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WATER RESOURCE ALLOCATION AND QUALITY OPTIMIZATION MODELING

FINAL REPORT

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Prepared for

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EXECUTIVE SUMMARY

PROJECT SCOPE

Due to the increasing demands placed on Florida's water resources, the state of Florida adopted legislation in 1990 to improve water resource management and to direct future growth through planning programs. This legislation requires each Water Management District to completely evaluate their water needs and sources through the year 2010 and delineate critical areas identified as water resource problems. Once completed, Districts were expected to develop possible alternative water supply scenarios which avoid adverse or otherwise unacceptable changes in the environment or the availability of water.

This report presents systematic modeling methods for determining optimum water supply strategies that satisfy various environmental and hydrological requirements. Five site specific water resource allocation optimization models were developed for Volusia County, Florida and were executed to investigate a variety of management objectives. These optimization models incorporate both water quantity and quality aspects of water resource management to determine optimum ground water allocation strategies which satisfy future water service demands and minimize adverse environmental impacts at specified areas. These areas include sensitive wetlands where projected water table declines are predicted to induce a high level of vegetative harm and well fields where excessive withdrawal is predicted to cause a degradation in water quality due to salt-water intrusion or upconing.

The five optimization models were designed to elucidate water resource allocation strategies for water service areas that would:

1) Satisfy the water demands of both municipal and agricultural water demands.

2) Explore development of both existing and proposed ground water supply areas.

3) Select wastewater effluent as a feasible supply to supplement agricultural demands.

Water quality aspects were incorporated in two models by constraining chloride concentrations changes at wells while simultaneously minimizing the maximum drawdown at sensitive wetland areas in one model and by minimizing relative chloride concentration increases at wells in a second model.

The optimization models were a product of a research project that sought to accomplish two goals. The first was to obtain a broad understanding of numerical optimization modeling and its recent applications to ground water resource management. The second goal was to demonstrate optimum resource allocation modeling at a selected site that would satisfy specified environmental and hydrological requirements. In order to achieve the established two goals and fulfill the scope of the project, the following three objectives were accomplished:

1) Review of current literature of numerical optimization modeling with respect to ground water resource management.

2) Construct site specific optimization models capable of generating water resource allocation scenarios which satisfy projected demands and environmental constraints for the year 2010.

3) Generate and summarize various optimum water resource allocation scenarios to meet specific objectives.

LITERATURE REVIEW

To meet the first objective a literature review was conducted to investigate applications of ground water resource management where tools of optimization were applied to demonstrate optimum scenarios of resource allocation. Chapter 2 presents a review of various water management models; a number of existing ground water optimization models were summarized in tabular form. Based on knowledge acquired through the literature review and the size and complexity of the problem, it was determined that the most efficient combined simulation/optimization model would be one developed using of the unit "response matrix" technique. This method consists incorporating a matrix of coefficients which represent the response of ground water to a specified change in withdrawal rate.

PROJECT SIMULATION MODELS

Chapter 3 presents details on the three-dimensional ground water flow and solute transport models used to generate the steady-state ground water system responses (i.e., predicted head values, solute concentrations, etc.) to various stresses (i.e., specified pumping and recharge rates) in the study area; these system responses are generated as needed to create response matrices that are later used to construct the larger optimization models. The study area includes most of Volusia County where in order to alleviate the problem of salt-water intrusion and satisfy water service demands, the general trend since the 1950's has been to locate additional wells to the west toward central Volusia County. The flow simulation model as originally acquired from the SJRWMD simulates the aquifer systems of Volusia County using MODFLOW (McDonald and Harbaugh 1988). This regional simulation model was revised to simulate flow in the study area. The transport model for the same study area was also developed by the District using the program DSTRAM (Huyakorn and Panday 1991).

WATER SUPPLY NEEDS AND SOURCES

Chapter 4 presents details on water supply needs and sources as currently viewed by SJRWMD under the District Water Management Plan. Projections of future water use for the year 2010 are presented that were based on historical trends, local government comprehensive plans, and direct communication with both public and private public supply utilities (SJRWMD, 1992). Major components of the water uses categories include municipal and agricultural service area demands. Some of the larger projected municipal demand increases include an increase in the Port Orange service area in excess of 70 percent of the current use, an increase in Daytona Beach service area of 62 percent, and an increase of 400 percent over current uses in the Smyrna and Samsula areas. The overall demand in the Volusia County area is projected to increased by approximately 75 percent from 1988 to 2010. Also included in Chapter 4 are maps and tables that depict wastewater treatment plant information used to incorporate water reuse in the development of the ground water management models.

GROUND WATER OPTIMIZATION MODELS

Chapter 5 presents the five aquifer optimization models developed to investigate optimal allocation of ground water to meet year 2010 demand in Volusia County project area. These models were formulated to investigate future water allocation strategies assuming feasible withdrawal scenarios must meet or exceed projected water service areas demands and not exceed available water supplies. It was assumed with these models that adverse environmental effects could be minimized at specific locations by constraining pressure head changes (i.e., drawdown) to meet specified environmental goals or standards. These models are essentially numerical optimization models comprised of an objective function, decision variables, and constraints. Of the five optimization models, three models dealt strictly with hydraulic constraints and two models incorporated both hydraulic and water quality constraints. Objective functions included the following: 1) minimizing the maximum drawdown value, 2) maximizing the minimum pressure head, 3) minimizing average drawdown over all sensitive wetland areas, 4) minimizing the maximum drawdown while limiting chloride concentrations, and 5) minimizing relative chloride concentration increases.

Data supplied by SJRWMD and gathered during the project were used to formulate the allocation models (i.e., in the development of an objective function and management constraints). This included well data specifying location, capacity, withdrawal rates, corresponding service areas, and additional data identifying potential well locations. Other data included information concerning effluent rates, capacities, and locations of wastewater treatment plants that could supplement the demand of agricultural areas. Water supply demand data consisted of municipal and agricultural water volumes used in year 1988 and projected water quantity requirements for the year 2010.

The District also supplied environmental impact data specifying locations where drawdown was expected to induce adverse effects on wetland vegetation. These target locations were used to constrain the optimization model to identify resource allocation scenarios that would achieve minimum environmental impacts.

Aquifer system response constraints were developed from steady-state unit response matrices containing influence coefficients generated from ground water flow and water quality simulation models described in Chapter 3. These steady-state unit response matrices represent steady-state changes in pressure head and water chloride concentrations with respect to pumpage. A ground water flow simulation model was executed once for each existing and potential well location to determine the head response at specific locations. A similar approach was taken with the solute transport simulation model to determine chloride changes at wells due to pumpage. Water level and water quality unit response matrices were developed for both municipal and agricultural wells. These response matrices summarize the influence of each well upon itself, upon all other wells, and also upon specified control points throughout Volusia County.

The GAMS program (Brooke, Kendrick, and Meeraus 1989) was used to formulate each allocation model into a linear program optimization model. A model specific objective function and various constraints (i.e., water supply, demand, and environmental constraints) constitute the basic framework of each resource allocation model. All the models contain specific constraints that define allowable connections between water supplies and demand areas. Three of the models incorporate data from a unit response matrix created for various hydraulic management constraints, while two models incorporates two response matrices needed to construct both hydraulic and water quality management constraints. Once formulated the optimization models were executed using the linear program solution algorithm available in the GAMS software.

Year 2010 water resource allocation strategies were examined with the optimization models. Results of each model revealed where future well fields should be developed and to what extent future and existing wells should be utilized. Aquifer responses induced by the optimized allocation strategies were compared to those induced by the projected year 2010 allocation strategy. Predicted pumping strategies identified from each optimization model were then reviewed and tabulated. To verify optimal resource allocation strategies identified by the five models the ground water flow and transport simulation models were used to simulate the optimal pumping scenarios generated. Predicted pressure heads, drawdowns, and chloride concentrations from the simulation models were then compared to optimization model results.

SUMMARY AND CONCLUSIONS

Chapter 6 summarizes the overall research effort and presents several salient conclusions. This work endeavor demonstrated the use of optimization modeling as a valuable tool for the management of water resources. Optimization modeling elucidated the potential for an abatement of adverse environmental impacts associated with declining water table elevations and this was verified when the optimized allocation strategies were simulated with ground water flow and transport models. It was also shown that the minimum discharge constraint is an important determinant of the identified optimum allocations. This constraint requires a minimum flow of at least 50 percent of the 1988 ground water pumping rate from all active wells. Several of the optimal allocation scenarios incorporate new wells in areas of Port Orange West well field, Daytona Beach West - South Daytona water treatment plant well field, and Ormond Beach State Road 40 / Hudson well fields. In general, each model identified a scenario that decreased ground water pumpage at the Daytona Beach West water service area and expanded pumpage in areas east and west where the native vegetation is less sensitive to ground water withdrawals.

Sensitivity analyses were performed with the optimization models to determine where management strategies could be changed to induce improvements in ground water levels. These analyses identified areas were the balance between supplies and demands could be changed to decrease the environmental impacts of ground water withdrawals. For example, it was shown that a ten percent decrease in the demand at the DBS (Daytona Beach West - South TP) water service area would induce a water level improvement of 17 percent. These types of analyses give the water resource manager valuable information on where conservation efforts and new well developments should be pursued and let the manager quickly determine how the resource allocation scenario given by each optimization model responds to changes in projected demands and withdrawal limits.

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$\alpha_{i,j}$ is the aquifer influence coefficient defining pressure head change at each sensitive wetland area control point j due to a change in discharge rate at each municipal well grid cell i.

$\beta_{n,j}$ is the aquifer influence coefficient defining pressure head change at each sensitive wetland area control point j due to a change in discharge rate at each agricultural well n.

$BOTELEV_h$ is the bottom elevation of the surficial aquifer in feet from mean sea level at each well grid cell control point h.

CA_n is the capacity rate in cfd of each agricultural well grid cell i.

CC_h is the chloride concentration in ten-thousandths of a milligrams per liter (mg/l) at each well grid cell control point h.

$CC0_h$ is the initial chloride concentration in mg/l at well grid cell control points h.

CI_h is the increase in chloride concentration in ten-thousandths of a milligrams per liter (mg/l) at each well grid cell control point h.

CL_h is the maximum chloride concentration limit in mg/l of each well grid cell control point h.

CM_i is the capacity rate in cfd of each municipal well grid cell i.

CW_m is the capacity rate in cfd of each wastewater treatment plant i.

DA_o is the demand rate in cfd of each agricultural water service area o.

DD_j is the drawdown at sensitive wetland control point j.

$DDPRIV_j$ is the drawdown in feet at each sensitive wetland area control point j due to private wells not incorporated in the optimization process.

DDW_h is the drawdown in millionths of a foot at each well grid cell h.

DM_k is the demand rate in cfd of each municipal water service area k.

$\gamma_{i,h}$ is the aquifer influence coefficient defining pressure head change at well grid cell control point h due to a change in discharge rate at each municipal well grid cell i.

h defines all the well grid points incorporated in the optimization model.

HD_j is the pressure head in millionths of a foot at each sensitive wetland area control point j.

HDW_h is the pressure head in millionths of a foot at each well grid cell h.

HO_j is the initial pressure head in feet at each sensitive wetland area control point j.

$HW0_h$ is the initial pressure head in feet at each well grid cell h.

i defines the 120 municipal well grid cells in the optimization model.

j defines the 100 grid cell points in the optimization model where there is a high potential of harm to vegetation in sensitive wetland areas.

k defines the 14 Municipal water service areas in the optimization model.

m defines the five wastewater treatment plants in optimization model.

n defines the 28 agricultural well grid cells in the optimization model.

o defines the 9 different agricultural water service areas in the optimization model.

$\phi_{n,h}$ is the aquifer influence coefficient defining chloride concentration change at each well grid cell control point h due to a change in discharge rate at each agricultural well n.

$QA_{n,o}$ is the discharge rate in cfd of each agricultural well grid cell n which supplies each agricultural water service area o.

$QA0_n$ is the initial discharge rate in cfd of each agricultural well grid cell n.

QAT_n is the total discharge rate in cfd of each agricultural well grid cell n.

$QM_{i,k}$ is the discharge rate in cubic feet per day (cfd) of each municipal well grid cell i which supplies each municipal water service area k.

$QM0_i$ is the initial discharge rate in cfd of each municipal well grid cell i.

QMT_i is the total discharge rate in cfd at each municipal well grid cell i.

$QW_{m,o}$ is the effluent reuse rate in cfd of each wastewater treatment plant m which supplements each agricultural water service area o.

QWT_m is the total effluent reuse rate in cfd of each wastewater treatment plant m.

RCl_h is the relative chloride concentration increase in ten-thousandths of a miligram per liter (mg/l) at each well grid cell control point h.

$SERVE1_{i,k}$ designates which municipal well grid cells i supply which municipal water service areas k.

$SERVE2_{n,o}$ designates which agricultural well grid cells n supply which agricultural water service areas o.

$SERVE3_{m,o}$ designates which wastewater treatment plants m can supplement the demand of which agricultural water service areas o.

TDD_j is the drawdown in feet at each sensitive wetland area control point j.

$TDDW_h$ is the drawdown in feet at each well grid cell h.

THD_j is the total pressure head in feet at each sensitive wetland area control point j.

$THDW_h$ is the total pressure head in feet at each well grid cell h.

$\theta_{n,h}$ is the aquifer influence coefficient defining pressure head change at well grid cell control point h due to a change in discharge rate at each agricultural well grid cell n.

TCI_h is the total increased chloride concentration, respectively, in mg/l at each well grid cell control point h.

TCC_h is the total chloride concentration in mg/l at each well grid cell control point h.

$\zeta_{i,h}$ is the aquifer influence coefficient defining chloride concentration change at each well grid cell control point h due to a change in discharge rate at each municipal well grid cell i.

CHAPTER 1
1.0 INTRODUCTION

In recent years, population growth, expanding agricultural and industrial activities, and rapid urban development have significantly increased the demand for clean, fresh water in the state of Florida. Within the boundaries of the St. Johns River Water Management District (SJRWMD), demand for public water has increased 66% from 1975 to 1990 and is predicted to increase another 120% by the year 2010 (SJRWMD 1992). This increased demand presents an ominous threat to ground water sources in terms of both water quantity and quality. Along coastal areas of the District, increased development has generated water quality problems related to high chloride concentrations due to salt-water intrusion and localized upconing. Sensitive wetland areas have also been adversely affected by recent declines in the water table.

Due to the increasing demands placed on Florida's water resources, the state of Florida adopted legislation in 1990 to improve water resource management and to direct future growth through planning programs. This legislation requires each Water Management District to completely evaluate their water needs and sources through the year 2010 and delineate critical areas identified as water resource problems. Districts are then expected to develop possible alternative water supply scenarios which avoid adverse or otherwise unacceptable changes in the environment or the availability of water.

A study of regional water supply needs and sources, could identify many possible resource allocation plans that meet the needs of the District. The distribution of available supplies to service areas is an allocation problem and can have several feasible solutions. It is also likely that the number of plans will be too extensive to permit detailed examination of each and every scenario. In order to identify a concise subset of water allocation plans which best meet environmental and developmental goals of the District, optimization modeling can be incorporated into the decision or plan elucidation process.

This project involves the development of a systematic method of determining optimum water supply strategies that satisfy various environmental and hydrological requirements. The purpose of this type of water supply strategy is to optimize the pattern of water supply development and usage to meet projected needs. This resource management problem requires the use of optimization modeling to identify desirable scenarios of resource allocation; otherwise, resources may not be used in the most effective and efficient manner. When environmental impacts are also incorporated, the allocation problem expands to include identifying feasible scenarios that must also satisfy environmental constraints (i.e., ground water quality standards, minimum water levels). To balance projected needs against available sources, it is possible that the management problem may become one of balancing projected development against adverse environmental impacts.

The objective of this project is to accomplish two goals. The first is to obtain a broad understanding of numerical optimization modeling and its recent applications to ground water resource management. The second goal is to demonstrate optimum resource allocation modeling at a selected site that would satisfy specified environmental and hydrological requirements. The demonstration will give the SJRWMD essential knowledge and experience to incorporate optimization technology into the District's decision-making framework. In order to achieve the established two goals and fulfill the scope of the project, the following three objectives were accomplished:

1) Review current literature of numerical optimization modeling with respect to ground water resource management.

2) Construct site specific optimization models capable of generating water resource allocation scenarios which satisfy projected demands and environmental constraints for the year 2010.

3) Generate and summarize various optimum water resource allocation scenarios which satisfy specific objectives.

To meet the first objective the literature review was limited to applications of ground water resource management and only where tools of optimization were applied to demonstrate optimum scenarios of resource allocation. Formulations of various water management models were reviewed and a number of existing ground water optimization models were summarized in tabular form. Based on knowledge acquired through the literature review and the size and complexity of the problem, it was determined that the most efficient simulation/optimization model would be one developed using the unit "response matrix" technique. This method consists of incorporating a matrix of coefficients which represent the response of ground water to a specified change in withdrawal rate. To achieve the second and third objectives it was necessary to select a study site within the district boundaries.

1.1 STUDY AREA BACKGROUND

Based on the extensive ground water flow and contaminant transport modeling effort completed by the SJRWMD, four distinct regions within the district were identified as candidates for project study. These areas included the Volusia County region, the Wekiva River Basin region, the East-Central Florida region, and the Geneva Area/Seminole County region. The regional models and areas were evaluated with respect to model accuracy and complexity, computer requirements and efficiency, data availability, and applicability. Based on this review, A study area within Volusia County Florida was selected as the site to apply the numerical optimization modeling techniques of this project, shown in Figure 1.1.

The study area includes most of Volusia County, thus a general geographical and hydrological characterization of this county is in order. Volusia County alone covers an area of approximately 1,200 square miles in the east-central region of Florida. As shown in Figure 1.1, the county is bounded by the Atlantic Ocean to the east, the St. Johns River to the West, Flagler and Putnam Counties to the north, and Brevard County to the south. Although the county consists of approximately 120 lakes larger than 5 acres in size, ground water supplies are currently the sole means of meeting public water demands (Knochenmus and Beard 1971, Kimrey 1990).

Most of the population of Volusia County occupies the region in and near the coastal cities of Daytona Beach, New Smyrna Beach, and Ormond Beach. Extensive development of these coastal areas has resulted in increased demands for ground water and an encroachment of the fresh-water/salt-water interface. In order to alleviate the problem of salt-water intrusion and satisfy water service demands, the general trend since

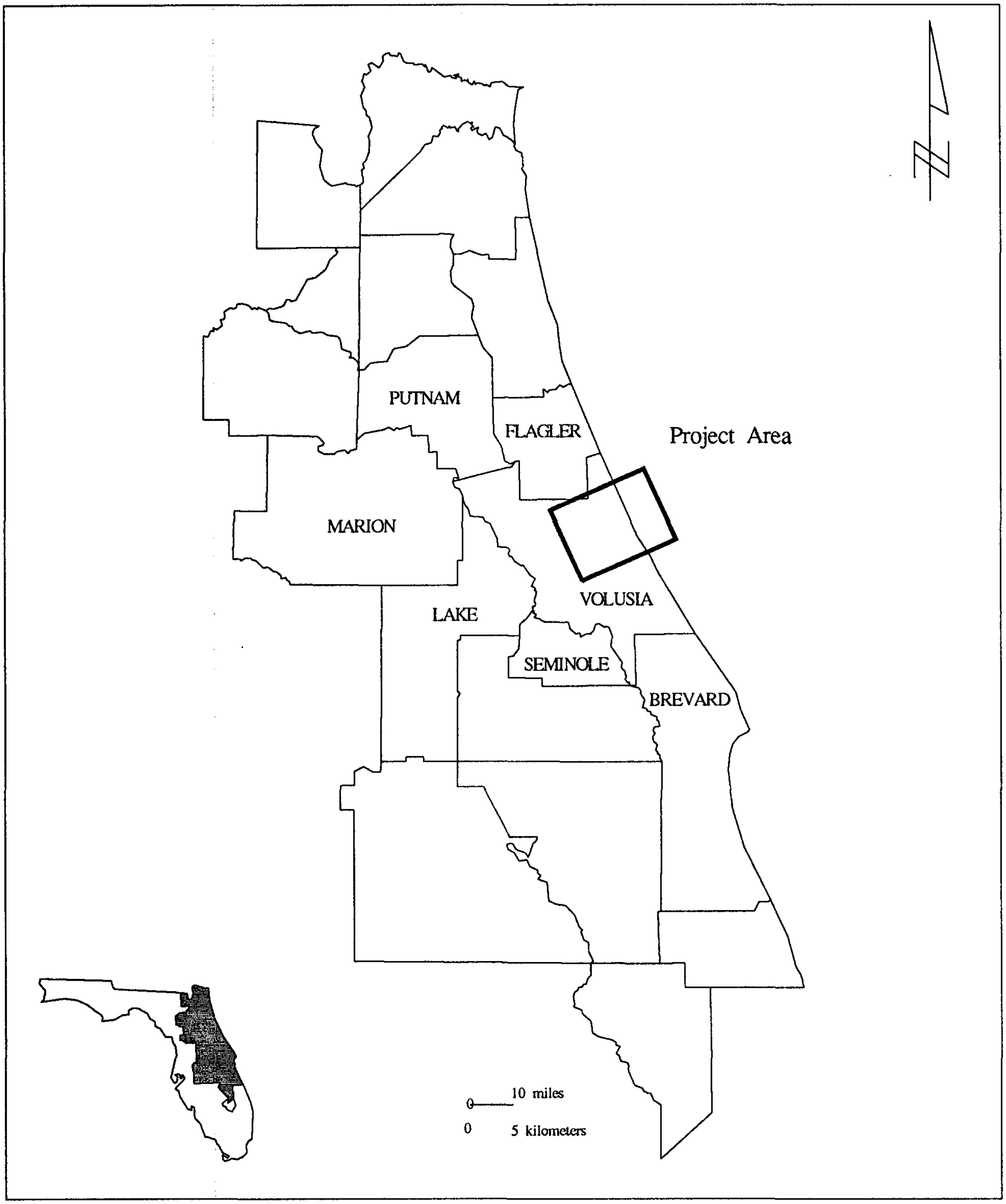


Figure 1.1

Location of Project in Volusia County, Florida within the St. Johns River Water Management District the 1950's has been to locate additional wells to the west towards central Volusia County.

The hydrogeologic system of Volusia County is made up of two aquifer systems, the surficial and the Floridan aquifers, separated by an intermediate confining layer (See Figure 1.2). The surficial aquifer is the uppermost formation, consists of silts, clays, cemented shell, and quartz sands, and is considered to be unconfined. The water table is usually at or near the surface in lowland and flatland regions and is generally a suppressed image of the ground level in highland regions. Precipitation, lakes, wetlands, and irrigation are the main sources of recharge into the surficial aquifer, which ranges in thickness between 50 and 100 feet. Via the intermediate confining layer, leakage can occur in and out of the aquifer depending on the difference in potentiometric head between the surficial and Floridan aquifer systems (Tibbals 1990).

The Floridan aquifer system contains the Upper Floridan and Lower Floridan aquifers and is divided by a middle confining unit. This confining layer of low permeability is located completely within the Avon Park limestone geologic formation. The lower portion of the Avon Park Formation along with the Oldsmar Formation comprise the Lower Floridan aquifer, which consists mainly of saline water. The Upper Floridan aquifer consists of the Ocala Limestone Formation and the upper portion of the Avon Park Formation (Miller 1986, Tibbals 1990). Although the Upper Floridan aquifer contains brackish water in the St. Johns River Valley, near the Atlantic coast, and north of Volusia County line, this ground water supply is the main means of meeting public water demands (Kimrey 1990). The entire thickness of the Floridan aquifer system ranges from approximately 1,800 to 2,300 feet in Volusia County.

Separating the surficial aquifer from the Floridan aquifer throughout most of the

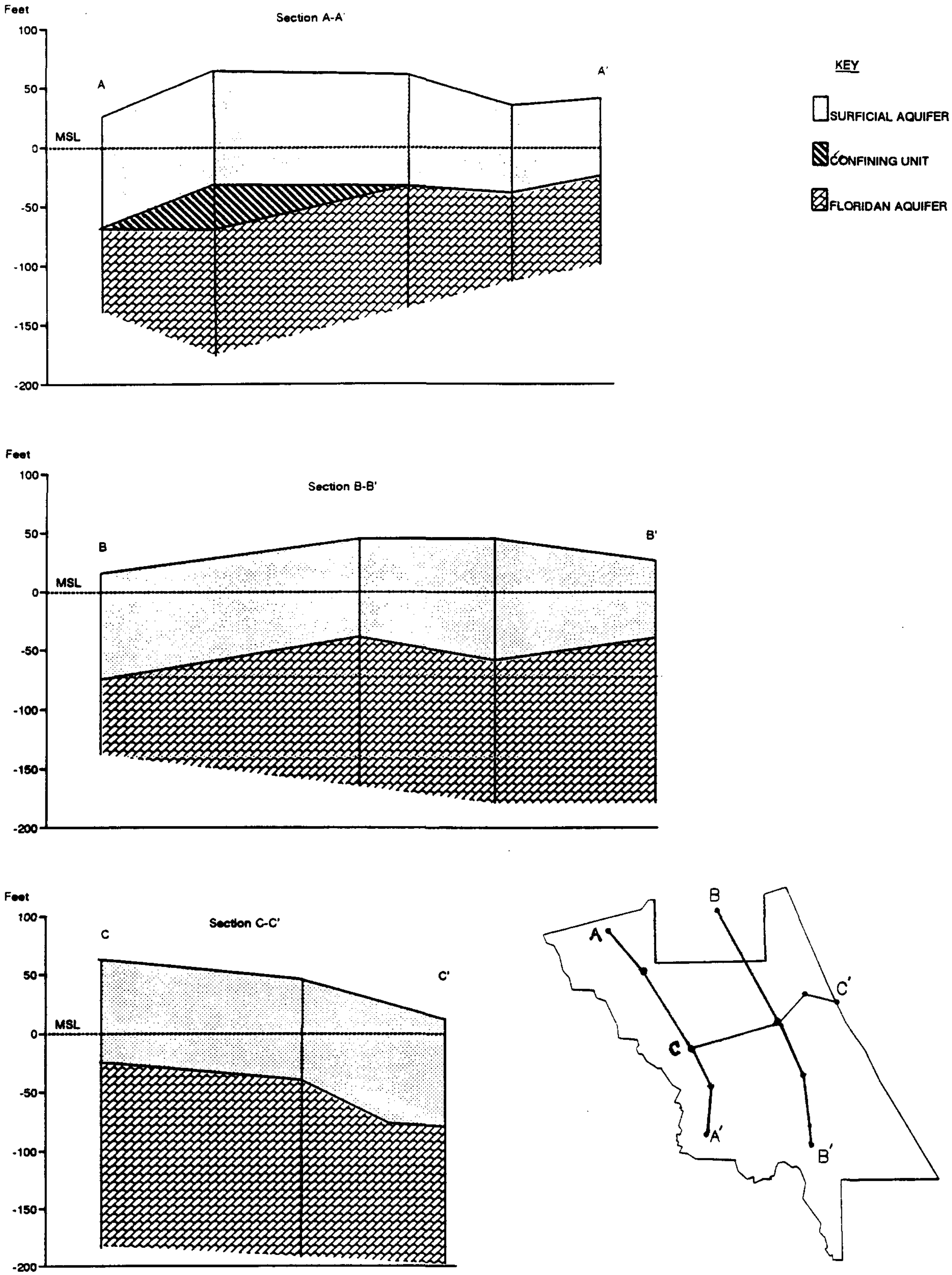


Figure 1.2 - Hydrologic cross sections of Volusia County, Florida.
(source: McKenzie-Arenberg, 1989)

county is the intermediate or upper confining layer. This layer consists of clay or silty sand of the Miocene to Pleistocene age and has a thickness range of 0 to 60 feet. The confining unit in the western portion of the county is thinner than the eastern portion and is sometimes totally absent. Although the low permeability layer is leaky, it is capable of confining the pressurized water in the Floridan aquifer system (Phelps 1990).

1.2 TECHNICAL APPROACH

Once the specific study site was selected, it was possible to initiate development of the water resource allocation models. These models were essentially numerical optimization models comprised of an objective function, decision variables, and constraints. A total of five optimization models were formulated to examine water resource allocations for year 2010, these included three models with hydraulic constraints and two models that incorporate both hydraulic and water quality constraints. The five model specific objective functions included: 1) minimizing the maximum drawdown value, 2) maximizing the minimum pressure head, 3) minimizing average drawdown over all sensitive wetland areas, 4) minimizing the maximum drawdown while limiting chloride concentrations, 5) minimizing the maximum chloride concentration, and 6) minimizing the maximum relative chloride concentration increase. From the literature review, it was determined that the formulation of these water resource allocation models would be predicated on the unit response matrix approach as described in the Literature Review.

Data supplied by SJRWMD and gathered during the project were used to develop the allocation models consisting of an objective function and management constraints.

The data supplied included well data specifying location, capacity, withdrawal rates, corresponding service areas, and additional data identifying potential well locations. Other data included information concerning effluent rates, capacities, and locations of wastewater treatment plants that could supplement the demand of agricultural areas. Water supply demand data consisted of municipal and agricultural water volumes used in year 1988 and projected water quantity requirements for the year 2010.

The District also supplied environmental impact data specifying locations where drawdown was expected to induce adverse effects on wetland vegetation. These target locations were used to constrain the optimization models. Thus, various ground water allocation scenarios could be examined that involve minimizing environmental impacts.

Aquifer system response constraints were incorporated in each optimization model. These constraints were developed from steady-state unit response matrices containing influence coefficients generated from ground water flow and solute transport simulation models. These steady-state unit response matrices represent steady-state changes in pressure head and water chloride concentrations with respect to pumpage. A ground water flow simulation model was executed once for each existing and potential well location to determine the head response at specific locations. A similar approach was taken with the solute transport simulation model to determine chloride changes at wells due to pumpage. Water level and water quality unit response matrices were developed for both municipal and agricultural wells. These response matrices summarize the influence of each well upon itself, upon all other wells, and also upon specified control points throughout Volusia County.

