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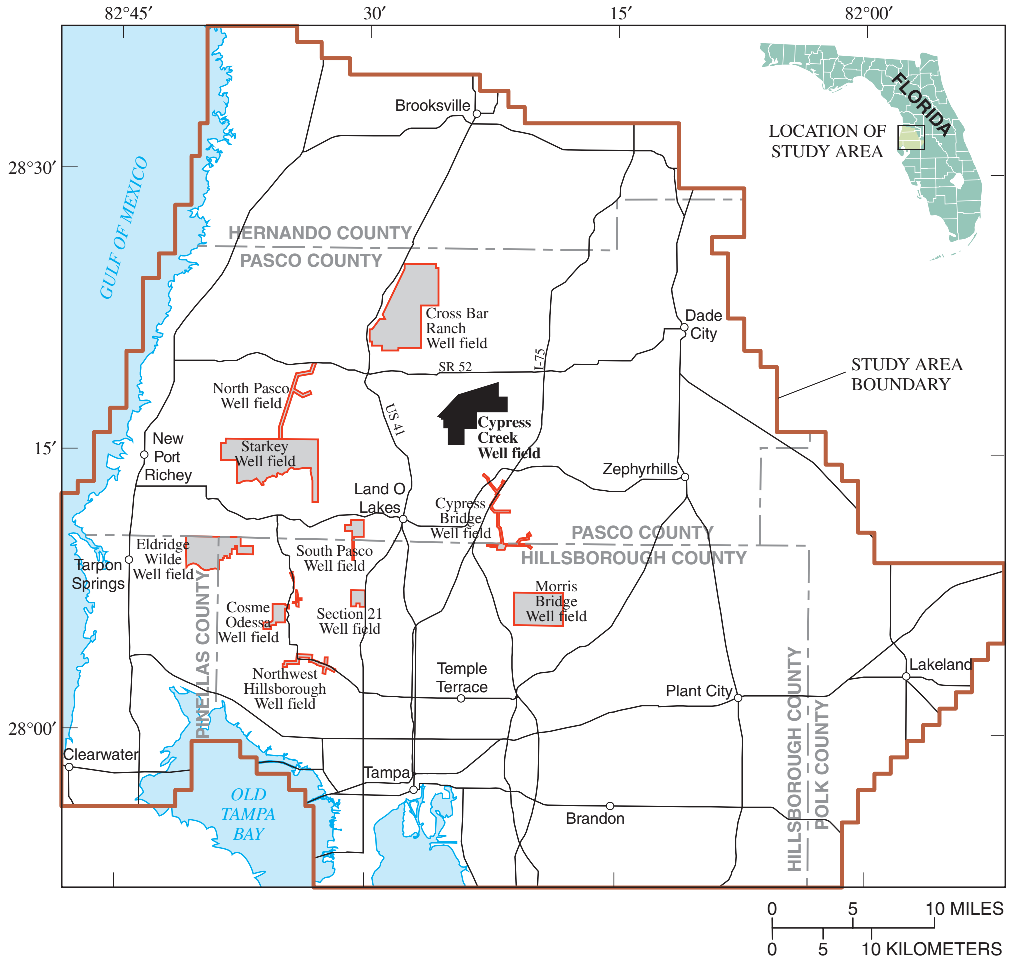
Testing the Sensitivity of Pumpage to Increases in Surficial Aquifer System Heads in the Cypress Creek Well-Field Area, West-Central Florida—An Optimization Technique

Water-Resources Investigations Report 02-4086



U.S. Geological Survey

Prepared in cooperation with

TAMPA BAY WATER

and the

SOUTHWEST FLORIDA WATER MANAGEMENT DISTRICT

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By Dann K. Yobbi

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Tallahassee, Florida
2002

U.S. DEPARTMENT OF THE INTERIOR
GALE A. NORTON, Secretary

U.S. GEOLOGICAL SURVEY
CHARLES G. GROAT, Director

For additional information write to:

District Chief
U.S. Geological Survey
Suite 3015
227 N. Bronough Street
Tallahassee, FL 32301

Copies of this report can be purchased from:

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CONVERSION FACTORS AND ABBREVIATIONS

Multiply inch-pound unit	By	To obtain
	Length	
foot (ft)	0.3048	meter
mile (mi)	1.6090	kilometer
	Area	
acre	0.4047	hectare
square foot (ft^2)	0.0920	square meter
square mile (mi^2)	2.5900	kilometer square
	Flow	
inch per year (in/yr)	25.4	millimeter per year
million gallons per day (Mgal/d)	0.04381	cubic meter per second
	Hydraulic Conductivity	
foot per day (ft/d)	0.3048	meter per day
	Transmissivity*	
foot squared per day (ft^2/d)	0.0920	meter squared per day
	Leakance	
foot per day per foot (ft/d)/ft	1.0	meter per day per meter

**Transmissivity:* The standard unit for transmissivity is cubic foot per day per square foot times foot of aquifer thickness [(ft^3/d)/ft^2]ft. In this report, the mathematically reduced form, foot squared per day (ft^2/d), is used for convenience.

Sea level: In this report, “sea level” refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929)---a geodetic datum derived from a general adjustment of the first-order level nets of the United States and Canada, formerly called “Sea Level Datum of 1929.” Horizontal coordinate information is referenced to the North American Datum of 1927 (NAD of 1927).

ACRONYMS USED IN THIS REPORT

CNTB	Central Northern Tampa Bay
ET	evapotranspiration
MODFLOW	U.S. Geological Survey Modular Three-Dimensional Ground-Water Flow Model
MODOPTIM	computer program linking MODFLOW with an optimization routine
OROP	Optimized Regional Operations Plan
SDI	SDI Environmental Services, Inc.
SS	sum-of-squares residuals
SWFWMD	Southwest Florida Water Management District
TBW	Tampa Bay Water
USGS	U.S. Geological Survey

Testing the Sensitivity of Pumpage to Increases in Surficial Aquifer System Heads in the Cypress Creek Well-Field Area, West-Central Florida–An Optimization Technique

By Dann K. Yobbi

Abstract

Tampa Bay depends on ground water for most of the water supply. Numerous wetlands and lakes in Pasco County have been impacted by the high demand for ground water. Central Pasco County, particularly the area within the Cypress Creek well field, has been greatly affected. Probable causes for the decline in surface-water levels are well-field pumpage and a decade-long drought. Efforts are underway to increase surface-water levels by developing alternative sources of water supply, thus reducing the quantity of well-field pumpage.

Numerical ground-water flow simulations coupled with an optimization routine were used in a series of simulations to test the sensitivity of optimal pumpage to desired increases in surficial aquifer system heads in the Cypress Creek well field. The ground-water system was simulated using the central northern Tampa Bay ground-water flow model. Pumping solutions for 1987 equilibrium conditions and for a transient 6-month timeframe were determined for five test cases, each reflecting a range of desired target recovery heads at different head control sites in the surficial aquifer system. Results are presented in the form of curves relating average head recovery to total optimal pumpage. Pumping solutions are sensitive to the location of head control sites formulated in the optimization problem and as expected, total optimal pumpage decreased when desired target head increased. The distribution of optimal pumpage for individual production wells also was significantly affected by the location of head control sites.

A pumping advantage was gained for test-case formulations where hydraulic heads were maximized in cells near the production wells, in cells within the steady-state pumping center cone of depression, and in cells within the area of the well field where confining-unit leakance is the highest. More water was pumped and the ratio of head recovery per unit decrease in optimal pumpage was more than double for test cases where hydraulic heads are maximized in cells located at or near the production wells. Additionally, the ratio of head recovery per unit decrease in pumpage was about three times more for the area where confining-unit leakance is the highest than for other leakance zone areas of the well field. For many head control sites, optimal heads corresponding to optimal pumpage deviated from the desired target recovery heads. Overall, pumping solutions were constrained by the limiting recovery values, initial head conditions, and by upper boundary conditions of the ground-water flow model.

INTRODUCTION

The Tampa Bay area relies heavily on ground water for public supply. Much of the public water supply in Hillsborough, Pinellas, and Pasco Counties is from a regional system of 11 interconnected well fields operated by Tampa Bay Water (TBW) (fig. 1). These well fields currently supply about 60 percent of the drinking water for the tri-county area. Current year (2001) withdrawals from the 11 well fields are permitted for 158 million gallons per day (Mgal/d). Numerous wetlands and lakes have disappeared in Pasco County since about 1989. One of the areas that has shown considerable decline in ground-water levels is in the Cypress Creek well field, located in central Pasco County (fig. 1). Probable causes for the decline in the surface-water levels are well-field pumpage and a decade-long drought. Efforts are underway to increase surface-water levels in the affected area by developing alternative sources of water supply so that well-field pumpage can be cut to 121 Mgal/d by the year 2002 and 90 Mgal/d by the year 2007.

