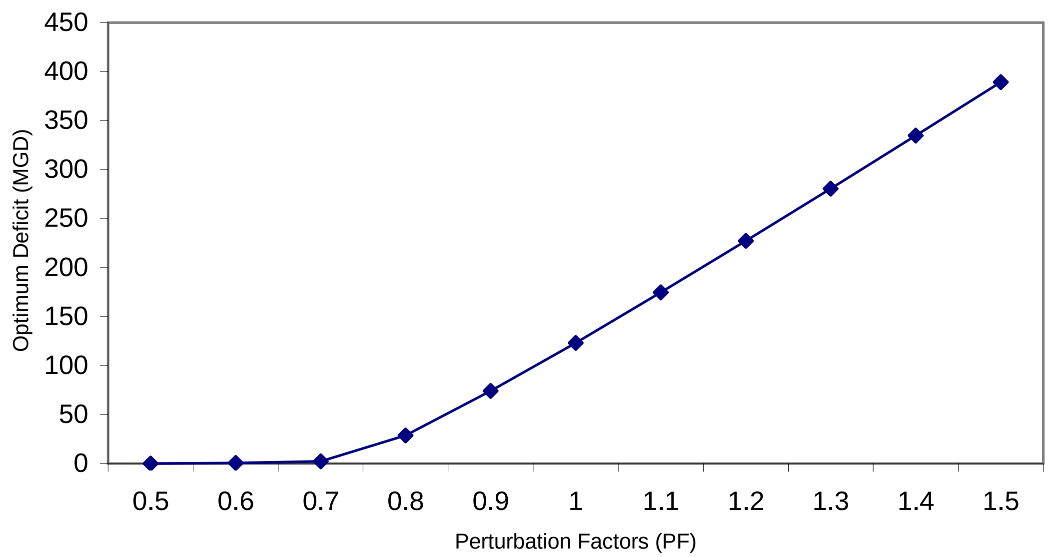

Figure 3.7 Sensitivity of demand at public demand areas

Case 4—Withdrawal Rate at Service Wells

The sensitivity of a non-optimized well withdrawal rate may be another parameter of interest since it indirectly influences the demand to be satisfied at the public demand area. The result of the sensitivity of this parameter is displayed in Table 3-3 along column 5. Increasing the well withdrawal rates above its nominal values forces the total deficit (objective function) to approach zero. This may however, not be a good management strategy to adopt in an attempt to solve the 2020 water resources allocation problem, since extreme withdrawal may in turn increase drawdown in the model region, which may subsequently affect vegetation at the wetland control points. The sensitivity output is shown in Figure 3-8.

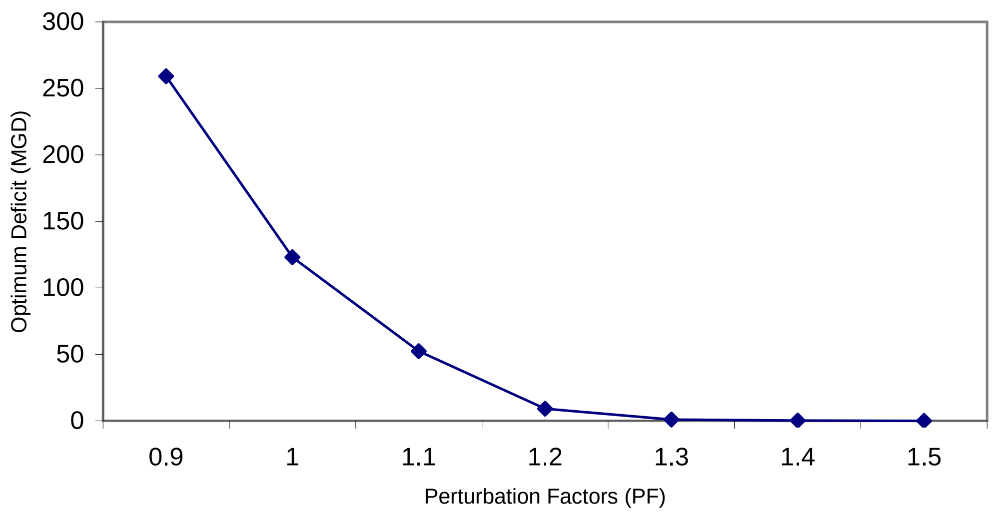

Figure 3.8 Sensitivity of withdrawal rates at public supply wells

In summary, this brief sensitivity analysis has provided some insight as to which parameter to control during a decision-making process. It has also highlighted that there are some limitations to the extent to which some controlling parameters may be varied. The infeasible solutions seen in Table 3-3 indicate that variation of the parameter beyond certain limits is not practical, given the values of the other optimization parameters and/or constraints in the entire model.

SUMMARY

The following findings can be addressed from the results of the runs conducted in this chapter.

The good agreement between the deficit model and the forward simulation model (MODFLOW) with respect to the hydraulic heads computed at the wetland control points and spring locations indicates that the use of the response matrix approach for the evaluation of heads in the deficit optimization model is a reasonable approach. The relatively large residuals (on the order of 0.1–0.2 foot) observed at some of the aforementioned locations may be due to (1) non-linearity and / or (2) possible truncation errors in the evaluation of the unit response matrix.

With the current water use practices, fresh groundwater withdrawals from the Floridan aquifer may exceed a sustainable limit by the year 2020. A timing analysis indicated that water allocation problems may begin to occur in year 2010.

For management decision purposes, a sensitivity analysis was conducted on some of the deficit model parameters. It was found that there was a limit to which some parameters could be perturbed. Perturbing beyond this limit resulted in an infeasible solution.

Equitable distribution of the available groundwater resource has been the objective of decision makers in almost all realms of resource management. The concept of equitable distribution is incorporated into the deficit model through the use of the so-called equity constraints, wherein every user is permitted to pump an additional percentage, x, of their respective 2020 allocation. The results of comparing an equity run with a non-equity run indicated that the use of the equity constraint is a legitimate means of introducing fairness in the equitable allocation of a scarce water resource.

DECISION MODEL FORMULATION

A decision model is developed as an extension of the deficit model to investigate the cost of exploring other alternative water sources to the traditionally used fresh groundwater. The actual linkage between the decision and the deficit model is one of inheritance. The decision model inherits all the constraint elements described previously in the deficit optimization model and then adds its unique constraints to complete the model. The elements inherited include the collection of the environmental and water management constraints.

METHODOLOGY

The general decision model formulation includes continuous variables for production flows or use rates, hydraulic heads, drawdowns, and costs. The decision model allows for a number of alternative water sources to be used in response to increased public water demands projected for year 2020. The model incorporates two types of cost: initial fixed costs and recurring unit costs. Initial fixed costs are incurred when a new source must be constructed. Recurring unit costs are similar to operation and maintenance costs but include only those costs which are directly dependent on the production flow or use rate of the source. For example, a technician's salary must be considered as part of the initial cost because salary does not depend on the flow rate, whereas power and chemical costs are unit costs because they do depend on the flow rate. Fixed and unit costs are represented in the model on an annualized basis. Consultants provided actual cost data on per unit basis (CH2M HILL 1997a; Law Engineering and HSW Engineering 1997). The objective function of the decision model is to minimize the total costs of water allocation strategies utilizing both existing and other potential water supply sources.

ALTERNATIVE WATER SUPPLY SOURCES

Alternative water supply sources investigated in the model include new public supply wells, surface water, and new public supply interconnects. Consultant cost data associated with the use and development of alternative water supply sources were provided. A brief description of each source is presented below.

New Public Supply Wells (Fresh and Brackish)

Reliable and inexpensive fresh groundwater is almost always the preferred source (within SJRWMD). New freshwater sources from the Floridan aquifer can potentially supply additional water, although the potential impact of such groundwater withdrawals on natural systems introduces major concerns. The approach taken here is to explore water allocation strategies that optimize the continued use of fresh groundwater from the Floridan aquifer, under constraints formulated to protect wetlands, lakes, and spring discharges.

Brackish groundwater from the Floridan aquifer is also a potential source of water supply, although it is not currently used to satisfy water supply needs. This type of groundwater is considerably more expensive to treat than fresh groundwater. Furthermore, concentrate management of brackish water treatment processes impedes development of this resource. The potential for developing brackish groundwater was evaluated at several sites, and the primary candidates were found to be located within the eastern portion of the area along the western side of the St. Johns River. CH2M HILL (1997b, 1998c) investigated treatment requirements and costs for these sites. The outcome of their investigation is used here to develop and evaluate utility-specific brackish groundwater options.

As shown in Figure 4-1, the decision model has 443 new well locations that are available as alternative water supplies. A separate analysis involving geographic information system (GIS) procedure was used to identify likely locations of new wells. In this analysis, the following GIS layers were used to screen areas of potential new groundwater (brackish water and freshwater) development: (a) locations of existing public supply wells, (b) wetlands, and (c) areas of high groundwater salinity. Fixed and unit costs for new groundwater wells are provided in a planning level cost estimates report by Law Engineering and HSW Engineering (1997).