Pumping strategies identified from each optimization model were reviewed and tabulated. To verify these optimum water allocation strategies, the ground water flow and the solute transport simulation models were used to simulate these pumping strategies. Predicted pressure head, drawdown, and water quality changes generated from the simulation models were then compared to predictions given by the five optimization models to determine how well the optimization models emulated the simulation models. When simulated and optimized model values for drawdown, pressure head, and chloride concentration differed severely, response matrices were recreated using initial conditions that better approximate the optimum pumping scenarios identified by each of the original optimization models.

CHAPTER 2
2.0 REVIEW OF CURRENT LITERATURE

Over the last three decades, ground water management models have been formulated using a variety of computational techniques to simulate and optimize the management of aquifer systems. These models have been produced by applying economic theory, heuristic or intuitive procedures, optimization algorithms, hydraulic flow equations, and complex combined simulation/optimization algorithms. Although each of these methods has its own advantages and disadvantages, the combined simulation/optimization approach is found to be the most powerful since it is capable of incorporating economic, physical, and policy considerations within a single model environment. The literature review that follows is limited to only the combined simulation/optimization approach and discusses the various methods and applications of this modeling technique. Also reviewed are applications where optimization modeling has been combined with salt-water intrusion models and computer programs which facilitate model formulation and act as an interface between the simulation and optimization models.

2.1 COMBINED SIMULATION/OPTIMIZATION MODELS

Models developed using the simulation/optimization approach contain both simulation equations and operations research style optimization algorithms. The simulation equations assure that the management model correctly emulates the aquifer responses to external and internal fluxes. The optimization algorithms allow the water management objective and restrictions to be specified as algebraic equations. The combined models are then capable of identifying ground water withdrawal scenarios which optimize given objective functions (Peralta and Willardson, 1992).

Ground water flow simulation models alone are only able to compute pressure heads and fluxes resulting from specified boundary conditions and pumping rates. Using the simulation model alone to determine an optimal pumping scenario can be tedious and error prone process; the simulation model must be executed many times with different pumping strategies in order to determine the best scenario. Unless all possible combinations of pumping strategies are evaluated, the optimum pumping strategy is not likely to be identified. The simulation/optimization models, however, directly compute the optimum strategies under identified management objectives subject to specified system constraints (i.e. assuring the heads and fluxes are constrained within desired limits).

Optimization methods which have been applied in the field of ground water management include linear programming, nonlinear programming, mixed-integer linear programming, quadratic programming, dynamic programming, differential dynamic programming, and goal programming. While these applications differ greatly in their mathematical basis, they may be grouped into general categories according to the method by which ground water flow equations are incorporated into the problem formulation.

Ground water flow equations have been combined with mathematical optimization algorithms through the use of various types of approaches, methods, or techniques. Historically, most simulation/optimization models represent the relationship between aquifer system response and stimuli by incorporating the "embedding" method or the "unit response matrix" approach. For ground water management models, pressure heads or fluid fluxes represent the system response and withdrawal rates represent the stimuli. The following is a review of current literature on the embedding and unit response matrix approaches.

2.1.1 Embedding Method

Models incorporating the "embedding" technique use numerical methods such as finite-differences or finite-elements to transform the partial differential ground water flow equations into a set of linear algebraic equations. These numerical methods both involve the use of a mesh or grid to discretize the aquifer system, with each grid cell specified as having homogenous properties. Pressure heads and discharge (and/or recharge) rates are then depicted by flow equations formulated at each cell node in terms of control or decision variables. These equations are then incorporated into the optimization model as constraints.

The embedding technique with respect to ground water management was first proposed by Aguado and Remson (1974). Their initial work demonstrated that linear programming could be used in conjunction with finite-difference approximations to study the physical response of ground water systems under conditions which define optimal resource management. The optimization models were developed to determine optimum well production scenarios which produce a maximized sum total of hydraulic heads over the area of interest. The method was demonstrated for both confined and unconfined aquifer systems under one-dimensional, two-dimensional, steady-state, and transient flow conditions. For the transient flow case, equations were developed for each time step and then solved simultaneously. For the unconfined aquifer system, the nonlinear equations were approximated by a succession of systems of linear difference equations.

Although the authors conclude that nonlinear flow equations and irregular boundary or initial conditions can be modeled adequately, they concede that the resulting linear programs are typically very large.

A site-specific application of the embedding technique was presented by Aguado et al. (1974) to determine an optimum aquifer dewatering strategy for a proposed dry dock. A linear programming model incorporating steady-state, finite-difference approximations of a two-dimensional unconfined aquifer was developed in order to predict the optimal number of wells, well locations, and corresponding withdrawal rates required to produce and maintain desired ground water levels. The optimization model formulation permits the identification of scenarios that minimize total pumping without restricting the number or location of utilized wells. Model execution led to a solution in which a maximum number of wells were positioned as close to the dewatering region as possible. To verify the accuracy of the optimization model, aquifer response results were compared to those predicted by unconstrained numerical models and also electrical analog ground water models (Remson, Aguado, and Remson, 1974). The authors concluded that a finer grid spacing in the management model would produce improved accuracy with respect to aquifer response, thus an improved optimum pumping strategy.

Alley et al. (1976) demonstrated the embedding technique by using a finite-difference approximation of two-dimensional transient flow. The optimization model determined optimum withdrawal strategies depicting well location and pumping rates for a 20 day time period. Since an existing withdrawal strategy was specified as an initial condition, the optimization model predicted the rate of increased withdrawal (as opposed to the absolute withdrawal rate). To optimize the transient condition, submodels were created which divided the time period into four equal time steps. These models were then executed sequentially to produce initial conditions for the subsequent time interval. This stepwise approach was found to reduce size dimensions of the model, and thus computation time, when compared to the lumped transient approach, which solves all time steps simultaneously. However, this method has been criticized due to conflict between long-range management goals and short-term, sequential optimization (Gorelick 1983). The availability of future water resources is affected by decisions made in earlier time periods.

The Galerkin-finite element method was incorporated by Willis and Newman (1977) to simulate a hypothetical aquifer consisting of heterogeneous anisotropic porous media. The two-dimensional flow equations were embedded into an optimal control ground water problem to determine well locations and pumping rates which would satisfy water demands at minimum cost over a number of planning periods. Since the objective function is non-linear, the algorithm was developed to solve a sequence of linear programming submodels until no improvement in the objective function could be obtained. Additional constraints and "penalty" costs were added to the management model to minimize deviations from the desired final system state. This modification enabled the model to find a more equally distributed pumping pattern, thus minimizing environmental impacts due to drawdown.

Yazicigil and Rasheeduddin (1987) extended the embedding approach to include multi-aquifer systems. Both a transient, single-objective approach and a steady-state, multiple-objective approach were presented. For both cases, a hypothetical, leaky confined two-layer aquifer system was discretized to represent the physical system. Quasi-three-dimensional finite- difference approximations were formulated for each active grid cell and embedded as constraints in the linear programming model.

For the transient case, the water management objective was specified to determine optimal well locations and corresponding withdrawal rates which would minimize drawdowns in the two aquifer system over a four season time period. Equations for all four time steps were effectively solved simultaneously, but an attempt to sequentially solve the four season problem generated an infeasible solution for the fourth time step. These results supported earlier discoveries by Alley et al. (1976) which suggested that the use of the embedding technique with sequential linear programming may result in unacceptable long range management decisions.

To further demonstrate the management model, the authors applied the embedding technique to steady-state, multi-objective analysis by replacing equations representing transient responses and adding a second objective function to the original problem. Constraint and weighing methods were then used to solve the resultant linear programming problem and to develop graphical trade-off curves between total withdrawal rate and total hydraulic head.

Jones et al. (1987) applied a modified embedding approach by incorporating a differential dynamic programming (DDP) algorithm to obtain optimal control of an unsteady, unconfined aquifer. DDP is a successive approximation technique for solving optimal control problems. Equations were incorporated which 1) represented the simulation model of the aquifer system, 2) constrained the hydraulic head and pumping parameters such as well capacity, drawdown, and supply demands, and 3) optimized objective functions such as maximize the sum of heads or minimize yield. The method was found to reduce the dimensionality problems associated with embedding hydraulic equations, linearize the exponential growth in computer time with respect to stages and time periods, and cause to solution to converge quadratically. The authors concluded DDP to be a powerful tool for management of transient ground water hydraulics.

The embedding technique has not only been applied to ground water quantity, but also to ground water quality. Willis (1979) demonstrated the use of ground water quality constraints in linear programming by developing an embedded algorithm for contaminant transport equations. The technique was applied to a ground water system utilized as both a water supply resource and storage reservoir for waste water residuals. The Galerkin method was used to transform the flow and transport differential equations, and the resulting equations were embedded as constraints into the management model to simulate the aquifer system. The model was developed to determine optimum pumping and waste injection rates which satisfy water service demands while maintaining ground water quality as mandated by current ground water standards. The analytical solutions to the transport equations are nonlinear with respect to the decision variables (pumping and injection rates). Therefore, the equations could not be embedded directly into a linear programming model. In order to approximate this nonlinear response and to determine optimal strategy, the management model was redefined to consist of two interdependent linear submodels. Optimal pumping and injection strategies were first determined by a flow submodel, then these results were input into a ground water quality submodel to predict the maximum waste injection concentrations for each operational cycle.

The embedding technique was again applied to ground water quality management by Shamir et al. (1984) for a coastal aquifer in Isreal. Aquifer response was simulated by two sets of finite-difference equations which represent the flow of ground water and the movement of the fresh-water/salt-water interface. These equations were included as constraints in the optimization model. However, in order to represent and constrain the location of the fresh-water/salt-water interface, a linearized model of salt-water intrusion was incorporated to approximate the nonlinear response. Multi-objective linear programming was incorporated to determine optimal annual operation of the coastal aquifer.

Multi-objective optimization involved performing single objective linear programming optimization and developing trade-off functions for conflicting objectives which are then utilized to obtain a final compromise solution. The management model contained four objective functions which minimized 1) changes in water levels due to pumping, 2) intrusion of the fresh-water/salt-water interface, 3) chloride concentrations, and 4) energy costs. Chloride mass balance equations and maximum upper concentration limits were also specified for each aquifer cell location.

Gorelick et al. (1979) applied a numerical finite-difference approximation method to solute transport convective-dispersive equations to determine chloride concentrations of a transient pollutant source. Numerical approximation was utilized to generate a system of linear equations which were then incorporated in a water quality optimization model as part of the formulated constraint set. Water quality standards were specified in the management model, and the model was executed to determine the maximum permissible single source pollutant concentration. The method demonstrated the feasibility of balancing water supply and waste disposal needs while satisfying water quality requirements over long time frames.

As discussed, the embedding technique has been successfully applied to a wide range of ground water management problems. The approach is capable of handling both linear and nonlinear ground water hydraulics; steady-state and transient problems with single or multipurpose objective functions. The primary advantage of the method is the ability to emulate the physical processes of an aquifer system at a detailed level by including ground water flow and/or solute transport equations for each grid cell directly as constraints in the optimization model.

A disadvantage of the method is the dimensionality associated with large scale management problems. The degree of dimensionality is a function of the number of decision variables, aquifer grid cells, and time steps. Therefore, spatially large or multi-time step management models could potentially have hundreds or even thousands of constraint equations. Attempts by Alley et al. (1976) and Yazicigil and Rasheedudd (1987) to redefine multi-time step embedded problems into a sequential set of smaller problems have resulted in management decisions biased towards the short-term. To avoid these difficulties, the embedded technique should be limited to local aquifer systems and steady-state management problems.

2.1.2 Unit Response Matrix Method

The "unit response matrix" method is utilized by simulation/optimization models by incorporating the use of influence coefficients, superposition, and linear systems theory. The simulation model is used initially to compute the system response to a unit stimuli. For ground water management models, the response matrix approach is based on the concept that the influence of discharging or recharging a single well on aquifer drawdown at selected locations may be expressed as simple algebraic functions. The individual influence functions are then combined using the principle of superposition to obtain the aquifer response due to multiple wells. Therefore, a "response matrix" can be developed which may be expressed algebraically as

$$d_j = \sum_{i=1}^{m} [\ a_{ij}q_i\ ] = [\ a_{ij}q_i + a_{(i+1)j}q_{(i+1)} \ \ldots\ a_{mj}q_m\ ]$$

where

d_j = drawdown at control point j
q_i = pumpage at well i
a_{ij} = unit response function of well i on point j
i = 1,2,..., m
j = 1,2,..., n
m = number of wells
n = number of control points

The response matrix of influence coefficients, a_{ij}'s, are generally determined from analytical or numerical ground water simulation models. Since the response equations need only be developed for selected points of interest, it is not necessary for equations to be developed for each grid cell within the aquifer system. This characteristic significantly reduces the dimensionality of the management problem when compared to other simulation/optimization techniques.

One of the earliest pioneers to combine the response matrix approach with ground water management, Deininger (1970) developed a management model which maximized well field production by optimally selecting well locations and withdrawal rates. A response matrix representing aquifer drawdown was developed from an analytical solution to the Theis nonequilibrium equation and incorporated in the linear programming model. Drawdown, pumping, and well characteristics were all specified as constraints in the optimization model. Maximum drawdown limits were specified at well field boundaries, and head versus discharge curves were approximated by piecewise linearization to incorporate drawdown constraints at wells. Although head losses in well casings were also linearized using a chordal approximation of the Manning formula, well screen head losses were assumed to be constant. Deininger demonstrated the formulated problem was easily solved by linear programming techniques.

Rosenwald and Green (1974) used the response matrix method in combination with branch and bound mixed-integer linear programming to identify optimal well sites in an "underground reservoir". Pressure coefficients, which represent potentiometric drawdowns at wells due to changes in discharge rate, were determined by executing a numerical simulation model. A set of constraint equations were then developed which combined the pressure response matrix with maximum allowable drawdown limits. A mixed-integer programming model was employed which assumed constant well flow rates and involved a binary switching variable allowing each potential well location to be either active or inactive.

One of the two cases used to demonstrate the technique involved the selection of well locations in a hypothetical ground water aquifer. Enumeration of the alternatives and comparison with simulation results verified that the response matrix approach based upon linear superposition was applicable to ground water hydraulics. In the second case, the authors applied the response matrix method to select well sites in a gas storage reservoir. The response matrix initially developed erroneously estimated pressure heads due to the nonlinear behavior of gas flow. Using various corrective techniques, attempts were made to improve the pressure response equations. Although the results of the optimization model were only slightly improved, the authors concluded that corrective procedures offer some potential for improving the optimization of nonlinear systems with response matrices.

Maddock (1972) discussed the use of transient algebraic technological functions (response functions) for aquifers whose transmissivities vary with drawdown. These algebraic functions were developed to relate seasonal pumping rates to drawdown levels at specified wells, and the relationship was derived from an analytical solution of the two-dimensional transient ground water flow system. A hypothetical example is used to demonstrate the method of combining response functions and quadratic programming to determine optimal semiannual well withdrawal strategies.

Algebraic response functions were again developed by Maddock (1974) to incorporate the optimization technique in nonlinear aquifer systems. Boussinesq's equation for unsteady flow due to pumping in an unconfined aquifer was approximated by an infinite power series. The total aquifer drawdown response function was then expressed as the finite sum of an infinite power series in pumping values. The approximation assumed fully penetrating wells, no vertical flow, and constant pumpage over a single time horizon. The number of terms required for an accurate water elevation estimate is dependent upon the ratio of drawdown to saturated thickness. To demonstrate the methodology, the author formulated and executed the nonlinear programming problem to determine least cost pumping distributions.

One of the first site specific applications of the response matrix technique was presented by Heidari (1982) and involved determining optimum ground water allocation strategies in Kansas. A two-dimensional ground water simulation model developed for the region was combined with the algebraic influence function proposed by Maddock (1972,1974) to develop the response matrix. For simplicity, sixty-one hypothetical well fields were created to represent total withdrawal from actual wells in the study area. Once the response matrix was incorporated into the management model, constraint equations were specified which limited drawdowns at each well to a fraction of the saturated aquifer thickness.

The optimization model was formulated to maximize total pumpage under two policy scenarios. With respect to the net appropriation (difference between appropriation and recharge), maximum limits were specified for the first scenario but no limits were specified for the second. The models were executed for five and ten year time periods and each contained five time steps. Results revealed that removing the net appropriation constraint created barely a noticeable increase in pumpage when the drawdown fraction was allowed to be equal to or greater than 20%. The model also indicated that under optimal conditions, only about 50% of the net appropriation could be satisfied over the ten year time period.

Willis and Liu (1984) incorporated response functions to develop a bi-objective optimization model to allocate ground water to competing irrigation demands of the Yun Lin basin in southwestern Taiwan. Objectives of the model were to maximize the sum of hydraulic head and to minimize the total water deficit in the basin. The simulation model, which was used to develop the response matrices, was developed using the Galerkin finite-element approximation method for ground water flow and was validated using field data from over 350 monitoring wells. The response matrices were incorporated as constraints and the model was initially executed as a steady-state linear programming problem. Comparison of results from this initial steady-state optimization to existing allocation strategies revealed that the total water deficit could be decreased substantially without decreasing the sum total of hydraulic heads. By assigning weights to the objective functions, solutions to the optimization problem were depicted in the form of trade-off curves to express the relationship between hydraulic head and total water deficit.

The authors reformulated the optimization model to perform a transient analysis. Piecewise linearization was used to incorporate time dependent boundary conditions for the development of the response functions. Using the same objectives specified for the steady-state condition, the model was executed and trade-off curves were again developed to depict the relationship between hydraulic head and water deficit. Results of the transient analysis indicated that the steady-state formulation overestimated the reduction in water deficit. However, the authors concluded that a significant water savings could still be achieved with the use of the optimization model.

Danskin and Gorelick (1985) were some of the first to implement response functions in the management of multi-layer aquifer systems. The study area consisted of a multi-layer aquifer system connected to a surface water system, and the model was formulated to evaluate the efficiency of flow between the two systems. Critical factors controlling basin management decisions were also identified through execution of the mixed-integer linear programming model. The response matrix was developed using a quasi-three-dimensional finite-difference simulation model, and the influence coefficients for the upper unconfined aquifer was linearized using an iterative approach. The response functions were used to develop water elevation constraints and additional limitations were specified with respect to surface water flow, vertical leakance, and water service demands. Mass balance constraints were also incorporated to regulate the concentration of dissolved solids.

Evaluation of historical basin management practices revealed that "the cost of operation during the study period was twice that of optimal basin management." However, the largest inefficiencies were localized and most activities were within 20% of optimal. Sensitivity analyses indicated that the controlling factor in basin operations involved surface water and ground water relationships.

Willis and Finney (1985) developed response equations using finite-difference methods, quasi-linearization, and matrix calculations. Nonlinear optimization for an unconfined aquifer system was performed by incorporating a quasi-linearization optimization algorithm and projected Langrangean methods. The model was structured as a discrete optimal control problem and determined the optimal pumping pattern while satisfying water demands. Quasi-linearization and MINOS (Modular In-Core Nonlinear Optimization System) algorithms were both found to be efficient for solving moderately sized nonlinear ground water management models. However, Willis and Finney concluded that large management problems could be solved more efficiently by applying quasi-linearization optimization.

Herrling and Heckele (1986) used both the embedding and response matrix techniques to couple a finite-element simulation model with an linear simplex optimization model. Management of the nonlinear ground water system was performed by optimizing well locations along with pumping and infiltration rates while satisfying ecological constraints. These constraints consisted of sustaining ground water levels and meeting contamination standards. The advantages of both coupling techniques were incorporated. The embedding method involved the implicit consideration of the flow model, and the influence function method reduced the amount of computer storage required.

The response matrix technique was combined with a stochastic approach by Tung (1986) for the management of ground water resources. The "chance constrained" management model was formulated using linear systems theory to determine optimal withdrawal rates in a well field subject to specified reliability constraints. Transient, nonleaky drawdown response functions were developed from an analytical solution to the Cooper-Jacob equation while transmissivity and storativity values were treated as independent random variables. First-order analysis was employed to estimate the statistical characteristics of drawdown at each control point. Stochastic drawdown response functions implicitly incorporating parameter uncertainty were then developed as linear constraints. Drawdown limitations were stated at each control point which required drawdowns to be less than a specified value times a reliability factor.

The management model was applied to a hypothetical confined aquifer where optimal withdrawal rates were determined for three potential wells over three time periods. While maximizing total well production, a sensitivity analysis was performed on the model to investigate the effect of parameter uncertainty and of varying reliability factors. Tung discovered that model results were insensitive to changes in storativity but quite sensitive to changes in transmissivity and the specified reliability factor. A postoptimal analysis indicated that the linear programming model produced acceptable results in terms of complying with reliability requirements, but only when the transmissivity uncertainty was small.

Lindner et al. (1988) applied the response matrix technique to a two-aquifer system to determine the optimum ground water withdrawal while meeting established environmental criteria. The two-aquifer system consisted of an upper unconfined and a lower confined aquifer with a leaky intermediate layer. The environmental criteria included meeting specified ecological conditions and ground water levels in the upper aquifer. Though ground water was withdrawn from the confined aquifer only, the unconfined aquifer was also affected due to leakage from the separating layer.

Galeati and Gambolati (1988) employed the response matrix technique to solve a three-dimensional aquifer dewatering problem. The water management model was formulated to identify the optimal spatial distribution and corresponding well rates required to maintain desired water levels during two planning periods. A three-dimensional finite-element ground water flow model was developed and repeatedly executed to develop a steady-state response for each abstraction well. The individual influence coefficients were then combined using the principle of superposition, and hydraulic head limits were specified to create a set of water elevation constraints. The model was formulated to minimize total withdrawal while still dewatering the study area. The authors assumed linear response since the dewatering area responds as a hydrodynamically closed system having essentially no influence on the surrounding unconfined aquifer.

Once the management model was executed and an optimal dewatering strategy was prescribed, it was discovered that some wells which were deemed active during the first planning period were deemed inactive during the second period, and vice versa.

Therefore, in order to avoid large installation costs and withdrawal rate nonuniformity associated with the solution, the model was reformulated as a mixed-integer linear programming model. Mixed-integer linear programming involved designating wells with a withdrawal rate of either zero or some constant value. Additional constraints were added which required wells be utilized in both time periods if they are utilized at all. Although the solution to this reformulated problem reduced the number of wells by 36%, the total pumping rate increased slightly. After performing yet another reformulation and execution with respect to minimizing costs, the authors concluded that the intermediate mixed-integer programming solution represented the best compromise between withdrawal and installation costs.

Similar research was performed by Lall and Santini (1989) to extend the response matrix approach to a nonlinear, multilayered, unconfined aquifer system. The Grinski potential concept was utilized to develop a linear approximation of the nonlinear aquifer system and was described as being analogous to the velocity potential for a confined aquifer system. For this case, hydraulic head is directly related to the vertical depth and hydraulic conductivity of each layer. The authors demonstrated that the steady-state continuity equation is a linear function of Grinski potential when hydraulic heads were converted to Grinski potentials. By incorporating the method of finite elements, the aquifer responses were then modeled as linear functions of the Grinski potential. The superposition principle was employed to sum the individual responses and obtain a response matrix in terms of the Grinski potential. The use of the Grinski potential for linear approximation was also shown to be applicable to transient ground water hydraulics under certain conditions. The method was also demonstrated using three different variations of the original dewatering problem.

Response functions and mixed integer programming were applied by Chau (1989) to analyze pressure relief systems. Optimal well sites and discharge schedules were determined while discharge was minimized and hydraulic heads were maintained with respect to soil stability and ground water flow. Response functions were determined from a simulation model and represented the drawdown as induced by discharge from another well and time period. The effects of varying performance parameters which represent aquifer characteristics, hydraulic head limits, well capacities, and discharge elevation at well locations were evaluated. Two hypothetical examples were analyzed to demonstrate the trade-offs between system parameters and system performance.

Yazicigil (1990) incorporated the response function approach to determine optimal planning and operating policies of a multiaquifer system in Eastern Saudi Arabia. MODFLOW, a three-dimensional finite-difference ground water flow model developed by McDonald and Harbaugh (1988), was executed under transient conditions and used to generate response coefficients for each well field. Since the flow model was formulated to simulate an eight year planning period with monthly time steps, over 20,000 individual influence coefficients were developed. These response functions were then combined with both linear and quadratic programming to determine optimal water management strategies for the 52 well fields in the basin. Three different objective functions were formulated which maximized withdrawals, minimized drawdowns, and minimized pumping costs. Trade-off costs which related withdrawal to drawdown, aquifer dewatering, and pumping costs were also determined from results of the optimization model.

The response matrix technique has not only been applied to management models with respect to ground water quantity, but also ground water quality. Colarullo et al. (1984) demonstrated this approach using a hypothetical unconfined aquifer which had historically been used for surface waste disposal, but was soon to be developed as a fresh-water supply. A two-dimensional ground water flow model was incorporated to develop linear response equations which represented the aquifer response due to a change in pumpage rate. Using this same method, response equations were also defined for pumpage induced velocities and incorporated in the model as constraints to limit contaminant flow. The optimization model was formulated to determine what quantity of water could be removed to supplement water service demands and how interception wells should be operated to avoid contamination of fresh-water supplies. A nonlinear optimization algorithm was utilized to identify optimal well discharges for supply and interception wells. The authors verified previous assumptions of linearity for the response matrices by executing the simulation model with the prescribed strategy.

Influence coefficients derived from an approximation of a solute transport model were applied by Datta and Peralta (1985) to revise a quantity optimization model to include quality. The authors applied the approach of Peralta and Killian (1985) to optimize the potentiometric surface and identify the water use strategy required to maintain the surface. Steady-state hydraulic stresses were determined from the simulation model, and steady-state ground water concentrations were determined from the solute transport model. The influence coefficients were determined based on the hydraulic head levels required to meet quality limits, and then used in establishing new hydraulic head constraints. The modified optimization model was created through the use of constrained derivatives of the quadratic optimization model. The methodology was found useful in determining concentrations in a subsystem of a larger model and/or the influence of water quantity changes on water quality.

Similar to the effort presented by Tung (1986), Wagner and Gorelick (1987) combined the response matrix approach with a stochastic optimization model for ground water quality management. A two-dimensional finite element flow and solute transport model was utilized to develop the head and concentration response, respectively, as a function of pumping and aquifer parameters. Simulation and multiple regression were used to develop parameter estimates and minimize the differences between simulated and observed values. In order to incorporate parameter uncertainty, the response equations were transformed from deterministic constraints to probabilistic constraints through first-order, first- and second-moment analysis. The chance constrained nonlinear optimization model was formulated to identify well locations and withdrawal rates for aquifer remediation under specified parameter uncertainty.

The authors applied the management model to both steady-state and transient conditions to determine which withdrawal strategy would satisfy water quality standards. Results revealed that the prescribed location, number, and pumping rate of wells were very sensitive to reliability level. Monte Carlo simulations verified that the true mean concentration and the concentration predicted by first-order analyses were nearly identical and normally distributed.

Finney et al. (1992) incorporated response equations into a management model for the control of salt-water intrusion in a multiaquifer system in Jakarta, Indonesia. Hydraulic response equations were also used by Willis and Finney (1988) in the development of a simulation/optimization model for the management of seawater intrusion in Yun-Lin ground water basin of southwestern Taiwan. These efforts along with others are discussed in the following literature review of salt-water intrusion models.

2.2 SALT-WATER INTRUSION MODELS

In many coastal areas, excessive abstraction of ground water has resulted in a decline of fresh-water potentiometric heads and therefore an intrusion of salt water. When this encroachment may have adverse effects on water supply quality, simulation/optimization models are applied to these coastal areas in order to minimize the effect of salt-water intrusion on fresh-water supplies. Simulation models which predict the location of the fresh-water/salt-water interface are combined with optimization models to minimize encroachment of the interface while meeting pumping demands.

Some of the initial research involving the optimum exploitations of ground water reserves located in coastal aquifers was performed by Cummings (1971) and Youngs (1971). Cummings created a ground water management optimization model which dealt with the economic impacts of aquifer exploitation and salt-water intrusion with respect to time. The model focused on the interrelated problems between annual ground water pumping rate and the annual cost of pumping. Although Cummings mentions how hydrological simulation models and optimization models complement each other, most of his research involved the economic side of ground water mining.

Youngs' (1971) research involved determining the optimum well conditions in order to maximize fresh-water output from coastal aquifers. By performing an analysis of the horizontal seepage, Youngs determined that the maximum pumping rate of fresh water could be calculated precisely and that the optimum conditions for well installation can be found which will produce the maximum continuous pumping rate. Knowing that pore water pressure is reduced near an operating well which causes an upconing of saline water, the analysis identifies the pumping rate required to raise the interface to the bottom of the well. Youngs then determined that maximum withdrawal is obtained when wells are positioned to a elevation equal to sea level and are pumped to maximum capacity. The analysis is presented using two hypothetical examples.

The dynamic programming approach was applied by Nutbrown et al. (1975) in a digital simulation/optimization model in order to effectively and efficiently manage a coastal aquifer near Brighton, England. The goal of their research was to predict the maximum yield of the aquifer while considering the limiting factor of salt-water intrusion. The simulation model was created to describe the transient effects of natural infiltration and abstraction of both fresh and saline ground water. The dynamic programming method was combined with the simulation model to generate abstraction regimes which would maximize the final storage of fresh water in the aquifer.

The approach was based on the methodology that at regular intervals of time, the spatial abstraction distribution for succeeding intervals could be calculated on the basis of existing water levels. The concept that pumping rate was proportional to the magnitude of local ground water flow was assumed. The model was then executed under average infiltration conditions until cyclic equilibrium was attained. Once validity had been determined, the model was applied to various drought and recharge conditions to determine the various optimum scenarios.

As mentioned earlier in the Embedding Method section of this literature review, Shamir et al. (1984) developed an annual operating plan for a coastal aquifer in Israel.

A linearized model of salt-water intrusion was combined with a flow simulation model to represent the movement of the salt-water interface. This simulation model was incorporated into a multi-objective linear programming model to produce the management model.

A planning model was developed by Willis and Finney (1988) to control salt-water intrusion and declining ground water levels in the Yun Lin ground water basin of southwestern Taiwan. The simulation/optimization model applied hydraulic response equations to relate the location and magnitude of ground water pumping and recharge to the movement of the salt-water interface. Finite-difference methods were used in the simulation model to approximate the aquifer's response to various management strategies. The model was based on the following hydraulic assumptions: 1) Hydrodynamic dispersion is negligible so the sharp interface theory is valid; 2) The Dupuit approximation is valid throughout the aquifer system; 3) The aquifer base is impermeable; 4) The hydraulic conductivity and storage coefficients are invariant with depth; and 5) Leakage to or from the aquifer is at steady-state.