A variety of techniques have been used to analyze the water-supply problems in west-central Florida, including the use of numerical ground-water flow models. A common use of the numerical models is to predict the response of an aquifer to planned

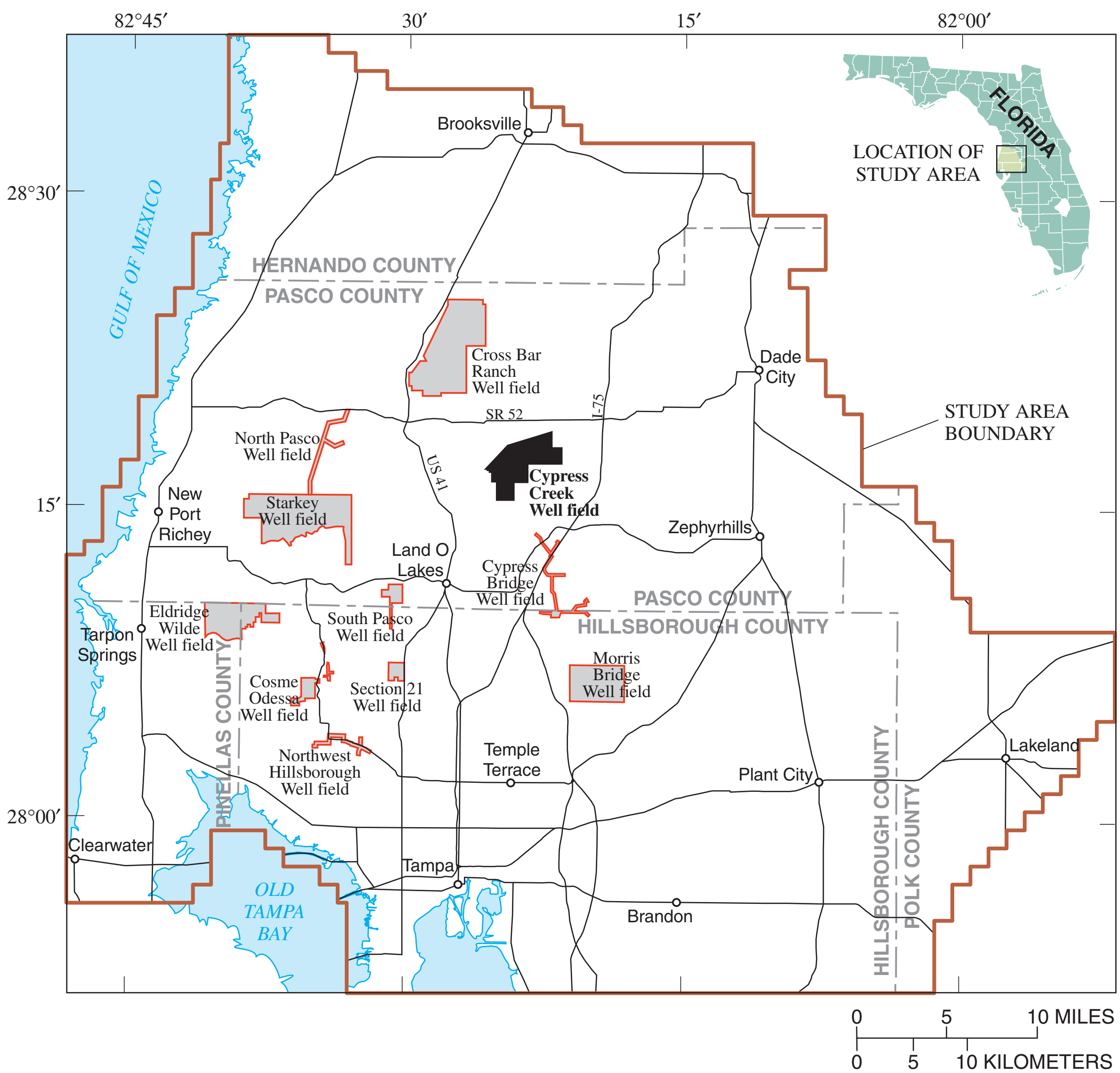

Figure 1. Location of study area and well fields.

stress. Use of these models involves simulating one strategy at a time, and from a group of related simulations, selecting the best strategy. However, simulating one strategy at a time does not guarantee an optimal or a completely effective strategy.

In 1998, TBW developed an Optimized Regional Operations Plan (OROP) for management of the 11 regional well fields (Tampa Bay Water, 1998). The OROP utilizes an integrated hydrologic optimization model to manage 172 production wells at the 11 well fields. The management objective of the optimization model is to maximize ground-water levels at a select set of surficial aquifer system monitoring sites, called control points, while meeting projected demands. The optimization model optimizes biweekly production schedules for each of the 11 well fields (172 wells) and seeks to maximize ground-water levels at a selected set of monitoring sites (currently 32 surficial aquifer system monitoring wells). The formulation accounts for variations in future water levels for a specified forecast period, and applies preferential weighting of prevailing ground-water levels to established target levels at monitor sites (Tampa Bay Water, 1998). The monitor wells were selected under the assumption that they would provide reasonable locations to monitor hydrologic stress due to well-field pumpage.

To facilitate the routine use of optimization techniques, the U.S. Geological Survey (USGS) in cooperation with Tampa Bay Water and the Southwest Florida Water Management District (SWFWMD), began a study in 1997 to apply these techniques to the existing central northern Tampa Bay (CNTB) area hydrologic flow model. The study included two phases:

(1) application of optimization techniques to the existing ground-water flow model of the central northern Tampa Bay area to determine parameter sensitivities and correlations and to estimate parameter values of the existing model; and,
(2) development of an optimization model to test the sensitivity of optimal pumpage to increases in surficial aquifer system heads at selected sites in and around the Cypress Creek well field.

Phase one was described by Yobbi (2000) and addresses the parameter estimation issue. Phase two is described in this report and addresses the management issue.

Calibration and management problems must be addressed because the relative worth of a ground-water management solution is dependent on how well the underlying ground-water flow model is calibrated. The use of one optimization tool, such as the one used in this study, eases the transition from model calibration to ground-water management and reduces the learning curve for users.

Purpose and Scope

The purpose of this report is to describe the use of an optimization technique to test the sensitivity of optimal pumpage to desired increases in surficial aquifer system heads at selected locations in the Cypress Creek well-field area of central Pasco County. The hydraulic response of the aquifer system was solved using an optimization algorithm (Halford, 1992) combined with MODFLOW (McDonald and Harbaugh, 1988). The report includes brief descriptions of the Cypress Creek well field, the ground-water flow system, and ground-water flow and optimization models. The remainder of the report is a discussion of optimization modeling results. Descriptive information about the hydrogeology and the ground-water flow model is contained in reports by SDI Environmental Services, Inc. (SDI, 1997) and Yobbi (2000).

Description of the Cypress Creek Well Field

The Cypress Creek well field is about 7 mi^2 in areal extent and is located in central Pasco County between U.S. Highway 41 to the west and Interstate 75 to the east (fig. 1). The Cypress Creek well field is bisected by Cypress Creek, which flows southward through the center of the well field (fig. 2). The creek is poorly defined as it meanders through a large, swampy area within the well field. Most of the well field lies within a natural, relatively undisturbed area, which is surrounded by agricultural and residential areas. About 70 percent of the well field is classified as a wetland or lake (Perry, 1987), because the water table is at or above the land surface for part of the year; the remaining area is dry uplands that gently slope to Cypress Creek. Several wetlands in the Cypress Creek well field are augmented with ground water to protect the native flora and fauna.

Ground-Water Flow System

The ground-water flow system in the area is a multilayered system consisting of a thick sequence of carbonate rocks overlain by clastic

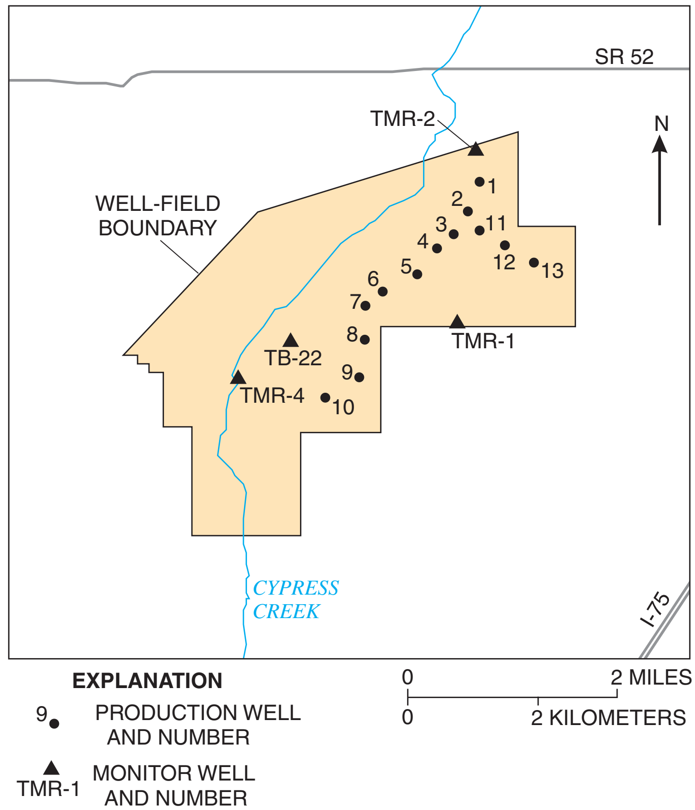

Figure 2. Location of production and selected monitor wells in the Cypress Creek well field.

deposits (fig. 3), and is described in detail by Ryder (1978). The deposits and carbonate rocks are subdivided into a hydrogeologic framework that forms a sequence of two aquifers and one confining unit. This framework includes the unconfined surficial aquifer system and the confined Floridan aquifer system. A leaky intermediate confining unit separates the aquifers. The Floridan aquifer system consists of the Upper and Lower Floridan aquifers that are separated by a middle confining unit. The middle confining unit contains saltwater in the study area, and freshwater flow is limited to the Upper Floridan aquifer. Recharge to the Upper Floridan aquifer is derived from the overlying surficial aquifer system by downward leakage through the intermediate confining unit. Part of this recharge is returned to the surficial deposits within the area as upward leakage, and most of the remainder leaves the area as it flows downgradient within the Upper Floridan aquifer.

About 30 million gallons of ground water is pumped daily from 13 production wells in the Cypress Creek well field (fig. 2). All the wells are completed in the Upper Floridan aquifer with depths ranging from about 500 to 700 ft. The major production zone is from two cavernous zones of a dolomitic section of the Avon Park Formation, approximately 400 to 500 ft below sea level (Ryder, 1978). Many surficial aquifer system and Upper Floridan aquifer monitor wells have been constructed to monitor ground-water levels in and around the well field.

DESCRIPTION OF MODELS

Ground-water flow was simulated using the USGS modeling software code MODFLOW (McDonald and Harbaugh, 1988). Optimization modeling is performed by linking the flow model code with an optimization routine (Halford, 1992). The combined ground-water flow code and optimization code is called MODOPTIM. The optimization routine determines the pumping rates that are passed in each call to MODFLOW and progresses toward a pumping solution by comparing heads determined by MODFLOW from one call to the next. MODFLOW is called repeatedly by the optimization routine until an optimal pumpage scheme is determined that maximizes the objective function.