Surface Water

Surface water is relatively abundant. It can potentially serve as a water supply source, specifically from the St. Johns and Ocklawaha river basins. Previous investigations by CH2M HILL (1996a, 1996b, 1996c, and 1997b) selected the following as the most likely sources for surface water supply:

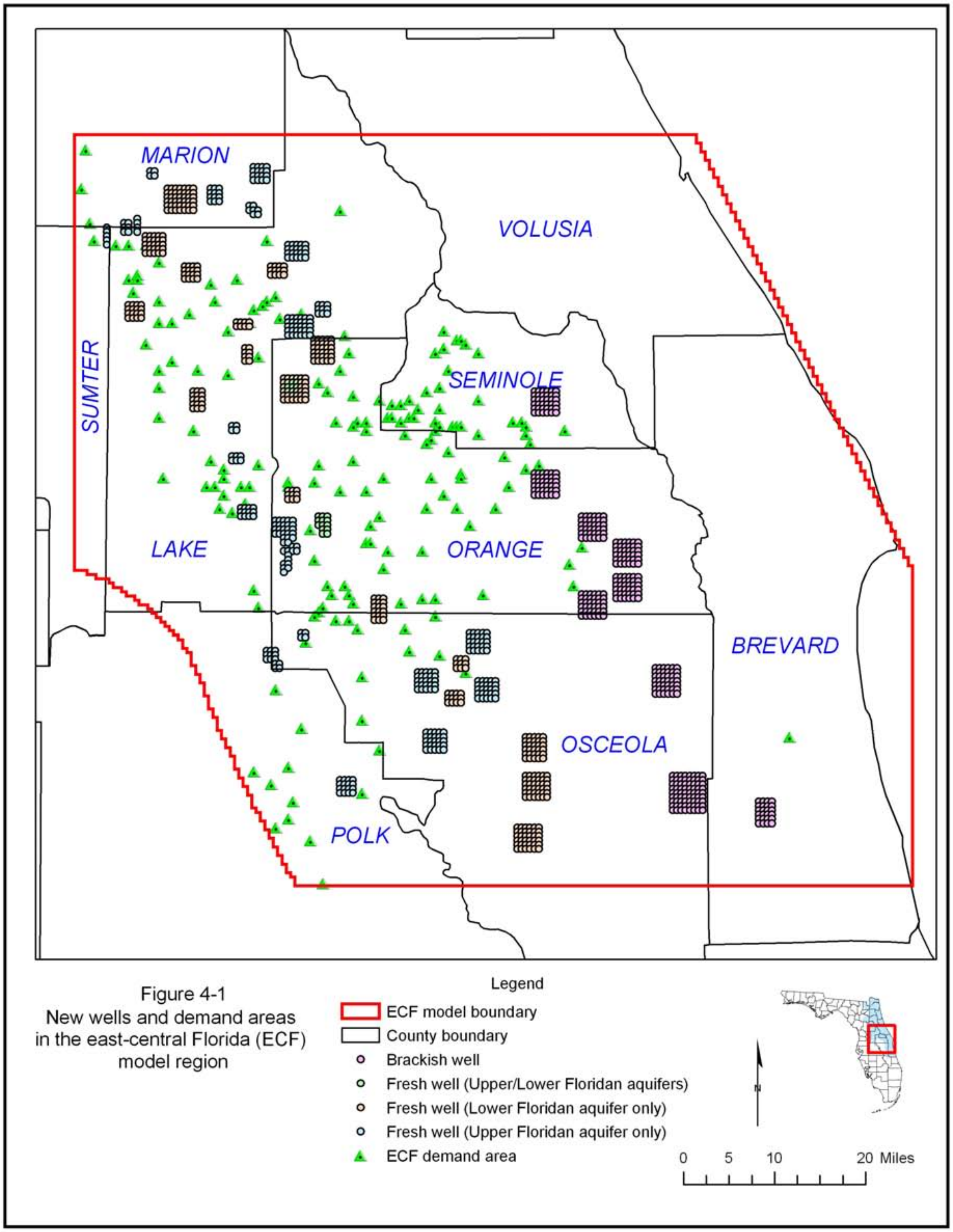

Figure 4-1
New wells and demand areas in the east-central Florida (ECF) model region

- Lake Griffin (Haines Creek) in Lake County near Leesburg
- Lake Apopka
- Ocklawaha River
- St. Johns River near Cocoa
- St. Johns River near Titusville
- St. Johns River near Sanford (Lake Monroe)
- St. Johns River near DeLand

Figure 4-2 shows some surface water sources in the ECF model region. Some of these surface water sources are used in the decision model as likely alternatives where surface water could be obtained for treatment and transport to demand areas. Details regarding the cost of treatment and transport of water from these sources have been reported by CH2M HILL (1997a). Also shown in the same figure are some important springs in the ECF model region. The drawdowns at these spring locations, just as those at the wetland control points, are critical parameters predicted by the optimization models.

Interconnects

Interconnects between public supply utility demand areas may transport groundwater or surface water from a source area to a destination area. Law Engineering and HSW Engineering (1997) reported interconnect component costs and provided a spreadsheet for calculating fixed and unit costs for different interconnect lengths and flow rates.

A more realistic assessment of fixed costs for existing interconnects would consider a flow rate for existing interconnects. Although these data were available for a limited number of distribution systems during model development, details in the critical public demand areas were not. Thus, additional detail in critical demand areas would be required to consider more realistic uses of existing interconnects. Future modeling efforts may include fixed costs when existing interconnects are utilized. However, the costs would be considerably lower than the costs for potential interconnects, as no purchase of land would be required.

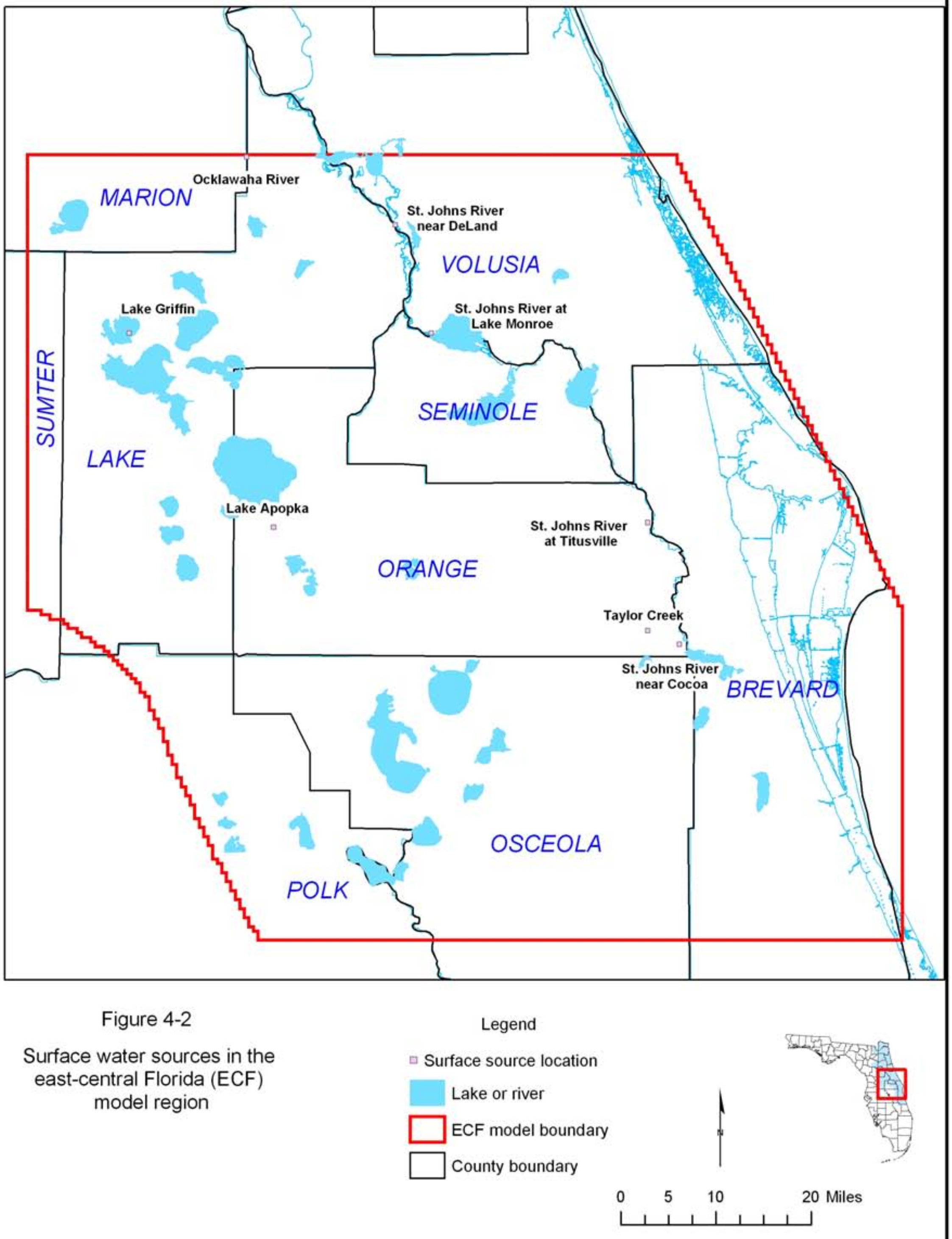

Figure 4-2

Surface water sources in the east-central Florida (ECF) model region

Annual costs used from CH2M HILL (1997a) were as follows:

- Transport systems cost (short—5 miles maximum)

(a) Fixed cost:
$$\text{Cost} = 22{,}460L^{1.232}ADF^{0.430} \quad [\$] \tag{4-1}$$

(b) Unit cost: negligible

- Transport systems cost (long—greater than 5 miles)

(a) Fixed cost:
1 pump station: $$\text{cost} = 32{,}040L^{1.044}ADF^{0.44} \quad [\$] \tag{4-2}$$
2 pump stations: $$\text{cost} = 43{,}180L^{0.935}ADF^{0.448} \quad [\$] \tag{4-3}$$

(b) Unit cost:
1 pump station: $$\text{cost} = 32{,}390ADF^{0.945} \quad [\$] \tag{4-4}$$
2 pump stations: $$\text{cost} = 51{,}420ADF^{0.969} \quad [\$] \tag{4-5}$$

where *L* is the length in miles and *ADF* is the average daily flow in million gallons per day.

Since the model formulation allows only linear functions of continuous decision variables, Equations 4-1 through 4-3 for fixed costs were approximated by linear equations. For unit cost equations, the decision model uses formulations where the exponents are rounded to unity. Furthermore, all fixed and unit costs are recast in a form having ADF expressed in cubic feet per day (cfd). The number of pump stations required is a function of total transport distance and average daily flow. In general, only one pump station is required, though flows or volumes transported over 25 miles require two or more pump stations. All public supply demand areas having large projected year 2020 demands (over 5 mgd) located within 20 miles of potential surface water sites were considered as primary destinations for surface water transport. Through public supply interconnects, additional water transport can be selected so as to supply other public supply demand areas acting as secondary surface water destinations.

DECISION MODEL FORMULATION

Model Indices

Decision model indices include *h*, the set of control points where drawdowns are constrained; *i*, the set of public supply utility wells or well grid cells; *j*, the set of public supply utility demand areas; *l*, the set of springs; *q*, the set of public demand areas which serve as interconnect destinations; and *p*, the set of surface water sources.

Model Parameters

Decision model parameters include the hydrogeologic parameters of the deficit model, additional service and distance maps for sources serving demand areas, and cost parameters for existing and alternative water supplies.

Decision Model Variables

Decision model continuous variables include the hydrogeologic and water management variables of the deficit model as well as use rates, fixed costs, and unit costs of alternative water supply strategies.

Model Constraints

The decision model includes all environmental constraints from the deficit model with additional constraints designed to address specific parts of the alternative water supply elements. Two types of constraints are used to define available quantities of surface water for public use.