The solution algorithms were based on the influence-coefficient method combined with quadratic programming and also the reduced-gradient method combined with a quasi-Newton algorithm. The influence-coefficient algorithm is based on hydraulic response equations of the ground water system, and the coefficients are applied to the quadratic programming optimization problem of the management model. The quadratic program determines the optimal direction vector which is then used to revise the current solution using a gradient-based algorithm. The reduced-gradient/quasi-Newton algorithm can also be used to determine optimal solutions of the planning model. This algorithm is implemented using MINOS and is able to determine a feasible decent direction vector at any iteration. The simulation/optimization model was created using historical ground water data for the basin in order to develop optimal pumping and recharge schedules.

The authors made the following conclusions based on the results of the optimization analysis: 1) Both algorithms produced stable and reliable solutions to the salt-water management problem; 2) Greatly differing pumping and recharge schedules produced essentially the same objective function values for the applied basin; 3) The management problem is characterized by local optimality problems; and 4) Different starting solutions for the algorithms produce different optimal schedules.

Finney et al. (1992) incorporated response equations into a management model for the control of salt-water intrusion in a multiaquifer system in Jakarta, Indonesia. Response functions relating pumpage to the location and movement of the salt-water interface were developed from a finite-difference simulation model. Within the model, multiple aquifer systems were linked through their recharge terms. The response equations were then included as part of a nonlinear optimization model which sought to minimize the total squared volume of salt water in each aquifer. Initial attempts to reach a solution with the MINOS programming package resulted in solutions that were not truly local optimum values. To correct the problem, Box's sequential search algorithm was used. This resulted in a 20% improvement over the MINOS generated values. Finney et al. concluded that the combined use of simulation and optimization was able to reduce the magnitude of salt-water intrusion in the Jakarta basin by 6%.

2.3 INTERFACE PROGRAMS

Due to the increasing popularity of the U.S. Geological Survey Modular Three-Dimensional Finite-Difference Ground water Flow Model (MODFLOW) and of ground water optimization models, recent efforts have been made by various groups to combine these models via interface computer programs. AQMAN3D and MODMAN are two such programs which simplify the interaction and iteration process between MODFLOW and particular standard optimization programs (GAMS, MINOS, etc.).

Puig et al. (1992) developed AQMAN3D by modifying the two-dimensional code AQMAN originally developed by Lefkoff and Gorelick (1987). The revised version is a mathematical programming system data set generator for aquifer management using MODFLOW as its ground water flow simulation subroutine. The FORTRAN-77 computer code can be used to formulate a variety of aquifer management models; depending on the chosen objective function to be optimized and the constraints imposed on hydraulic conditions. The program creates input files to be used by any standard optimization program using Mathematical Programming System (MPS) input format.

When AQMAN3D is used with an optimization program, the optimum pumping and/or recharge strategy can be determined while ground water hydraulic conditions are maintained within specified limits. The applied management function may be linear or nonlinear and restrictions can be applied to ground water heads, gradients, and/or velocities. The program is limited to confined or quasi-confined aquifer systems and to nonlinear sources/sinks in the ground water flow model.

The AQMAN3D aquifer management modeling process is initiated by qualitatively and quantitatively conceptualizing the aquifer system by using MODFLOW.

The simulation flow model is then calibrated to steady-state or transient conditions and then executed to simulate particular unmanaged aquifer conditions. The optimization portion of the model is then initiated by developing a objective function and a system of constraints which reflect the aquifer management condition. The AQMAN3D program is now executed to interact with MODFLOW to produce the MPS input file containing the objective function, constraints, and response coefficient matrix. The MPS file is applied to the standard optimization model to determine the optimal scenario for the specified management conditions. This optimal scenario should then be applied back into the MODFLOW model to observe and verify the response of the aquifer system.

Another model which has been developed to perform the interface operations between MODFLOW and a standard optimization program is recognized as MODMAN (MODFLOW Management - An Optimization Module for MODFLOW). GeoTrans, Inc. (1990) developed MODMAN for use by the Southwest Florida Water Management District to assist in determining optimum pumping scenarios in their region. MODMAN is analogous to AQMAN3D in that it is a modified extension of AQMAN (Lefkoff and Gorelick 1987) and accommodates three-dimensional problems. However, unlike AQMAN3D, MODMAN is actually a linked module or subroutine of a revised version of MODFLOW and can be executed in two modes. The MODFLOW code was modified by the creators of MODMAN in order to facilitate the process of determining the response coefficient matrix.

When used together with standard optimization software, MODMAN determines the optimum strategy depicting the location and rate of extraction and/or injection wells while satisfying user-specified constraints. One of many objective functions may be maximized or minimized depending on the goals of the ground water manager. Once the optimal pumping/injection strategy has been determined, MODMAN is incorporated again by acting as a postprocessor of the optimization output data. MODMAN automatically inserts the optimal well rates into the MODFLOW input file, executes a simulation based on the optimal well strategy, indicates which constraints are binding in the optimization model, and warns the user if nonlinearities have significantly affected the optimization process.

2.4 SUMMARY

The approach of combining simulation and optimization models was first used in the development of management models in the late 1960's and early 1970's. This type of an approach offers the water resource manager a means of considering physical, policy, and environmental factors simultaneously. Generally, the combined approaches can be divided into two separate categories according to the method in which ground water flow equations are incorporated into mathematical optimization models.

In the "embedding" technique, numerical or analytical solutions to ground water flow and/or transport equations are written for each control node and are subsequently included as constraints in the optimization model. This method has been successfully applied to a large number of ground water management problems and has been demonstrated to handle both steady-state and transient conditions. This approach has also been used with problems involving linear and nonlinear ground water hydraulics and single and multipurpose objective functions. Although the ability to accurately simulate physical processes of an aquifer system is a major advantage, there is a dimensionality problem associated with the embedding approach when applied to large scale management problems. Therefore, this technique is generally only applicable to small aquifer systems and steady-state management problems.

The "response matrix" approach has also been demonstrated as a technique to combine simulation and optimization models for ground water management. As opposed to the embedding technique however, this method is capable of handling large scale management problems. The response matrix is composed of coefficients which represent the influence of specified wells on the aquifer system. The aquifer response in terms of change in hydraulic head, or drawdown, is usually derived from a simulation model. The response is then expressed as simple algebraic functions in the mathematical model using the principle of superposition. Constraints are incorporated into the algebraic functions to limit drawdown at specified control points. Because equations are developed only for the selected control points, use of the response matrix method generally results in a reduced problem.

Additional ground water optimization methods including differential dynamic programming, multi-objective programming, and quadratic programming were reviewed. Techniques such as combining embedding and response matrix approaches for a simulation/optimization model and revising an existing quantity optimization model to incorporate quality were also discussed.

An assortment of optimization models were reviewed which incorporated varying methods of managing coastal aquifers while minimizing the adverse effects of salt-water intrusion. Many of the models were combined with simulation models which applied finite-difference methods to determine the aquifer's response to varying pumping strategies. All of the models assumed that the effect of hydraulic dispersion between fresh and salt water was negligible so that the "sharp interface" concept was acceptable.

In order to simplify the interaction process between the ground water simulation model and the numerical programming software, interface programs have been developed which link the ground water flow model MODFLOW with a standard optimization program. Two such models which were reviewed are AQMAN3D and MODMAN. Both models are extended modifications of AQMAN (a previously developed two-dimensional interface model), incorporate the response matrix approach, and accommodate three-dimensional problems. MODMAN differs from AQMAN3D in that it is actually a module of MODFLOW, it can be applied in a second mode as a postprocessor, and it is more "user-friendly".

Existing simulation/optimization models pertaining to ground water resource allocation and management are displayed in tabular form in Table 2.1. The model name, author(s), type, aquifer condition, optimization technique, objective function, and constraints are shown for easy comparison.

Table 2.1 - Ground water optimization models.

Author [Year]	Type [Dimension]	Aquifer Condition	Equation / Solution Technique	Objective Function [Liner / Nonlinear]	Constraints [Linear / Nonlinear]	Optimization Technique	Model Name
Ahlfeld [1988]	Quantity / Quality [2-D]	Saturated Confined Steady / Unsteady	Linear / Finite Element	Minimize Total Pumping [Linear]	Pumping Rates, Hydraulic Head, Magnitude & Direction of Groundwater Velocity [Linear]	Linear Programming	VCON
Ahlfeld & Pinder [1988]	Quantity / Quality [1 & 2-D]	Saturated Confined Steady / Unsteady	Linear / Finite Element	Minimize Total Volume of Pumping [Linear]	Minimum Concentrations at All Nodes at Future Times [Non-linear]	Nonlinear Programming	GW2SEN
Alley et al. [1976]	Quantity [2-D]	Unsteady - Confined Saturated	Linear / Finite Difference	Maximize Hydraulic Heads [Linear]	Well Flow Rates, Hydraulic Heads [Linear]	Linear Programming	AQMG
Aly & Peralta	Quantity [2-D]	Steady / Unsteady Confined / Unconfined	Theis Analytical [Well Function] Deterministic / Stochastic	Minimize Extraction, Injection [Linear / Nonlinear]	Hydraulic Gradients, Extraction or Injection Rates, Hydraulic Head [Linear / Nonlinear]	Linear, Quadratic, or Nonlinear Programming	US/WELL
Aquado & Remson [1974]	Quantity [2-D]	Steady - Unconfined Saturated	Linearized / Finite Difference	Minimize Total Pumping [Linear]	Hydraulic Heads, Pumping Rates [Linear]	Linear Programming	OPAQD
Deninger [1970]	Quantity Radial	Unsteady,Unconfined Saturated	Analytical [Well Formula]	Maximize Well Production [Linear]	Drawdown, Well Facility [Linear]	Linear Programming	SAWSS
Elango & Rouve [1980]	Quantity [2-D]	Steady - Confined Saturated	Linear / Finite Element	Maximize Total Pumpages [Linear]	Flow Equilibrium, Piezometric Heads, Pumping Capacities [Linear]	Linear Programming	-----

Table 2.1 - continued

Author [Year]	Type [Dimension]	Aquifer Condition	Equation / Solution Technique	Objective Function [Liner / Nonlinear]	Constraints [Linear / Nonlinear]	Optimization Technique	Model Name
Gorelick & Remson [1982]	Quality [2-D]	Steady State Flow and Transport Confined Saturated	Linear / Finite Difference	Maximize Waste Disposal [Linear]	Water Quality [Linear]	Linear Programming	OLMWDF
Haimes & Dretzen [1977]	Quantity [2-D]	Aquifer / Stream System, Unsteady-Unconfined Saturated	Linearized / Cell Model	Maximize User's Net Benefit [Quadratic]	Water Requirements, Lift & Pumping limits, Capacity of Recharge Facility [Linear]	Quadratic Programming	MGWSW
Heidari [1982]	Quantity [2-D]	Unsteady,Unconfined Saturated	Linearized / Finite Difference	Maximize Pumping Rates over Time [Linear]	Pumping Demands, Drawdown [Linear]	Linear Programming	LSTLP
Koltermann [1983]	Quantity [2-D]	Unsteady - Confined Saturated	Linear / Finite Difference	Maximize Hydraulic Head and Minimize Water Transfer and Recharge [Linear]	Groundwater Flow, Piezometric Heads, Pumping Capacities [Linear]	Linear Programming	-----
Larson et al. [1977]	Quantity [2-D]	Steady - Unconfined	Linearized / Finite Difference	Maximize Steady State Pumping [Linear]	Pumping Rates, Number of Wells, Drawdown [Linear]	Mixed Integer Linear Programming	OTGWD
Molz & Bell [1977]	Quantity [2-D]	Steady - Confined Saturated	Linear / Finite Difference	Minimize Total Pumping [Linear]	Hydraulic Heads, Head Gradients [Linear]	Linear Programming	HGCAQ
Peralta et al. [1987]	Quantity / Quality [2-D]	Confined Saturated Steady / Unsteady	Linearized Boussinesq, Linearized & Nonlinear Transport/ Method for Characteristics	Minimize Changes in Piezometric Head [Linear]	Steady/Unsteady Flow and Transport Water Quality [Linear]	Linear Goal Programming	MODCON
Peralta et al. [1992]	Quantity [3-D]	Steady / Unsteady Confined / Unconfined Saturated/Unsaturated	Linear / Finite Difference	Minimize Withdrawal and Recharge Rates [Linear]	Head, Head Gradient, Flow Velocities, Demand, Capacity & Pumping [Linear]	Linear Programming	US/REMAX

Table 2.1 - continued

Author [Year]	Type [Dimension]	Aquifer Condition	Equation / Solution Technique	Objective Function [Liner / Nonlinear]	Constraints [Linear / Nonlinear]	Optimization Technique	Model Name
Rosenwald & Green [1974]	Quantity [2-D]	Unsteady-Unconfined Saturated	Linearized / Finite Difference	Minimize Total Pumping [Linear]	Production Demand, Number of Wells [Linear]	Mixed Integer Linear Programming	OLWR
Shamir et al. [1983]	Quantity [2-D]	Unsteady-Unconfined Saturated	Linearized / Finite Difference	Minimize Energy Demands for Pumping / Recharge [Linear]	Pumping Demands, Import/Export Fluxes, Drawdown Position, Water Quality Limits [Linear]	Linear Programming	OPOCAQ
Takahashi & Peralta [1991]	Quantity [Quasi 3-D]	Confined/Unconfined Steady	Linear / Finite Difference	Maximize Extraction [Linear]	Potentiomentric Head, Pumping Rate [Linear]	Linear Programming	USUGWM
Willis [1979]	Quantity / Quality [2-D]	Confined Saturated Unsteady Transport Quasi-Steady Flow	Linear / Finite Element	Maximize Lowest of Waste Concentration [Linear]	Water Target, Pumping & Injection, Water Quality [Linear]	Linear Programming	PMMGWQ
Willis [1983]	Quantity [2-D]	Unsteady Confined/ Unconfined	Boussinesq Equation/ Finite Element Analytical	Maximize Sum of Heads, Minimze deficit [Linear]	Agricultural Demand, Heads, Well Capacity [Linear]	Linear Programming	UARGWM
Yazicigil et al. [1987]	Quantity [2-D]	Unsteady - Confined Saturated	Linear / Finite Difference	Minimize Total Pumping [Linear]	Water Demands, Maximum & Minimum Pumping Rates, Drawdowns [Linear]	Linear Programming	OPMRA
Yazicigill & Rashee-duddin [1987]	Quantity [3-D]	Saturated Confined Steady / Unsteady	Linear / Finite Difference	Maximize Total Hydraulic Head [Linear]	Water Demands, Hydraulic head bounds, Maximize Pump Rates [Linear]	Linear Programming	OPMGW

Reference: El-Kadi et al. (1991)

CHAPTER 3
3.0 GROUND WATER SIMULATION MODELS

From the literature review, it was determined that the most efficient general formulation of the allocation models would be one containing aquifer response constraints developed from steady-state unit response matrices created with ground water flow and transport simulation models. Two- and three-dimensional finite-difference ground water simulation models have been employed extensively for the last twenty years by the United States Geological Survey and other ground water professionals. These numerical simulation models apply particular boundary conditions to a spatially discretized aquifer system in order to predict potentiometric heads, fluxes, and solute concentrations throughout a specified region. These boundary conditions include the size, shape, hydrogeologic framework, hydraulic characteristics, and particular known fluxes affecting the aquifer system. Although there are various ground water flow simulation programs available, MODFLOW was used in this project, because a simulation model had already been developed for the project area. Similarly the solute transport model program DSTRAM was incorporated into the project because it had been successfully implemented in the project area.

MODFLOW is a modular three-dimensional finite-difference ground water flow program created by the United States Geological Survey (McDonald and Harbaugh 1988). The model is based on the following well known governing equations describing the movement of an incompressible fluid through porous material:

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial X}\right) + \frac{\partial}{\partial Y}\left(K_{yy}\frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right) - W = S_s\frac{\partial h}{\partial t} \tag{3.1}$$

where K_{xx}, K_{yy}, and K_{zz}, are values of hydraulic conductivity along the x, y, and z coordinate axes, which are assumed to be parallel to the primary axes of hydraulic conductivity (Lt^{-1}). H is the potentiometric head (L), W is a volumetric flux per unit volume and represents sources and/or sinks of water (t^{1}), S_s is the specific storage of the porous material (L^{-1}), and t is time (t).

Equation 3.1 approximates flow under non-equilibrium conditions in a heterogeneous and anisotropic medium, assuming the principal axes of hydraulic conductivity are aligned with the coordinate directions. Since analytical solutions to this equation are rarely possible, the finite-difference numerical method is implemented to procure approximate solutions. In this method, the continuous system described by the partial-differential equation is replaced by a finite set of discretized equations in time and space. Continuous partial derivatives are replaced by discrete algebraic functions describing the change in pressure head at the distinct points. A system of linear algebraic difference equations results from this methodology, and its solution produces values of pressure head at specific points and time.

MODFLOW is able to simulate aquifer systems with layers that are confined, unconfined, or a combination of both. Each cell of every layer is specified as being inactive (no flow), active (variable head), or having a constant head. Boundary conditions such as specific flux, specific head, or a head-dependant flux can be applied in the model. The model is capable of simulating external flows such as discharge and injection wells, rivers and streams, evapotranspiration, precipitation, and agricultural drains. Because of its diversity and effectiveness, MODFLOW has become one of the most popular ground water flow simulation models amongst hydrogeologic professionals.

DSTRAM developed by HydroGeologic Inc. is a three-dimensional numerical finite element program that simulates fluid flow and solute transport saturated porous media. The code is capable of performing several types of analysis. These include ground water flow analyses, trace concentration solute transport analyses, and density dependent coupled flow and transport analyses. The code is based on the following governing equation for three-dimensional density-dependent flow and transport in an aquifer system:

$$\frac{\partial}{\partial x_i}\left[\rho\frac{k_{ij}}{\mu}\left(\frac{\partial p}{\partial x_j}+\rho g e_j\right)\right] = \frac{\partial}{\partial t}\phi\rho \qquad i,j = 1,2,3 \tag{3.2}$$

where p is fluid pressure, k_{ij} is the intrinsic permeability tensor, ρ is the fluid density, μ is the dynamic viscosity, g is the gravitational acceleration, e_j is the unit vector in the upward vertical direction, and ϕ is the porosity of the porous medium.

DSTRAM analysis can be performed in an areal plane, a vertical cross-section, an axisymmetric configuration, or a fully three-dimensional mode. Because of its special design features, DSTRAM is capable of handling a wide range of complex three-dimensional, steady-state or transient, field problems and producing values of solute concentration at specific points and times (Huyakorn and Panday, 1991).

3.1 REGIONAL FLOW SIMULATION MODEL

A regional model covering the Volusia County study area was acquired from SJRWMD. The model was originally developed by Geraghty & Miller, Inc. (1991), but several modifications were incorporated by SJRWMD to increase simulation accuracy (Williams, 1993). The model as received is capable of simulating three different steady-state stress conditions with respect to time, predevelopment, year 1988, and year 2010.

The finite-difference grid of the regional model area consists of 86 columns, 91 rows, and 5 layers for a total of 39,130 cells. The model covers an area of approximately 1,850 square miles and consists of planar cell spacings varying from 0.25 to 2.0 miles. The model is bounded by the St. John's river on the west and extends approximately seven miles off the Atlantic Coast on the east.

The 5 layers of the model aquifer system consist of the surficial aquifer in layer 1, the Upper Floridan aquifer in layer 2, the middle semi-confining unit of the Floridan aquifer in layer 3, and the Lower Floridan aquifer in layers 4 and 5. The upper confining unit which separates the surficial and Upper Floridan aquifers are interconnected through the use of leakance coefficients which represent the hydraulic connection between the two layers. The surficial aquifer is modeled as unconfined and the Floridan aquifer system is modeled as confined.

The boundary conditions used in the development of the regional model included constant flux, constant head, and head-dependant flux boundaries. Constant head boundary conditions were applied to regions where impacts of pumping were assumed to be negligible. These areas include the Atlantic Ocean, Halifax River, St. Johns River, Blue Springs vicinity, and miscellaneous lakes within the county (See Figure 3.1). Constant heads were also applied along the edge of the model near Blue Springs in the surficial aquifer to allow flow from Lake County. Constant flux boundary conditions were used in the model to represent the various discharge and recharge areas. The discharge flow rates for each cell were calculated by summing the pumping rates from all wells located within that cell for a specific layer.

Head-dependant flux boundaries are often known as mixed-type boundary conditions since they are essentially a combination of specified head and constant flux conditions. They are applied to represent an unknown flux which is dependant on a specified, hydraulically connected, and externally-located head. Blue Spring, Ponce De Leon Springs, Gemini Springs, and a variety of small creeks were modeled using head-dependant flux boundaries. The western and northern boundaries of the Floridan Aquifer system were defined with a type of head-dependant boundary called general head boundaries in MODFLOW. These conditions provide adequate inflow to the model while minimizing boundary effects due to varying pumping rates.

3.2 FLOW SIMULATION MODEL FOR PROJECT AREA

The five optimization models produced through the efforts of this project were applied to the region defined by SJRWMD (See Figure 3.2). This project area covers a smaller subarea of the above described regional flow simulation model. To facilitate the development of this model, the Volusia County regional flow simulation model was modified to create a MODFLOW subregional flow simulation model for the project area.

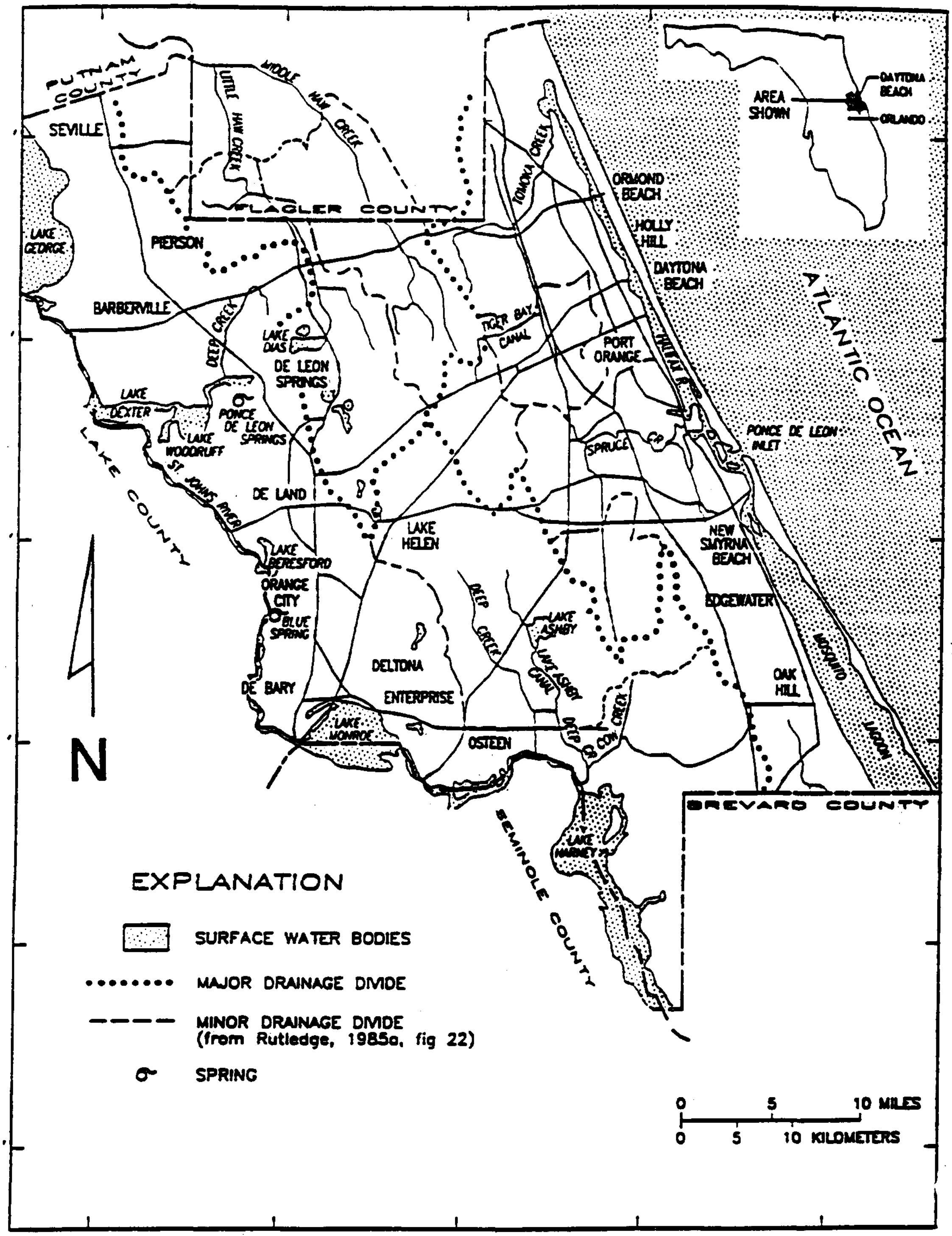


Figure 3.1

Map of Volusia County, Florida.

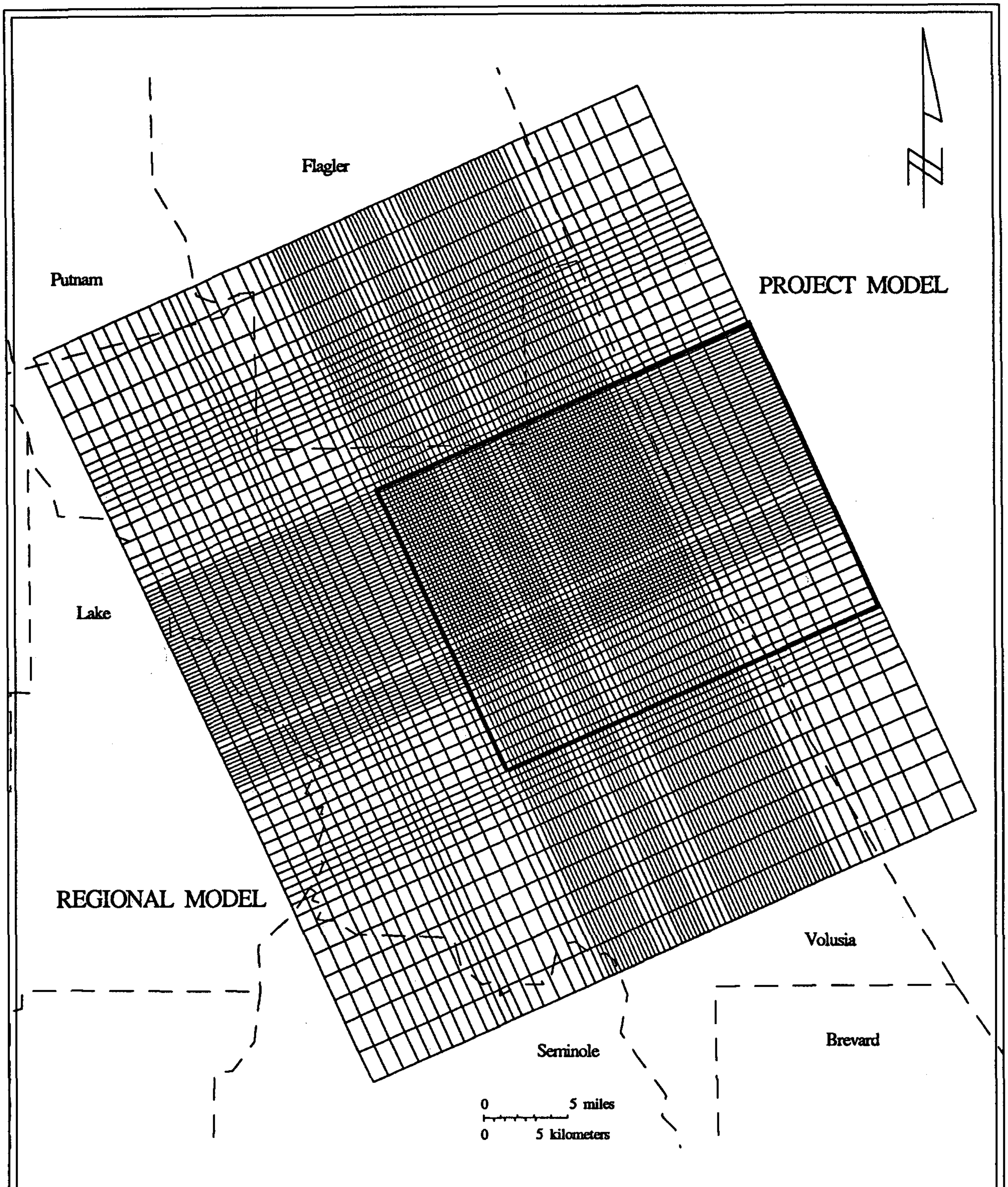


Figure 3.2
Volusia County regional and project numerical model grid discretizations.

The process of developing the project flow simulation model was initiated by essentially cropping the northern, western, and southern portions of the regional model. (The eastern border coincides with the larger regional model). This process involved modifying all the regional model input files so they would correspond with the project geometry. Computer programs written in FORTRAN code were developed to facilitate this process. Removed from the regional model were 19 columns at the western border and 20 and 11 rows from the northern and southern borders respectively. The location of the project model with respect to the regional model is depicted in Figure 3.2. Figure 3.3 provides a reference for flow model cell locations.

The next step in the modification process involved applying the appropriate boundary conditions to the project model in order to simulate the regional model as closely as possible. Boundary conditions were applied using the general head boundary option on the three boundaries of the model which do not coincide with regional boundaries. This was performed by applying a constant external head value and a value which represents the conductance between the external location and the boundary.