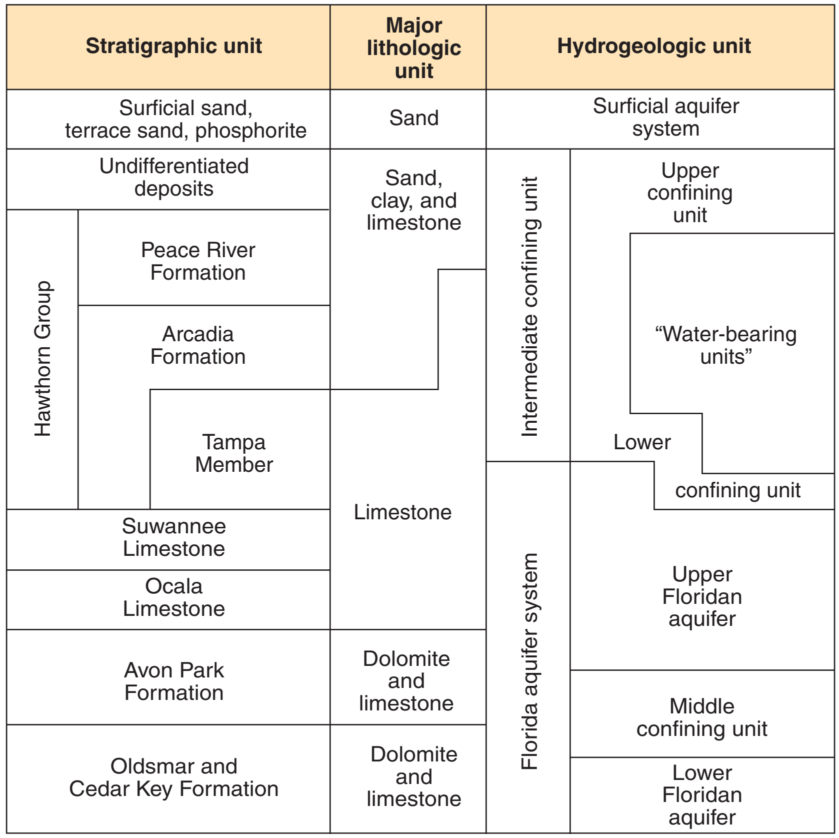

Figure 3. Generalized stratigraphic and hydrogeologic section, west-central Florida. (From Yobbi, 2000.)

Ground-Water Flow Model

No optimization model can be developed without first having a calibrated ground-water flow model. The CNTB area hydrologic model developed by SDI (1997) and modified by Yobbi (2000) was used in this study. The SDI model was calibrated to transient conditions using hydrologic data from 1971 to 1993 (approximately 1,200 weeks). The calibration period was the 12-year period from 1976 through 1987. Simulation of the 5-year period from 1971 through 1975 prior to calibration was used to stabilize water levels and flows in the model. The 6-year period from 1988 through 1993, following the calibration period was chosen as the model verification period.

The model has an area of approximately 2,000 mi^2 that includes all of Pasco County, most of Hernando, Pinellas and Hillsborough Counties, and part of Polk County. Six rivers (Alafia, Anclote, Hillsborough, Pithlachascotee, Weeki Wachee, and Withlacoochee) and their tributaries, several small streams along the coast, and some internally drained systems that flow only during extreme rainfall events, define the surface-water system of the model area (fig. 4). The two largest systems are the Hillsborough and Withlacoochee Rivers. Hundreds of lakes, swampy plains, and intermittent ponds are dispersed throughout the study area, ranging in size from less than 1/4 acre to more than 2,500 acres. A total of 17 springs are located in the model area and are either found inland flowing to adjacent rivers or along the coast discharging directly to the Gulf of Mexico.

Areally, the model is divided into a grid of 131 rows by 121 columns (fig. 5). The smallest cells, located in the center of the model, cover 0.25 mi^2; largest cells cover

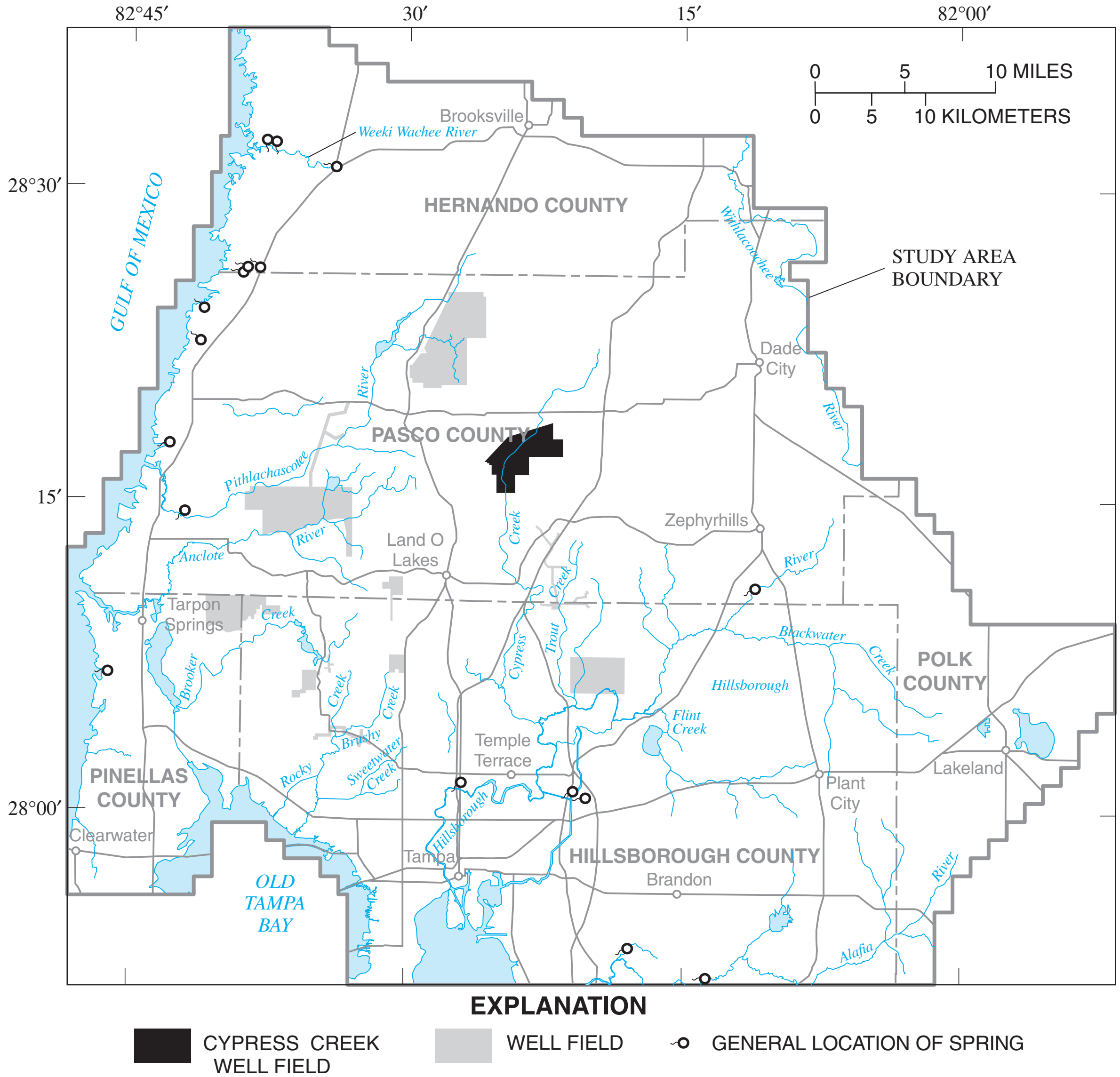

Figure 4. Location of rivers and springs.

1 mi². The ground-water flow model consists of two layers. The upper layer (layer 1) represents the surficial aquifer system as an unconfined layer. The lower layer (layer 2) represents the Upper Floridan aquifer as a confined/unconfined layer. Vertical leakage through the intermediate confining unit is simulated implicitly using a leakance array. A high leakance value (0.35 (ft/d)/ft) simulates the absence of the confining unit.

Rivers are simulated as river cells in layer 1 and, in those locations in model layer 2 where rivers are probably in direct hydraulic connection with the Upper Floridan aquifer. Wetlands are modeled as river cells in layer 1. Springs are represented by a head-dependent drain function, and discharge is linearly related to head differences between the spring pool and the potentiometric surface in layer 2.

Several boundary conditions are used to define the lateral extent of the simulated ground-water flow system. Based on regional ground-water flow, most of the lateral extent in layer 1 is a no-flow boundary coinciding with flow lines. Conditions coinciding with the coastline of the Gulf of Mexico and Tampa Bay are represented in the model as specified heads. In layer 2, the southeastern and most of the northern boundaries are no-flow bound-