First, surface water use may not exceed source capacity:

$$QS_p \leq Scap_p \tag{4-6}$$

where

QS_p = use rate of surface water source *p*, and

$Scap_p$ = capacity of surface source *p*.

Second, the cumulative use rate of a surface water source *p*, QS_p, must equal the sum supplied to all public areas:

$$QS_p = \sum_j QQS_{p,j} \tag{4-7}$$

where

$QQS_{p,j}$ = use rate of surface source *p* at public demand area *j*.

The total use rate of all surface water sources cannot exceed the projected year 2020 demand of the receiving public area:

$$\sum_p QQS_{p,j} Servs_{p,j} \le DM_j \tag{4-8}$$

where

$Servs_{p,j}$ = service map from surface source *p* to demand area *j*.

$Servs_{p,j}$ is 0 if surface water source *p* does not supply demand area *j*. Otherwise, $Servs_{p,j}$ is equal to 1.

The demand constraint for public supply areas is

$$\begin{aligned} &\sum_i Q1_{i,j} Servm_{i,j} + \sum_p QQS_{p,j} Servs_{p,j} \\ &+ \sum_q Q2_{q,j} at_{q,j} \ge DM_j + \sum_q Q2_{j,q} at_{j,q} \end{aligned} \tag{4-9}$$

where

$Q1_{i,j}$ = pump rate of well *i* supplying demand area *j*, and
$Q2_{j,q}$ = public interconnect use rate from source area *j* to destination *q*.

Note that *Q2* appears on both sides of the above equation. On the left side of the inequality, when summed over its first index, it refers to the total quantity of water received from other public areas. On the right side of the inequality, *Q2* is summed over its second index, referring to the amount of water serving other public areas.

Fixed Cost of New Alternatives

The next equations are equality constraints that define the fixed charge components of the objective function. Fixed costs are incurred when the decision model selects alternatives to existing water supplies. Alternatives to existing supplies include new wells, new water treatment plants, new interconnects, and surface water sources. Fixed costs are defined as follows:

New wells and treatment plants:

$$FW_j = \sum_i Servm_{i,j} QP_i [0.0164 + ful_i + fll_i + ftp_i] \qquad \textbf{(4-10)}$$

where

- FW_j = fixed and treatment cost of new wells to public area *j*,
- ful_i = unit fixed cost for new Upper Floridan aquifer well per unit QP_i,
- fll_i = unit fixed cost for new Lower Floridan aquifer well per unit QP_i, and
- ftp_i = unit fixed cost for urban lime softening treatment plant, including aeration and filtration of water per unit QP_i.

New public interconnects:

$$FIC_q = \sum_j (icept_{j,q} + slope_{j,q} Q2_{j,q}) \text{ if } at_{j,q} = 1 \qquad \textbf{(4-11)}$$

where

- FIC_q = fixed cost of interconnect to destination area *q*, from source *j* [$],
- $icept_{j,q}$ = fixed cost of linear approximation intercept of interconnect to destination area *q* from source *j*,
- $slope_{j,q}$ = unit fixed cost of linear approximation slope per unit $Q2_{i,j}$, and
- $at_{j,q}$ = integer parameter mapping source area *j* to destination area *q*.

New surface water sources:

$$FP1_j = \sum_p fsu_p QQS_{p,j} \qquad \textbf{(4-12)}$$

where

$FP1_j$ = fixed cost of surface water site *p* serving public area *j*, and
fsu_p = fixed cost for surface water source *p*.

Connections from surface water sources to public demand areas:

$$FP2_j = \sum_p (icepts_{p,j} + slopes_{p,j} \ \ QQS_{p,j}) \quad \textbf{(4-13)}$$

where

$FP2_j$ = fixed cost of surface water transport from all sources connected to area *j*,
$icepts_{p,j}$ = fixed cost linear approximation intercept of surface water transport from surface source *p* to demand area *j*, and
$slopes_{p,j}$ = fixed cost linear approximation slope of surface water transport from source *p* to demand area *j*.

Unit Costs of All Selections, Existing or Proposed

The following equality constraints calculate unit costs of all sources, both existing and potential, selected by the decision model.

For public supply demand areas (not considering interconnects):

$$UP_j = \sum_i QP_i Servm_{i,j} uw_i + \sum_p QQS_{p,j} Servs_{p,j} us_p \quad \textbf{(4-14)}$$

where

UP_j = unit cost of supplies serving public area *j*,
uw_i = unit cost of supply and treatment of well *i*, supplying public area *j*, and
us_p = unit cost of surface water source *p*, supplying demand area *j*.

For public interconnects:

$$UI_q = \sum_j 0.38 Q2_{j,q} \ \ \text{if } dists_{j,q} \geq 5 \text{ miles} \quad \textbf{(4-15)}$$

where

UI_q = unit cost of interconnect from all destinations connected to source area *q*, and
$dists_{j,q}$ = distance from source area *j* to destination area *q*.

The unit costs are similar for surface water transport, but indexed by public area *j* and summed over index *p,* the set of surface water sites:

$$UST_j = \sum_p 0.38 QQS_{p,j} \quad \text{if } distsm_{p,j} \geq 5 \text{ miles} \tag{4-16}$$

where

UST_j = unit cost of surface water transport to demand area *j* from all sources, and
$distsm_{p,j}$ = distance from surface water source *p* to public demand area *j*.

Objective Function

The objective of the decision model is to minimize

$$Z = \sum_j [FW_j + FP1_j + FP2_j + UP_j + UST_j] + \sum_q [FIC_q + UI_q] \tag{4-17}$$

subject to the following constraints:

- Hydrogeologic constraints

$$D_h = Ho_h - Hno_h + \sum_i \alpha_{i,h} [QP_i - Qno_i] \tag{4-18}$$

$$D_h \leq D\lim_h \tag{4-19}$$

$$HS_l = Hsno_l - \sum_i \gamma_{i,l} [QP_i - Qno_i] \tag{4-20}$$

$$HS_l \geq Ps_l (Hso_l - El_l) + El_l \tag{4-21}$$

$$Ps_l = \frac{HS_l - El_l}{Hso_l - El_l} \tag{4-22}$$

$$SD_l = Cd_l (Hs_l - El_l) \tag{4-23}$$

$$SD_l \geq Scrit_l \tag{4-24}$$

- Public water management constraints

$$QP_i = \sum_j Q1_{i,j} \tag{4-25}$$

$$QP_i \le Mcap_i \tag{4-26}$$

$$QS_p \le Scap_p \tag{4-27}$$

$$QS_p = \sum_j QQS_{p,j} \tag{4-28}$$

$$\sum_p QQS_{p,j} servs_{p,j} \le DM_j \tag{4-29}$$

$$\begin{aligned} &\sum_i Q1_{i,j} Servm_{i,j} + \sum_p QQS_{p,j} Servs_{p,j} \\ &+ \sum_q Q2_{q,j} at_{q,j} \ge DM_j + \sum_q Q2_{j,q} at_{j,q} \end{aligned} \tag{4-30}$$

DECISION MODELING PROCEDURE

The method used to determine optimum water allocation scenarios is an iterative process and can be divided into the following five steps:

1. Solve the decision model with first estimates of influence coefficients. The optimization model represents a system of linear equations solved during the first step using GAMS.

2. Compare the decision model results to MODFLOW simulation predictions. This comparison is accomplished by evaluating hydraulic heads at the control points in MODFLOW using the optimum well rates predicted by the decision model and checking the results against the hydraulic heads estimated by the decision model.

3. If the hydraulic head results are below allowable tolerances, calculate a revised set of influence coefficients using different perturbations.

4. Solve the decision model to obtain the optimized pumping rates with revised influence coefficients.

5. Repeat steps 2–4 until an acceptable level of agreement is reached between the decision and MODFLOW simulation results.

SUMMARY

The formulation of the decision model, designed as an extension of the deficit model, has been presented. Subject to some constraints, and given the availability of other alternative sources of water such as new wells, surface water sources, and interconnect possibilities, the decision model attempts to minimize the cost of using water from a combination of these sources as an alternative solution to water deficit problems. Its application to water supply management problems is presented in chapter 5.

DECISION MODEL RUNS

As its name implies, the decision model ultimately makes decisions. It tells the user the appropriate measures to consider in the event of water deficit concerns. Its objective function considers the minimization of the costs associated with the use of alternative sources of water. These costs include the costs of construction and operation of new, fresh, and brackish wells, the cost of the use of interconnects, and the cost of the use of surface water sources. When deficits are encountered, it selects the appropriate sources of possible water in an optimized manner based on the cost from the following alternatives: (1) the possible construction of new wells, (2) the use of new interconnects, and (3) the extraction of water from surface sources. This chapter presents first a brief introduction to uncertainty analysis with reference to the vertical leakance parameter, followed by a series of runs performed on a number of alternatives to demonstrate some of the outputs of the decision model. Other parameters, such as hydraulic conductivity, could have been chosen for the uncertainty analysis, but the leakance responded fairly well to sensitivity analysis of the ECF flow model. The capabilities of the decision model are similarly limited by the same conditions enumerated for the deficit model. It must be emphasized once again that all the model runs presented below are examples to illustrate the way the optimization model works and the information that could be obtained from its results for decision-making when the need arises.

LEAKANCE UNCERTAINTY ANALYSES

Sensitivity analyses conducted with the ECF groundwater model show that the leakance input can be varied within a certain range without rendering the model calibration unacceptable (McGurk and Presley 2002). Leakance defines the hydraulic conductivity through a semiconfining unit lying between two aquifer units. In the ECF model, leakance measures connectivity between the surficial aquifer and the Upper Floridan aquifer, across the upper semiconfining unit. Specifically, modelwide changes using a coefficient ranging from a minimum of 0.67 to a maximum of 1.5 times the optimal leakance values produce simulated water levels and spring flows acceptably close to actual measurements. The predicted change in surficial aquifer levels caused by pumping will vary, depending upon the uncertainty in the upper semiconfining unit leakance values.

The ranges of leakance values were bracketed in two model runs using the ECF groundwater model. The model runs predicted the change in surficial aquifer levels for 2020 Floridan aquifer pumping, in relationship to the average 1995 conditions. The model calculations assumed recharge to the surficial aquifer equaled 1995 recharge levels, except where changes in either reuse water application or irrigation with groundwater from the Floridan aquifer system have been projected. The difference between the two resulting predictions yielded a range of potential change in surficial aquifer water levels. This difference was used as an estimate of the effect of leakance uncertainty.