Effective conductance values between the regional and project borders were calculated for each layer. Initially, the conductance of each cell located outside the model was calculated in the direction perpendicular to the border. The effective conductances were then calculated between the regional and project borders using the individual cell conductances, or

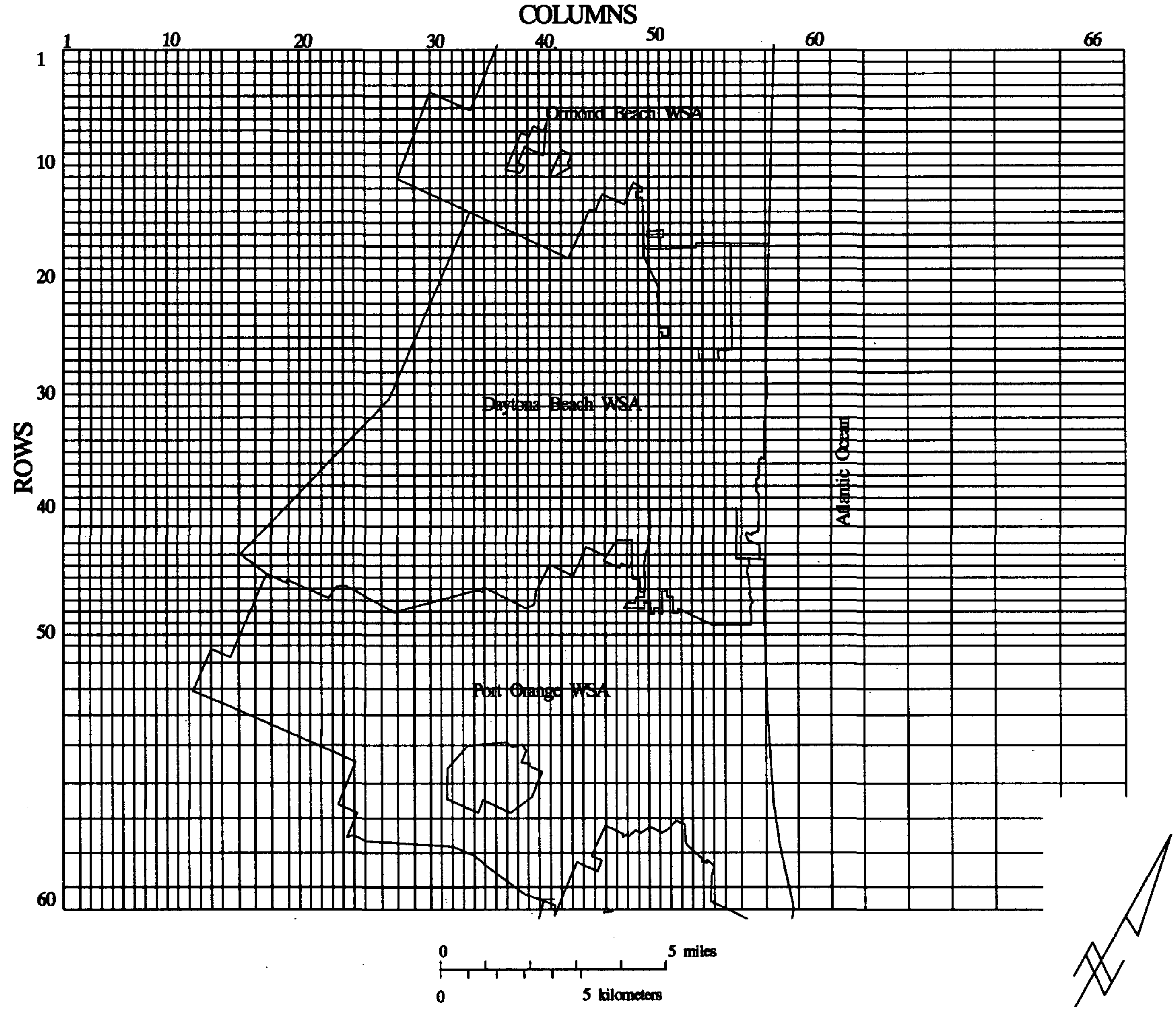


Figure 3.3
Project Flow Model
Cell Location Reference Map

$$\frac{1}{C} = \frac{1}{C_1} + \frac{1}{C_2} + \ldots + \frac{1}{C_n} \tag{3.2}$$

where C is conductance, K is hydraulic conductivity, A is cross sectional area perpendicular to flow, L is length of flow path, and n is the number of cells.

FORTRAN programs were developed to facilitate the process of gathering and manipulating the geometric and hydraulic data for these calculations. Results from the execution of the regional model were also used to determine the conductance and external head values for the general head boundary conditions. These boundary conditions varied depending on the simulated condition of the regional model (predevelopment, year 1988, or year 2010). For layers 2 through 5 of the regional model, general head boundary conditions were applied to the northern and western borders. Therefore, external head values used for the regional model were applied to the project model, and conductance values applied to the project model were determined by combining the conductance values used for the regional model with the conductance between the borders of the two models. To determine the external head values to be applied at the southern border of the project model, the regional model was executed for specific conditions to determine head values along its southern border. This process assures that the influences of the regional models on the smaller project model is incorporated in the simulation.

For the surficial aquifer (layer 1), there are often many bodies of water between the regional and project borders which are modeled as constant heads (especially on the western border). In these cases, the constant head values which were closest and external to project border were used as the external head values for the project model.

The effective conductances were calculated in these cases between the project border and the location of the external head. In cases where constant heads did not exist between the regional and project borders, effective conductances were calculated between the two borders as described previously. External head values for the project model were determined by using the corresponding head values along the regional border.

3.3 SOLUTE TRANSPORT SIMULATION MODEL

In order to incorporate water quality constraints into several of the optimization models a solute transport simulation model was needed. The DSTRAM program was used for the project area was chosen for this application because SJRWMD had already developed a model for another project which covers area common to the subregion modeled with MODFLOW. The DSTRAM model incorporates an uniform grid of 30 columns by 51 row over the project area. This model uses 15 layers to simulate the Upper Floridan aquifer, Lower Floridan aquifer, and the confining units in the system. The majority of the information needed to create the DSTRAM project model was taken from the regional MODFLOW model (Williams, 1994). This information includes boundary conditions, the modeled region size and shape, hydrogeologic framework, hydraulic characteristics, and known fluxes affecting the aquifer system. DSTRAM simulation runs for the project were made by SJRWMD as needed during the optimization model development.

CHAPTER 4
4.0 WATER SUPPLY NEEDS AND SOURCES

Water supply needs and sources are currently being evaluated by SJRWMD under the District Water Management Plan. The needs and sources of Volusia County have been recorded and projected by SJRWMD to the year 2010 for several use categories. Projections of future water use are based on historical trends, local government comprehensive plans, and direct communication with both public and private public supply utilities (SJRWMD, 1992). Computer data files depicting municipal, agricultural, and miscellaneous well information and water use needs have been obtained from the District. Maps and tables have also been obtained which depict wastewater treatment plant information. This data was used to incorporate water reuse in the development of the ground water management model.

Municipal well data include the municipality and water treatment plant name, the water treatment plant permitted capacity, existing and proposed wells which service these areas, location of these wells in state-planar and numerical-grid coordinates, pumping rates for years 1988 and 1990, and the predicted pumping rate for year 2010. See Table 4.1. Within the project area, there are 97 existing and 74 proposed wells in 17 separate well zones which supply water to 8 different municipalities or water service areas. Nine different water treatment plants supply these service areas. These municipalities or water service areas include Holly Hill, Port Orange, Spruce Creek, Daytona Beach, Ormond Beach, Tymber Creek Utilities, The Trails, Inc., and a section of New Smyrna Beach,

Table 4.1 - Volusia County municipal well data for Project area.

Municipal Water Service Area	State Planar Coord. Location X	Y	Project Flow Model Cell Location row	col.	Well Grid Cell Name	1988 Pumpage (cu ft/mo)	1990 Pumpage (cu ft/mo)	2010 Projected Pumpage (cu ft/mo)	Water Treatment Plant Name	Permitted Capacity (mgd)
*** PORT ORANGE ***									Port Orange	6.21
WESTERN WELLFIELD	457956	1736744	45	22	MWELL1	1079040	1184841	961230		
	457862	1732805	48	21	MWELL9	1079040	1184841	961230		
	457594	1730684	49	20	MWELL5	1079040	1184841	961230		
	457505	1730381	49	20	MWELL5	1079040	1184841	961230		
	457592	1729068	50	19	MWELL4	1079040	1184841	961230		
	457946	1728664	51	19	MWELL10	1079040	1184841	961230		
	456001	1734120	46	20	MWELL3	1079040	1184841	961230		
	455911	1733009	47	19	MWELL6	1079040	1184841	961230		
	455999	1731999	48	19	MWELL7	1079040	1184841	961230		
	456175	1730888	49	19	MWELL11	1079040	1184841	961230		
	453961	1734022	46	18	MWELL12	1079040	1184841	961230		
	454491	1732708	47	18	MWELL8	1079040	1184841	961230		
	454046	1731597	47	18	MWELL8	1079040	1184841	961230		
	457948	1729674	50	20	MWELL13	1079040	1184841	961230		
	454222	1730486	48	18	MWELL2	1079040	1184841	961230		
POW - Proposed	453874	1735638	45	19	MWELL14	0	0	961230		
	453874	1735638	45	19	MWELL14	0	0	961230		
	453874	1735638	45	19	MWELL14	0	0	961230		
	456084	1729070	50	18	MWELL15	0	0	961230		
	456084	1729070	50	18	MWELL15	0	0	961230		
	456084	1729070	50	18	MWELL15	0	0	961230		
POW - Proposed	422871	1756899	21	6	MWELL16	0	0	0		
	420211	1756905	20	4	MWELL17	0	0	0		
	425531	1756894	22	8	MWELL18	0	0	0		
	422877	1759929	19	7	MWELL19	0	0	0		
	420218	1759935	18	5	MWELL20	0	0	0		
	425537	1759924	20	9	MWELL21	0	0	0		
	422865	1753869	23	5	MWELL22	0	0	0		
	420205	1753874	22	3	MWELL23	0	0	0		
	425525	1753864	24	7	MWELL24	0	0	0		
POW - Proposed	457946	1729674	51	20	MWELL25	0	0	961230		
	457946	1732805	51	21	MWELL26	0	0	961230		
	458355	1729068	52	19	MWELL27	0	0	961230		
	458355	1729674	52	20	MWELL28	0	0	961230		
	458355	1732805	52	21	MWELL29	0	0	961230		
	458672	1729068	53	19	MWELL30	0	0	961230		
	458672	1729674	53	20	MWELL31	0	0	961230		
	458672	1732805	53	21	MWELL32	0	0	961230		
EASTERN WELLFIELD	491132	1745711	49	46	MWELL34	124503	136712	221822		
	491132	1746216	49	46	MWELL34	124503	136712	221822		
	491132	1746721	49	46	MWELL34	124503	136712	221822		
	491132	1746014	49	46	MWELL34	124503	136712	221822		
	490600	1745610	49	45	MWELL35	124503	136712	221822		
	491133	1748236	47	46	MWELL38	124503	136712	221822		
	491133	1748741	47	47	MWELL36	124503	136712	221822		
	491133	1749246	47	47	MWELL36	124503	136712	221822		
	490690	1749448	47	47	MWELL36	124503	136712	221822		
	490335	1749448	46	46	MWELL33	124503	136712	221822		
	489891	1749448	46	46	MWELL33	124503	136712	221822		
	489093	1749448	46	45	MWELL37	124503	136712	221822		
	488916	1749448	46	45	MWELL37	124503	136712	221822		

Table 4.1 - continued

Municipal Water Service Area	State Planar Coord. Location X	Y	Project Flow Model Cell Location row	col.	Well Grid Cell Name	1988 Pumpage (cu ft/mo)	1990 Pumpage (cu ft/mo)	2010 Projected Pumpage (cu ft/mo)	Water Treatment Plant Name	Permitted Capacity (mgd)
*** DAYTONA BCH ***									Daytona Beach	35.17
EASTERN WELLFIELD	478108	1765916	32	43	none	0	0		Marion Street	
(7 wells inactive)	477221	1765513	32	42	none	0	0			
	476512	1765109	32	42	none	0	0			
	475182	1765110	32	41	none	0	0			
	474473	1764606	32	40	none	0	0			
	473852	1764303	32	40	none	0	0			
	472522	1763799	32	38	MWELL39	0	0			
	471990	1763496	32	38	MWELL39	2388368	2508913	1246680		
	471192	1763093	32	37	MWELL42	2388368	2508913	1246680		
	469862	1762589	32	36	MWELL40	2388368	2508913	1246680		
	464897	1761482	31	32	MWELL41	2388368	2508913	1246680		
	464188	1760978	31	32	MWELL41	2388368	2508913	1246680		
WESTERN WELLFIELD	462769	1760272	31	31	MWELL46	3796082	2092352	3110003	Ralph Brennan	
	461794	1761081	30	30	MWELL47	3796082	2092352	3110003		
	458589	1747752	39	26	MWELL44	3796082	2092352	3110003		
	458145	1747551	39	25	MWELL52	3796082	2092352	3110003		
	457169	1747148	39	25	MWELL52	3796082	2092352	3110003		
	456371	1746846	39	24	MWELL43	3796082	2092352	3110003		
	455306	1746342	39	23	MWELL45	3796082	2092352	3110003		
	454507	1746040	39	22	MWELL48	3796082	2092352	3110003		
	454242	1746848	38	23	MWELL49	3796082	2092352	3110003		
	453800	1747960	37	23	MWELL50	3796082	2092352	3110003		
	453269	1748769	36	22	MWELL51	3796082	2092352	3110003		
WESTERN WELLFIELD	451854	1751800	34	22	MWELL53	0	2092352	3110003	South Daytona	
(1988 construction)	451146	1752407	33	22	MWELL54	0	2092352	3110003		
	450969	1753115	33	22	MWELL54	0	2092352	3110003		
	451148	1753922	32	23	MWELL55	0	2092352	3110003		
	450972	1754933	31	23	MWELL56	0	2092352	3110003		
	451062	1756044	31	23	MWELL56	0	2092352	3110003		
	449910	1756449	30	23	MWELL57	0	2092352	3110003		
	453265	1745638	38	21	MWELL58	0	2092352	3110003		
	452917	1750587	35	23	MWELL59	0	2092352	3110003		
	452297	1751093	34	23	MWELL60	0	2092352	3110003		
DBW - Proposed	451242	1757861	29	24	MWELL61	0	0	3110003		
	450977	1758872	29	24	MWELL61	0	0	3110003		
	450712	1759882	28	24	MWELL62	0	0	3110003		
	450448	1760893	27	24	MWELL63	0	0	3110003		
	450183	1761903	26	24	MWELL64	0	0	3110003		
*** SPRUCE CREEK ***	483674	1724907	56	34	MWELL66	658423	493538	326649	Spruce Creek	7.73
	484295	1725513	56	35	MWELL65	658423	493538	326649		
SC - Proposed	484118	1725311	56	34	MWELL66	0	0	326649		
	483496	1724705	56	34	MWELL66	0	0	326649		
	484739	1725815	56	35	MWELL65	0	0	326649		
*** HOLLY HILL ***									Holly Hill	1.43
EASTERN WELLFIELD	484940	1784396	22	53	MWELL68	69192	66195	79694		
	485117	1784093	22	54	MWELL67	69192	66195	79694		
	485294	1783689	22	53	MWELL68	69192	66195	79694		
	485472	1784800	21	54	MWELL69	69192	66195	79694		
	484409	1784700	21	53	MWELL70	69192	66195	79694		
	482991	1784094	21	52	MWELL71	69192	66195	79694		
WESTERN WELLFIELD	461629	1773101	22	34	MWELL72	593074	567314	597705		
	461896	1773606	22	34	MWELL72	593074	567314	597705		
	462251	1774009	22	35	MWELL73	593074	567314	597705		
	462694	1774211	22	35	MWELL73	593074	567314	597705		
	461632	1775424	21	35	MWELL74	593074	567314	597705		
	462163	1774918	21	35	MWELL74	593074	567314	597705		
	463402	1773705	22	35	MWELL73	593074	567314	597705		
HHW - Proposed	462783	1775019	21	35	MWELL74	0	0	597705		

Table 4.1 - continued

Municipal Water Service Area	State Planar Coord. Location X	Y	Project Flow Model Cell Location row	col.	Well Grid Cell Name	1988 Pumpage (cu ft/mo)	1990 Pumpage (cu ft/mo)	2010 Projected Pumpage (cu ft/mo)	Water Treatment Plant Name	Permitted Capacity (mgd)
*** NEW SMYRNA BCH ***									New Smyrna	3.60
SAMSULA WELLFIELD	471322	1701078	60	20	MWELL75	1207921	1290193	1521308	Beach	
	470701	1701483	60	20	MWELL75	1207921	1290193	1521308		
SR44 WF - Proposed	461913	1702702	59	16	MWELL76	0	0	1521308		
	461380	1702804	59	15	MWELL77	0	0	1521308		
	462358	1704318	58	16	MWELL78	0	0	1521308		
	461470	1704117	58	16	MWELL78	0	0	1521308		
	461915	1704722	58	16	MWELL78	0	0	1521308		
	461560	1705127	58	16	MWELL78	0	0	1521308		
*** ORMOND BCH ***									Ormond Beach	8.00
DIVISION AVE WELLFIELD	479276	1795409	12	53	MWELL80	917447	645276	659368		
	479365	1795308	12	53	MWELL80	917447	645276	659368		
	478833	1793692	13	52	MWELL81	917447	645276	659368		
	478479	1794197	13	52	MWELL81	917447	645276	659368		
	478302	1794399	13	52	MWELL81	917447	645276	659368		
	478124	1794096	13	52	MWELL81	917447	645276	659368		
	476176	1793794	12	50	MWELL79	917447	645276	659368		
	475644	1793290	13	50	MWELL83	917447	645276	659368		
	475024	1793088	13	49	MWELL84	917447	645276	659368		
	478480	1796116	12	53	MWELL80	917447	645276	659368		
	478125	1795914	12	52	MWELL82	917447	645276	659368		
	475822	1794704	12	50	MWELL79	917447	645276	659368		
SR 40 WELLFIELD	467319	1793397	10	44	MWELL86	2582442	645276	659368		
	464038	1789966	11	41	MWELL87	2582442	645276	659368		
	467940	1794912	9	45	MWELL88	0	645276	659368		
	468294	1794305	10	45	MWELL89	0	645276	659368		
	462089	1789564	11	39	MWELL85	2582442	645276	659368		
HUDSON WELLFIELD	450488	1791092	6	32	MWELL90	0	645276	659368		
	450489	1792002	6	32	MWELL90	0	645276	659368		
	450490	1792911	5	32	MWELL91	0	645276	659368		
	450491	1793719	4	33	MWELL92	0	645276	659368		
	450492	1794628	4	33	MWELL92	0	645276	659368		
	450494	1795537	3	33	MWELL93	0	645276	659368		
	450495	1796446	3	33	MWELL93	0	645276	659368		
	451553	1792909	5	33	MWELL94	0	645276	659368		
	451997	1793818	5	34	MWELL95	0	645276	659368		
	451998	1794727	4	34	MWELL96	0	645276	659368		
	452001	1797151	3	35	MWELL97	0	645276	659368		
	449516	1792912	5	32	MWELL91	0	645276	659368		
	448719	1792913	4	31	MWELL98	0	645276	659368		
OB - Proposed CENTRAL RECHARGE WELLFIELD	419697	1764986	15	6	MWELL99	0	0	659368		
	420758	1764075	16	7	MWELL100	0	0	659368		
	421643	1763265	16	7	MWELL100	0	0	659368		
	421995	1761951	17	7	MWELL101	0	0	659368		
	422080	1760436	18	7	MWELL102	0	0	659368		
	422166	1758820	20	6	MWELL103	0	0	659368		
	422163	1757708	20	6	MWELL103	0	0	659368		
	422160	1756193	21	5	MWELL104	0	0	659368		
	422159	1755385	22	5	MWELL105	0	0	659368		
	423769	1762452	18	8	MWELL106	0	0	659368		
	424833	1762652	18	9	MWELL107	0	0	659368		
	425719	1762549	18	10	MWELL108	0	0	659368		
	426872	1762749	18	11	MWELL109	0	0	659368		
	425544	1763762	17	10	MWELL110	0	0	659368		
	425280	1764671	17	10	MWELL110	0	0	659368		
	420039	1759430	19	5	MWELL111	0	0	659368		
	419152	1759129	18	4	MWELL112	0	0	659368		
	418000	1759333	18	3	MWELL113	0	0	659368		
	417820	1757920	19	3	MWELL114	0	0	659368		
	417639	1756405	20	2	MWELL115	0	0	659368		
	417548	1755395	21	2	MWELL116	0	0	659368		
OB - Proposed RIMA RIDGE	441343	1777571	13	23	MWELL117	0	0	659368		
	441168	1778784	12	24	MWELL118	0	0	659368		
*** TYMBER CREEK UTIL ***	459436	1792900	8	39	MWELL119	372103	409982	728944	Tymber Creek	160000
*** THE TRAILS INC. ***	453145	1790887	7	34	MWELL120	1299484	1299484	2355224	????	????
					Total	100230505	105037423	174947036		

(Figure 4.1).

Agricultural water service data include both agricultural and golf course well data. This data depicts well locations in state-planar and numerical-grid coordinates, water application types, and pumping rates for year 1990. See Table 4.2. These rates have been predicted to remain approximately constant through the year 2010 (Geraghty & Miller 1992). According to data obtained from SJRWMD, there are 80 agricultural and golf course wells throughout the Volusia County subregion. The ground water management model was formulated to supply a portion of the ground water needs with effluent from wastewater treatment plants. The agricultural areas and wastewater treatment plants are shown in Figure 4.2.

The miscellaneous well data includes the well description, the well location in state-planar and numerical-grid coordinates, and the 1988 and 1990 pumping rates. The Volusia County subregion consists of 9 miscellaneous wells which supply the Florida Mining and Materials Department, the Florida Department of Education, and the Tomoka Correctional Facility. Additional data depicting location and discharge rates of private wells within the region for years 1988 and 2010 are also included. Data were supplied by SJRWMD in a form to facilitate execution of the simulation model and subsequent incorporation into the optimization model. However, these non-municipal wells were not optimized during this study.

In order for the simulation/optimization model to predict a feasible water management strategy, water service demands must be incorporated into the model. Municipal and agricultural area water supply demands for the year 2010 were calculated by taking the sum of all projected well discharge rates which supplied a specific water

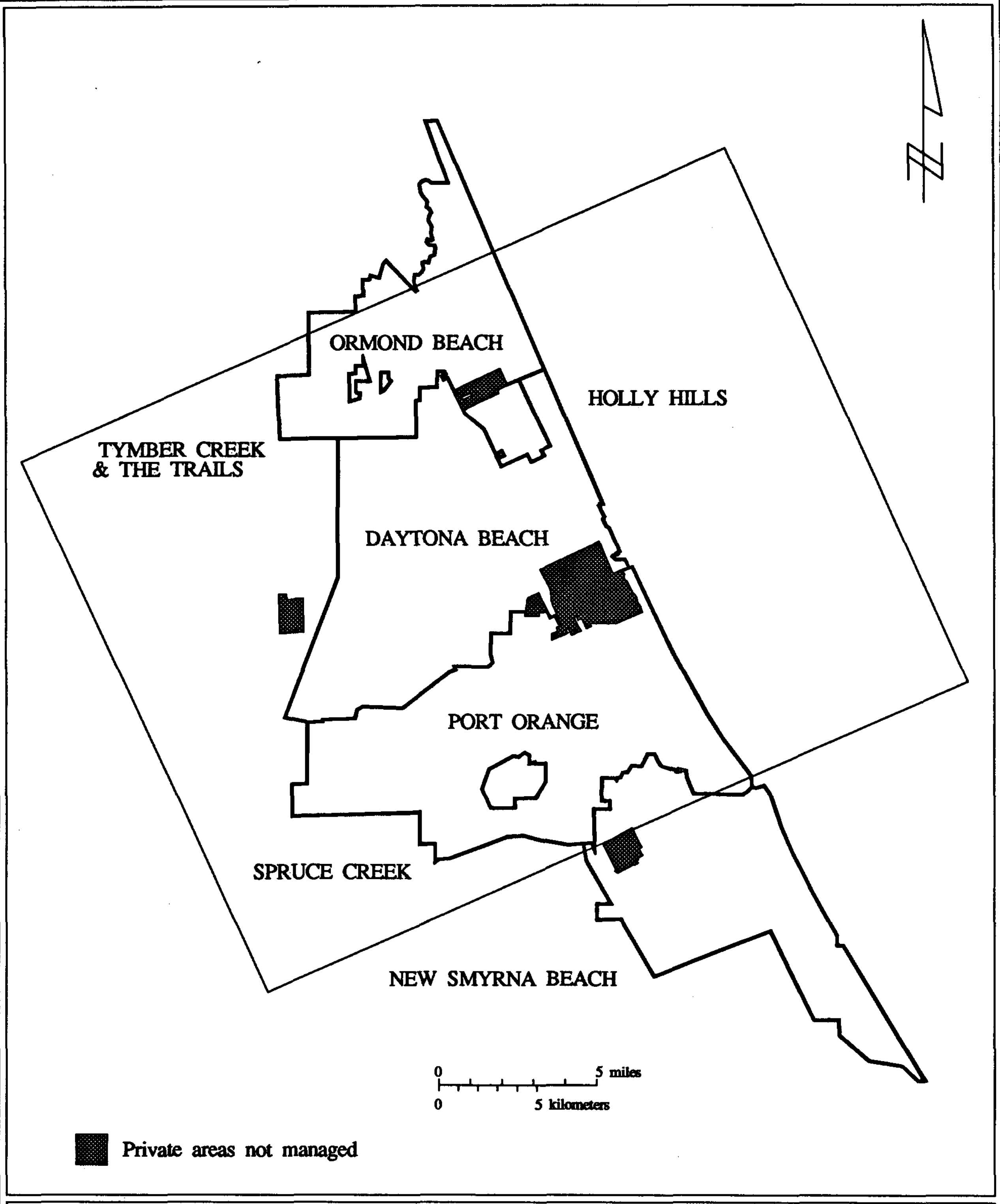


Figure 4.1
Location of Municipal Water Service Areas within Project Area.

Table 4.2 - Volusia County agricultural well data project area.