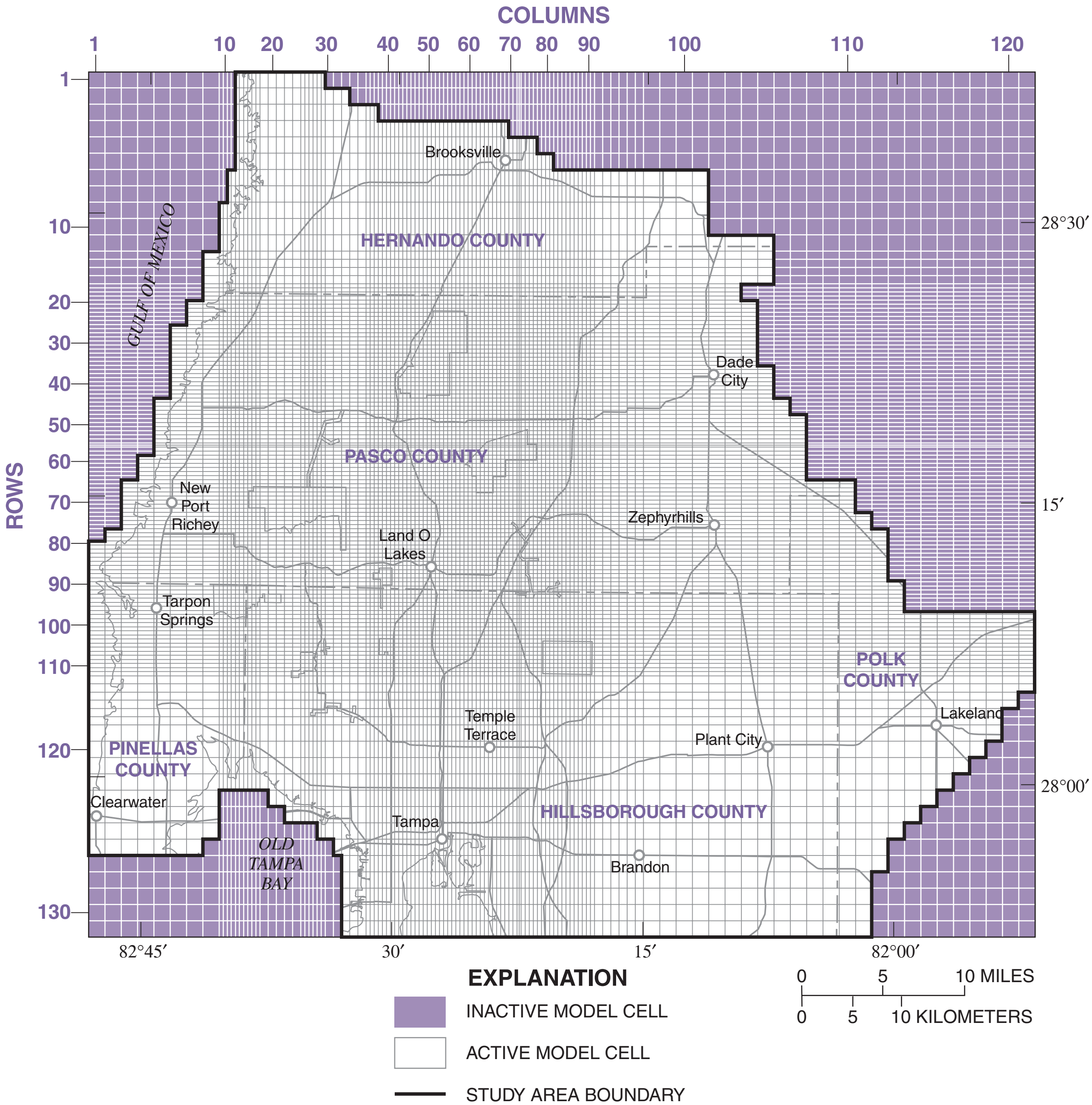

Figure 5. Model grid used in the simulation of ground-water flow system. (Modified from SDI Environmental Services, Inc., 1997).

aries representing flow lines in the Upper Floridan aquifer. Part of the northern boundary is represented by a general head boundary in layer 2 simulating flow out of the model. The extreme eastern edge of the model is represented as a specified head boundary. The coastline is represented as a no-flow boundary. Figures 7 and 8 in Yobbi (2000, pp. 9-10) show distribution of the boundary cells.

Recharge to the ground-water flow system averages 9.6 in/yr. Part of the recharge (2.0 in/yr) contributes to additional evapotranspiration (ET) and surface runoff. Simulated net leakage between the surficial aquifer system and the Upper Floridan aquifer was 6.8 in/yr downward, which represented 50 percent of the total flows in the surficial aquifer system. The quantity of total flow is in agreement with high leakance characteristics of the intermediate confining unit. Consequently, hydrologic conditions in the surficial aquifer system can significantly affect conditions in the Upper Floridan aquifer. Likewise, hydrologic conditions in the Upper Floridan aquifer can substantially affect conditions in the surficial aquifer system.

Input data were obtained directly from the SDI (1997) and the USGS (Yobbi, 2000) models and included: starting water-level values, hydraulic conductivity of the surficial aquifer system, bottom altitude of the surficial aquifer system, riverbed conductance values and altitudes, intermediate aquifer system leakance, transmissivity of the Upper Floridan aquifer, horizontal anisotropy values of the Upper Floridan aquifer, boundary heads and boundary conductance values for the Upper Floridan aquifer, pumpage rates, and drain altitudes and conductances for the surficial aquifer system and the Upper Floridan aquifer. Input data arrays also included specified recharge and discharge rates to/from the surficial aquifer system. Net recharge rates were calculated separately by subtracting ET from recharge in each model cell, thus in cells where ET exceeds recharge, a negative value of recharge was obtained. Rejected recharge simulated in the SDI (1997) model was simulated in this model by drains, where the elevation of the drain was set at land surface.

Hydraulic conductivity values for the surficial aquifer system (layer 1) range from 0.1 to15 ft/d and transmissivity values for the Upper Floridan aquifer (layer 2) range from 10,000 to 500,000 ft^2/d. Leakance values for the intermediate confining unit range from 3.5 x 10^{-1} to 1.0 x 10^{-6} (ft/d)/ft with the higher value applied in areas where the confining unit is absent. A detailed description of model input parameters and calibration results are presented in SDI (1997) and Yobbi (2000).

Optimization Model

The optimization model constructed for this study combines the CNTB ground-water flow model (Yobbi, 2000) with an optimization algorithm (Halford, 1992). The method minimizes the sum-of-squares residuals (SS) between simulated and desired heads and is based on a modified Gauss-Newton method (Gill and others, 1981). This method was selected over other optimization methods (such as linear programming) because it does not require a feasible solution to maximize the objective function. This approach is particularly suitable for solution of the management problems formulated in this study because the optima can lie in the infeasible region. The SS is defined as:

$$SS = \sum_{j=1}^{n} [w_i(h_{js} - h_{jm})]^2 \quad ,(1)$$

where

h_{js} is the j^{th} simulated head, in feet;

h_{jm} is the j^{th} target recovery head, in feet;

w_i is the weighting factor; and

n is the number of head comparisons.

The optimization procedure uses several gradient-search minimization algorithms, which assume that the objective function defines a smooth continuous surface. The parameter change vector is solved using a quasi-Newton algorithm (Gill and others, 1981). If the second-order information fails or there is no prior parameter change information, then a variant of the Levenberg-Marquardt algorithm (Marquardt, 1963) is used. The optimization algorithms start with an initial pumping rate and converge to a local optimum. The magnitude and direction of change is based on minimization of the objective function (eq. 1), which is the smallest set of differences between the simulated and desired heads. The first step in the optimization process is to perform one execution of the model to establish the initial differences (residuals) between simulated and desired heads. The residuals are squared and summed to produce the SS objective function, which is used by the optimization program to measure model fit to the desired heads. In the next step, the sensitivity coefficients (derivatives of simulated water-level change with respect to parameter change) are calculated by the influence coefficient method (Yeh, 1986), using the initial model results. After the residuals and the sensitivities are calculated, a single pumping-estimation iteration is performed. The model is updated to reflect the latest pumping estimates and a new set of residuals are calculated. The entire process of changing pumpage in the model, calculating new residuals, and computing a new value for pumpage is continued iteratively until the SS is reduced to a specified level, or until a specified number of iterations is made (Halford, 1992). The major steps performed during each optimization iteration are summarized in figure 6.

An advantage of this optimization technique is the applicability for solving ground-water model calibration and ground-water management problems. Implementation of the method is straightforward and the computer code used for this application can be employed with data and operational requirements used for parameter estimation. Another advantage of this method is that model execution, input preparation, and output can be integrated into a spreadsheet and users can quickly set up and graphically compare simulation results. Most importantly, this method provides an alternate tool that the knowledgeable hydrologist can employ to better evaluate the ground-water resources.

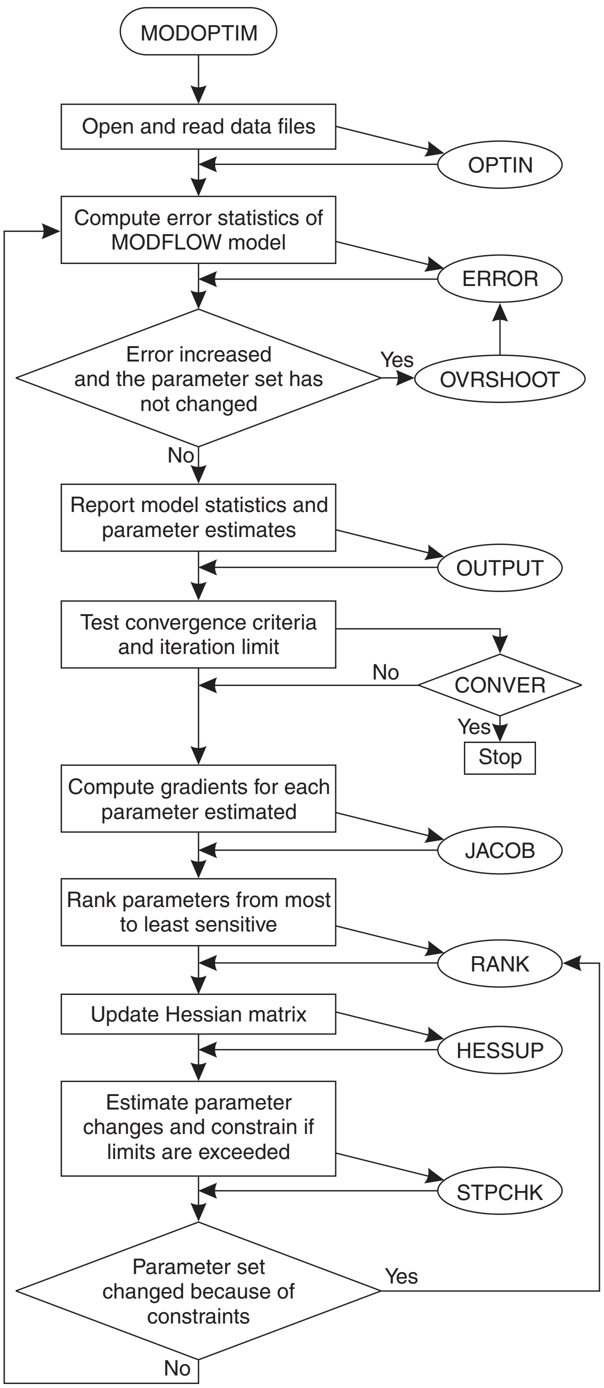

Figure 6. Flow chart of MODOPTIM main program.