To study the leakance uncertainty effect in the optimization models, two optimization runs, each with its unique set of influence coefficients, is required. A first set of influence coefficient values is obtained by running the flow model the required number of times using a low-end vertical leakance value of 0.67 times the calibrated leakance. Using these influence coefficients values, a decision (or deficit) model is developed. A second decision (or deficit) model is developed using a high-end leakance value obtained as 1.5 times the calibrated value. If, for a given demand area A_j, the deficit for the first model run is QD_{jl} and that for the second is QD_{jh}, then the degree of uncertainty R_j may be defined as the ratio of the sum of the deviations of the two model runs from the baseline deficit QD_j to the deficit QD_j.

$$R_j = \frac{\left|QD_j - QD_{jl}\right| + \left|QD_j - QD_{jh}\right|}{QD_j}, \qquad \textbf{(5-1)}$$

where QD_j is the baseline deficit value obtained using the influence coefficients generated by the flow model with the calibrated leakance. A zero (or near zero) value of R_j indicates the lack of significance of leakance uncertainty in this area. Areas with high values of R_j may exhibit a high degree of uncertainty effect. The numerator in Equation 5-1 measures the degree of variation in deficits for demand area *j*, based on the uncertainty that the vertical leakance in the model may vary between a factor of 0.67 and 1.50. Scaling the numerator by QD_j allows for the comparison of the degree of uncertainty of a number of demand areas. Thus, column 7 of Table 5-1 shows that the degree of uncertainty in estimated deficit results is very pronounced for utilities in Polk County[1] areas of the ECF model domain, whereas those in Seminole and Orange counties experience rather low uncertainty in estimated deficits. Uncertainty computations for Brevard, Marion, and Sumter counties are skipped because these areas have zero deficits for the limits of leakance parameters used for evaluation of the influence coefficients.

Table 5-1. Summary of uncertainty analyses

County	QD_j (mgd)	QD_{jh} (mgd)	QD_{jl} (mgd)	$\lvert QD_j - QD_{jl} \rvert$ (mgd	$\lvert QD_j - QD_{jh} \rvert$ (mgd)	R_j (dimensionless)
Brevard	0.00	0.00	0.00	0.00	0.00	NA
Lake	19.64	23.33	0.00	19.64	3.69	1.187
Marion	0.00	0.00	0.00	0.00	0.00	NA
Orange	109.14	101.92	0.00	109.14	7.22	1.066
Osceola	4.94	6.25	0.00	4.94	1.31	1.265
Polk*	1.44	0.92	0.00	1.44	0.52	1.361
Seminole	55.36	54.35	0.00	55.36	1.01	1.018
Sumter	0.00	0.00	0.00	0.00	0.00	NA

Note: mgd = million gallons per day
NA = not applicable

*As of July 1, 2003, the portion of the St. Johns River Water Management District that was in Polk County became part of the Southwest Florida Water Management District.

It must be noted that the above uncertainty analysis can be performed using other independent parameters in the ECF simulation model; particularly hydraulic conductivity. However, vertical leakance is chosen because it caused an extreme degree of response in the measure of performance during the sensitivity analysis of the ECF flow model (McGurk and Presley 2002). This section explains how our lack of knowledge about the values of certain parameters can affect the results of the optimization model. In other words, the results of the optimization models may not be certain until we have an accurate knowledge about the parameters used in the forward simulation model to generate the desired influence coefficients for the optimization models. To reduce this level of uncertainty in the optimization models, an attempt is made

[1]As of July 1, 2003, the portion of the St. Johns River Water Management District that was in Polk County became part of the Southwest Florida Water Management District.

to incorporate uncertainty effects in the optimization models through adjustments of the drawdown constraint limits.

INCORPORATING UNCERTAINTY IN OPTIMIZATION MODELS

Since uncertainty is a probable event, its effect is not so simple to account for in an optimization model. However, a simple approach has been adopted to implicitly account for the effect of uncertainty in the current SJRWMD optimization model.

The technique is to adjust the drawdown limits imposed at the wetland control points in the ECF model region by the degree of water level changes expected, based on the understanding that the calibrated vertical leakance (of the flow model) for the region may vary between a factor of 0.67 and 1.50. The changes in water levels are estimated by first performing a flow simulation by multiplying the calibrated leakance of the flow model by 0.67 and collecting the water levels estimated for the wetland control points. This procedure is repeated in a second run by multiplying the calibrated leakance by 1.50. The difference between the resulting predictions at the wetland control points (the drawdowns of which are being constrained in the optimization models) yields a range of potential change in surficial aquifer water levels. This expected potential change is accounted for in the optimization models by adjusting the constrained drawdown limits set for the wetland control points by these potential water level changes. Thus, if the drawdown limit imposed at the wetland control point h is $d\lim_h$ and the corresponding potential change in water level is f_h, then the actual drawdown constraint imposed at this control point is adjusted $a\lim_h$, which is expressed as

$$a\lim_h = d\lim_h + f_h \qquad \textbf{(5-2)}$$

$f_h = H_1 - H_{0.67}$ in areas where leakance is expected to be low or
$f_h = H_1 - H_{1.5}$ in areas where leakance is expected to be high,

where

H_1 = head based on expected (calibrated) parameters,
$H_{0.67}$ = head based on calibrated leakance multiplied by 0.67, and
$H_{1.5}$ = head based on calibrated leakance multiplied by 1.50.

Hence, if $a\lim_h$ is used as the adjusted drawdown constraint in the optimization model, then uncertainty effects are assumed to be implicitly accounted for in the estimation of deficit (and other variables) for the water demand areas in the model. Allowing for leakance uncertainty in this manner permits the use of high adjusted constraint values in areas where the potential water level changes are high and low values in areas where the potential changes are low.

SIMULATION ALTERNATIVES AND RUNS

In support of the SJRWMD water supply planning process, many potential modeling alternatives were considered using the decision model. Of these, six potentially viable regional solutions were identified that satisfy both projected demands and Water 2020 goals. These alternatives are briefly defined in Table 5-2.

Table 5-2. Decision modeling alternatives

Alternative	Sources	Surface Water	Political Boundary Constraints	Uncertainty
2	All existing, proposed, and new sources	Yes	No	No
2u	All existing, proposed, and new sources	Yes	No	Yes
2WOLG	All existing, proposed, and new sources	Yes	No	No
4	All existing, proposed, and new sources, without Lake Griffin	Yes	County and SJRWMD	No
6	All existing, proposed, and new sources	Yes	SJRWMD	No
8	All existing, proposed, and new sources; no brackish water	Yes	County and SJRWMD	No

Decision Model Run for Alternative 2

Alternative 2 considers all existing, proposed, and candidate sources with surface sources. The transport of water is not restricted to counties or water management district political boundaries.

Alternative 2 meets 2020 demands by creating 23.2 mgd of new fresh groundwater, 4.2 mgd of brackish groundwater, and 122.2 mgd of surface water to meet the majority of additional demand. The new sources and associated demand sources are shown in Table 5-3a.

With new fresh groundwater and surface water sources available, alternative 2 meets 2020 demands by expanding existing wellfields and treatment plants and developing five new wellfields (three Upper Floridan aquifer, one Lower Floridan aquifer, and one brackish) with new treatment plants and two surface water withdrawal and treatment plants. Of the total of 150 mgd, about 81.7% comes from surface water and the remaining 18.3% from expanding existing groundwater facilities and new wellfields. Table 5-3b provides a summary of the cost associated with this alternative. A visual representation of the decision model results for alternative 2 is shown in Figure 5-1.

Decision Model Run for Alternative 2u

Alternative 2u considers all existing, proposed, and candidate sources included in alternative 2 plus the incorporation of uncertainty.

Alternative 2u meets 2020 demands by expanding existing and proposed wellfields and identifying 127.1 mgd of new supply. This new supply includes 26.1 mgd of new fresh groundwater, 4.2 mgd of brackish water, and 96.8 mgd of surface water. Accounting for the effect of uncertainty makes greater amounts of groundwater available from existing and proposed wellfields (from 23 mgd in alternative 2 to 26 mgd). Also, the dramatic change in demand from surface water sources (from 122 mgd in alternative 2 to 96.8 mgd) when the effect of uncertainty is accounted for in a model run is worthy of note.

With new fresh groundwater and surface water sources available, alternative 2u meets 2020 demand by expanding existing wellfields and treatment plants, developing seven new wellfields (four fresh Upper Floridan aquifer, two fresh Lower Floridan aquifer, and one brackish) with new treatment plants and developing two surface water withdrawal and treatment plants. Table 5-4a shows the new sources with their associated demand, for average day demand.

Table 5-4b provides a summary of the estimated cost for alternative 2u. Equivalent annual cost estimates total $131.7 million/year for this alternative. This total includes annualized costs for expanding existing facilities and developing new groundwater and surface water sources. Developing the new

Table 5-3a. Decision model results for alternative 2 for the east-central Florida planning area

Alternative	New Supply Source	New Supply Used (mgd)	New Water Use by Demand Area (mgd)	
2—Existing and proposed Floridan aquifer wells, new Floridan aquifer wells, and surface water sources	Freshwater wells	23.2	Haines City	2.4
			Kissimmee	9.2
			OUC SF	7.6
			Poinciana	2.0
			St. Cloud	2.0
	Brackish wells	4.2	Kissimmee	4.2
	Surface water	122.2	Leesburg	9.1
			Tavares	1.6
			Altamonte Springs	3.7
			Apopka	14.4
			Lake Mary	2.8
			Longwood	3.2
			Mt. Dora	2.7
			OCU West	10.7
			OUC SF	23.8
			OUC SJ	43.4
			Seminole County PWD	6.8

Note: mgd = million gallons per day
OCU = Orange County Utilities
OUC = Orlando Utilities Commission (SF indicates the service area is in the South Florida Water Management District; SJ indicates the service area is in the St. Johns River Water Management District)
PWD = Public Works Department

Table 5-3b. Cost summary for alternative 2

Source	Construction Cost ($M)	Capital Cost ($M)	O&M Cost ($M)	Equivalent Annual Cost ($M/yr)
EPFW	199.0	297.4	12.4	37.8
NULW	57.1	120.0	2.1	11.4
NBRW	25.5	40.8	2.3	5.9
NSWS	376.7	567.0	42.8	9.3
Total	658.3	1,025.2	59.6	64.4

Note: EPFW = existing and proposed Floridan aquifer wells
NBRW = new brackish wells
NSWS = new surface water sources
NULW = new Upper and Lower Floridan aquifer wells
O&M = operation and maintenance