Agricultural Service Area	Application Number	State Planar Coord. Location X	State Planar Coord. Location Y	Project Flow Model Cell Location row	Project Flow Model Cell Location col	Well Grid Cell Name	1990 Estimated Pumpage (cu ft/mo)	2010 Projected Pumpage (cu ft/mo)
AGAREA1	2-127-0396AN	495388	1736822	54	46	AGWELL2	55355	55355
	2-127-0396AN	496541	1736519	54	46	AGWELL2	55355	55355
	2-127-0396AN	495388	1737024	54	46	AGWELL2	55355	55355
	2-127-0396AN	496452	1737327	54	47	AGWELL1	55355	55355
	2-127-0396AN	496452	1736923	54	47	AGWELL1	55355	55355
AGAREA2	2-127-0269AN	479539	1789550	16	51	AGWELL4	191139	191139
	2-127-0269AN	481045	1789953	17	53	AGWELL5	191139	191139
	2-127-0269AN	482639	1790256	17	54	AGWELL3	191139	191139
AGAREA3	2-127-0279AU	505496	1758941	45	58	AGWELL6	14255	14255
	2-127-0279AU	505674	1757931	45	58	AGWELL6	14255	14255
	2-127-0279AU	506117	1758032	45	58	AGWELL6	14255	14255
	2-127-0279AU	505674	1758638	45	58	AGWELL6	14255	14255
	2-127-0279AU	505496	1758941	45	58	AGWELL6	14255	14255
	2-127-0279AU	505851	1758436	45	58	AGWELL6	14255	14255
	2-127-0279AU	505762	1758739	45	58	AGWELL6	14255	14255
	2-127-0279AU	505940	1758234	45	58	AGWELL6	14255	14255
	2-127-0279AU	505851	1758436	45	58	AGWELL6	14255	14255
	2-127-0279AU	505585	1758840	45	58	AGWELL6	14255	14255
	2-127-0279AU	505674	1758739	45	58	AGWELL6	14255	14255
	2-127-0279AU	505585	1757931	45	58	AGWELL6	14255	14255
	2-127-0279AU	506738	1756921	46	58	AGWELL7	14255	14255
	2-127-0279AU	506206	1757830	46	58	AGWELL7	14255	14255
	2-127-0279AU	506294	1757729	46	58	AGWELL7	14255	14255
	2-127-0279AU	506383	1757527	46	58	AGWELL7	14255	14255
	2-127-0279AU	506206	1756719	46	58	AGWELL7	14255	14255
	2-127-0279AU	506472	1757325	46	58	AGWELL7	14255	14255
	2-127-0279AU	506826	1757022	46	58	AGWELL7	14255	14255
	2-127-0279AU	505940	1756820	46	58	AGWELL7	14255	14255
	2-127-0279AU	506560	1757123	46	58	AGWELL7	14255	14255
	2-127-0279AU	506383	1757628	46	58	AGWELL7	14255	14255
	2-127-0279AU	506206	1757224	46	58	AGWELL7	14255	14255
	2-127-0279AU	506383	1757325	46	58	AGWELL7	14255	14255
	2-127-0279AU	506117	1756618	46	58	AGWELL7	14255	14255
AGAREA4	2-127-0565ANV	484418	1806921	6	58	AGWELL12	92010	92010
	2-127-0565ANV	484506	1807022	6	59	AGWELL11	92010	92010
	2-127-0565ANV	485480	1805708	7	59	AGWELL10	92010	92010
	2-127-0565ANV	485745	1804799	8	59	AGWELL8	92010	92010
	2-127-0565ANV	485833	1804395	8	59	AGWELL8	92010	92010
	2-127-0565ANV	485834	1804698	8	59	AGWELL8	92010	92010
	2-127-0565ANV	485833	1803183	9	58	AGWELL9	92010	92010
AGAREA5	2-127-0647AUS	469188	1804304	3	49	AGWELL16	193220	193220
	2-127-0647AUS	468566	1802688	4	48	AGWELL14	193220	193220
	2-127-0647AUS	470692	1803596	4	50	AGWELL13	193220	193220
	2-127-0647AUS	470071	1802182	5	49	AGWELL15	193220	193220

Table 4.2 - continued

Agricultural Service Area	Application Number	State Planar Coord. Location X	Y	Project Flow Model Cell Location row	col	Well Grid Cell Name	1990 Estimated Pumpage (cu ft/mo)	2010 Projected Pumpage (cu ft/mo)
AGAREA6	2-127-0147AU	483584	1722786	56	33	AGWELL17	302712	302712
	2-127-0147AU	485536	1723189	56	35	AGWELL18	302712	302712
	2-127-0147AU	485536	1723189	56	35	AGWELL18	302712	302712
	2-127-0147AU	484118	1726321	56	35	AGWELL18	302712	302712
AGAREA7	2-127-0236AN	465995	1701284	59	17	AGWELL19	857472	857472
	2-127-0236AN	468304	1701787	59	18	AGWELL20	857472	857472
	2-127-0236AN	466614	1698456	60	17	AGWELL21	857472	857472
AGAREA8	2-127-0237AN	487399	1719754	57	35	AGWELL22	141258	141258
	2-127-0237AN	488907	1717128	58	35	AGWELL24	141258	141258
	2-127-0237AN	488907	1717128	58	35	AGWELL24	141258	141258
	2-127-0237AN	488907	1717128	58	35	AGWELL24	141258	141258
	2-127-0237AN	488907	1717128	58	35	AGWELL24	141258	141258
	2-127-0237AN	488907	1717128	58	35	AGWELL24	141258	141258
	2-127-0237AN	488907	1717128	58	35	AGWELL24	141258	141258
	2-127-0237AN	490416	1719652	58	37	AGWELL23	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
	2-127-0237AN	490681	1715511	59	36	AGWELL25	141258	141258
AGAREA9	2-127-0085AN	483427	1768641	32	47	AGWELL26	24536	24536
	2-127-0085AN	483782	1768843	32	48	AGWELL28	24536	24536
	2-127-0085AN	483516	1768742	32	48	AGWELL28	24536	24536
	2-127-0085AN	484047	1768943	32	48	AGWELL28	24536	24536
	2-127-0085AN	483427	1768944	32	48	AGWELL28	24536	24536
	2-127-0085AN	483604	1769247	32	48	AGWELL28	24536	24536
	2-127-0085AN	484668	1770155	31	49	AGWELL27	24536	24536
						Total	9686206	9686206

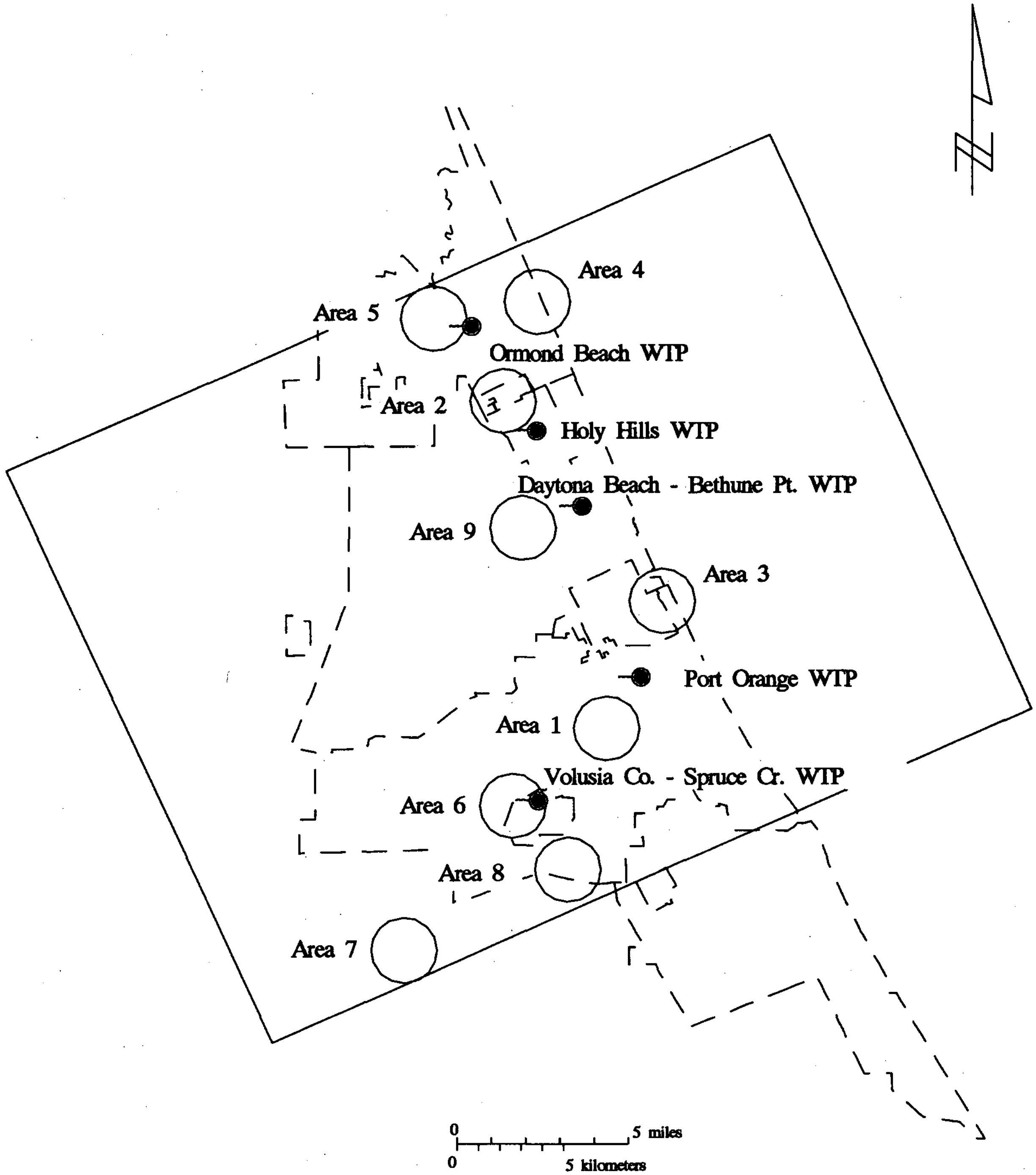


Figure 4.2
Agricultural Areas and Wastewater Treatment Plants
with respect to Numerical Model Grid Location

service area. Municipal and agricultural service area demands are summarized in Tables 4.3 and 4.4, respectively, for years 1988 and 2010.

Municipal water service areas were generally defined by individual well fields and/or water treatment plants which supply ground water to the municipalities. Proposed well fields which were not in service before 1991 were not considered as individual water service areas. They were incorporated in the optimization model as possible well sites and their year 2010 withdrawal rates contributed to projected demand. However, these locations were not specified as demand areas. The Central Recharge and Rima Ridge well fields are two such proposed areas which will supply the Ormond Beach municipality by the year 2010. Therefore, the year 2010 projected demand of these areas were equally divided between the State Road 40 and Hudson well fields. These well fields were selected because of the proximity to the proposed Central and Rima well fields.

Defining water service areas as individual well fields is only one method of designating demand areas. The definition of a water service area within the optimization model can easily be revised depending on the objectives of the water resource manager. This includes defining each municipality, or the entire Volusia County region for that matter, as a single water service area.

Effluent from wastewater treatment plants can often be used to replace ground water currently being used to irrigate agricultural areas and golf courses. The wastewater treatment plant data includes tables depicting facility names and locations, current permitted capacities and mean flow rates, and predicted permitted capacities and mean flow rates for the year 2010. There are 9 wastewater treatment facilities in the

Table 4.3 - Year 1988 and projected year 2010 demand rates for municipal water service areas.

Well ID	Water Service Area Name	Year 1988 Demand Rate (cfd)	Projected Year 2010 Demand Rate (cfd)
POW	Port Orange - West Wellfield	532127	916458
POE	Port Orange - East Wellfield	53211	94807
DBM	Daytona Beach East - Marion TP	392609	204934
DBB	Daytona Bch West - Brennan TP	1372832	1124716
DBS	Daytona Beach West - South TP	0	1533702
SCK	Spruce Creek	43294	53695
HHE	Holly Hill - East Wellfield	13650	15720
HHW	Holly Hill - West Wellfield	136489	157205
NSB	Smyrna Beach / Samsula	79425	400124
OBD	Ormond Beach - Division Ave WF	361952	260135
OB4	Ormond Beach - State Rd. 40 WF	254706	346848
OBH	Ormond Beach - Hudson Wellfield	0	541950
TCU	Tymber Creek Utilities	12234	24298
TTI	The Trails, Inc.	42723	78507
	Total	3295252	5753099

Table 4.4 - Year 1988 and projected year 2010 demand rates for agricultural water service areas.

Well ID	Agricultural Application Number	Year 1988 Demand Rate (cfd)	Projected Year 2010 Demand Rate (cfd)
AGAREA1	AG-0396AN	9100	9100
AGAREA2	AG-0269AN	18852	18852
AGAREA3	AG-0279AU	11717	11717
AGAREA4	AG-0565ANV	21175	21175
AGAREA5	AG-0647AUS	25408	25408
AGAREA6	AG-0147AU	39808	39808
AGAREA7	AG-0236AN	84573	84573
AGAREA8	AG-0237AN	102170	102170
AGAREA9	AG-0085AN	5645	5645
	Total	318448	318448

Volusia County subregion with permitted capacities of 0.1 mgd or greater. The wastewater and agricultural data was reviewed and compared to determine which agricultural areas could supplement their ground water demand with wastewater plant effluent. From this review and based on proximity to agricultural areas, it was determined that 5 of the 9 treatment plants could feasibly supplement the agricultural demands. Constraints for the ground water optimization model were then formulated based on the results of the review and comparison. See Table 4.5.

The needs and sources evaluation of Volusia County by SJRWMD includes a water resources impact assessment. This assessment involves identifying potential problem areas when the year 2010 projected allocation strategy is implemented. One such problem area involves the possibility of hydrophytes being harmed as a result of declines in water table. Based on the evaluation, several areas of Volusia County have a high potential for vegetative harm if proposed strategies were incorporated.

In order to reduce the possibility of harm to wetland vegetation, areas depicted as having high potential for harm were incorporated into the optimization model as control points. These control points are locations where pressure head and/or drawdown values were constrained or optimized. To reduce the number of control points and therefore the size of the optimization model, 100 out of approximately 500 points were selected throughout the high potential harm areas. Figure 4.3 displays the control point locations along with areas of high, medium, and low potential harm.

Table 4.5 - Volusia County subregional model wastewater treatment plant data.

Supply ID	Plant Name	Coordinate Location		Numerical Grid Cell Location		Permit Capacity		Wastewater Reuse Service Area
		latitude	longitude	Row	Col.	(mgd)	(cfd)	
WASTE1	Port Orange WWTP	290812	805949	51	53	12.00	1604400	AGAREA1 and/or AGAREA3
WASTE2	Holly Hill WWTP	291426	810240	21	54	2.40	320880	AGAREA2
WASTE3	Daytona Beach-Bethune Pt. WWTP	291205	810031	31	55	12.00	1604400	AGAREA3 and/or AGAREA9
WASTE4	Ormond Beach WWTP	291720	810426	6	52	6.00	802200	AGAREA4 and/or AGAREA5
WASTE5	Volusia Co. - Spruce Creek WWTP	290443	810318	55	36	0.35	46795	AGAREA6 and/or AGAREA8

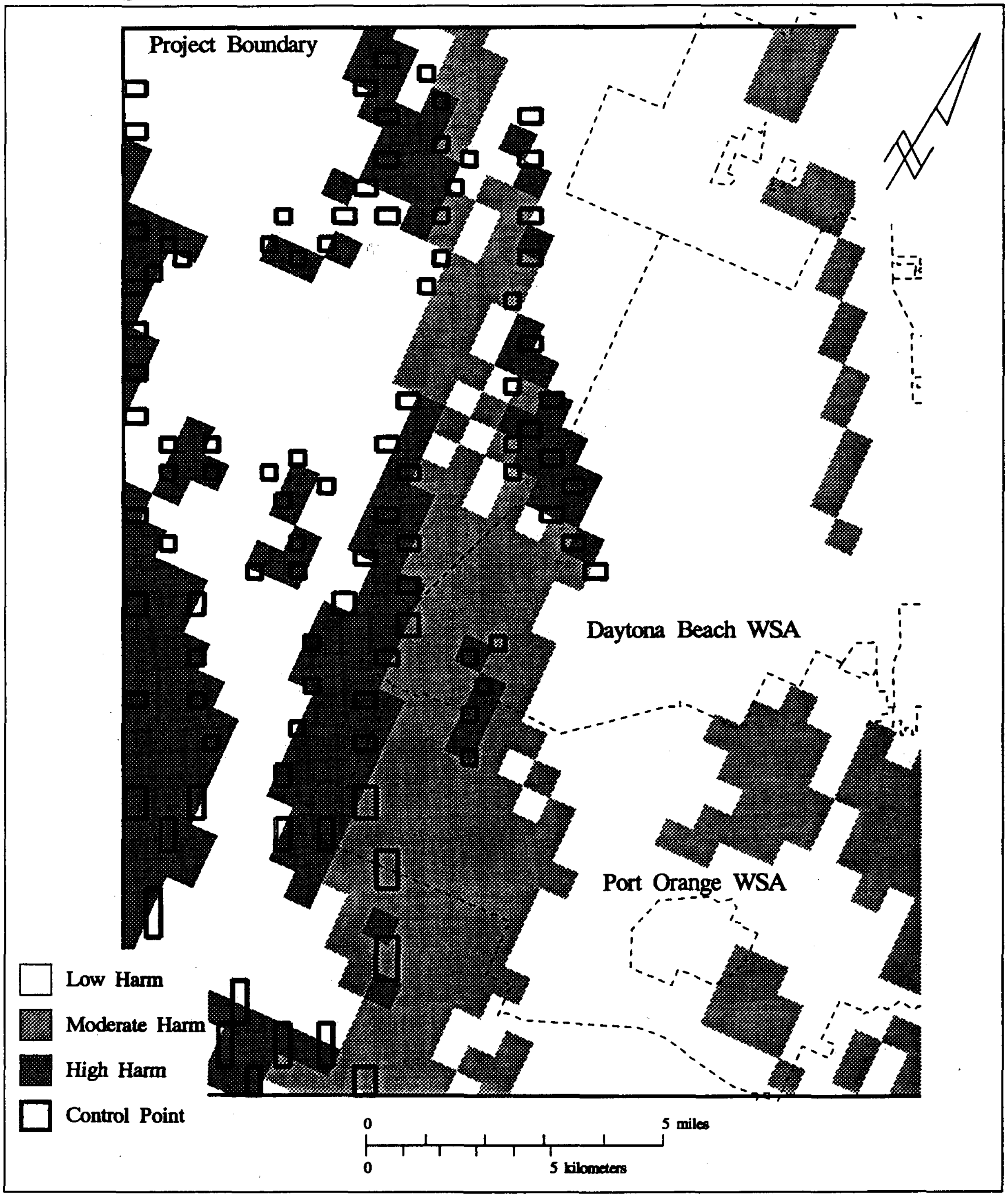


Figure 4.3

Areas of potential impacts to plant communities resulting from projected changes in ground water withdrawals between 1988 and 2010.

CHAPTER 5
5.0 GROUND WATER OPTIMIZATION MODELS

Five aquifer optimization models were developed to investigate optimal allocation of ground water to meet year 2010 demand in the project area. These models were developed to investigate future water allocation strategies assuming that feasible withdrawal scenarios meet or exceed projected water service area demands and do not exceed available water resource supplies. It was assumed with these models that adverse environmental effects could be minimized at specific locations by constraining pressure head changes (i.e. drawdown) to meet specified environmental goals or standards. For example, one hundred control point locations were chosen at which ground water levels changes were constrained. These points were in areas where native vegetation could be harmed by declines in the surficial aquifer due to pumping. In addition to the ground water level constraints two optimization models developed that incorporate a set of constraints to allocate water in a manner that preserves ground water quality.

It is important to note the optimization models were developed using data generated from both numerical flow and transport simulation models (e.g., information describing aquifer responses to changing stresses such as pumping). The modeling grid from the flow simulation model was incorporated in the formulations of all the optimization models (see Chapter 3). Information on needs and sources was also included in the formulation of each the optimization model (see Chapter 4). For example, elemental discharge rates and pressure heads given by each optimization model correspond to elemental cumulative discharges (from wells located in a grid cell) and elemental average pressure heads in associated cells defined in the flow simulation model. In addition, the term "well grid cells" refers to numerical model cells where one or more wells are located, and is used herein to reflect the fact that the optimization model identifies the cumulative well flows in each grid cell (in contrast to individual well flows).

5.1 OPTIMIZATION MODEL DECISION VARIABLES

Each model is comprised of combinations of several groups of decision variables. One group defines steady-state drawdowns and pressure heads at specified control points. These are DD_j, the drawdown at sensitive wetland control point j; DDW_h, the drawdown at each well grid cell h; HD_j, the pressure head at sensitive wetland control point j; and HDW_h, the pressure head at each well grid cell h.

Another group defines cumulative and elemental flows from well grid cells used to meet service area demands. These are $QM_{i,k}$, the discharge rate of each municipal well grid cell i which supplies each municipal water service area k; $QA_{n,o}$, the discharge rate of each agricultural well grid cell n which supplies each agricultural water service area o; $QW_{m,o}$, the effluent reuse rate each wastewater treatment plant m which supplements each agricultural water service area o; QMT_i, the total discharge rate at each municipal well grid cell i; QAT_n, the total discharge rate of each agricultural well grid cell n; and QWT_m, the total effluent reuse rate of each wastewater treatment plant m.

Additional decision variables used in one optimization model that incorporate water quality constraints define steady-state chloride concentrations and changes in concentrations at well points. These are CI_h, the increase in chloride concentration at each well grid cell control point h and CC_h, the chloride concentration at each well grid cell control point h.

5.2 OPTIMIZATION MODEL OBJECTIVE FUNCTIONS

Five optimization model formulations were identified under the assumption that it was desirable to determine feasible ground water allocation in which minimum aquifer system responses are maximized or maximum aquifer system responses are minimized. For example, when the objective is to minimize the maximum drawdown then the value of the objective function must be less than or equal to all drawdowns in the management area. However, if the objective is to maximize a minimum pressure head, the objective function must have a value greater than or equal to all the pressure heads in the management area. In the optimization models presented, objective functions will appear as statements specifying that the value of the objective function S is to be maximize or minimize.

The objective function of the first model is to minimize the maximum drawdown at all sensitive wetland control points (i.e. minimize S). For this model, the following constraint was used to define S:

$$S \geq DD_j \quad for\ all\ j \tag{5.1}$$

where DD_j is the drawdown at sensitive wetland control point j.

The objective of the second model is to maximize the minimum pressure head at all sensitive control points (i.e. maximize S). The appertain constraint associated with this objective is:

$$S \leq HD_j \quad for\ all\ j \tag{5.2}$$

where HD_j is the pressure head at sensitive wetland control point j.

The objective of the third model is to minimize the average drawdown of all sensitive wetland control points (i.e. minimize S). This model objective, which is equal to maximizing average pressure heads, requires the following constraint to define S:

$$S \geq \frac{\sum_{j}^{p} DD_j}{p} \quad for\ all\ j \tag{5.3}$$

where p is the number of sensitive wetland control points.

The objective of the fourth model is to minimize the maximum drawdown while constraining concentration levels. The objective function of this model is formulated the same as the first model, equation 5.1. The difference between model one and model four is the addition of water quality constraints to model four. These additional constraints function to elucidate pumping strategies which satisfying specified water quality standards.

Finally, the objective of the fifth model is to minimize the maximum relative chloride concentration increase. This objective function is written as follows:

$$S \leq RCL_h \tag{5.4}$$

where RCL_h is the relative chloride concentration increase at all well grid cells h.

5.3 OPTIMIZATION MODEL CONSTRAINTS

Other than model specific objective functions and their appurtenant constraint equations (i.e., Equations 5.1, 5.2, 5.3, and 5.4) the six optimization model share a large common block of constraint equations. This block of equations includes aquifer response constraints, management constraints, and nonnegativity constraints. The common block and water quality constraints were used in different combinations to formulate the five different optimization models with unique goals to identify optimal allocation strategies. The following sections discuss these constraint in greater detail.

5.3.1 Aquifer Response Constraints

The optimization models were developed to allow pressure head and/or drawdown to be constrained or optimized. Drawdown constraints at the specified control points were developed using influence coefficients that describe pressure head changes at each control point created by ground water pumpage at each well grid cell. The following general drawdown constraint for a control point includes a linear combination of aquifer responses to the municipal, agricultural, and private wells.

$$DD_j = \sum_i \alpha_{i,j} \, (QMT_i - QM0_i) + \sum_n \beta_{n,j} \, (QAT_n - QA0_n) + DDPRIV_j \, (10^{+6}) \quad \textit{for all } i, j, \textit{ and } n \tag{5.5}$$

where DD_j is the drawdown at sensitive wetland control point j, $\alpha_{i,j}$ is the aquifer influence coefficient defining pressure head change at each sensitive wetland area control point j due to a change in discharge rate at each municipal well grid cell i, QMT_i is the total discharge rate in cfd at each municipal well grid cell i, $QM0_i$ is the initial discharge rate in cfd of each municipal well grid cell i, $\beta_{n,j}$ is the aquifer influence coefficient defining pressure head change at each sensitive wetland area control point j due to a change in discharge rate at each agricultural well n, QAT_n is the total discharge rate in cfd of each agricultural well grid cell n, $QA0_n$ is the initial discharge rate in cfd of each agricultural well grid cell n, and $DDPRIV_j$ is the drawdown in feet at each sensitive wetland area control point j due to private wells not incorporated in the optimization process.

QMT_i values were calculated by summing the discharge rates at a specific well grid cell over all municipal service areas for which it supplies. QAT_n values were calculated by summing the discharge rates at a specific well grid cell over all agricultural service areas for which it supplies. $QM0_i$ and $QA0_n$ values were determined using the year 1988 water allocation strategy. $QM0_i$ and $QA0_n$ were used also in calculating the drawdown from years 1988 to 2010 at the control points. $DDPRIV_j$ is the drawdown from year 1988 to 2010 under steady-state conditions. These values were calculated by increasing the discharge of private wells alone in the simulation model and were used in calculating the total drawdown at the control points. To facilitate the linear programming solver, the original influence coefficients were multiplied by 10^{+6} to give resulting coefficients values on the order of one. As a result, calculated DD_j drawdown values were divided by 10^{+6} to produce the decision variable TDD_j, which has units of feet:

$$TDD_j = \frac{DD_j}{10^{+6}} \qquad \textit{for all } j \tag{5.6}$$

where TDD_j is the total drawdown at each sensitive wetland area control point j.

Pressure head at the sensitive wetland control points, HD_j, are determined by a constraint equation subtracting the drawdown at control point j from the initial pressure head:

$$HD_j = HO_j\,(10^{+6}) - DD_j \qquad \textit{for all } j \tag{5.7}$$

where HD_j is the pressure head in millionths of a foot at each sensitive wetland area control point j and HO_j is the initial pressure head in feet at each sensitive wetland area control point j.

Initial pressure heads at sensitive wetland area control points, HO_j, were determined by executing the simulation model at year 1988 conditions. Again, a second decision variable was created to describe pressure head in feet. The total pressure head, THD_j, values were calculated by dividing HD_j by 10^{+6}:

$$THD_j = \frac{HD_j}{10^{+6}} \tag{5.8}$$

where THD_j is the total pressure head in feet at each sensitive wetland area control point j.

Similar to the aquifer influence coefficient matrices use to create constraint equations 5.6 and 5.8 for sensitive wetland control points, influence coefficient matrices were also developed for constraints expressing the aquifer response at each well grid locations due to pumpage within a well grid cell and at every other well grid cell. The following constraint equations define the pressure head at well grid cell h:

$$DDW_h = \sum_i \gamma_{i,h} (QMT_i - QM0_i) + \sum_n \theta_{n,h} (QAT_n - QA0_n) \tag{5.9}$$

$$HDW_h = HW0_h (10^{+6}) - DDW_h \quad for\ all\ h \tag{5.10}$$

where DDW_h is the drawdown in millionths of a foot at each well grid cell h, $\gamma_{i,h}$ is the aquifer influence coefficient defining pressure head change at well grid cell control point h due to a change in discharge rate at each municipal well grid cell i, $\theta_{n,h}$ is the aquifer influence coefficient defining pressure head change at well grid cell control point h due to a change in discharge rate at each agricultural well grid cell n, HDW_h is the pressure head in millionths of a foot at each well grid cell h, and $HW0_h$ is the initial pressure head in feet at each well grid cell h.

Initial pressure heads at well grid cells, $HW0_h$, were determined by executing the simulation model at year 1988 conditions and were used in calculating the year 2010 pressure head at the well grid cells. Again, $\gamma_{i,h}$ and $\theta_{n,h}$ were multiplied by one million to obtain values on the order of one. $THDW_h$ and $TDDW_h$ values were calculated by dividing HDW_h and DDW_h by 10^{+6}:

$$TDDW_h = \frac{DDW_h}{10^{+6}} \tag{5.11}$$

$$THDW_h = \frac{HDW_h}{10^{+6}} \quad \textit{for all } h \tag{5.12}$$

where $TDDW_h$ is the drawdown in feet at each well grid cell h and $THDW_h$ is the total pressure head in feet at each well grid cell h.

An additional constraint was incorporated into the optimization model to preclude pumpage that would dewater the surficial aquifer. The following constraint was implemented to ensure water table elevations do not decrease below a level of one foot above the bottom of the surficial aquifer. This constraint enables the pressure heads to be constrained in order to avoid drying out of well grid cells:

$$HDW_h \geq BOTELEV_h + 1.0 \quad \textit{for all } h \tag{5.13}$$

where $BOTELEV_h$ is the bottom elevation of the surficial aquifer in feet from mean sea level at each well grid cell control point h. $BOTELEV_h$ values were obtained from the input file of the simulation model.

5.3.2 Management Constraints

Management constraints used in these optimization models define the capacity of available resources, the demand for available resources, and the source to demand links. The first set of constraints specify limits on capacities associated with the production of water from aquifer systems. These following constraints were incorporated in the model to limit the maximum withdrawal rates at the well grid cells:

$$QMT_i = \sum_k QM_{i,k} \leq CM_i \tag{5.14}$$

$$QAT_n = \sum_o QA_{n,o} \leq CA_n \tag{5.15}$$

$$QWT_m = \sum_o QW_{m,o} \leq CW_m \quad \textit{for all } i, k, m, n, \textit{ and } o \tag{5.16}$$

where $QM_{i,k}$ is the discharge rate in cubic feet per day (cfd) of each municipal well grid cell i which supplies each municipal water service area k, CM_i is the total capacity rate in cfd of each municipal well grid cell i, $QA_{n,o}$ is the discharge rate in cfd of each agricultural well grid cell n which supplies each agricultural water service area o, CA_n is the total capacity rate in cfd of each agricultural well grid cell n, QWT_m is the total effluent reuse rate in cfd of each wastewater treatment plant m, $QW_{m,o}$ is the effluent reuse rate in cfd of each wastewater treatment plant m which supplements each agricultural water service area o, and CW_m is the capacity rate in cfd of each wastewater treatment plant i.

Municipal well grid cell capacities, CM_i (for all i), were set at 600,000 cfd. CA_n values for each agricultural well grid cell were set at the service area demand for which the cell supplied. CW_m limits reflect the available wastewater effluent which could be used to supplement the agricultural demand.

Minimum withdrawal rates on municipal well grid cell were also incorporated into the optimization model to prevent the shutting off of existing wells. This is achieved by placing lower discharge limits on the well grid cells. These constraints were formulated to require minimum flows that equal a percentage of the 1988 withdrawal rates. From a review of the projected year 2010 water allocation strategy, discharges at existing well grid cells were allowed to decrease to approximately 50 percent of the 1988 distribution discharge rate estimates. Thus, the following constraints incorporated in the optimization models, which requires year 2010 discharges at municipal well grid cells to be greater than or equal to half of the year 1988 rates:

$$QMT_i = \sum_k QM_{i,k} \geq 0.50 \ (QMO_i) \tag{5.17}$$

In order to be a feasible water allocation strategy, the strategy must meet or exceed the demands of the service areas. The following demand constraints ensure that water needs of municipal and agricultural service areas are satisfied:

$$\sum_i QM_{i,k} \geq DM_k \tag{5.18}$$

$$\sum_n QA_{n,o} + \sum_m QW_{m,o} \geq DA_o \quad \textit{for all } i, k, m, n, \textit{and } o \tag{5.19}$$

where DM_k is the demand rate in cfd of each municipal water service area k and DA_o is the demand rate in cfd of each agricultural water service area o.

Municipal demand, DM_k, and agricultural demand, DA_o, were calculated using projected year 2010 discharge rates (See Chapter 4). These demands ensure that the model identifies discharge from well grid cells that meet future demands of the water service areas. It may be seen from constraint equations 5.19 that agricultural well grid cells are not constrained to lower limits as long as demand can be satisfied with wastewater effluent.

Also, since every water supply area does not (and can not) supply every water service area, the optimization model was constructed to link specific water sources with specific demand areas. The following constraints were formulated to specify which well grid cells and wastewater treatment plants can not supply water to water service demand areas where source-demand links are nonexistent:

$$QM_{i,k} = 0.0 \tag{5.20}$$

$$QA_{n,o} = 0.0 \tag{5.21}$$

$$QW_{m,o} = 0.0 \quad \textit{for all } i,\ k,\ m,\ m, \textit{ and } o \tag{5.22}$$

5.3.3 Chloride Concentration Constraints

The water quality optimization models that were developed allow chloride concentrations to be calculated, constrained, and/or optimized. Chloride concentration constraints at well grid cells were developed using additional influence coefficient matrices. These matrices consist of influence coefficients which reflect the aquifer response in terms of chloride concentration changes due to a change in pumpage. Similar to the method of calculating the drawdown values, the increase in concentration is calculated by multiplying the coefficients by the change in pumpage predicted by the optimization model. As shown below, the general increase-in-concentration constraint at given well grid cells includes the linear combination of aquifer responses to the municipal and agricultural wells:

$$CI_h = \sum_i \zeta_{i,h}(QMT_i - QM0_i) + \sum_n \phi_{n,h}(QAT_n - QA0_n) \tag{5.23}$$

$$\textit{for all } h,\ i, \textit{ and } n$$

where CI_h is the increase in chloride concentration in 10^{-4} of a milligrams per liter (mg/l) at each well grid cell control point h, $\zeta_{i,h}$ is the aquifer influence coefficient defining chloride concentration change at each well grid cell control point h due to a change in discharge rate at each municipal well grid cell i, and $\phi_{n,h}$ is the aquifer influence coefficient defining chloride concentration change at each well grid cell control point h due to a change in discharge rate at each agricultural well n.