The principal disadvantage of this optimization technique is that constraints are simply "targets" for the optimization model to shoot for and not "hard limits" as defined by other optimization methods, such as a linear programming. Thus, constraints can be violated. This is likely to happen when the objective function is poorly defined or when solution of the management problem is infeasible, such as with the present management application. This shortcoming can be addressed by removing those cells with large violations from the analysis or by increasing the weight given to that component of the least-squares objective to decrease the magnitude of the violation.

The management objective for this study is to maximize pumpage (Z) from the 13 production wells in the Cypress Creek well field such that the SS between simulated and desired recovery heads at selected control sites within and adjacent to the well field are minimized. Equal weights were applied to each site, which eliminates the subjectivity of finding the proper value of weight to preferentially minimize head deviations at one control site versus head deviations at another control site. Weights are generally used to adjust the relative sensitivity of the optimization routine; however, the optimization problem becomes difficult to solve if a proper value of weight is not selected.

The decision variable (variables to be determined as part of the optimization solution) provides the withdrawal quantities and are either the lumped or individual pumpage rates that represent the total pumpage from wells within the well field.

The optimization model is subject to an inbuilt upper bound on hydraulic heads. This bound represents ground surface. The mathematical representation of the management objective is:

$$\text{maximize } Z = \sum_{i}^{m}\sum_{j}^{n} Qij \,, \quad (2)$$

where

Qij is the ground-water pumpage at site i during time period j,

m is the total number of pumping sites; and

n is the total number of time periods.

The optimization model of the Cypress Creek well field and adjacent areas in central Pasco County was assigned calibrated parameter values (SDI, 1997) and given initial and boundary conditions (Yobbi, 2000). Initial conditions were the average annual hydrologic conditions for the 1987 calendar year. The 1987 hydrologic conditions were considered suitable for several reasons.

1. The frequency of data collection was sufficient for calculation of representative annual averaged conditions.
2. Measured annual precipitation in 1987 was similar to the long-term average annual value.
3. The small net change in water levels measured in wells indicates that the change in storage in the aquifer systems was small during 1987, reflecting equilibrium conditions of the aquifer.

APPLICATION OF THE OPTIMIZATION MODEL

The optimization model was used in a series of simulations to test the sensitivity of optimal pumpage to the number of head control locations (4 to 462 sites) and desired recovery heads (1 to 5 ft) in the Cypress Creek well field. Five scenarios (or cases) were formulated and pumping solutions for steady-state and transient conditions were simulated. The model was used to simulate steady-state ground-water flow for constant pumping solutions sustainable for an infinite length of time. Transient ground-water flow was simulated for short-term pumping solutions for a 6-month management timeframe. The following model applications were formulated.

- **Case A—**Four head control cells corresponding to TMR-1, TMR-2, TMR-4, and TB-22 surficial aquifer monitor well locations are used in this model application (fig. 7). These monitor wells are a subset of control points that are currently used by the OROP for the Cypress Creek well field. The monitor wells are located north, south, and west of the production wells.
- **Case B—**Thirteen head control cells corresponding to production well locations are used in this model application (fig. 7). These cells serve as a head control and pumping site.
- **Case C—**Eighty-seven head control cells corresponding to the area containing the 2-ft steady-state cone of depression pumping center are used in this model application (fig. 7). Case C considers the area most sensitive to well-field pumpage.
- **Case D—**Four hundred and sixty-two head control cells corresponding to areas within and adjacent to the well field are used in this simulation (fig. 7). Case D illustrates the sensitivity of the pumping solution for the entire well field.
- **Case E—**Six separate simulations are used in this model application. Each simulation is formulated to include only head control cells contained within its respective confining unit leakance zone. Therefore, the number of head control cells differs for each simulation because the number of model cells in each leakance zone is different (fig. 7). Case E illustrates the sensitivity of the pumping solution to changes in the desired target recovery head for separate leakance zone areas of the well field.

Steady-State Formulation

The management objective for these simulations was to maximize pumpage from the 13 production wells while minimizing deviations from the desired target recovery heads under equilibrium conditions. Results of the optimization simulations are given in table 1 and presented in the form of curves relating steady-state head recovery to optimal pumpage in figure 8.

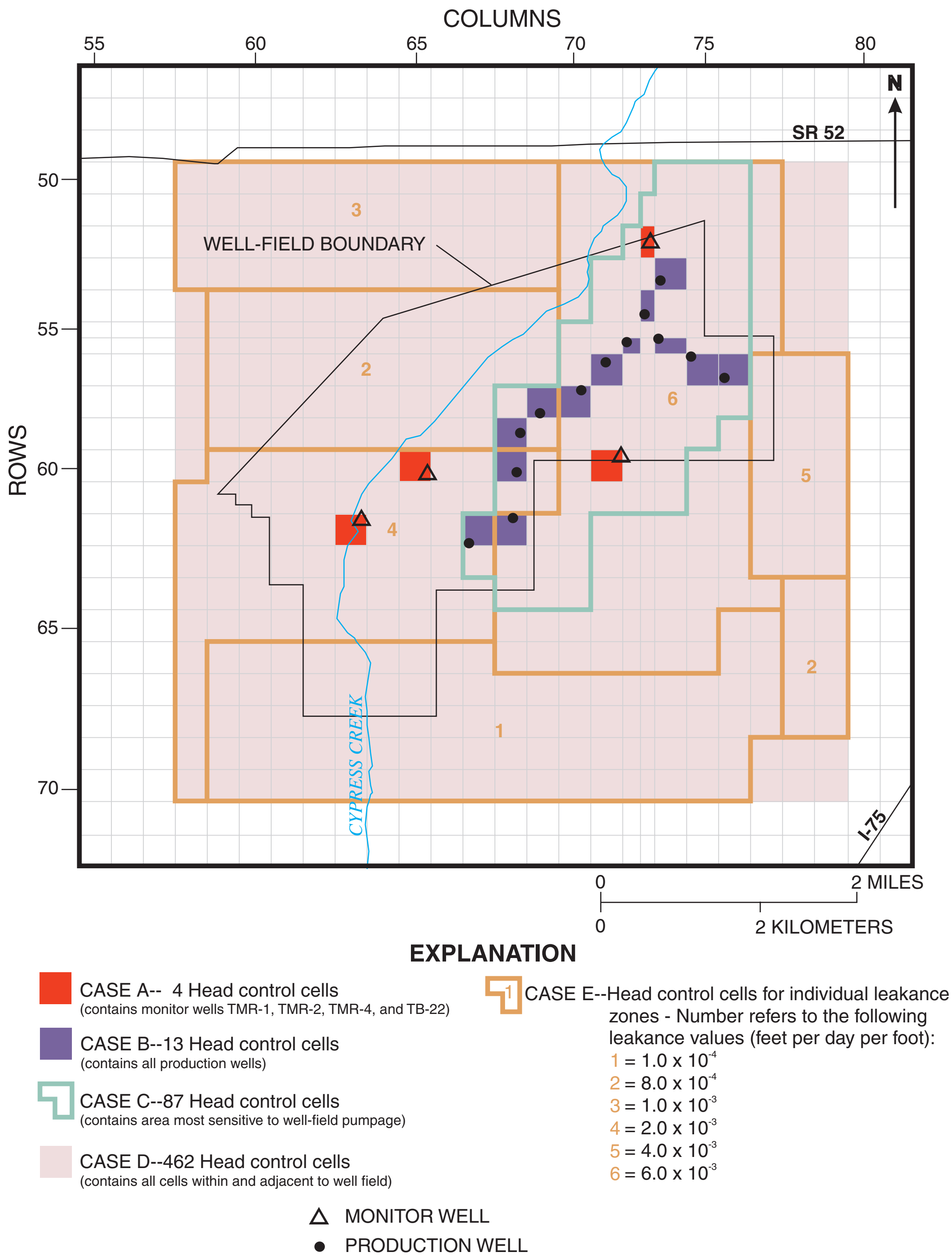

Figure 7. Head control cells used in the optimization model.

Table 1. Optimal pumpage and average simulated head recovery in the surficial aquifer system considering a 1 to 5 foot target head recovery for 1987 steady-state conditions

[Q, optimal pumpage; Mgal/d, million gallons per day]

Target head recovery, in feet	Case A (4 head control cells)		Case B (13 head control cells)		Case C (87 head control cells)		Case D (462 head control cells)	
	Q, in Mgal/d	Average simulated head recovery, in feet	Q, in Mgal/d	Average simulated head recovery, in feet	Q, in Mgal/d	Average simulated head recovery, in feet	Q, in Mgal/d	Average simulated head recovery, in feet
1	27.1	0.6	28.2	1.0	27.8	0.9	27.4	0.3
2	23.8	1.3	26.6	1.8	25.7	1.8	24.8	0.6
3	20.5	2.0	24.9	2.6	23.2	2.8	21.8	0.9
4	15.9	2.7	22.7	3.6	20.6	3.6	19.0	1.1
5	9.8	3.5	20.7	4.5	18.2	4.3	15.9	1.3