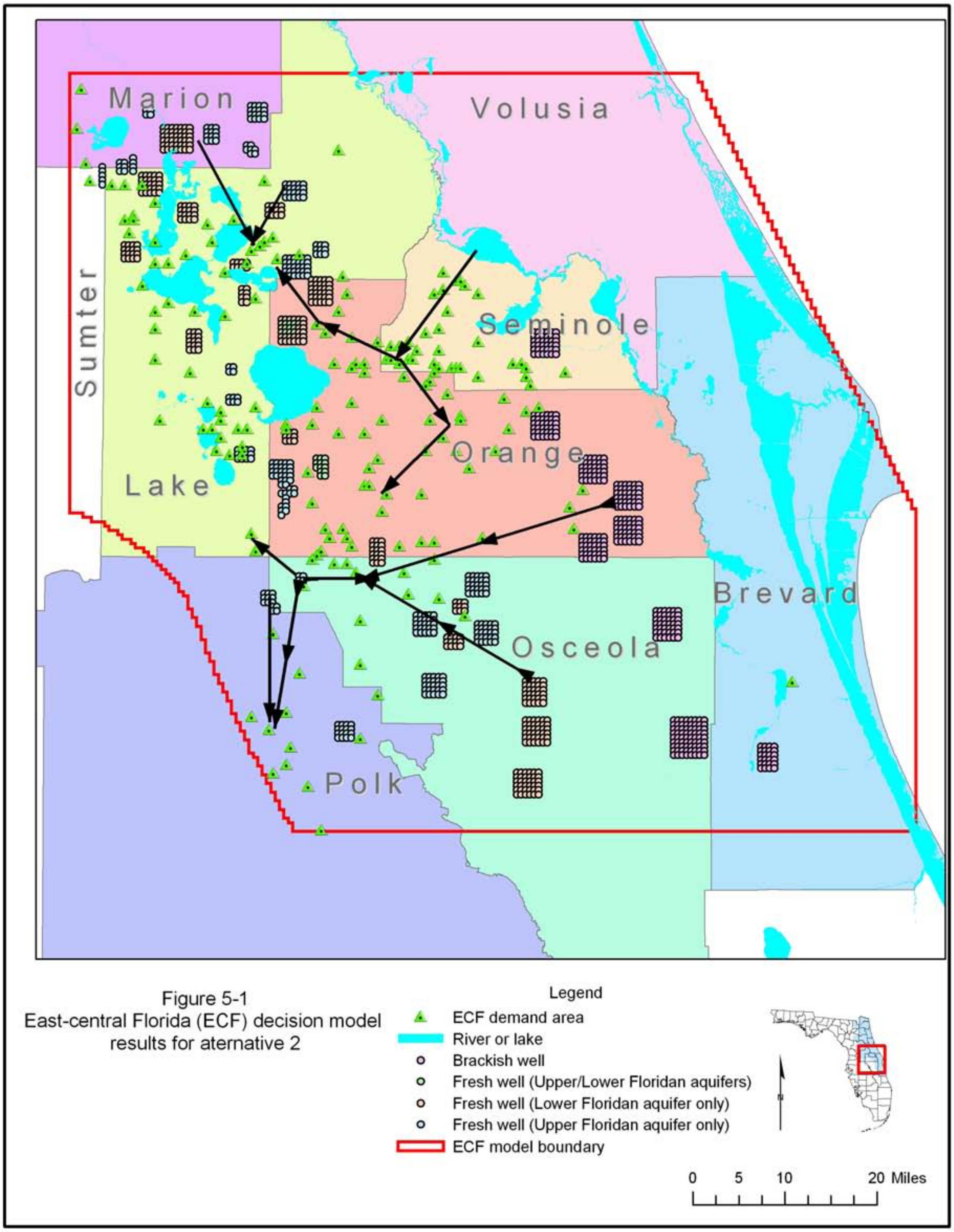

Figure 5-1
East-central Florida (ECF) decision model results for aternative 2

Table 5-4a. Decision model results for alternative 2u for the east-central Florida planning area

Alternative	New Supply Source	New Supply Used (mgd)	New Water Use by Demand Area (mgd)	
2u—Existing and proposed Floridan aquifer wells, new Floridan aquifer wells, and surface water sources (with leakance uncertainty)	Freshwater wells	26.1	Haines City	2.4
			Kissimmee	13.5
			Lake Grove Utilities	2.2
			Poinciana	1.8
			South Utilities Inc., South Lake	2.7
			St. Cloud	2.0
			Tavares	1.5
	Brackish wells	4.2	Kissimmee	4.2
	Surface water	96.8	Leesburg	11.1
			Apopka	14.4
			Lake Mary	2.8
			Longwood	3.2
			OCU West	14.7
			OUC SF	7.2
			OUC SJ	43.4

Note: mgd = million gallons per day
OCU = Orange County Utilities
OUC = Orlando Utilities Commission (SF indicates the service area is in the South Florida Water Management District; SJ indicates the service area is in the St. Johns River Water Management District)

Table 5-4b. Cost summary for alternative 2u

Source	Construction Cost ($M)	Capital Cost ($M)	O&M Cost ($M)	Equivalent Annual Cost ($M/yr)
EPFW	201.8	302.0	12.7	38.4
NULW	97.7	162.6	2.4	14.9
NBRW	25.5	40.8	2.3	5.9
NSWS	296.4	445.2	33.6	72.5
Total	621.4	950.6	51.0	131.7

Note: EPFW = existing and proposed Floridan aquifer wells
NBRW = new brackish wells
NSWS = new surface water sources
NULW = new Upper and Lower Floridan aquifer wells
O&M = operation and maintenance

surface water sources accounts for about 55% of the annualized cost. A visual representation of the decision model results for the alternative 2u run is shown in Figure 5-2.

Decision Model Run for Alternative 2WOLG

Alternative 2WOLG differs from alternative 2 only by the elimination of Lake Griffin as a candidate water source.

This alternative creates 26.4 mgd of new groundwater; it also relies on surface water sources to meet most of the 2020 demand. Using new fresh and brackish groundwater and surface water sources, alternative 2WOLG meets the 2020 demand by expanding existing new wellfields and treatment plants, developing seven new wellfields (five Upper Floridan aquifer, one Lower Floridan aquifer, and one brackish) with new treatment plants, and developing one surface water withdrawal and treatment plant. The three sources supply a total of 154.5 mgd. Of this total, about 80% comes from surface water. Table 5-5a shows the new sources with their associated demand, for average daily demand.

A summary of the cost estimates for alternative 2WOLG is shown in Table 5-5b. Equivalent annual cost totals about $150.4 million / year for this alternative. This total includes annualized costs for expanding existing facilities and developing new groundwater and surface water sources. Developing significant sources of new surface water accounts for about 62% of the annualized cost. A visual representation of the decision model's results for alternative 2WOLG is shown in Figure 5-3.

Decision Model Run for Alternative 4

Alternative 4 considers all existing, proposed, and potential new well sources and surface water. In addition to this, it incorporates a political constraint restricting the transport of water from new sources across a county boundary. Thus, new sources can be used to satisfy water demands only within the county of origin. This constraint does not affect existing cross-county transfers.

Alternative 4 meets 2020 demand by creating 182.1 mgd of additional water and expanding existing wellfields and treatment plants. It develops three new wellfields with new treatment plants and four surface water withdrawal

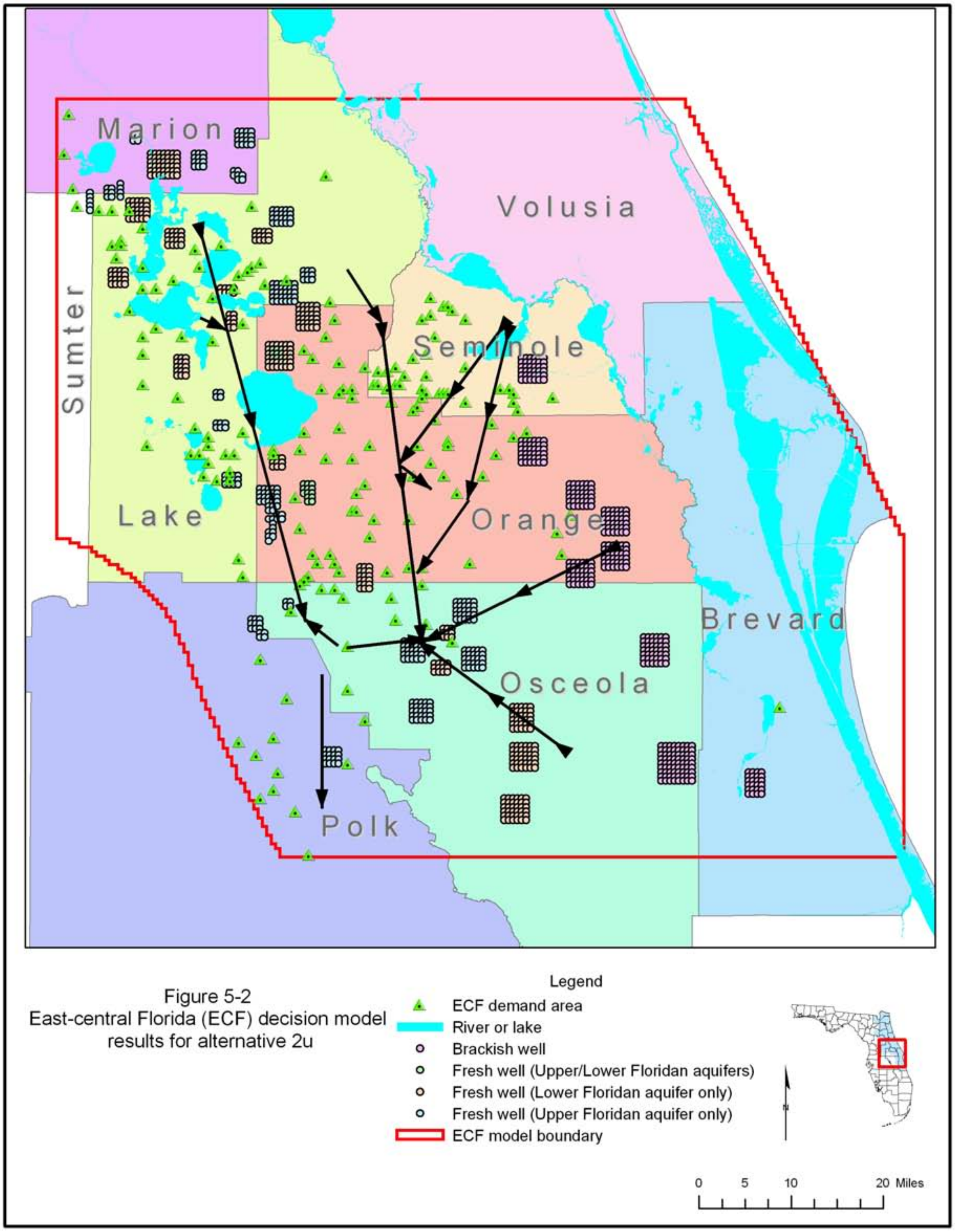

Figure 5-2
East-central Florida (ECF) decision model results for alternative 2u

Table 5-5a. Decision model results for alternative 2WOLG for the east-central Florida planning area