Similar to the hydraulic influence coefficients, the water quality influence coefficients were multiplied by 10^{+4} to obtain values on the order of one. Therefore, the calculated increase concentration values must be divided by this constant to change the units to mg/l. The total concentration increase, TCI_h, values were calculated by dividing CI_h by 10^{+4}:

$$TCI_h = \frac{CI_h}{10^{+4}} \quad \textit{for all } h \tag{5.24}$$

where TCI_h is the total increased chloride concentration, respectively, in mg/l at each well grid cell control point h.

Once the concentration increases were determined, absolute concentrations at the well grid cells were calculated by adding the values to the initial year 1988 concentrations as shown below:

$$CC_h = CC0_h(10^{+4}) + CI_h \qquad \textit{for all } h \tag{5.25}$$

where CC_h is the chloride concentration in ten-thousandths of a milligrams per liter (mg/l) at each well grid cell control point h and $CC0_h$ is the initial chloride concentration in mg/l at well grid cell control points h. $CC0_h$ values were calculated by executing the steady-state solute transport simulation model using year 1988 conditions. Again, the total chloride concentrations, TCC_h, were calculated by dividing CC_h by 10^{+4}.

$$TCC_h = \frac{CC_h}{10^{+4}} \quad \textit{for all } h \tag{5.26}$$

where TCC_h is the total chloride concentration in mg/l at each well grid cell control point h.

Similar to the hydraulic management constraints, maximum limits were set on the chloride concentration levels at particular wells. The following constraint was applied to the optimization model to allow concentration levels to be controlled at specified well grid cells:

$$TCC_h \leq (\ CL_h\ ,\ CC0_h\ ),\ \ \textit{whichever is greater}, \ \ \textit{for all } h \tag{5.27}$$

where CL_h is the maximum chloride concentration limit in mg/l of each well grid cell control point h. The constraint states that concentration levels at all well grid cells must be less than or equal to the specified limit <u>or</u> the initial year 1988 concentration, whichever is greatest. This constraining method was chosen since preexisting concentrations at some well grid cells already exceed the chloride standard. Values of CL_h were set to 1000 mg/l for municipal wells and 9500 mg/l for agricultural wells, since several wells in both municipal and agricultural areas were already operating near these levels.

5.3.4 Nonnegativity Constraints

If any of the decision variables have lower or upper bounds which are not negative or positive infinite, then the bounds must be defined in the optimization model. Well grid cell discharge rates and wastewater treatment plant effluent rates were defined as positive values. Since all the wells are withdrawing water and not injecting, the following constraints were placed on the values of municipal, agricultural, and wastewater treatment plant discharge decision variables:

$$QM_{i,k} \geq 0.0 \tag{5.28}$$

$$QA_{n,o} \geq 0.0 \tag{5.29}$$

$$QW_{m,o} \geq 0.0 \quad \textit{for all } i, k, m, n, \textit{ and } o \tag{5.30}$$

Since chloride concentration cannot be negative the following nonnegativity constraint was also placed on all well grid cells:

$$CC_h \geq 0.0 \quad \textit{for all } h \tag{5.31}$$

5.4 OPTIMIZATION MODEL FORMULATION

5.4.1 General Optimization Model Formulation

The general ground water optimization model is comprised of the following components:

Model component A: Objective Function.

Model component B: Apparent constraints defining S, such as equations 5.1, 5.2, 5.3, and 5.4. For example model 1 would use equation 5.1 to define S.

$$S \geq DD_j \quad \textit{for all } j \tag{5.1}$$

Model component C: Hydraulic constraints including:

Aquifer response constraints for all h, i, j, n, and o:

$$DD_j = \sum_i \alpha_{i,j} \, (QMT_i - QM0_i) + \sum_n \beta_{i,j} \, (QAT_n - QA0_n) + DDPRIV_j \, (10^{+6}) \tag{5.5}$$

$$TDD_j = \frac{DD_j}{10^{+6}} \tag{5.6}$$

$$HD_j = H0_j \, (10^{+6}) - DD_j \tag{5.7}$$

$$THD_j = \frac{HD_j}{10^{+6}} \tag{5.8}$$

$$DDW_h = \sum_i \gamma_{i,h} \, (QMT_i - QM0_i) + \sum_n \theta_{n,h} \, (QAT_n - QA0_n) \tag{5.9}$$

$$HDW_h = HW0_h \, (10^{+6}) - DDW_h \tag{5.10}$$

$$TDDW_h = \frac{DDW_h}{10^{+6}} \tag{5.11}$$

$$THDW_h = \frac{HDW_h}{10^{+6}} \tag{5.12}$$

$$HDW_h \geq BOTELEV_h + 1.0 \tag{5.13}$$

Management constraints for all h, i, j, k, m, n, and o:

$$QMT_i = \sum_k QM_{i,k} \leq CM_i \tag{5.14}$$

$$QAT_n = \sum_o QA_{n,o} \leq CA_n \tag{5.15}$$

$$QWT_m = \sum_o \leq CW_m \tag{5.16}$$

$$QMT_i = \sum_k QM_{i,k} \geq 0.50 \, (QMO_i) \tag{5.17}$$

$$\sum_{i} QM_{i,k} \geq DM_k \tag{5.18}$$

$$\sum_{n} QA_{n,o} + \sum_{m} QW_{m,o} \geq DA_o \tag{5.19}$$

Allocation constraints for all i, k, m, n, and o where source-demand links are nonexistent:

$$QM_{i,k} = 0.0 \tag{5.20}$$

$$QA_{n,o} = 0.0 \tag{5.21}$$

$$QW_{m,o} = 0.0 \tag{5.22}$$

Model component D: Water quality constraints including:

Chloride concentration constraints for all h, i, and n:

$$CI_h = \sum_{i} \zeta_{i,h}(QMT_i - QM0_i) + \sum_{n} \phi_{n,h}(QAT_n - QA0_n) \tag{5.23}$$

$$TCI_h = \frac{CC_h}{10^{+4}} \tag{5.24}$$

$$CC_h = CC0_h(10^{+4}) + CI_h \tag{5.25}$$

$$TCC_h = \frac{CC_h}{10^{+4}} \tag{5.26}$$

Management constraints for all h:

$$TCC_h \leq (CL_h \text{ or } CC0_h), \quad \textit{whichever is greater} \tag{5.27}$$

Model Component E: Nonnegativity constraints for all h, i, k, m, n, and o:

$$QM_{i,k} \geq 0.0 \tag{5.28}$$

$$QA_{n,o} \geq 0.0 \tag{5.29}$$

$$QW_{m,o} \geq 0.0 \tag{5.30}$$

$$CC_h \geq 0.0 \tag{5.31}$$

5.4.2 Specific Optimization Model Formulations

Table 5.1 - Optimization Model Formulations

Model Component	Model 1	Model 2	Model 3	Model 4	Model 5
A	Min. S	Max. S	Min. S	Min. S	Min. S
B	Eq. 5.1	Eq. 5.2	Eq. 5.3	Eq. 5.1	Eq. 5.4
C	Eqs. 5.5 thru 5.22	Eqs. 5.5 thru 5.22	Eqs. 5.5 thru 5.22	Eqs. 5.5 thru 5.22	Eqs 5.5 thru 5.22
D	Not Included	Not Included	Not Included	Eqs. 5.23 thru 5.27	Eqs 2.23 thru 5.27
E	Eqs. 5.28 thru 5.30	Eqs. 5.28 thru 5.30	Eqs. 5.28 thru 5.30	Eqs. 5.28 thru 5.31	Eqs. 5.28 thru 5.31

5.5 PROCEDURE FOR IDENTIFYING STRATEGIES

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

1) Solve the optimization model with initial estimates of influence coefficients. The optimization model represents a system of linear equations solved during the first step using GAMS. If a feasible solution to the problem exists, GAMS identifies values for decision variables, minimizes (or maximizes) the value of the objective function and satisfies all specified constraints. Output from the optimization

models includs data depicting an optimum water allocation strategy (i.e. pumpage rates) and the aquifer system's response to the strategy (i.e. pressure heads, drawdowns, concentrations, and concentration increases). The model identifies which municipal and agricultural well grid cells and wastewater treatment plants should be used, their respective withdrawal or discharge rates, the pressure heads and drawdown values at the sensitive wetland control points, and chloride concentrations and increases at the well grid cells.

2) Execute simulation model with prescribed allocation strategy. Because the optimization model is based on linear response theory and is not a true simulation model, the optimum pumping strategy must be incorporated into an updated input file for a MODFLOW or DSTRAM simulation model to permit the second step, a simulation to determine the actual response to the strategy. When these simulation model responses were compared to the responses predicted by the optimization model, it was shown that the aquifer system response to pumping is generally linear in the aquifer system. However, there were occasions where control points were found to respond in a nonlinear fashion.

3) Compare optimization model results to simulation model predications.

4) If hydraulic and/or water quality results from the optimization models are not in agreement with the simulation models, calculate a revised set of influence coefficients.

5) Solve the optimization model with revised influence coefficients.

6) Execute simulation model with the prescribed allocation strategy.

7) Compare optimization model results to simulation model predications.

8) Repeat if necessary.

Nonlinear responses were found to be due to both 1) nonlinearly decreasing pressure head with increasing discharge rates and 2) a nonadditive effect of drawdown and chloride concentration when utilized well grid cells are in close proximity. An example of the first nonlinear response is shown in Figure 5.1 where the slope of the line increases negatively with increased pumping. These nonlinear responses were corrected

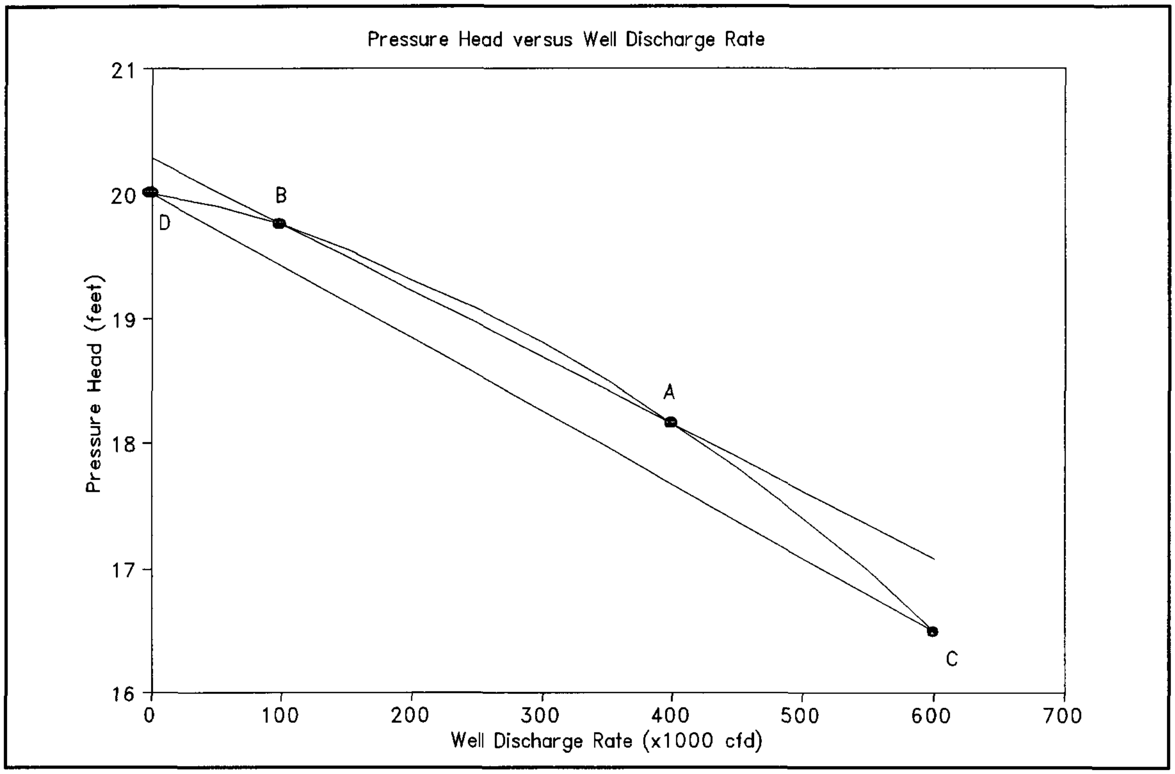


Figure 5.1 - Example of nonlinear aquifer response with increased discharge rate at a surficial aquifer control point.

by creating new sets of response matrices based on the previously predicted strategy and incorporating these into the optimization model. As previously discussed, the correlation between the optimization and simulation models with respect to aquifer system response increases when the response matrix calculation process incorporates an initial allocation strategy which is close to the actual optimum strategy. The revised set of response matrices were determined using revised initial conditions that essentially match the first estimates of the optimum water allocation strategy. Minor differences involved designating a maximum withdrawal rate of 400,000 cfd at well grid cells in the flow simulation model where the previous strategy predicted a withdrawal rate of greater than 400,000 cfd. Using Figure 5.1 as an example, the linear approximation from point (a) to point (b) is a better estimation of the nonlinear approximation than the linear response from point (c) to point (d). This procedure reduces the possibility of grid cells completely draining and improves the correlation between the optimization and simulation models.

Using the revised initial conditions, influence coefficients were then determined by perturbating the withdrawal rates of individual well grid cells. The following methodology was used to determine the perturbations in withdrawal rate with respect to magnitude and direction:

1) If the initial withdrawal rate was less 200,000 cfd, the rate at individual well grid cells was equated to the demand of the service area being supplied or 400,000 cfd, whichever is lower.

2) If the initial withdrawal rate was greater than 200,000 cfd and less than 300,000 cfd, the rate at individual well grid cells was increased by 100,000 cfd.

3) If the initial withdrawal rate was greater than 300,000 cfd and less than 400,000 cfd, the rate at individual well grid cells was decreased by 100,000 cfd.

4) If the initial withdrawal rate was greater than 400,000 cfd, the rate at individual well grid cells was decreased by 50,000 cfd.

Once generated using the above process with the simulation models, the revised set of response matrices were used to obtain a revised optimization model. Well grid cells not utilized during the previous execution of the optimization model were stated as unavailable in the revised optimization model. This was accomplished by revising the SERVE tables specifying which well grid cells and wastewater treatment plants supply the various service areas. The appropriate table elements were changed by subsituting the value of one for a value of zero. With these changes, the revised optimization model, was executed to determine an improved water allocation strategy. Finally, the water allocation scenario predicted by a revised optimization model was examined using both simulation models to verify aquifer responses predicted by the optimization model.

Another method of reducing the adverse effect associated with nonlinear aquifer responses, but was not implemented in this project, involves limiting the difference in withdrawal rate from the first to the second execution of the optimization model. By limiting this difference, the withdrawal rate range is reduced which causes the aquifer system to respond in a more linear fashion. This could improve the correlation between the optimization and simulation models although additional parameters and constraints would have to be incorporated in the optimization model.

5.6 RESULTS AND DISCUSSION

5.6.1 Model 1

The objective of the first resource allocation model was to minimize the maximum drawdown at all specified control points designated as having a high potential for vegetative harm. The tabulated optimum water allocation strategy is summarized in Tables 5.2 and 5.3. Along with the optimum withdrawal rates, these tables describe the

Table 5.2 - Municipal water supply grid cell locations and withdrawal rates.

Supply I.D.	Numerical Grid Cell Location		Water Service Area Name	Year 1988 Withdrawal Rate (cfd)	Projected Year 2010 Withdrawal Rate (cfd)	Optimized Year 2010 Withdrawal Rate (cfd)				
	Row	Column				Model 1: Min Max Drawdown	Model 2: Max Min Head	Model 3: Min Avg Drawdown	Model 4: Min Max Drawdown	Model 5: Min Max Relative CC
MWELL1	45	22	Port Orange - West Wellfield	35475	31602	17738	17738	17738	48792	17738
MWELL2	48	18	Port Orange - West Wellfield	35475	31602	17770	17770	17770	17770	17770
MWELL3	46	20	Port Orange - West Wellfield	35475	31602	17738	17738	17738	17738	17738
MWELL4	50	19	Port Orange - West Wellfield	35475	31602	17738	17738	17738	110935	17738
MWELL5	49	20	Port Orange - West Wellfield	70951	63204	35476	35476	35476	35476	35476
MWELL6	47	19	Port Orange - West Wellfield	35475	31602	17738	17738	17738	17738	17738
MWELL7	48	19	Port Orange - West Wellfield	35475	31602	17738	17738	17738	17738	17738
MWELL8	47	18	Port Orange - West Wellfield	70951	63204	35476	35476	35476	35476	35476
MWELL9	48	21	Port Orange - West Wellfield	35475	31602	17738	17738	17738	17738	49306
MWELL10	51	19	Port Orange - West Wellfield	35475	31602	17738	17738	112872	17738	81303
MWELL11	49	19	Port Orange - West Wellfield	35475	31602	17738	17738	17738	17738	17738
MWELL12	46	18	Port Orange - West Wellfield	35475	31602	17738	17738	17738	17738	17738
MWELL13	50	20	Port Orange - West Wellfield	35475	31602	17738	17738	17738	78723	17738
MWELL14	45	19	Port Orange - West Wellfield	0	94806	0	0	0	0	0
MWELL15	50	18	Port Orange - West Wellfield	0	94806	0	0	0	0	0
MWELL16	21	6	Port Orange - West Wellfield	0	0	0	0	0	0	0
MWELL17	20	4	Port Orange - West Wellfield	0	0	0	0	0	0	0
MWELL18	22	8	Port Orange - West Wellfield	0	0	0	0	0	0	0
MWELL19	19	7	Port Orange - West Wellfield	0	0	0	95135	0	0	0
MWELL20	18	5	Port Orange - West Wellfield	0	0	219952	0	0	414940	0
MWELL21	20	9	Port Orange - West Wellfield	0	0	0	0	600000	0	0
MWELL22	23	5	Port Orange - West Wellfield	0	0	0	0	0	0	0
MWELL23	22	3	Port Orange - West Wellfield	0	0	475183	0	0	94958	600000
MWELL24	24	7	Port Orange - West Wellfield	0	0	0	0	0	0	0
MWELL25	51	20	Port Orange - West Wellfield	0	31602	0	0	0	0	0
MWELL26	51	21	Port Orange - West Wellfield	0	31602	0	0	0	0	0
MWELL27	52	19	Port Orange - West Wellfield	0	31602	0	0	0	0	0
MWELL28	52	20	Port Orange - West Wellfield	0	31602	0	0	0	0	0
MWELL29	52	21	Port Orange - West Wellfield	0	31602	0	0	0	0	0
MWELL30	53	19	Port Orange - West Wellfield	0	31602	0	0	0	0	0
MWELL31	53	20	Port Orange - West Wellfield	0	31602	0	0	0	0	0
MWELL32	53	21	Port Orange - West Wellfield	0	31602	0	600000	0	0	0

Table 5.2 - continued

Supply I.D.	Numerical Grid Cell Location Row	Column	Water Service Area Name	Year 1988 Withdrawal Rate (cfd)	Projected Year 2010 Withdrawal Rate (cfd)	Optimized Year 2010 Withdrawal Rate (cfd) Model 1: Min Max Drawdown	Model 2: Max Min Head	Model 3: Min Avg Drawdown	Model 4: Min Max Drawdown	Model 5: Min Max Relative CC
MWELL33	46	46	Port Orange - East Wellfield	8186	14586	4145	4145	4145	4145	4145
MWELL34	49	46	Port Orange - East Wellfield	16373	29171	77621	8187	8187	16808	8187
MWELL35	49	45	Port Orange - East Wellfield	4093	7293	2047	2047	2047	13131	2047
MWELL36	47	47	Port Orange - East Wellfield	12280	21878	6140	75574	75574	55868	44410
MWELL37	46	45	Port Orange - East Wellfield	8186	14586	4093	4093	4093	4093	35257
MWELL38	47	46	Port Orange - East Wellfield	4093	7293	2079	2079	2079	2079	2079
MWELL39	32	38	Daytona Beach East - Marion TP	78522	40987	50673	50673	50673	39261	306882
MWELL40	32	36	Daytona Beach East - Marion TP	78522	40987	39261	39261	39261	39261	39261
MWELL41	31	32	Daytona Beach East - Marion TP	157043	81973	78522	78522	78522	78522	256276
MWELL42	32	37	Daytona Beach East - Marion TP	78522	40987	39325	39325	39325	99640	39325
MWELL43	39	24	Daytona Bch West - Brennan TP	124803	102247	62555	62555	62555	62555	62555
MWELL44	39	26	Daytona Bch West - Brennen TP	124803	102247	62555	62555	62555	62555	167746
MWELL45	39	23	Daytona Bch West - Brennan TP	124803	102247	62504	62504	62504	62504	62504
MWELL46	31	31	Daytona Bch West - Brennan TP	124803	102247	515859	378782	515859	515859	62402
MWELL47	30	30	Daytona Bch West - Brennan TP	124803	102247	62402	62402	62402	62402	286411
MWELL48	39	22	Daytona Bch West - Brennan TP	124803	102247	62402	199478	62402	62402	62402
MWELL49	38	23	Daytona Bch West - Brennan TP	124803	102247	62402	62402	62402	62402	62402
MWELL50	37	23	Daytona Bch West - Brennan TP	124803	102247	62402	62402	62402	62402	186658
MWELL51	36	22	Daytona Bch West - Brennan TP	124803	102247	62402	62402	62402	62402	62402
MWELL52	39	25	Daytona Bch West - Brennan TP	249605	204493	124854	124854	124854	124854	124854
MWELL53	34	22	Daytona Beach West - South TP	0	102247	0	0	0	0	0
MWELL54	33	22	Daytona Beach West - South TP	0	204493	0	0	0	0	0
MWELL55	32	23	Daytona Beach West - South TP	0	102247	0	0	0	0	0
MWELL56	31	23	Daytona Beach West - South TP	0	204493	0	0	0	0	0
MWELL57	30	23	Daytona Beach West - South TP	0	102247	0	0	0	0	0
MWELL58	38	21	Daytona Beach West - South TP	0	102247	574408	600000	600000	600000	506932
MWELL59	35	23	Daytona Beach West - South TP	0	102247	380593	355001	355001	489057	576028
MWELL60	34	23	Daytona Beach West - South TP	0	102247	0	0	0	0	0
MWELL61	29	24	Daytona Beach West - South TP	0	204493	0	0	0	0	0
MWELL62	28	24	Daytona Beach West - South TP	0	102247	0	0	0	0	0
MWELL63	27	24	Daytona Beach West - South TP	0	102247	0	0	0	0	0
MWELL64	26	24	Daytona Beach West - South TP	0	102247	600000	600000	600000	465944	600000
MWELL65	56	35	Spruce Creek	21647	21478	11012	43471	11012	32330	11012
MWELL66	56	34	Spruce Creek	21647	32217	43429	10971	43429	10971	32288

Table 5.2 - continued

Supply I.D.	Numerical Grid Cell Location Row	Column	Water Service Area Name	Year 1988 Withdrawal Rate (cfd)	Projected Year 2010 Withdrawal Rate (cfd)	Optimized Year 2010 Withdrawal Rate (cfd) Model 1: Min Max Drawdown	Model 2: Max Min Head	Model 3: Min Avg Drawdown	Model 4: Min Max Drawdown	Model 5: Min Max Relative CC
MWELL67	22	54	Holly Hill - East Wellfield	2275	2620	10188	10188	10188	12026	1170
MWELL68	22	53	Holly Hill - East Wellfield	4550	5240	2307	2307	2307	2307	2307
MWELL69	21	54	Holly Hill - East Wellfield	2275	2620	1170	1170	1170	1170	1170
MWELL70	21	53	Holly Hill - East Wellfield	2275	2620	1138	1138	1138	1138	10156
MWELL71	21	52	Holly Hill - East Wellfield	2275	2620	1138	1138	1138	1138	1138
MWELL72	22	34	Holly Hill - West Wellfield	38997	39301	19499	19499	19499	19499	19499
MWELL73	22	35	Holly Hill - West Wellfield	58495	58952	120391	120391	120391	29248	29248
MWELL74	21	35	Holly Hill - West Wellfield	38997	58952	19499	19499	19499	110642	110642
MWELL75	60	20	Smyrna Beach / Samsula	79425	100031	39713	39713	405682	39713	80000
MWELL76	59	16	Smyrna Beach / Samsula	0	50016	0	0	0	0	0
MWELL77	59	15	Smyrna Beach / Samsula	0	50016	365970	365970	0	40288	0
MWELL78	58	16	Smyrna Beach / Samsula	0	200062	0	0	0	0	0
MWELL79	12	50	Ormond Beach - Division Ave WF	60325	43356	30163	30163	30163	57686	30163
MWELL80	12	53	Ormond Beach - Division Ave WF	90488	65034	45276	45276	127856	45276	127856
MWELL81	13	52	Ormond Beach - Division Ave WF	120650	86711	60357	60357	60357	60357	60357
MWELL82	12	52	Ormond Beach - Division Ave WF	30163	21678	97693	97693	15114	15114	15114
MWELL83	13	50	Ormond Beach - Division Ave WF	30163	21678	15114	15114	15114	15114	15114
MWELL84	13	49	Ormond Beach - Division Ave WF	30163	21678	15145	15145	15145	70201	15145
MWELL85	11	39	Ormond Beach - State Rd. 40 WF	84902	21678	42503	42503	42503	42503	108182
MWELL86	10	44	Ormond Beach - State Rd. 40 WF	84902	21678	42535	42535	42535	42535	42535
MWELL87	11	41	Ormond Beach - State Rd. 40 WF	84902	21678	42451	42451	42451	42451	42451
MWELL88	9	45	Ormond Beach - State Rd. 40 WF	0	21678	0	0	0	0	0
MWELL89	10	45	Ormond Beach - State Rd. 40 WF	0	21678	0	0	224175	0	0
MWELL90	6	32	Ormond Beach - Hudson Wellfield	0	43356	0	0	0	0	0
MWELL91	5	32	Ormond Beach - Hudson Wellfield	0	43356	0	0	0	0	0
MWELL92	4	33	Ormond Beach - Hudson Wellfield	0	43356	0	0	0	0	0
MWELL93	3	33	Ormond Beach - Hudson Wellfield	0	43356	0	0	0	0	0
MWELL94	5	33	Ormond Beach - Hudson Wellfield	0	21678	0	0	0	0	0
MWELL95	5	34	Ormond Beach - Hudson Wellfield	0	21678	0	0	0	0	0
MWELL96	4	34	Ormond Beach - Hudson Wellfield	0	21678	0	0	0	0	0
MWELL97	3	35	Ormond Beach - Hudson Wellfield	0	21678	0	0	549473	0	0
MWELL98	4	31	Ormond Beach - Hudson Wellfield	0	21678	0	0	0	0	0

Table 5.2 - continued

Supply I.D.	Numerical Grid Cell Location		Water Service Area Name	Year 1988 Withdrawal Rate (cfd)	Projected Year 2010 Withdrawal Rate (cfd)		Optimized Year 2010 Withdrawal Rate (cfd)				
	Row	Column					Model 1: Min Max Drawdown	Model 2: Max Min Head	Model 3: Min Avg Drawdown	Model 4: Min Max Drawdown	Model 5: Min Max Relative CC
MWELL99	15	6	Ormond Bch - SR40 &/or Hudson	0	21678		600000	600000	0	173648	600000
MWELL100	16	7	Ormond Bch - SR40 &/or Hudson	0	43356		173648	0	0	600000	107969
MWELL101	17	7	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL102	18	7	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL103	20	6	Ormond Bch - SR40 &/or Hudson	0	43356		0	0	0	0	0
MWELL104	21	5	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL105	22	5	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL106	18	8	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL107	18	9	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL108	18	10	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL109	18	11	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL110	17	10	Ormond Bch - SR40 &/or Hudson	0	43356		0	0	0	0	0
MWELL111	19	5	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL112	18	4	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL113	18	3	Ormond Bch - SR40 &/or Hudson	0	21678		0	173648	0	0	0
MWELL114	19	3	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL115	20	2	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL116	21	2	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL117	13	23	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL118	12	24	Ormond Bch - SR40 &/or Hudson	0	21678		0	0	0	0	0
MWELL119	8	39	Tymber Creek	12234	24298		24298	24298	24298	24298	24298
MWELL120	7	34	The Trails, Inc.	42723	78507		78507	78507	78507	78507	78507
			Total	3295252	5753100		5863625	5863625	5863624	5863630	6088706

Table 5.3 - Agricultural water supply grid cell locations and withdrawal rates.