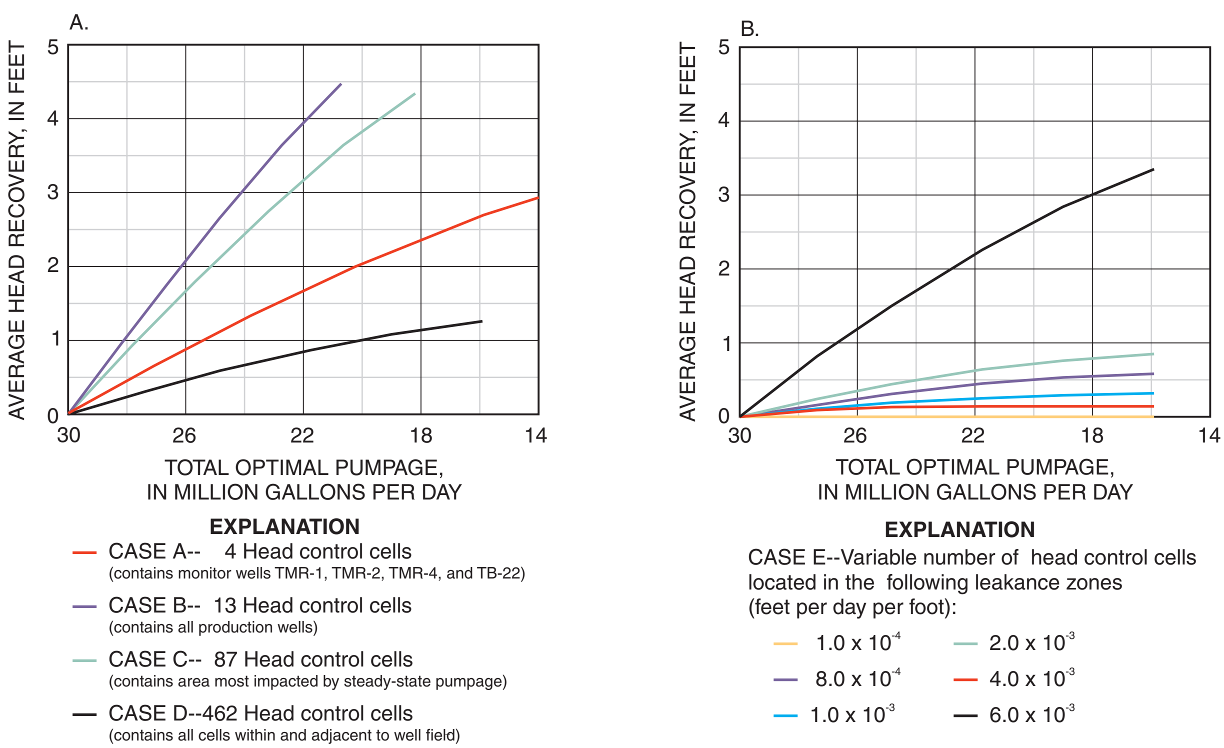

Figure 8. Comparison between total optimal pumpage and average head recovery in the surficial aquifer system for selected management cases, simulating 1987 steady-state conditions.

Four curves, one for management Cases A, B, C, and D, are shown in figure 8A and six curves corresponding to individual confining-unit leakance zones (Case E) are shown in figure 8B. In these figures, the pumpage values represent the average optimal pumping solution for the 13 production wells, and the head recovery represents the average optimal head recovery that was simulated at the specified locations. Several aspects of the curves are apparent.

1. Pumping solutions are sensitive to the location of head control sites formulated in the optimization problem. More water is withdrawn for test cases where hydraulic heads were maximized in cells near production wells and within the pumping center cone of depression (Cases B and C).
2. Total pumpage decreases as head recovery increases.
3. Changes in head in the surficial aquifer system do not respond linearly to changes in pumpage.
4. Case B (control sites at production well cells) provides the highest rate of pumpage and the highest recovery of ground-water heads. The rate of head recovery per unit decrease in optimal pumpage for Case B (control sites at production well cells) is about twice as much as Case A (TBW control sites), about 25 percent more than Case C (control sites within pumping center cells), and about 5 times more for Case D (control sites in all cells of well field).
5. The rate of head recovery per unit decrease in pumpage is greater for areas of the well field where permeability of the confining unit is higher than other areas of the well field where permeability of the confining unit is lower.

The highest amount of withdrawn water and the highest head recovery are simulated for the two cases (Cases B and C) with head control sites located in cells near the production wells and within the pumping center cone of depression. This suggests that a pumping advantage could be gained by maximizing hydraulic heads in cells near the production wells. The head recovery per unit decrease in optimal pumpage is more than twice as much for Cases B and C than for Cases A and D. A ratio of about 2 Mgal/d reduction per foot of average recovery is simulated for Cases B and C, while a ratio of about 5 Mgal/d and 10 Mgal/d reduction per foot of average head recovery was simulated for Cases A and D, respectively. In other words, for 1 ft of average head recovery for Cases B and C, a decrease in optimal pumpage of about 2 Mgal/d (from 30 Mgal/d to about 28 Mgal/d) is simulated. In contrast, for 1 ft of average head recovery for Case C, a decrease in optimal pumpage of about 5 Mgal/d (from 30 to about 25 Mgal/d) is simulated, and a decrease of about 10 Mgal/d (from 30 to about 20 Mgal/d) is simulated for Case D.

A pumping advantage also could be gained by maximizing hydraulic heads in cells where permeability of the confining unit is higher. For example, the rate of head recovery per unit decrease in pumpage is more than triple for the highest leakance zone (zone 6, fig. 7) than for other leakance zones (zones 1-5, fig. 7) of the well field. For 1 ft of average head recovery at head control sites in leakance zone 6, optimal steady-state pumpage decreased from 30 Mgal/d to about 25 Mgal/d. For 1 ft of average head recovery at head control sites in zones 1-6, optimal steady-state pumpage decreased from 30 to less than 15 Mgal/d.

The areal distribution of head recovery for the 5-ft target level using the 462 head control cells (Case D) is illustrated in figure 9. From a management perspective, it is unrealistic to control pumping using this large number of head control sites; however, simulation of this optimization problem is helpful in identifying areas of the Cypress Creek well field where maximizing hydraulic heads would most likely or least likely affect pumping solutions. Areas not meeting the 5-ft recovery target are of particular interest in figure 9. The western part of the well field and most areas outside of the well field are significantly below the 5-ft target head recovery. Several reasons for this simulation result are: (1) the selected head control sites are near the periphery of the steady state pumping cone of depression, (2) the initial steady-state heads are close to ground surface preventing the simulated head from changing substantially for different desired recovery heads, and (3) most of this area is represented in the ground-water flow model as either a river or drain boundary cell, which influences the upper part of the simulated flow system. A pumping advantage probably would not be gained by maximizing hydraulic heads within these areas of the well field. In contrast, a pumping advantage could be gained by maximizing hydraulic heads in areas of the well field meeting the 4- to 5-ft recovery target (fig. 9). Cells with head recoveries of 4 ft or more primarily correspond to the northern

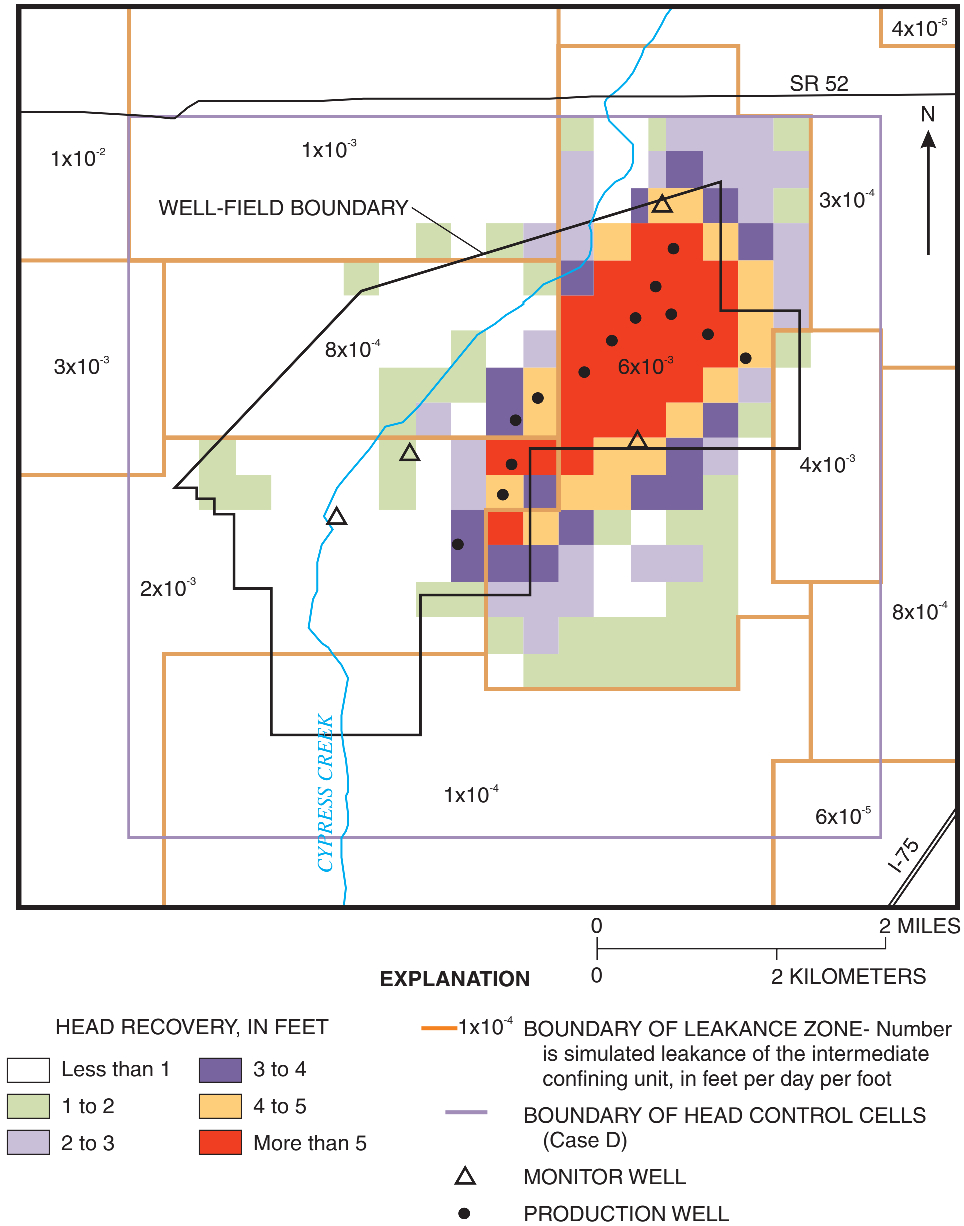

Figure 9. Spatial distribution of head recovery in the surficial aquifer system for Case D (462 head control cells) and a 5-foot target recovery, simulating 1987 steady-state conditions.

production well area where the pumping influence is greatest, confining-unit leakance is highest, and surficial aquifer system heads are substantially below land surface.