Alternative	New Supply Source	New Supply Used (mgd)	New Water Use by Demand Area (mgd)	
2WOLG—Existing and proposed Floridan aquifer wells, new Floridan aquifer wells, and surface water sources (without Lake Griffin)	Freshwater wells	26.4	Haines City	2.4
			Kissimmee	7.3
			Lake Grove Utilities	2.1
			Leesburg	6.2
			Poinciana	2.1
			South Utilities Inc., South Lake	2.7
			St. Cloud	2.0
			Tavares	1.6
	Brackish wells	4.2	Kissimmee	4.2
	Surface water	123.9	Altamonte Springs	3.1
			Apopka	14.4
			Eatonville	1.1
			Lake Mary	2.8
			Longwood	3.2
			Mt. Dora	2.7
			OCU West	15.0
			OUC SF	31.4
			OUC SJ	43.4
			Seminole County PWD	5.3
			Winter Haven	1.5

Note: mgd = million gallons per day
OCU = Orange County Utilities
OUC = Orlando Utilities Commission (SF indicates the service area is in the South Florida Water Management District; SJ indicates the service area is in the St. Johns River Water Management District)
PWD = Public Works Department

Table 5-5b. Cost summary for alternative 2WOLG

Source	Construction Cost ($M)	Capital Cost ($M)	O&M Cost ($M)	Equivalent Annual Cost ($M/yr)
EPFW	195.4	292.5	12.2	37.0
NULW	90.0	150.5	2.2	13.8
NBRW	25.5	40.8	2.3	5.9
NSWS	367.1	550.6	45.5	93.7
Total	678.0	1,034.4	62.2	150.4

Note: EPFW = existing and proposed Floridan aquifer wells
NBRW = new brackish wells
NSWS = new surface water sources
NULW = new Upper and Lower Floridan aquifer wells
O&M = operation and maintenance

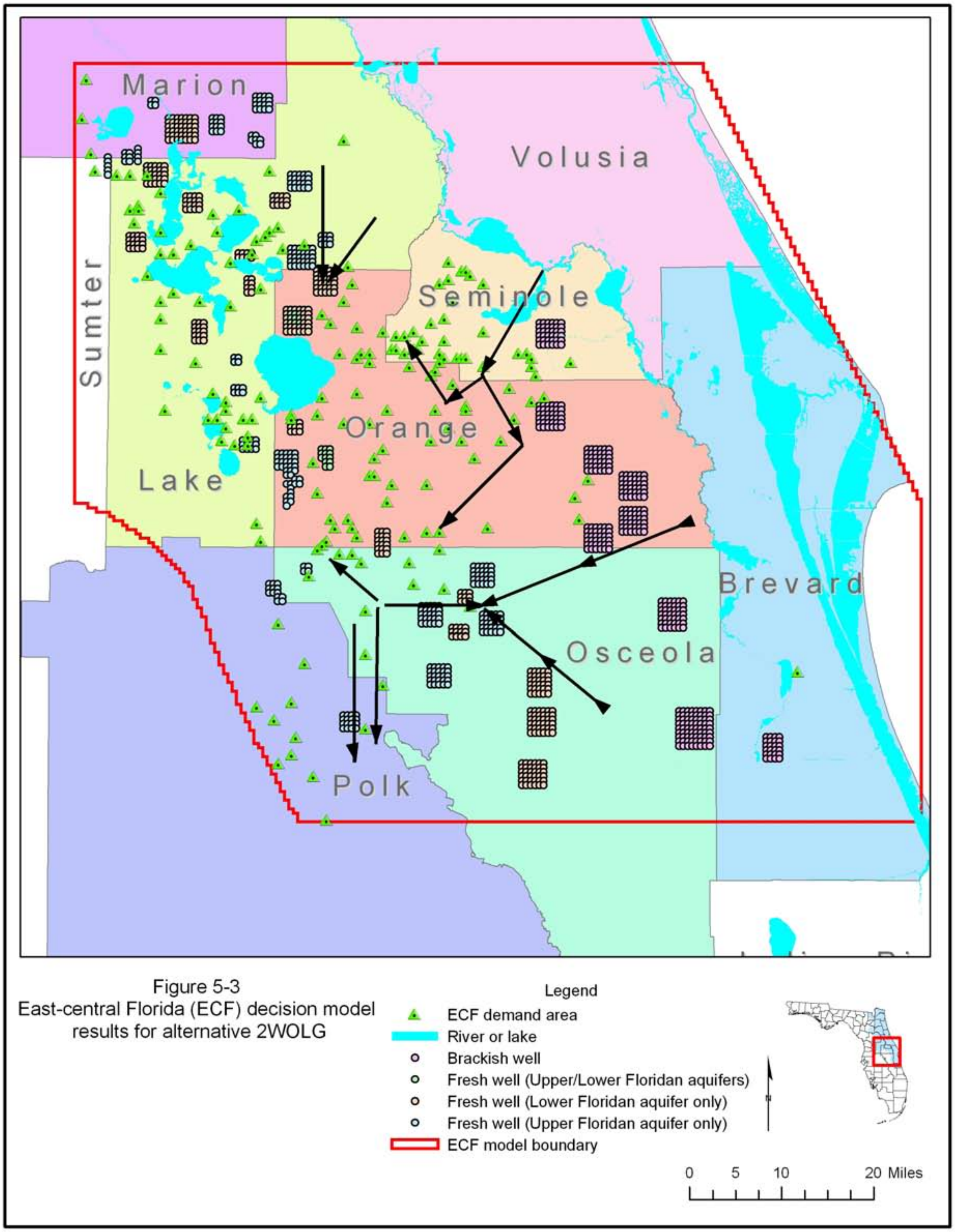

Figure 5-3
East-central Florida (ECF) decision model results for alternative 2WOLG

sources and treatment plants. Of the 180.1 mgd total, about 90.1% is obtained from surface water sources. Table 5-6a shows the new sources with their associated demand, for average daily demand.

A summary of the cost estimates for alternative 4 is shown in Table 5-6b. Equivalent annual cost totals about $172 million / year for this alternative. This total includes annualized costs for expanding existing facilities and developing new groundwater and surface water sources. Developing significant amounts of new surface water accounts for about 71% of the annualized cost. A visual representation of the decision model's results for alternative 4 is shown in Figure 5-4.

Decision Model Run for Alternative 6

Alternative 6 considers all existing, proposed, and candidate sources. It also considers political constraints which prohibit the transport of new water sources across water management district boundaries. This transport constraint does not affect any existing cross-boundary transfers.

Alternative 6 meets 2020 demand by relying heavily on the use of surface water sources. Out of the 207 mgd created, about 87% is supplied by surface water. Using these available new freshwater and surface water sources, alternative 6 meets demand by expanding existing wellfields and treatment plants. It develops four new wellfields with new treatment plants and three surface water sources and treatment plants. Table 5-7a shows the new sources with their associated demand, for average daily demand.

A summary of the cost estimates for alternative 6 is shown in Table 5-7b. Equivalent annual cost totals $180.5 million / year for this alternative. This total includes annualized costs for expanding existing facilities and developing new groundwater and surface water sources. Developing significant amounts of new surface water accounts for about 77% of the annualized cost. A visual representation of the decision model's results for alternative 6 is shown in Figure 5-5.

Table 5-6a. Decision model results for alternative 4 for the east-central Florida planning area

Alternative	New Supply Source	New Supply Used (mgd)	New Water Use by Demand Area (mgd)	
4—Existing and proposed Floridan aquifer wells, new Floridan aquifer wells, and surface water sources (with prohibition against transfer of water from new sources across county boundaries)	Freshwater wells	13.9	St. Cloud	2.0
			Kissimmee	9.8
			Poinciana	2.1
	Brackish wells	4.2	NA	NA
	Surface water	164.0	Altamonte Springs	4.3
			Apopka	14.4
			Eatonville	1.1
			Lake Mary	2.8
			Longwood	3.2
			Mt. Dora	2.7
			Reedy Creek Utilities	17.9
			OCU East	15.5
			OCU West	10.1
			OUC SF	31.4
			OUC SJ	43.4
			Seminole County PWD	5.8
			Eustis	1.2
			Leesburg	4.3
			Tavares	1.6
			Casselberry	4.3

Note: mgd = million gallons per day
NA = not applicable
OCU = Orange County Utilities
OUC = Orlando Utilities Commission (SF indicates the service area is in the South Florida Water Management District; SJ indicates the service area is in the St. Johns River Water Management District)
PWD = Public Works Department

Table 5-6b. Cost summary for alternative 4

Source	Construction Cost ($M)	Capital Cost ($M)	O&M Cost ($M)	Equivalent Annual Cost ($M/yr)
EPFW	198.6	297.3	12.4	37.7
NULW	37.4	61.4	1.4	6.1
NBRW	19.4	30.4	2.3	5.1
NSWS	503.4	756.8	57.0	122.9
Total	758.8	1,145.9	73.1	171.8

Note: EPFW = existing and proposed Floridan aquifer wells
NBRW = new brackish wells
NSWS = new surface water sources
NULW = new Upper and Lower Floridan aquifer wells
O&M = operation and maintenance