Supply I.D.	Numerical Grid Cell Location Row	Numerical Grid Cell Location Column	Agricultural Application # / Name	Year 1988 Withdrawal Rate (cfd)	Projected Year 2010 Withdrawal Rate (cfd)	Optimized Year 2010 Withdrawal Rate (cfd) Model 1: Min Max Drawdown	Model 2: Max Min Head	Model 3: Min Avg Drawdown	Model 4: Min Max Drawdown	Model 5: Min Max Relative CC
AGWELL1	54	47	AG-0396AN / AGAREA1	3640	3640	0	0	0	0	0
AGWELL2	54	46	AG-0396AN / AGAREA1	5460	5460	0	0	0	0	0
AGWELL3	17	54	AG-0269AN / AGAREA2	6284	6284	0	0	0	0	0
AGWELL4	16	51	AG-0269AN / AGAREA2	6284	6284	0	0	0	0	0
AGWELL5	17	53	AG-0269AN / AGAREA2	6284	6284	0	0	0	0	0
AGWELL6	45	58	AG-0279AU / AGAREA3	5624	5624	0	0	0	0	0
AGWELL7	46	58	AG-0279AU /AGAREA3	6093	6093	0	0	0	0	0
AGWELL8	8	59	AG-0565ANV /AGAREA4	9075	9075	0	0	0	0	0
AGWELL9	9	58	AG-0565ANV / AGAREA4	3025	3025	0	0	0	0	0
AGWELL10	7	59	AG-0565ANV / AGAREA4	3025	3025	0	0	0	0	0
AGWELL11	6	59	AG-0565ANV / AGAREA4	3025	3025	0	0	0	0	0
AGWELL12	6	58	AG-0565ANV / AGAREA4	3025	3025	0	0	0	0	0
AGWELL13	4	50	AG-0647AUS / AGAREA5	6352	6352	0	0	0	0	0
AGWELL14	4	48	AG-0647AUS / AGAREA5	6352	6352	0	0	0	0	0
AGWELL15	5	49	AG-0647AUS / AGAREA5	6352	6352	0	0	0	0	0
AGWELL16	3	49	AG-0647AUS / AGAREA5	6352	6352	0	0	0	0	0
AGWELL17	56	33	AG-0147AU / AGAREA6	9952	9952	0	0	0	0	0
AGWELL18	56	35	AG-0147AU / AGAREA6	29856	29856	0	0	0	0	0
AGWELL19	59	17	AG-0236AN / AGAREA7	28191	28191	85747	85747	0	85747	85747
AGWELL20	59	18	AG-0236AN / AGAREA7	28191	28191	0	0	85747	0	0
AGWELL21	60	17	AG-0236AN / AGAREA7	28191	28191	0	0	0	0	0
AGWELL22	57	35	AG-0237AN / AGAREA8	4644	4644	0	0	0	0	0
AGWELL23	58	37	AG-0237AN / AGAREA8	4644	4644	97155	97155	97155	97155	103589
AGWELL24	58	35	AG-0237AN / AGAREA8	27865	27865	0	0	0	0	0
AGWELL25	59	36	AG-0237AN / AGAREA8	65017	65017	0	0	0	0	0
AGWELL26	32	47	AG-0085AN /AGAREA9	806	806	0	0	0	1758	0
AGWELL27	31	49	AG-0085AN / AGAREA9	806	806	0	0	0	0	0
AGWELL28	32	48	AG-0085AN / AGAREA9	4033	4033	0	0	0	0	5725
WASTE1	51	53	AGAREA1 and/or AGAREA3	0	0	21105	21105	21105	21105	21105
WASTE2	21	54	AGAREA2	0	0	19114	19114	19114	19114	19114
WASTE3	31	55	AGAREA3 and/or AGAREA9	0	0	5725	5725	5725	5725	5725
WASTE4	6	52	AGAREA4 and/or AGAREA5	0	0	47232	47232	47232	47232	47232
WASTE5	55	36	AGAREA6 and/or AGAREA8	0	0	46795	46795	46795	46795	46795
			Total	318448	318448	322873	322873	322873	324631	335032.12

location and corresponding water service area for each municipal well, agricultural well, and wastewater treatment plant grid cell for optimization models 1, 2, 3, 4, and 5.

The allocation strategies determined by model 1 were incorporated into the ground water simulation model to determine the response of the aquifer system. Table 5.4 displays the pressure head and drawdown determined by both the optimization and simulation models for the first execution. The optimization model predicted a maximum drawdown of 12.1 feet while the simulation model predicted a value of 11.9 feet.

To display the correlation of aquifer responses between simulation model predictions and optimization predictions, pressure head and drawdown predicted by the optimization model was plotted against those of the simulation model as shown in Figures 5.2a and 5.2b respectively. Ideally, predicted pressure head and drawdown from the optimization model should match those generated with the simulation model when using the optimum pumping scenario as input to the simulation model so that all points fall on the line having a slope and coefficient determination of one. However, because of the nonlinear effects previously discussed, a few points deviate from the "ideal" line. The coefficient of determination, R^2, for the head and drawdown comparisons were found to be 0.960 and 0.847 respectively. A mean difference of 0.71 feet and a standard deviation of 1.50 feet were determined to exist between predicted head and drawdown responses.

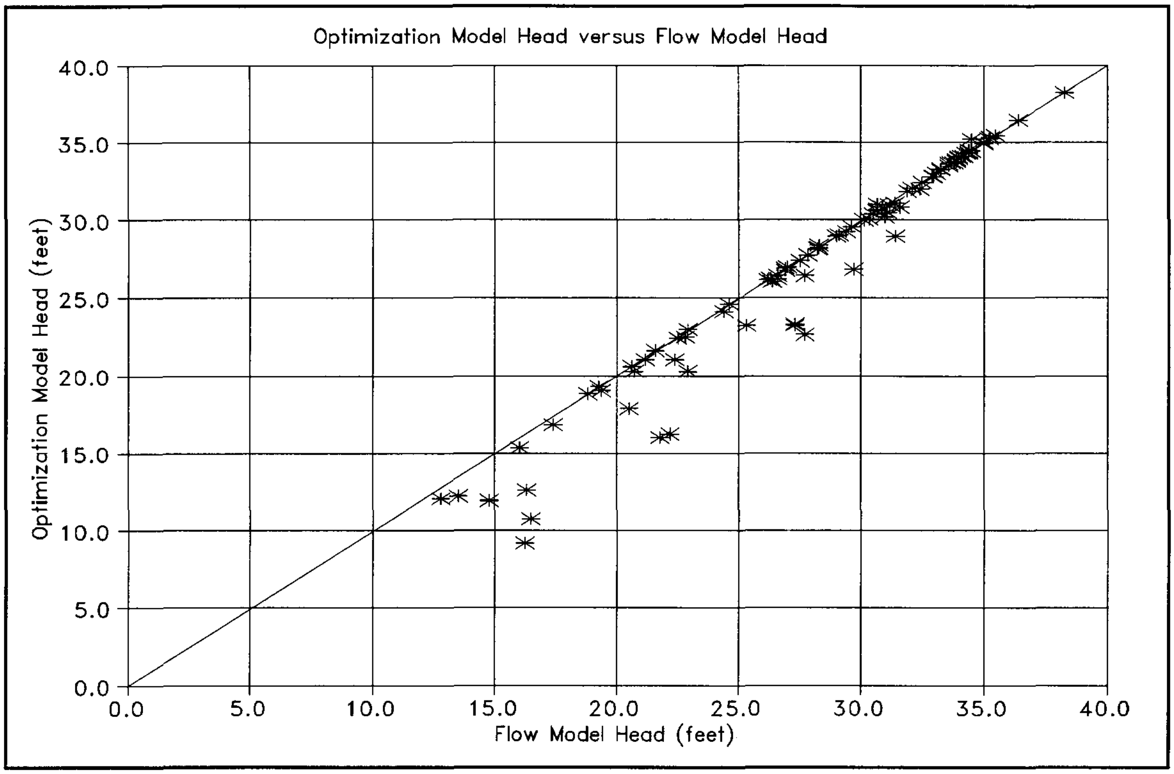
For the initial formulation, deviations occurred on one side of the "ideal" line. This side of the line reflects "conservative errors" since predicted pressure heads are higher and drawdown values are lower for the simulation model than the optimization model. This phenomena is due to the conservative method in which the response

Table 5.4 - Calculated pressure head and drawdown values while minimizing maximum drawdown at sensitive wetland control points (Results from Initial Model 1 Formulation).

Control Point	Numerical Grid Cell Location row	column	Simulation Model head (feet)	drawdown (feet)	Optimization Model 1 head (feet)	drawdown (feet)	Difference head (feet)	drawdown (feet)
1	5	1	28.3	1.33	28.37	1.33	-0.07	0.00
2	8	1	28.2	1.71	28.17	1.73	0.03	-0.02
3	15	1	26.9	2.94	26.81	2.99	0.09	-0.05
4	19	1	30.3	3.38	30.06	3.54	0.24	-0.16
5	22	1	31.4	3.34	31.07	3.63	0.33	-0.29
6	25	1	34.5	2.04	34.46	2.14	0.04	-0.10
7	28	1	33.1	1.26	32.99	1.31	0.11	-0.05
8	35	1	33.9	0.45	33.82	0.48	0.08	-0.03
9	41	1	34.1	0.30	34.08	0.32	0.02	-0.02
10	47	1	34.2	0.28	34.11	0.30	0.09	-0.02
11	53	1	34.4	0.32	34.48	0.32	-0.08	0.00
12	18	2	26.6	5.47	26.27	5.83	0.33	-0.36
13	56	2	35.5	0.41	35.48	0.42	0.02	-0.01
14	16	3	24.6	5.33	24.60	5.30	0.00	0.03
15	30	3	34.0	1.24	33.99	1.31	0.01	-0.07
16	32	3	34.3	0.94	34.30	1.00	0.00	-0.06
17	37	3	35.2	0.52	35.23	0.57	-0.03	-0.05
18	54	3	34.0	0.37	34.03	0.37	-0.03	0.00
19	17	4	21.2	11.94	20.96	12.14	0.24	-0.20
20	41	5	34.4	0.47	34.29	0.51	0.11	-0.04
21	44	5	33.9	0.40	33.87	0.43	0.03	-0.03
22	47	5	35.3	0.35	35.33	0.37	-0.03	-0.02
23	53	5	35.0	0.33	34.96	0.34	0.04	-0.01
24	30	6	34.4	1.20	34.34	1.26	0.06	-0.06
25	32	6	35.0	0.94	34.90	1.00	0.10	-0.06
26	50	6	36.4	0.33	36.45	0.35	-0.05	-0.02
27	59	7	33.8	0.68	33.71	0.79	0.09	-0.11
28	58	8	33.9	0.64	33.76	0.74	0.14	-0.10
29	39	9	32.5	0.58	32.36	0.64	0.14	-0.06
30	60	9	33.0	1.09	32.79	1.31	0.21	-0.22
31	16	10	27.5	3.69	27.39	3.81	0.11	-0.12
32	32	10	33.6	0.93	33.60	1.00	0.00	-0.07
33	14	11	27.8	2.90	27.73	2.97	0.07	-0.07
34	34	11	30.9	0.84	30.79	0.91	0.11	-0.07
35	52	11	33.6	0.29	33.47	0.33	0.13	-0.04
36	54	11	33.7	0.30	33.66	0.34	0.04	-0.04
37	59	11	32.4	1.69	31.97	2.13	0.43	-0.44
38	17	12	30.7	2.50	30.62	2.59	0.08	-0.09
39	31	12	30.5	1.01	30.40	1.10	0.10	-0.09
40	37	12	30.5	0.81	30.41	0.90	0.09	-0.09
41	39	12	32.9	0.71	32.81	0.79	0.09	-0.08
42	49	12	31.9	0.30	31.86	0.34	0.04	-0.04
43	43	13	30.5	0.53	30.42	0.58	0.08	-0.05
44	46	13	30.7	0.38	30.67	0.43	0.03	-0.05
45	16	14	28.3	1.93	28.20	2.01	0.10	-0.08
46	33	14	30.1	1.09	30.00	1.21	0.10	-0.12
47	54	14	33.3	0.30	33.24	0.36	0.06	-0.06
48	59	14	27.3	6.38	23.27	10.43	4.03	-4.05
49	14	15	26.2	1.57	26.16	1.64	0.04	-0.07
50	41	15	29.7	1.10	26.88	3.92	2.82	-2.82
51	5	16	27.0	0.89	27.00	0.90	0.00	-0.01
52	12	16	29.6	1.28	29.57	1.33	0.03	-0.05
53	38	16	25.3	4.59	23.21	6.69	2.09	-2.10

Table 5.4 - continued

	Numerical Grid			Simulation Model			Optimization Model 1			Difference	
Control	Cell Location			head	drawdown		head	drawdown		head	drawdown
Point	row	column		(feet)	(feet)		(feet)	(feet)		(feet)	(feet)
54	47	16		34.5	0.24		35.20	-0.50		-0.70	0.74
55	50	16		31.0	0.17		30.22	0.88		0.78	-0.71
56	53	16		33.3	0.23		33.29	0.31		0.01	-0.08
57	60	16		27.3	4.67		23.34	8.66		3.96	-3.99
58	3	17		22.9	0.85		22.95	0.85		-0.05	0.00
59	7	17		24.6	0.90		24.58	0.92		0.02	-0.02
60	10	17		26.3	0.98		26.27	1.03		0.03	-0.05
61	14	17		29.4	1.15		29.28	1.22		0.12	-0.07
62	30	17		27.7	2.93		22.62	7.98		5.08	-5.05
63	35	17		22.4	7.25		21.02	8.68		1.38	-1.43
64	44	17		30.7	0.65		31.03	0.37		-0.33	0.28
65	55	17		32.1	0.38		32.08	0.42		0.02	-0.04
66	57	17		31.4	1.48		29.01	3.89		2.39	-2.41
67	27	18		31.2	1.67		30.98	1.92		0.22	-0.25
68	32	18		22.9	7.04		20.26	9.64		2.64	-2.60
69	37	18		16.0	9.88		15.34	10.56		0.66	-0.68
70	40	18		17.4	7.62		16.81	8.19		0.59	-0.57
71	42	18		20.7	4.64		20.21	5.09		0.49	-0.45
72	4	19		27.8	0.74		27.76	0.74		0.04	0.00
73	19	19		26.6	1.25		26.42	1.39		0.18	-0.14
74	6	20		29.0	0.68		28.99	0.71		0.01	-0.03
75	9	20		29.1	0.73		29.03	0.77		0.07	-0.04
76	14	20		27.5	0.89		27.43	0.97		0.07	-0.08
77	17	20		27.0	1.11		26.97	1.23		0.03	-0.12
78	12	21		26.9	0.83		26.90	0.90		0.00	-0.07
79	10	22		38.3	0.94		38.21	1.10		0.09	-0.16
80	44	22		27.7	1.90		26.42	3.18		1.28	-1.28
81	48	22		24.4	0.95		24.13	1.27		0.27	-0.32
82	51	22		31.6	0.19		30.85	0.95		0.75	-0.76
83	46	23		31.1	1.31		30.47	1.93		0.63	-0.62
84	43	24		19.4	3.55		18.99	4.01		0.41	-0.46
85	20	25		26.4	1.35		26.04	1.66		0.36	-0.31
86	26	25		21.8	4.28		16.03	10.07		5.77	-5.79
87	30	25		16.5	6.36		10.76	12.14		5.74	-5.78
88	32	25		16.3	8.43		12.56	12.14		3.74	-3.71
89	7	26		19.3	0.50		19.27	0.53		0.03	-0.03
90	10	26		18.8	0.56		18.80	0.61		0.00	-0.05
91	14	26		21.6	0.71		21.61	0.79		-0.01	-0.08
92	17	26		22.5	0.87		22.42	0.98		0.08	-0.11
93	23	26		22.2	3.62		16.21	9.60		5.99	-5.98
94	29	26		16.2	4.78		9.17	11.73		7.03	-6.95
95	27	27		22.8	1.96		22.51	2.30		0.29	-0.34
96	31	27		14.8	6.17		11.88	9.12		2.92	-2.95
97	35	27		12.8	7.11		12.06	7.84		0.74	-0.73
98	33	28		13.5	6.14		12.21	7.39		1.29	-1.25
99	37	28		20.5	1.76		17.82	4.48		2.68	-2.72
100	39	29		20.6	0.96		20.55	1.05		0.05	-0.09
	Maximum			38.30	11.94		38.21	12.14		7.03	0.74
	Minimum			12.80	0.17		9.17	-0.50		-0.70	-6.95
	Average			28.21	2.10		27.50	2.81		0.71	-0.71
	Standard Deviation			5.86	2.38		6.68	3.36		1.50	1.49
	Variance			34.37	5.68		44.60	11.28		2.24	2.23



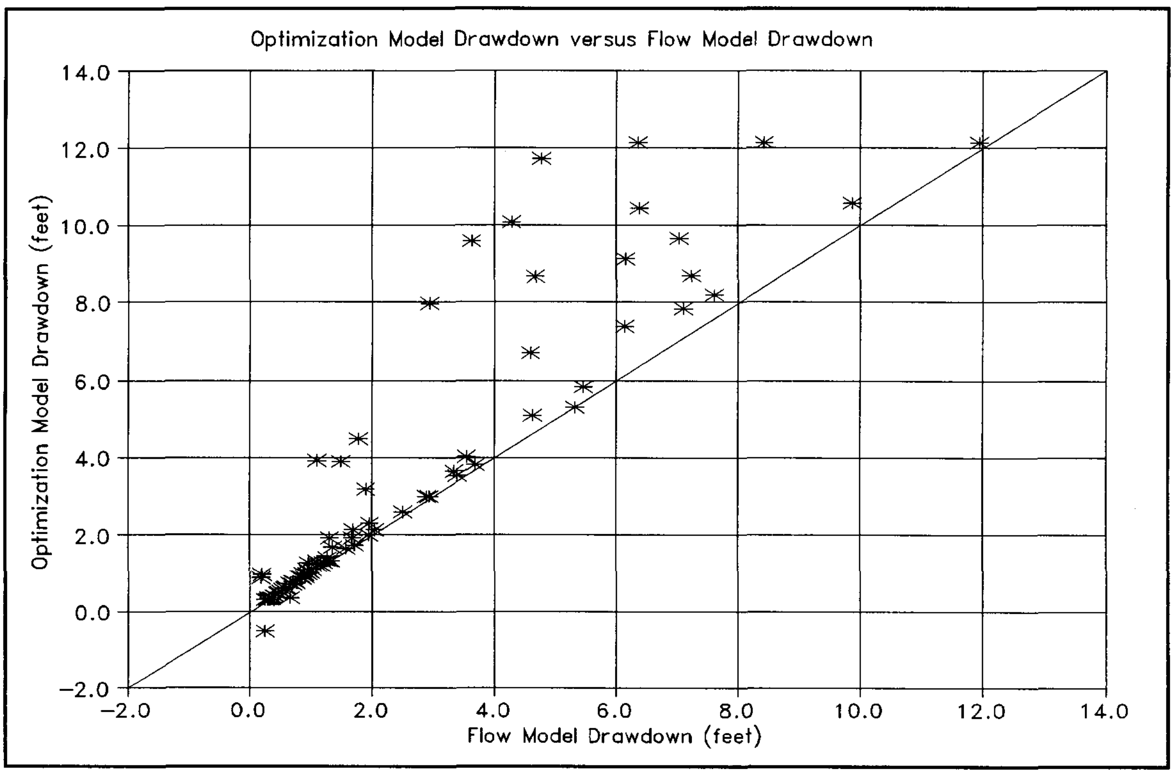


Figure 5.2 - Correlation between simulation model results and initial optimization model 1 formulation results.
a) Pressure heads for year 2010; b) Drawdowns from year 1988 to 2010.

matrices were calculated and was found to hold true for the five models demonstrated in this study.

To reduce the effects of the nonlinear behavior of the aquifer system, a revised model 1 was created and executed that incorporates updated influence coefficient matrices as described in section 5.5. When compared to the initial formulation of model 1 the revised model produced an improvement in the accuracy of the estimated objective function value (i.e. a smaller maximum drawdown for model 1 over the project area). Table 5.5 displays the pressure head and drawdown values determined by both the optimization and simulation models for the revised optimization model execution. The optimization model predicted a maximum drawdown of 7.9 feet while the simulation model predicted a value of 10.4 feet. The maximum drawdown for the revised model does not fall on the "conservative side" like the majority of predicted points. The maximum drawdown for the optimization and simulation models are in the same location, which shows a strong correlation between simulation model predictions and optimization predictions. The improvement of the revised model is shown by a decrease between the initial formulation and revised optimization models drawdown predictions which represents a 12.6 percent (1.5 feet) decrease in the objective function of the revised model.

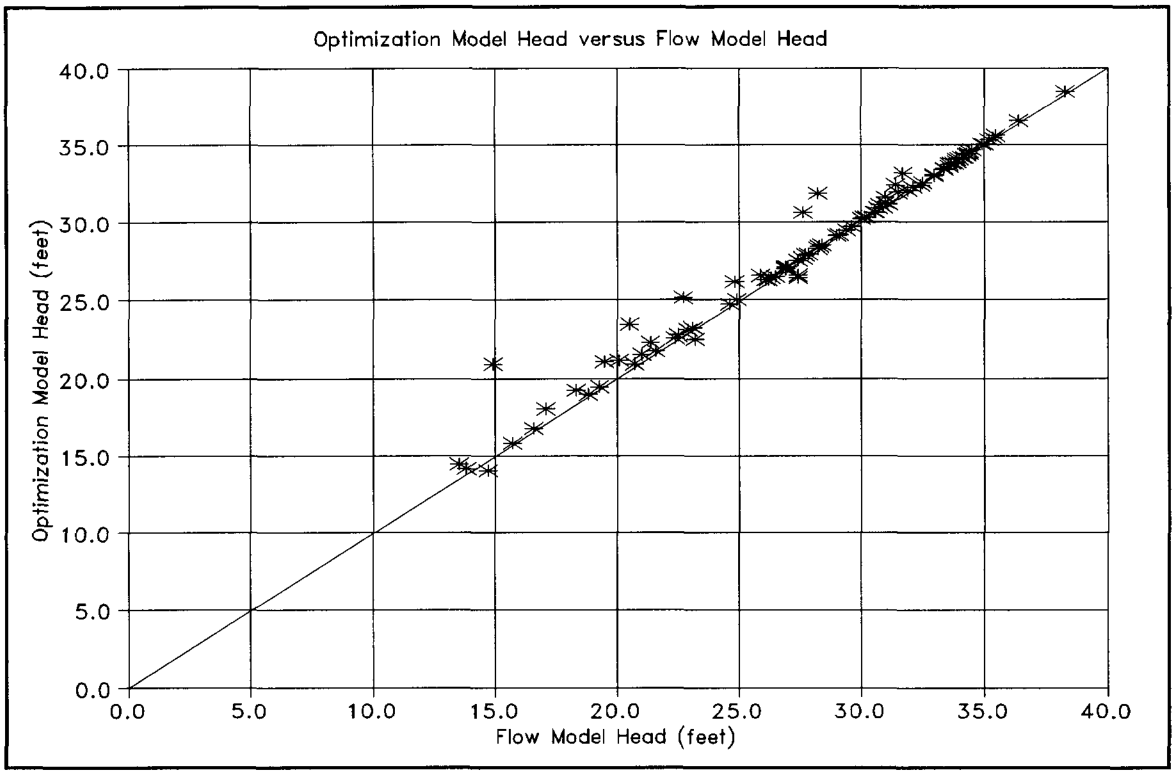
Pressure head and drawdown predictions for the revised optimization model were plotted against those of the simulation model as shown in Figures 5.3a and 5.3b, respectively. The coefficients of determination for the revised model 1 were 0.978 for head comparisons and 0.855 for the drawdown comparisons. This shows small prediction improvements for the revised model when compared to the initial formulation

Table 5.5 - Calculated pressure head and drawdown values while minimizing maximum drawdown at sensitive wetland control points (Results from Revised Model 1)

	Numerical Grid		Simulation Model		Optimization Model 1		Difference	
Control	Cell Location		head	drawdown	head	drawdown	head	drawdown
Point	row	column	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
1	5	1	28.4	1.31	28.50	1.21	-0.10	0.10
2	8	1	28.2	1.67	28.32	1.58	-0.12	0.09
3	15	1	26.9	2.86	27.03	2.77	-0.13	0.09
4	19	1	30.2	3.44	30.26	3.34	-0.06	0.10
5	22	1	31.0	3.68	31.20	3.50	-0.20	0.18
6	25	1	34.3	2.23	34.45	2.15	-0.15	0.08
7	28	1	33.0	1.35	33.03	1.27	-0.03	0.08
8	35	1	33.9	0.47	33.89	0.41	0.01	0.06
9	41	1	34.1	0.30	34.16	0.24	-0.06	0.06
10	47	1	34.2	0.27	34.19	0.22	0.01	0.05
11	53	1	34.4	0.32	34.54	0.26	-0.14	0.06
12	18	2	26.9	5.19	27.12	4.98	-0.22	0.21
13	56	2	35.5	0.40	35.57	0.33	-0.07	0.07
14	16	3	24.9	5.05	25.01	4.89	-0.11	0.16
15	30	3	34.0	1.32	34.08	1.23	-0.08	0.09
16	32	3	34.3	0.98	34.40	0.90	-0.10	0.08
17	37	3	35.2	0.53	35.34	0.46	-0.14	0.07
18	54	3	34.0	0.36	34.12	0.28	-0.12	0.08
19	17	4	22.7	10.42	25.18	7.92	-2.48	2.50
20	41	5	34.4	0.47	34.43	0.37	-0.03	0.10
21	44	5	33.9	0.39	34.00	0.30	-0.10	0.09
22	47	5	35.4	0.34	35.45	0.25	-0.05	0.09
23	53	5	35.0	0.32	35.06	0.24	-0.06	0.08
24	30	6	34.3	1.25	34.45	1.15	-0.15	0.10
25	32	6	34.9	0.97	35.03	0.88	-0.13	0.09
26	50	6	36.4	0.32	36.57	0.23	-0.17	0.09
27	59	7	33.8	0.67	33.85	0.65	-0.05	0.02
28	58	8	33.9	0.64	33.90	0.60	0.00	0.04
29	39	9	32.5	0.57	32.53	0.47	-0.03	0.10
30	60	9	33.0	1.08	33.01	1.09	-0.01	-0.01
31	16	10	27.6	3.58	27.71	3.49	-0.11	0.09
32	32	10	33.6	0.94	33.76	0.84	-0.16	0.10
33	14	11	27.8	2.84	27.98	2.72	-0.18	0.12
34	34	11	30.9	0.83	30.97	0.73	-0.07	0.10
35	52	11	33.6	0.28	33.62	0.18	-0.02	0.10
36	54	11	33.7	0.29	33.79	0.21	-0.09	0.08
37	59	11	32.4	1.68	32.39	1.71	0.01	-0.03
38	17	12	30.7	2.46	30.87	2.33	-0.17	0.13
39	31	12	30.5	1.02	30.59	0.91	-0.09	0.11
40	37	12	30.6	0.77	30.63	0.67	-0.03	0.10
41	39	12	33.0	0.67	33.04	0.56	-0.04	0.11
42	49	12	31.9	0.28	32.03	0.17	-0.13	0.11
43	43	13	30.5	0.48	30.64	0.36	-0.14	0.12
44	46	13	30.7	0.34	30.88	0.22	-0.18	0.12
45	16	14	28.3	1.92	28.40	1.80	-0.10	0.12
46	33	14	30.1	1.07	30.25	0.95	-0.15	0.12
47	54	14	33.4	0.28	33.40	0.20	0.00	0.08
48	59	14	27.4	6.35	26.45	7.25	0.95	-0.90
49	14	15	26.2	1.57	26.34	1.46	-0.14	0.11
50	41	15	30.0	0.79	30.23	0.57	-0.23	0.22
51	5	16	27.0	0.89	27.13	0.77	-0.13	0.12
52	12	16	29.6	1.29	29.73	1.17	-0.13	0.12
53	38	16	25.9	4.04	26.56	3.34	-0.66	0.70

Table 5.5 - continued

	Numerical Grid		Simulation Model		Optimization Model 1		Difference	
Control	Cell Location		head	drawdown	head	drawdown	head	drawdown
Point	row	column	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
54	47	16	34.5	0.18	34.68	0.02	-0.18	0.16
55	50	16	31.0	0.12	31.57	-0.47	-0.57	0.59
56	53	16	33.4	0.21	33.51	0.09	-0.11	0.12
57	60	16	27.4	4.64	26.62	5.38	0.78	-0.74
58	3	17	22.9	0.85	23.08	0.72	-0.18	0.13
59	7	17	24.6	0.91	24.71	0.79	-0.11	0.12
60	10	17	26.3	1.00	26.42	0.89	-0.12	0.11
61	14	17	29.4	1.18	29.43	1.07	-0.03	0.11
62	30	17	27.6	3.02	30.60	0.00	-3.00	3.02
63	35	17	23.1	6.60	23.27	6.43	-0.17	0.17
64	44	17	30.8	0.53	31.06	0.34	-0.26	0.19
65	55	17	32.1	0.36	32.23	0.27	-0.13	0.09
66	57	17	31.5	1.44	31.88	1.02	-0.38	0.42
67	27	18	31.1	1.82	31.18	1.72	-0.08	0.10
68	32	18	23.2	6.74	22.46	7.44	0.74	-0.70
69	37	18	17.1	8.79	17.98	7.92	-0.88	0.87
70	40	18	18.3	6.69	19.21	5.79	-0.91	0.90
71	42	18	21.4	3.97	22.28	3.02	-0.88	0.95
72	4	19	27.7	0.75	27.89	0.61	-0.19	0.14
73	19	19	26.5	1.37	26.55	1.25	-0.05	0.12
74	6	20	29.0	0.69	29.12	0.58	-0.12	0.11
75	9	20	29.1	0.75	29.17	0.63	-0.07	0.12
76	14	20	27.4	0.95	27.56	0.84	-0.16	0.11
77	17	20	26.9	1.21	27.12	1.08	-0.22	0.13
78	12	21	26.9	0.87	27.05	0.75	-0.15	0.12
79	10	22	38.3	0.99	38.48	0.82	-0.18	0.17
80	44	22	28.2	1.41	31.83	-2.23	-3.63	3.64
81	48	22	24.8	0.58	26.17	-0.77	-1.37	1.35
82	51	22	31.7	0.10	33.11	-1.31	-1.41	1.41
83	46	23	31.4	0.99	32.42	-0.02	-1.02	1.01
84	43	24	20.1	2.93	21.13	1.87	-1.03	1.06
85	20	25	26.1	1.58	26.28	1.42	-0.18	0.16
86	26	25	19.5	6.56	21.05	5.06	-1.55	1.50
87	30	25	15.7	7.19	15.80	7.10	-0.10	0.09
88	32	25	16.6	8.10	16.78	7.92	-0.18	0.18
89	7	26	19.3	0.52	19.40	0.40	-0.10	0.12
90	10	26	18.8	0.59	18.93	0.47	-0.13	0.12
91	14	26	21.6	0.77	21.75	0.66	-0.15	0.11
92	17	26	22.4	0.97	22.55	0.85	-0.15	0.12
93	23	26	20.5	5.32	23.45	2.35	-2.95	2.97
94	29	26	14.9	6.00	20.93	-0.03	-6.03	6.03
95	27	27	22.5	2.28	22.79	2.02	-0.29	0.26
96	31	27	14.7	6.28	14.01	6.99	0.69	-0.71
97	35	27	13.5	6.39	14.41	5.49	-0.91	0.90
98	33	28	13.8	5.78	14.18	5.42	-0.38	0.36
99	37	28	21.0	1.31	21.51	0.79	-0.51	0.52
100	39	29	20.7	0.86	20.90	0.70	-0.20	0.16
Maximum			38.30	10.42	38.48	7.92	0.95	6.03
Minimum			13.50	0.10	14.01	-2.23	-6.03	-0.90
Average			28.24	2.07	28.61	1.71	-0.36	0.36
Standard Deviation			5.84	2.29	5.62	2.21	0.88	0.88
Variance			34.05	5.25	31.54	4.88	0.77	0.77



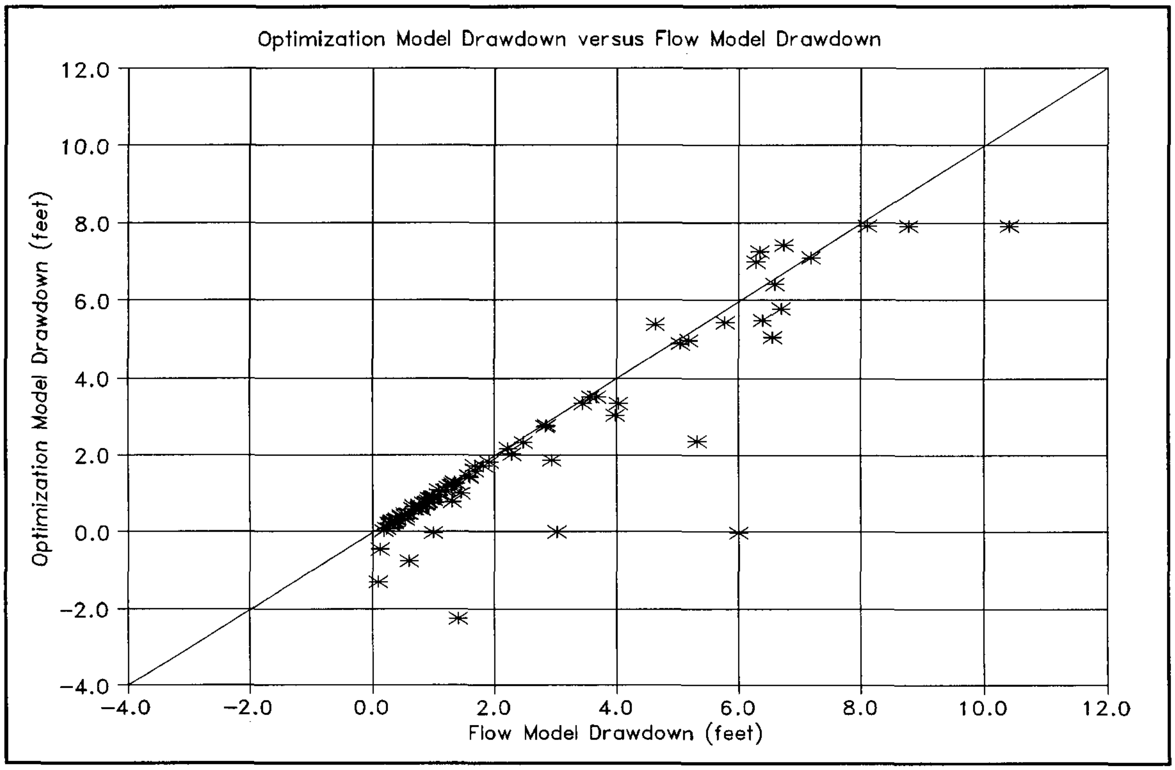


Figure 5.3 - Correlation between simulation model results and revised optimization model 1 results.
a) Pressure heads for year 2010; b) Drawdowns from year 1988 to 2010.

results. For both head and drawdown, mean difference and standard deviation between the two models decreased by 50 percent (0.35) and 41 percent (0.62 feet), respectively. Similar to the initial formulation, points which deviate from the "ideal" line generally fall on the "non-conservative" side. Again, this is due to the non-linear characteristics of the aquifer system. This phenomena was also found to hold for models 2, 3, 4, and 5.