The distribution of optimal pumpage for individual production wells was significantly affected by the location of head control sites formulated in the optimization problem. The sensitivity of the pumping solution to individual production wells can be determined from graphical analysis such as presented in figure 10, which shows the distribution of optimal pumping among the 13 production wells for simulation of the 2-ft target recovery head under steady-state conditions. Similar pumping patterns were observed for the 1-, 3-, 4-, and 5-ft target recovery head simulations. As shown in figure 10, the distribution of optimal pumpage for Case A is quite different from the others. For Case A, most of the pumpage is focused in the northern portion of the well field close to head control sites TMR-1 and TMR-2, using production wells 3-5 and 11-13. In contrast, optimal pumpage for Cases B and C is more broadly distributed with wells 1, 12, and 13 at the periphery of the well field pumping the least amount of water.

The optimal steady-state heads for control sites used in formulations of Case A (TBW control sites) and Case B (control sites at production well sites) compared to optimal pumpage are presented in figures 11 and 12, respectively. The graphs were generated using simulation results for target recovery heads of 1 to 5 ft above the initial heads. The graphs can serve as approximate estimates of the required optimal pumpage to meet optimal steady-state heads in the cell containing the head control sites. Heads increase with a decrease in pumpage for all locations. The small recovery in optimal head simulated for sites TMR-4 and TB-22 is of particular interest. The reason for the small change in head is due primarily to initial steady-state heads that are near land surface, thus preventing the simulated head from changing substantially for different desired recovery heads. The sites also are located in cells that are near the periphery of the pumping influence, which also limits the head recovery. Flattening of the line for production well 7 cell (fig. 12) indicates that heads are constrained by land surface.

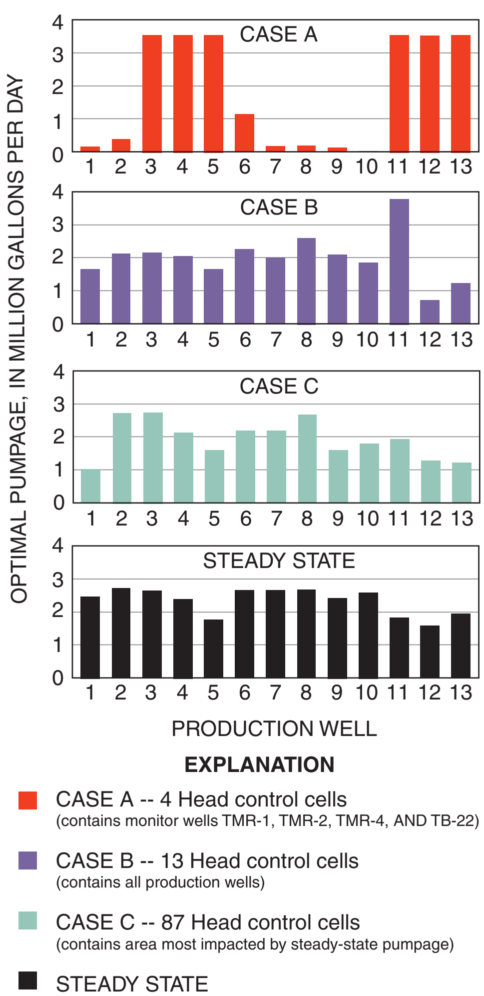

Figure 10. Distribution of optimal pumpage among the 13 production wells for selected management cases and a 2-foot target head recovery, simulating 1987 steady-state conditions.

Transient Formulation

The test period was chosen specifically to investigate the effects on the aquifer system that could be expected within a short period of time (180 days) with no recharge. Simulation of this management problem is useful in demonstrating the temporal performance of the optimization model and providing a comparison with the steady-state solution. Three of the five management formulations of the optimization problem (Cases A, B, and C) were evaluated. The objective function maximizes ground-water mining (withdrawal of more water than is recharged for a period of time), subject to desired target recovery heads. The optimization model was designed to maximize head recovery within a 6-month management timeframe and was divided into six time periods, each of 30 days (30, 60, 90, 120, 150, and 180 days total elapsed time). The analysis was done in two steps. First, the 1987 hydraulic head distribution was used as the initial conditions and run for 180 days with no recharge. The simulated hydraulic-head distribution at each of the six time periods was used as the basis for assigning the desired recovery target heads at head control sites. Second, 30-day pumping rates were determined by specifying recovery heads of 1 to 5 ft at selected cells with a linear rate of recovery at respective time periods. These target recovery heads were specified based on the simulated response of the aquifer system using the 180-day transient simulation (step one).

Pumping solutions at respective time periods considering a linear rate of recovery for Case A (TBW control sites), Case B (control sites at production well cells), and Case C (control sites within pumping center

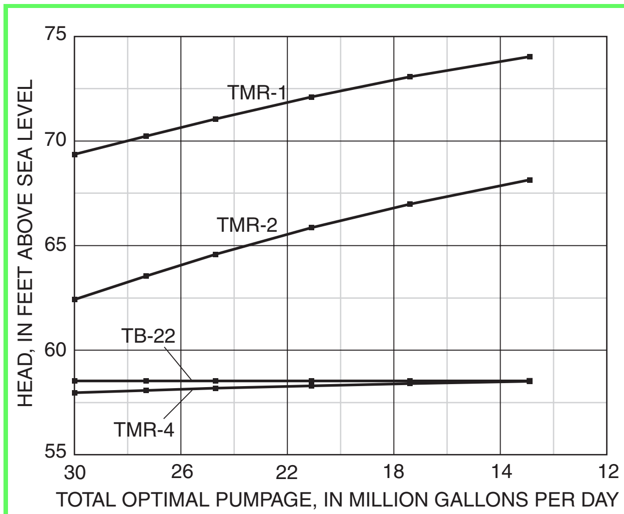

Figure 11. Comparison between total optimal pumpage and head in the surficial aquifer system at TMR-1, TMR-2, TMR-4, and TB-22 monitor well cells (Case A), simulating 1987 steady-state conditions.

cells) are given in table 2 and graphically shown in figures 13 and 14, which relate average optimal head recovery at specified time periods to total optimal pumpage. The transient rate of pumpage for the planning period decreases when the target head recovery is raised (fig. 13). As with the steady-state results, formulating head control sites at or near production well cells (Cases B and C) in the optimization problem provide the highest rate of withdrawn water and the highest average optimal head recovery. Optimal pumpage averages about 23.7 Mgal/d for Case B, about 22.5 Mgal/d for Case C, and about 19.6 Mgal/d for Case A. The change in head per unit decrease in pumpage is more than triple for Cases B and C than for Case A. A ratio of about 3 Mgal/d reduction of optimal pumpage per foot of recovery is calculated for Cases B and C, while a ratio of 9 Mgal/d reduction of optimal pumpage per foot of recovery is simulated for Case A. Results also indicate that the decrease in optimal pumpage is rather significant for higher target recovery heads than at lower target recovery heads. For example, the pumping solution at 180 days for Case A for the 1-ft recovery is 25.1 Mgal/d whereas the pumping solution at 180 days for the 5-ft target recovery head is 4.6 Mgal/d.

The temporal distributions of head at TMR-1 and TMR-2 control sites for simulation of the 3-ft target recovery head are presented in figure 15. The target recovery heads and the "unmanaged" simulated heads at the two sites also are included in this figure for comparison with "optimized" heads. The head deviation between the optimized heads and the desired target recovery heads is of particular interest. For well TMR-1, the optimized head at the end of 180 days is about 0.5 ft higher than the target recovery head and about 0.2 ft lower than the target recovery head for the cell containing observation well TMR-2. For most head control cells, the recovery head corresponding to optimal pumpage at the respective time period deviated somewhat from the target recovery head at the respective time period. This is a shortcoming of the optimization technique used in the study because head violations are allowed. Overall, pumping solutions were constrained by the limiting recovery values, initial head conditions, and by upper boundary conditions of the ground-water flow model.

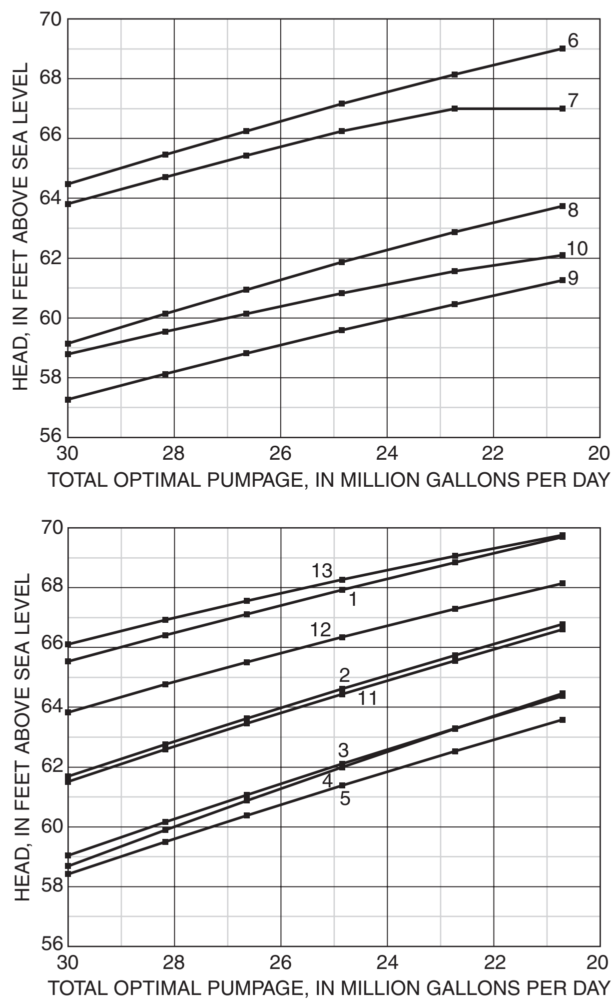

Figure 12. Comparison between total optimal pumpage and head in the surficial aquifer system at the 13 production well cells (Case B), simulating 1987 steady-state conditions.