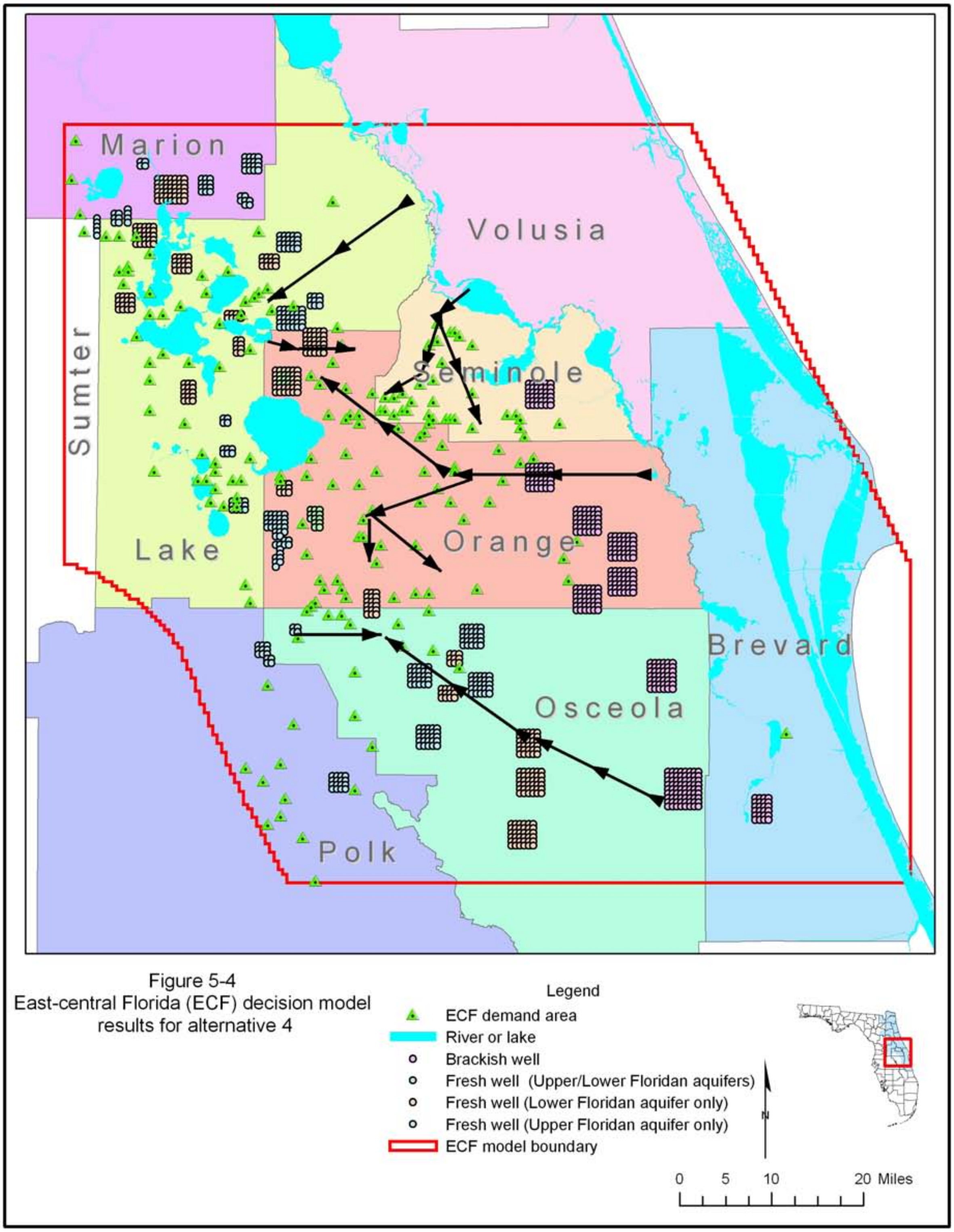

Figure 5-4
East-central Florida (ECF) decision model results for alternative 4

Table 5-7a. Decision model results for alternative 6 for the east-central Florida planning area

Alternative	New Supply Source	New Supply Used (mgd)	New Water Use by Demand Area (mgd)	
6—Existing and proposed Floridan aquifer wells, new Floridan aquifer wells, and surface water sources (with prohibition against transfer of water from new sources across water management district boundaries)	Freshwater wells	27.5	Poinciana	2.1
			Kissimmee	17.7
			Reedy Creek Utilities	5.0
			Southlake Utilities	2.7
	Surface water	179.5	Altamonte Springs	6.0
			Apopka	14.4
			Eatonville	1.1
			OCU	2.1
			OCU West	17.6
			OUC SF	31.4
			OUC SJ	43.4
			Oviedo	3.2
			Sanford	6.7
			Sanlando Utilities Corp.	6.3
			Seminole County PWD	13.9
			Eustis	3.0
			Leesburg	9.0
			Tavares	1.6
			Wildwood (Coleman FED)	4.8
			Winter Haven	9.9
			Winter Park	5.1

Note: mgd = million gallons per day
OCU = Orange County Utilities
OUC = Orlando Utilities Commission (SF indicates the service area is in the South Florida Water Management District; SJ indicates the service area is in the St. Johns River Water Management District)
PWD = Public Works Department

Table 5-7b. Cost summary for alternative 6

Source	Construction Cost ($M)	Capital Cost ($M)	O&M Cost ($M)	Equivalent Annual Cost ($M/yr)
EPFW	162.5	343.6	9.2	30.1
NULW	75.4	125.0	2.4	12.1
NBRW	NA	NA	NA	NA
NSWS	528.5	793.3	68.8	138.3
Total	766.4	1,261.9	80.4	180.5

Note: EPFW = existing and proposed Floridan aquifer wells
NBRW = new brackish wells
NSWS = new surface water sources
NULW = new Upper and Lower Floridan aquifer wells
O&M = operation and maintenance

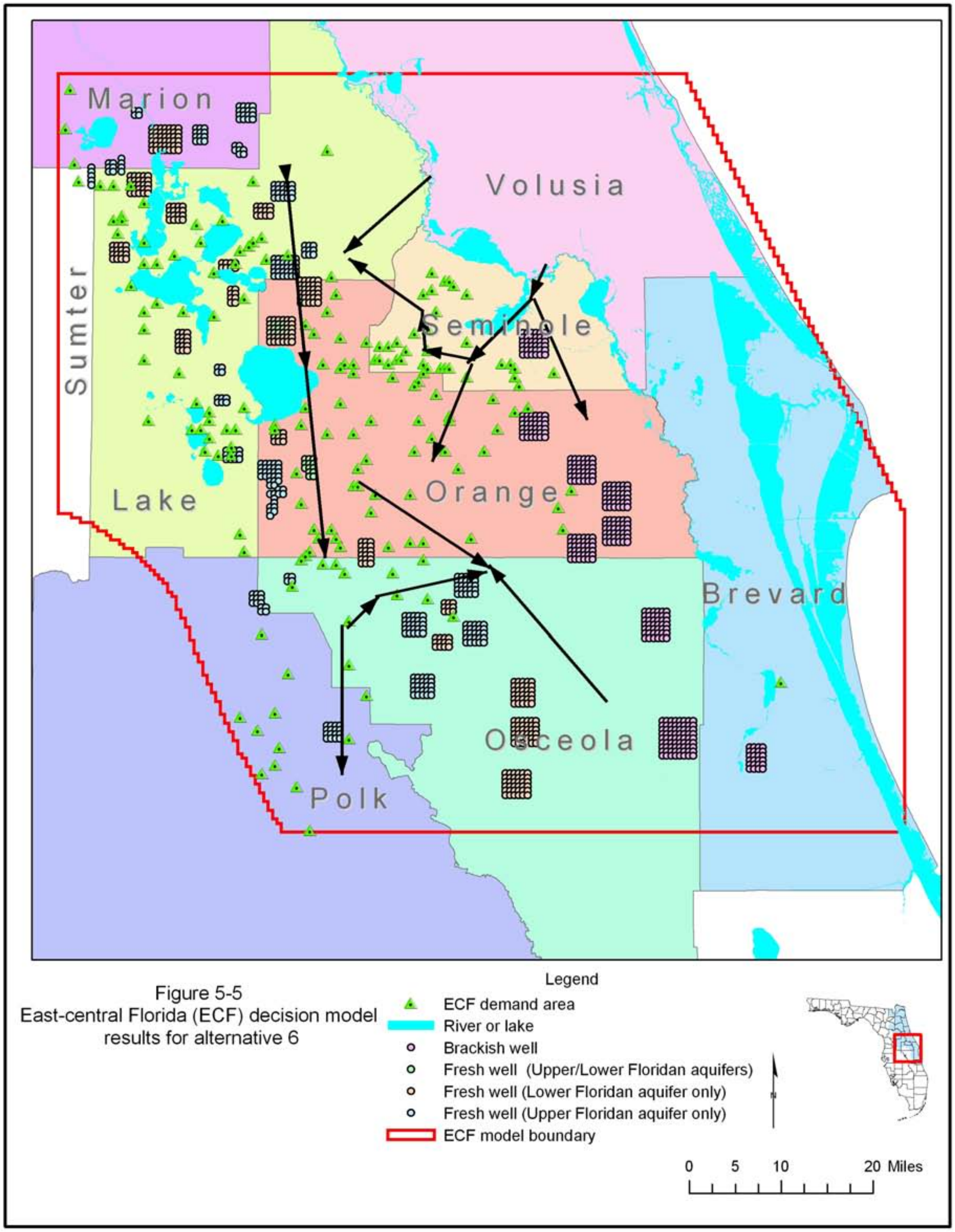

Figure 5-5
East-central Florida (ECF) decision model results for alternative 6

Decision Model Run for Alternative 8

Alternative 8 considers all existing, proposed, and candidate sources. It also prohibits the transport of water supplies across a county line or water management district boundary and the use of brackish water. This alternative is the same as alternative 4 except for the limitation of the use of brackish water.

Alternative 8 meets 2020 demand by relying on 239.2 mgd of surface water sources and 31.8 mgd of new groundwater. New supply is provided by three new wellfields and four surface water withdrawal points. Using available new fresh groundwater and surface water sources, alternative 8 meets 2020 demands by expanding existing wellfields and treatment plants. Table 5-8a shows the new sources with their associated average daily demand.

A summary of the cost estimates for alternative 8 is shown in Table 5-8b. Equivalent annual cost totals $200.7 million / year for this alternative. This total includes annualized costs for expanding existing facilities and developing new groundwater and surface water sources. Developing significant amounts of new surface water supplies accounts for about 80% of the annualized cost. A visual representation of the decision model's results for alternative 8 is shown in Figure 5-6.

SUMMARY

The following conclusions can be drawn from the decision model runs analyses:

1. The results suggest that during the Water 2020 planning horizon, given the projected quantity and spatial distribution of groundwater withdrawals, east-central Florida will potentially reach the limit of sustainable withdrawals from the Floridan aquifer within the next 10 years.

2. The decision model seldom chose brackish groundwater as a source option if either groundwater or surface water, or both, were available. This may be due to two reasons: (a) the high cost associated with the treatment of brackish groundwater and (b) the fact that the withdrawal of brackish water from the Upper Floridan aquifer, as with normal production wells, adds to the cumulative drawdown at control points.

Table 5-8a. Decision model results for alternative 8 for the east-central Florida planning area

Alternative	New Supply Source	New Supply Used (mgd)	New Water Use by Demand Area (mgd)	
8—Existing and proposed Floridan aquifer wells, new Floridan aquifer wells, and surface water sources (with prohibition against transfer of water from new sources across either county or water management district boundaries)	Freshwater wells	31.8	Poinciana	2.1
			OCU South	1.2
			Kissimmee	17.7
			Reedy Creek Utilities Co.	8.8
			St. Cloud	2.0
	Surface water	239.2	Clermont	4.5
			Eustis	3.0
			Lake Grove Utilities	2.2
			Mt. Dora	2.7
			South Lake Utilities Inc.	2.7
			Tavares	1.6
			Eatonville	1.1
			Maitland	1.7
			Ocoee	4.0
			OCU West	17.6
			Orange County Utilities	2.1
			OUC SJ	43.4
			Zyderveld Joost P. Johns	19.1
			Leesburg	8.9
			Altamonte Springs	6.0
			Apopka	14.4
			Casselberry	4.3
			Clermont	4.5
			Eatonville	1.1
			Lake Mary	2.8
			Longwood	3.2
			Maitland	1.7
			OCU East	19.1
			Duplicate	17.6
			Oviedo	3.2
			Sanford	6.7
			Sanlando Utilities Corp.	6.3
			Seminole County PWD	13.9
			Wildwood (Coleman FED)	4.8
			Winter Haven	9.9
			Winter Park	5.1

Note: mgd = million gallons per day
OCU = Orange County Utilities
OUC = Orlando Utilities Commission (SJ indicates the service area is in the St. Johns River Water Management District)
PWD = Public Works Department

Table 5-8b. Cost summary for alternative 8

Source	Construction Cost ($M)	Capital Cost ($M)	O&M Cost ($M)	Equivalent Annual Cost ($M/yr)
EPFW	156.3	234.4	8.7	28.8
NULW	62.4	101.5	2.7	10.6
NBRW	NA	NA	NA	NA
NSWS	673.0	1,017.0	73.1	161.3
Total	891.7	1,352.9	84.5	200.7

Note: EPFW = existing and proposed Floridan aquifer wells
NA = not applicable
NBRW = new brackish wells
NSWS = new surface water sources
NULW = new Upper and Lower Floridan aquifer wells
O&M = operation and maintenance

3. Future demands in areas of western Seminole County and in adjacent Orange County tend to be met by surface water while those in Lake County tend to be met by a combination of groundwater and surface water.

4. The effect of applying political boundary restrictions is to increase the cost of developing new sources. Applying restrictions against either intercounty or inter-District transfer increases cost by approximately 10–15% in each case. Applying both restrictions together results in a 30% cost increase.

5. Generally, the model recommends the use of surface water sources to meet the demands of the 2020 projected year. Only about 15–20% of the deficit could be satisfied through the construction of new wells. A possible reason as to why the decision model favors the use of surface water is to avoid possible harm to the ecosystem that could arise through the excessive use of groundwater.

6. Whenever new wells were selected, a greater proportion of the flow was allocated to the use of freshwater wells than to the use of brackish water wells, the reason being the cost associated with the treatment of brackish water.

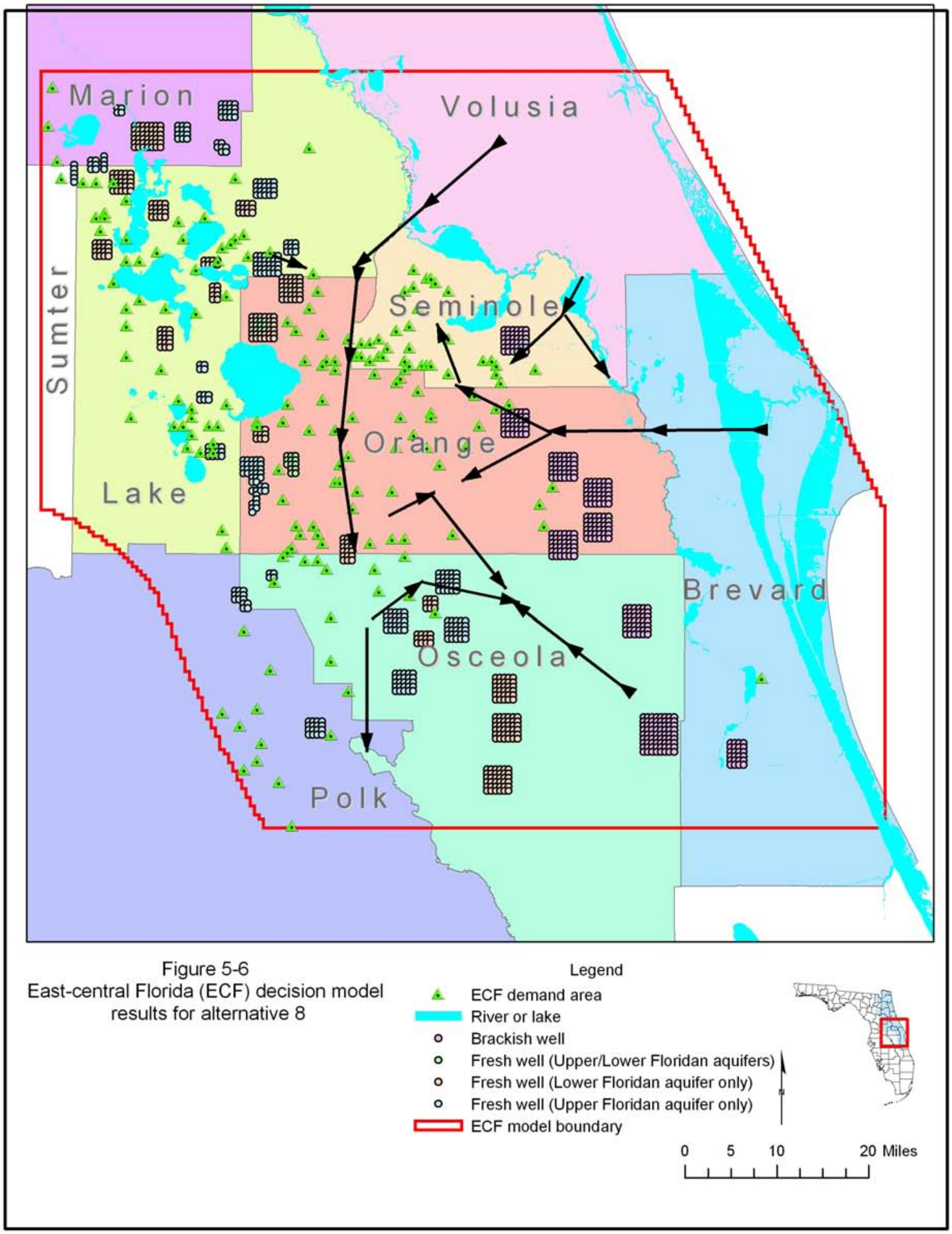

Figure 5-6
East-central Florida (ECF) decision model results for alternative 8

CONCLUSIONS AND RECOMMENDATIONS

CONCLUSIONS

Results from the optimization study have shown that east-central Florida may reach the limit of sustainable water supply from its aquifer by the year 2020. While the deficit model showed a significant amount of water deficit, the results of the decision model indicated that the use of surface water may be a likely alternative to alleviate the water deficit problem. Irrespective of the cost of construction and maintenance, the decision model indicated in all of its runs that most of the deficit that may accrue from present to the 2020 planning horizon can be accounted for by withdrawing water from the abundant surface water sources along the St. Johns River. Although the cost of constructing and maintaining water treatment facilities for surface sources may be more expensive than for groundwater sources, users of the ECF groundwater resource must be encouraged to at least supplement their day-to-day groundwater use with surface water from the abundant sources in the ECF region if future problems of water deficits are to be avoided.

RECOMMENDATIONS

Based on the results and experience from the model runs, the following may be recommended for future optimization study in the ECF model region.

The results of the model runs indicated that the use of surface water sources is a very likely alternative if water deficits in the ECF model region are to be avoided. It is therefore recommended that seekers of water resources be encouraged to consider surface water as a suitable alternative to relying exclusively on groundwater.

Water quality is a very important issue in the aquifers of east-central Florida. Because some portions of the ECF model region share boundaries with the coast, the problem of the occurrence of saltwater upconing due to freshwater pumping has been known to exist in some parts of the model region. It is therefore recommended that the current constraints in the optimization model be extended to incorporate water quality issues in future studies.

The response of the surficial aquifer system to pumping and recharge has been known to be nonlinear. The flow model used to generate the influence coefficients for the optimization models assumes this response to be linear. To improve upon future predictions of both optimization and flow models, this nonlinear response should be simulated as it is. It is recommended that a flow model that can handle the nonlinearities in the surficial aquifer response be used in future studies.

It is very common for a short-term optimization project to be proposed. If, for example, a project is proposed, the time period of which spans a horizon of 5 years, it will not be meaningful to generate the unit response matrix with a steady-state flow model. Such a short-term horizon project may exhibit considerable transient or dynamic effects that must be accounted for in the modeling process. Therefore, it is recommended that, in future, the current models be extended to incorporate transient or dynamic effects in their structure.

Generation of the unit response matrix (or influence coefficients) required for developing the optimization models (deficit and decision) is the major time-consuming part of the entire optimization model process. This is because the steady-state version of the ECF flow model has to be run a large number of times in order to generate the unit response matrix for the public supply wells with respect to the wetland control points and spring locations. The length of time (which easily runs into days) required to generate the unit response matrix depends on the number of public supply wells considered in the optimization model. To reduce the time associated with this, it is recommended that a new approach of generating the unit response matrix be adopted and incorporated into the entire modeling process. Adopting the parallel computing approach by sharing the computational burden for other processors can be an attractive option for future consideration.

Nowadays, graphical representation of data or modeling results is common. It offers a very fast, easy, and convenient means of interpreting data in an enhanced visual format. It is strongly recommended that as the optimization modeling practices of SJRWMD move forward into the future, the modeling software be improved to incorporate tools that will enhance both the pre- and post-processing of input and output data, respectively.

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