Table 2. Temporal change in optimal pumpage considering a 1 to 5 foot linear rate of target head recovery in the surficial aquifer system for selected management cases, transient-state conditions (180 days with no recharge)

[Mgal/d, million gallons per day; ft, feet]

Time, in days	Target head recovery				
	1 ft	2 ft	3 ft	4 ft	5 ft
	Case A (4 head control cells) Pumpage in Mgal/d				
30	27.7	25.6	23.4	20.9	19.0
60	27.1	24.6	22.0	18.8	15.3
90	26.7	23.9	20.8	16.9	13.7
120	26.3	23.1	19.6	14.6	11.3
150	25.8	22.3	18.4	12.9	8.6
180	25.1	21.2	16.8	10.0	4.6
	Case B (13 head control cells) Pumpage in Mgal/d				
30	28.5	27.0	25.6	24.1	22.6
60	28.4	26.7	25.0	23.4	21.8
90	28.1	26.1	24.1	22.2	20.2
120	27.8	25.6	23.4	21.1	18.9
150	27.6	25.1	22.6	20.1	17.7
180	27.4	24.7	22.0	19.4	16.7
	Case C (87 head control cells) Pumpage in Mgal/d				
30	28.1	26.2	24.3	22.4	20.5
60	27.9	25.8	23.8	21.7	19.6
90	27.6	25.3	22.9	20.5	18.0
120	27.4	24.7	22.0	19.3	16.8
150	27.2	24.2	21.3	18.4	15.5
180	27.0	23.9	20.8	17.7	14.6

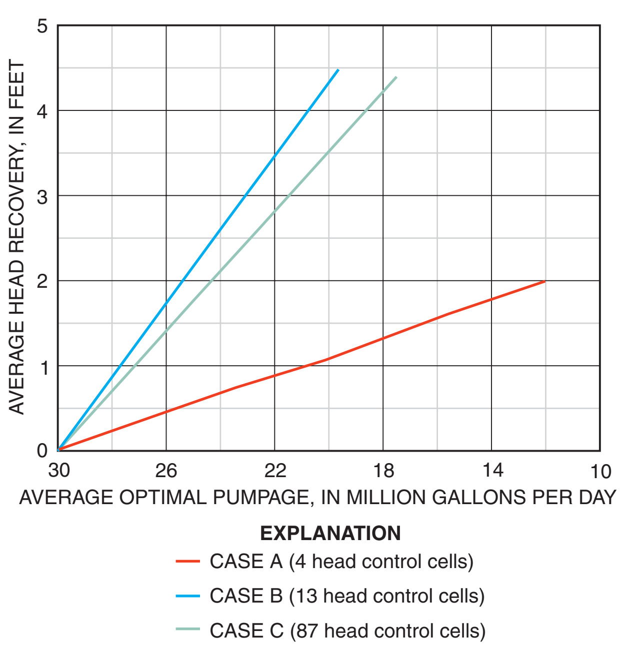

Figure 13. Comparison between average optimal pumpage and average head recovery in the surficial aquifer system for selected management cases, simulating transient-state conditions (180 days with no recharge).

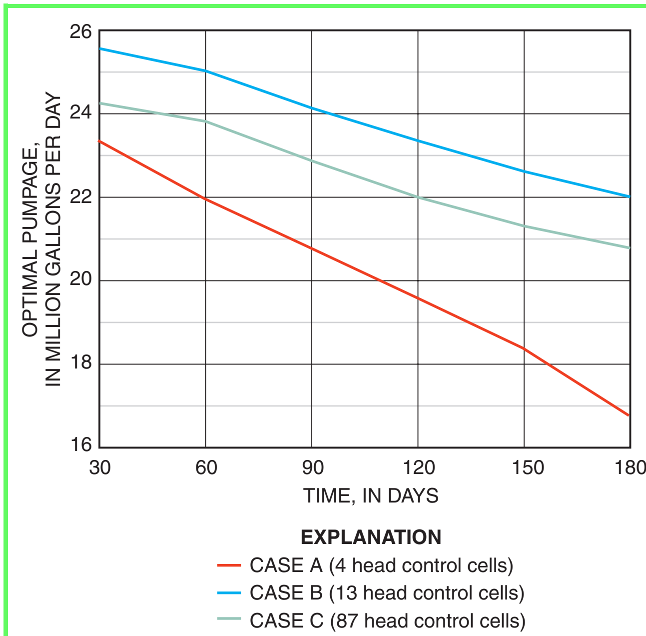

Figure 14. Temporal distribution of optimal pumpage considering a 3-foot linear rate of head recovery in the surficial aquifer system for selected management cases, simulating transient-state conditions (180 days with no recharge).

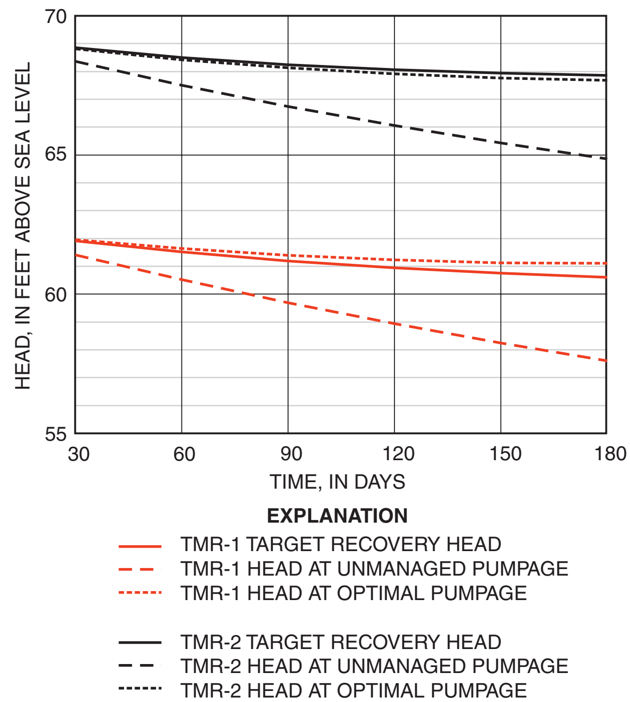

Figure 15. Temporal distribution of target recovery head, head at optimal pumpage, and head at unmanaged pumpage considering a 3-foot head recovery at TMR-1 and TMR-2 surficial aquifer system well cells, simulating transient-state conditions (180 days with no recharge).

Limitations

This optimization model is designed not as a predictive tool, but as an interpretative one. The model applications are intended to provide insight about maximizing hydraulic heads and pumpage in the Cypress Creek well field given the proposed management formulations. However, the optimization method used in this study is not without limitations. (1) Practical uncertainties in the simulation (arising from model parameter uncertainties) and optimization (arising from the algorithms) components of the technique make it impossible to guarantee that the unique optimum solution will be found (if one exists). (2) The technique does not insure feasibility. As indicated earlier, recovery heads used in this model application are simply “targets” for the optimization model to shoot for and not “hard limits” as are defined in a linear programming model. As such, target recovery heads in this model can be violated. (3) Results of this optimization model are not unique, and different pumping solutions could be obtained with other optimization methods and different formulations of the optimization problem. Regardless of these limitations, the results presented here indicate that the technique is a useful alternative method for evaluating a variety of management alternatives for maximizing ground-water levels and pumpage in the Cypress Creek well field.

SUMMARY

The Tampa Bay area relies heavily on ground water for water supply. Numerous wetlands and lakes have been impacted in Pasco County, in particular, within the Cypress Creek well field in central Pasco County. An optimization model developed by Tampa Bay Water is being used to assist in the management of ground-water resources in the area. The optimization model relies on surficial aquifer wells to monitor hydrologic stress and to provide managed heads for the optimization model. Plans have been made to develop alternate sources for water supply so that pumpage from the well fields can be cut to 90 Mgal/d by the year 2007. Identification of optimal pumping solutions will provide better management of ground-water resources.

The ground-water flow system in the Cypress Creek well-field area consists of an unconsolidated surficial aquifer system, an intermediate confining unit, and a carbonate rock comprising the Upper Floridan aquifer. The thickness and leakance characteristics of the clay beds in the lower part of the surficial aquifer system are the principle controls on the vertical movement of water.

An optimization technique was used to test the sensitivity of optimal pumpage to increases in surficial aquifer system heads at selected locations in the Cypress Creek well field. The ground-water system was simulated using the USGS model MODFLOW and linked to an optimization routine. The ground-water flow model is based on data and information presented by SDI Environmental Services, Inc., and the USGS. Five management cases, which involved different head control sites, were evaluated using the optimization technique. The management objective was to maximize total pumpage in the Cypress Creek well field such that the sum-of-squares differences between simulated and target recovery heads at selected sites are minimized.

Pumping solutions for selected management cases were determined for 1987 steady-state conditions and for a 6-month management timeframe. Results of the optimization simulations are presented in the form of curves relating average head recovery to total optimal pumpage. Pumping solutions are sensitive to the location of head control sites formulated in the optimization problem and as expected, total optimal pumpage decreased as optimal recovery head increased. Results suggest that a pumping advantage could be gained when hydraulic heads are maximized in cells near the production wells, in cells within the steady-state pumping center cone of depression, and in cells within the area of the well field where confining unit leakance is the highest. More water was pumped and the ratio of average head recovery per unit decease in withdrawn water was more than twice as much for test cases where hydraulic heads are maximized in cells located at or near the production wells. Additionally, the ratio of average head recovery per unit decrease in withdrawn water is about three times greater for the area where the confining unit leakance is the highest than for other areas of the well field. For most cells, recovery heads corresponding to pumping solutions are less than the maximum specified. This is a shortcoming of the optimization algorithms used in this study because head violations are allowed. Recovery heads are simply "targets" for the optimization model to shoot for and not "hard limits" as are defined in constrained linear programming techniques.

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