Figures 5.4 through 5.11 depict the final optimum water allocation strategy for the eight municipal water service areas as predicted by revised model 1. Each figure describes the well grid locations and corresponding year 1988, projected year 2010, and optimized year 2010 water allocation strategies. The final optimum water allocation strategy for the nine agricultural areas are shown in Figures 5.12 through 5.14. Along with the agricultural well grid cell locations and withdrawal rates for years 1988 and 2010, these figures include the locations of wastewater treatment plants which supplement the agricultural water demand.

Although the water management model incorporates controls for aquifer responses at over 100 specified locations, the entire project area is affected by changes in allocation strategy. Figures 5.15 and 5.16 display the predicted pressure head elevations and drawdown in the surficial aquifer, respectively, when the optimum strategy is simulated by MODFLOW.

5.6.2 <u>Model 2</u>

The objective incorporated into the optimization model 2 was to maximize the minimum pressure head at all specified control points designated as having a high potential for vegetative harm. Determining an optimum strategy with this objective is different than the first in that absolute water table elevations are considered rather than

WELL FLOW RATES (CF/D)

Well#	Q 1988	Q 2010	Q optimize
33	8289	14586	4145
34	16373	29171	77621
35	4093	7293	2047
36	12280	21878	6140
37	8186	14586	4093
38	4157	7293	2079

Total 2010 Projected Demand = 92032 (CF/D)

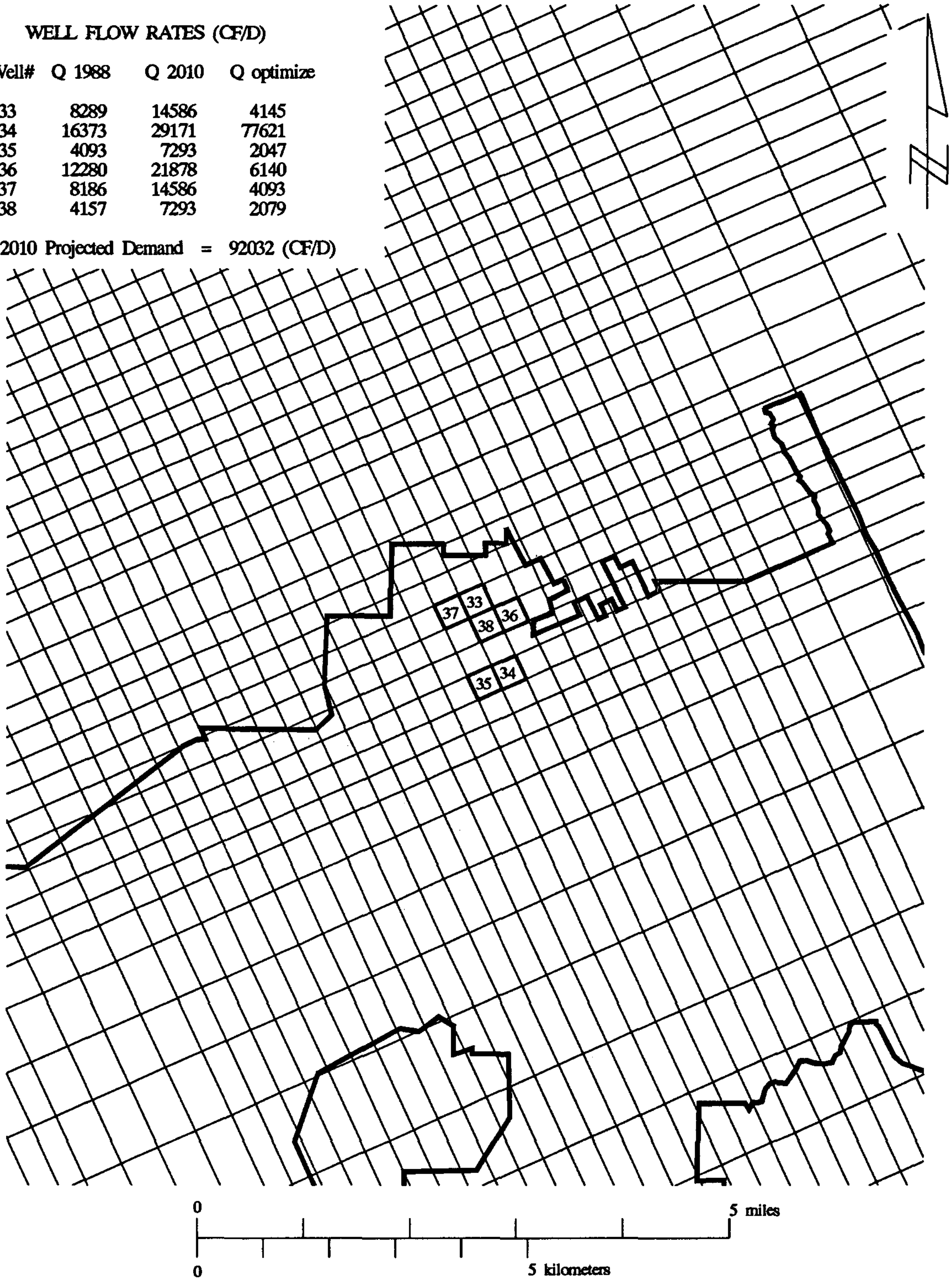


Figure 5.4

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for Port Orange East water service area.

WELL FLOW RATES (CF/D)

Well#	Q 1988	Q 2010	Q optimize
1-4	35475	31602	17738
5	70951	63204	35476
6,7	35475	31602	17738
8	70951	63204	35476
9,13	35475	31602	17738
14,15	0	94806	0
16-19	0	0	0
20	0	0	219952
21,22	0	0	0
23	0	0	475183
24	0	0	0
25-32	0	31602	0

Total 2010 Projected Demand = 819301 (CF/D)

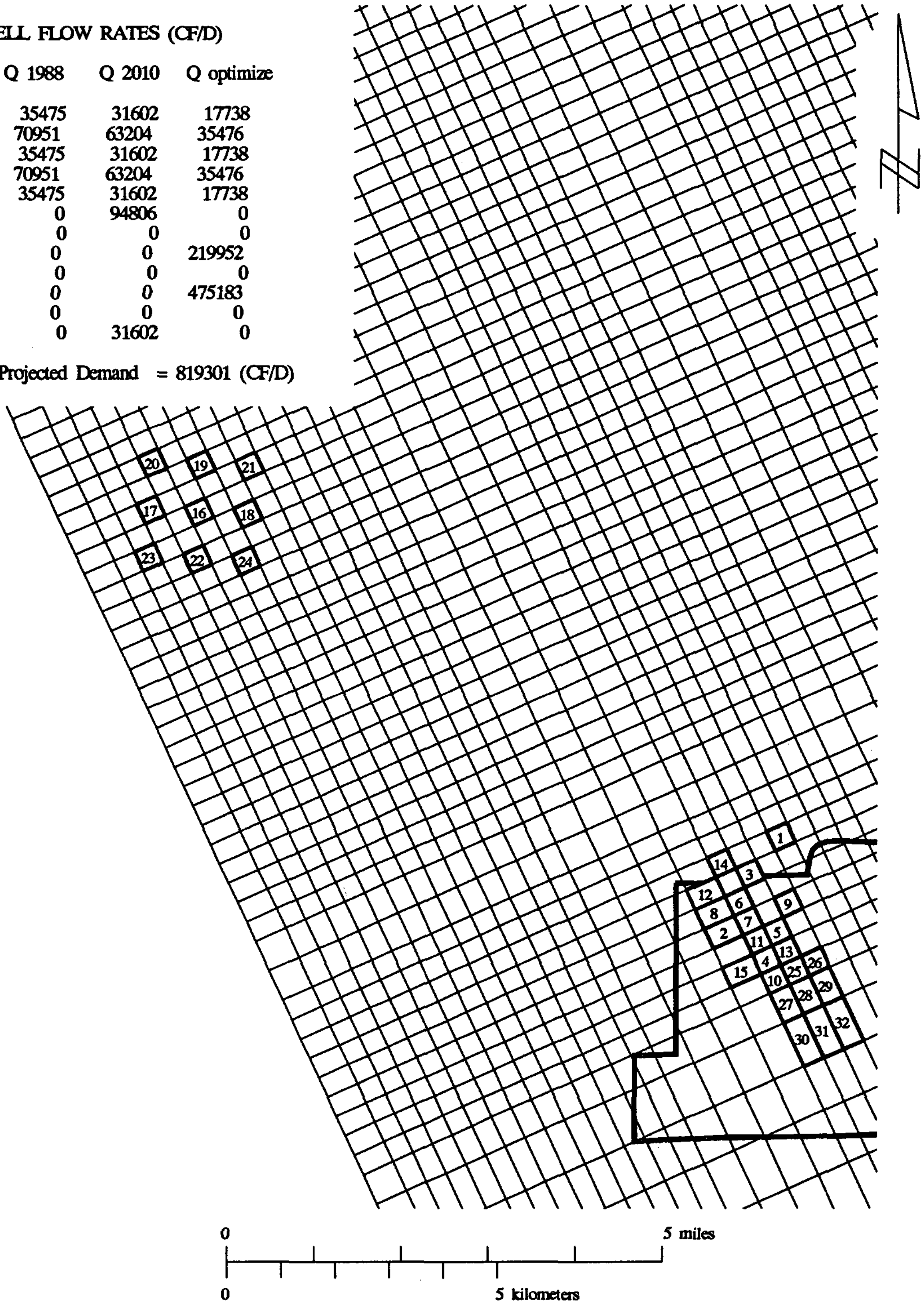


Figure 5.5

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for Port Orange West water service area.

WELL FLOW RATES (CF/D)

Well#	Q-1988	Q-2010	Q-optimize
39	78522	40987	50673
40	78522	40987	39261
41	157043	81973	78522
42	78522	40987	39325
43,44	124803	102247	62555
45	124803	102247	62504
46	124803	102247	515859
47-51	124803	102247	62402
52	249708	204493	124854
53	0	102247	0
54	0	204493	0
55	0	102247	0
56	0	204493	0
57	0	102247	0
58	0	102247	574408
59	0	102247	380593
60	0	102247	0
61	0	204493	0
62,63	0	102247	0
64	0	102247	600000

Total 2010 Projected Demand = 2903119 (CF/D)

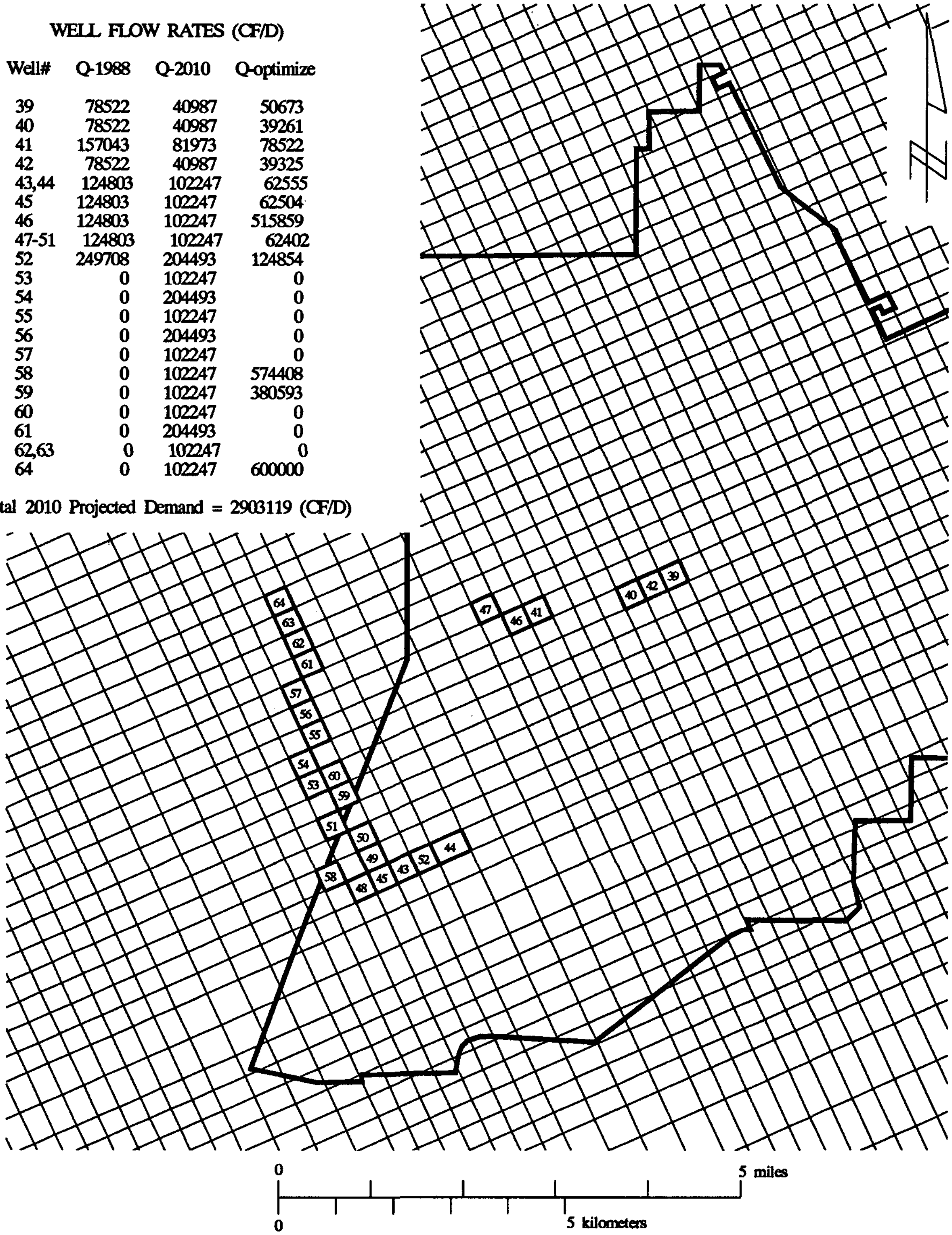


Figure 5.6

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for Daytona Beach water service area.

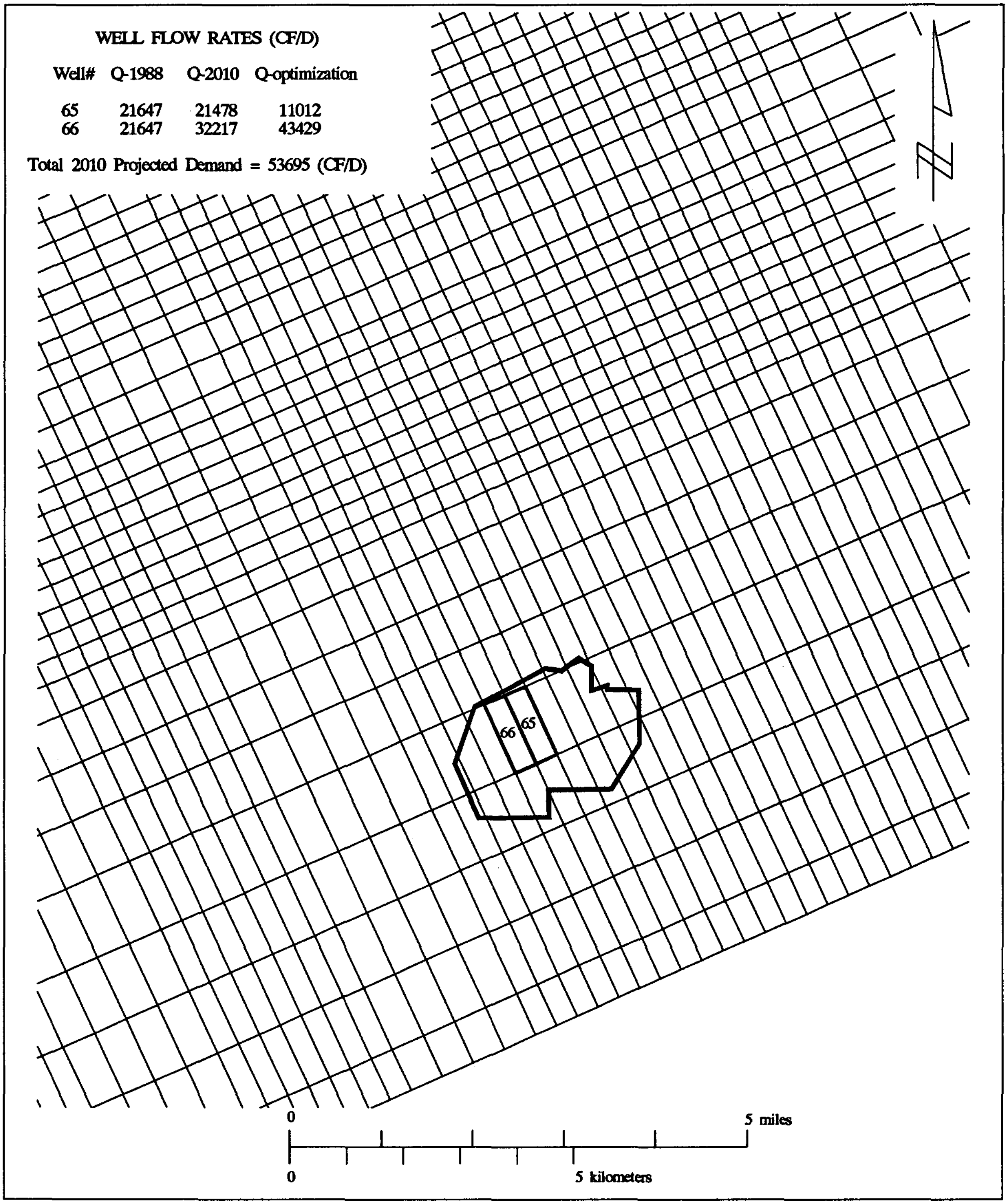
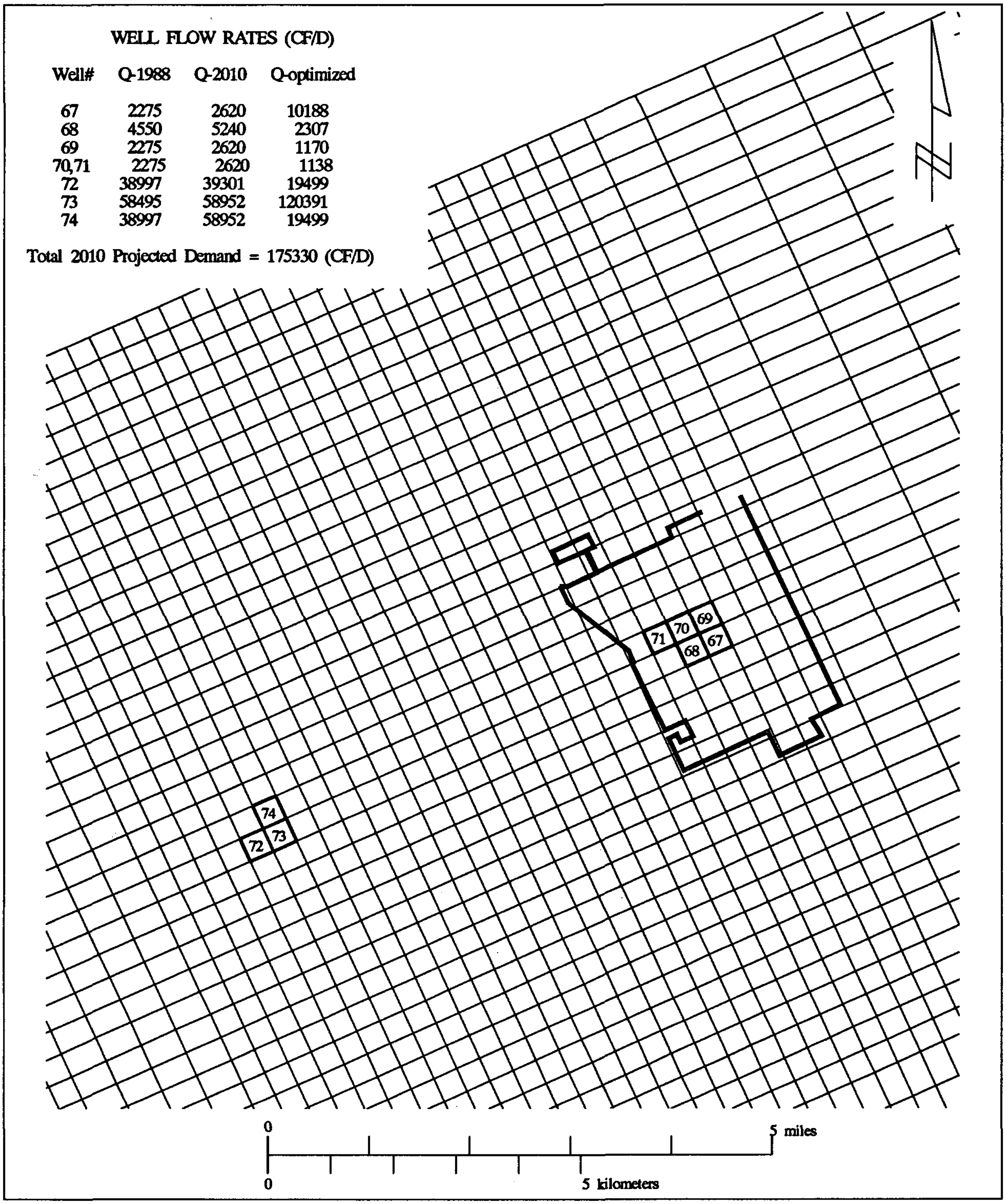


Figure 5.7
Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for Spruce Creek water service area.



Well#	Q-1988	Q-2010	Q-optimized
67	2275	2620	10188
68	4550	5240	2307
69	2275	2620	1170
70,71	2275	2620	1138
72	38997	39301	19499
73	58495	58952	120391
74	38997	58952	19499

Figure 5.8

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for Holy Hills water service area.

WELL FLOW RATES (CF/D)

well#	Q-1988	Q-2010	Q-Optimized
75	79425	100031	39713
76	0	50016	0
77	0	50016	365970
78	0	200062	0

Total 2010 Projected Demand = 405683 (CF/D)

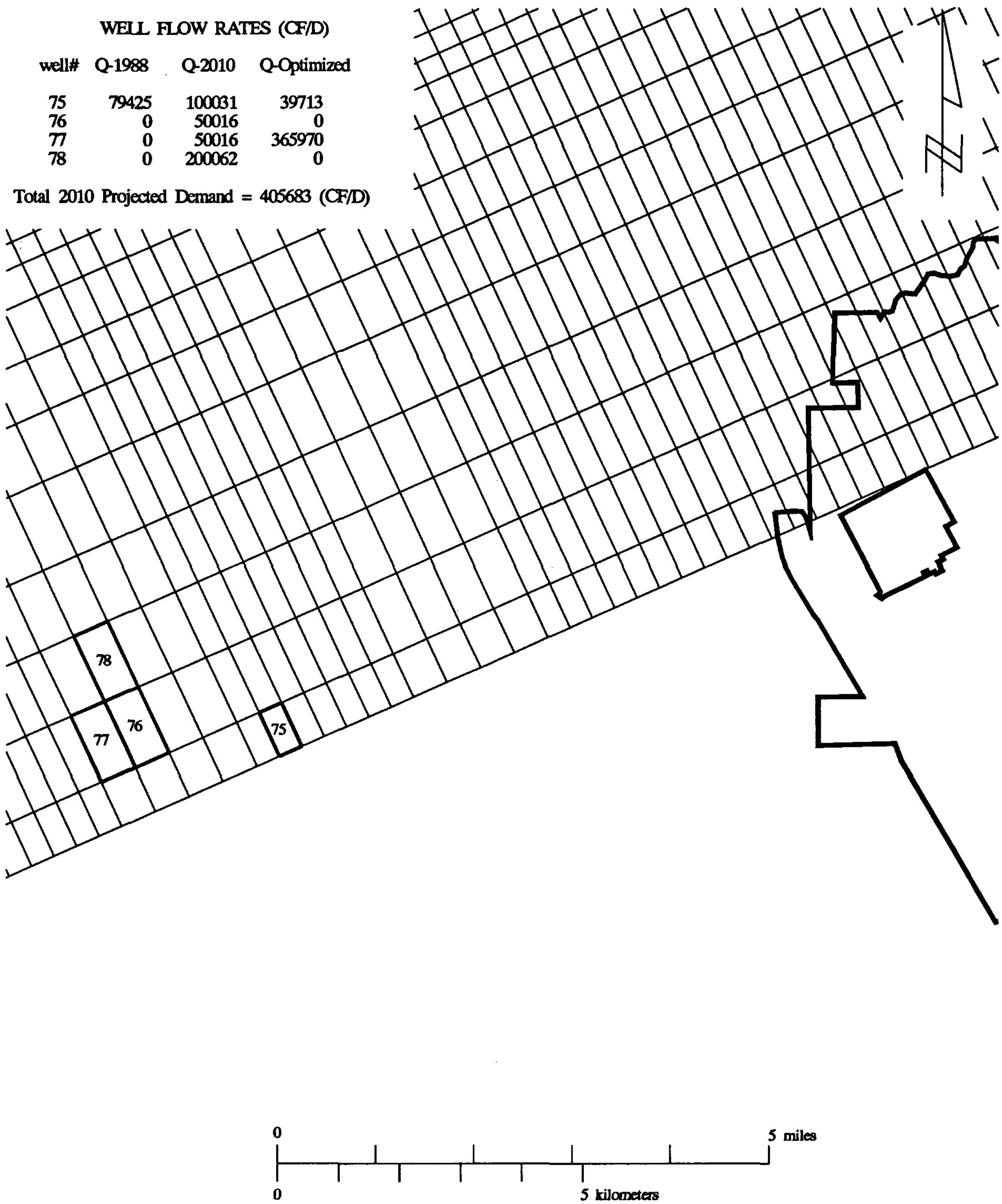


Figure 5.9

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for New Smyrna Beach water service area.

WELL FLOW RATES (CF/D)

Well#	Q-1988	Q-2010	Q-Optimized
79	60325	43356	30163
80	90448	65097	45276
81	120650	86775	60357
82	30163	21678	97693
83	30163	21678	15114
84	30163	21678	15145
85	84902	21678	42503
86	84902	21678	42535
87	84902	21678	42451
88,89	0	21678	0
90-93	0	43356	0
94-98	0	21678	0
99	0	21678	600000
100	0	43356	173648
101,02	0	21678	0
103	0	43356	0
104-09	0	21678	0
110	0	43356	0
111-18	0	21678	0

Total 2010 Projected Demand = 1148934 (CF/D)

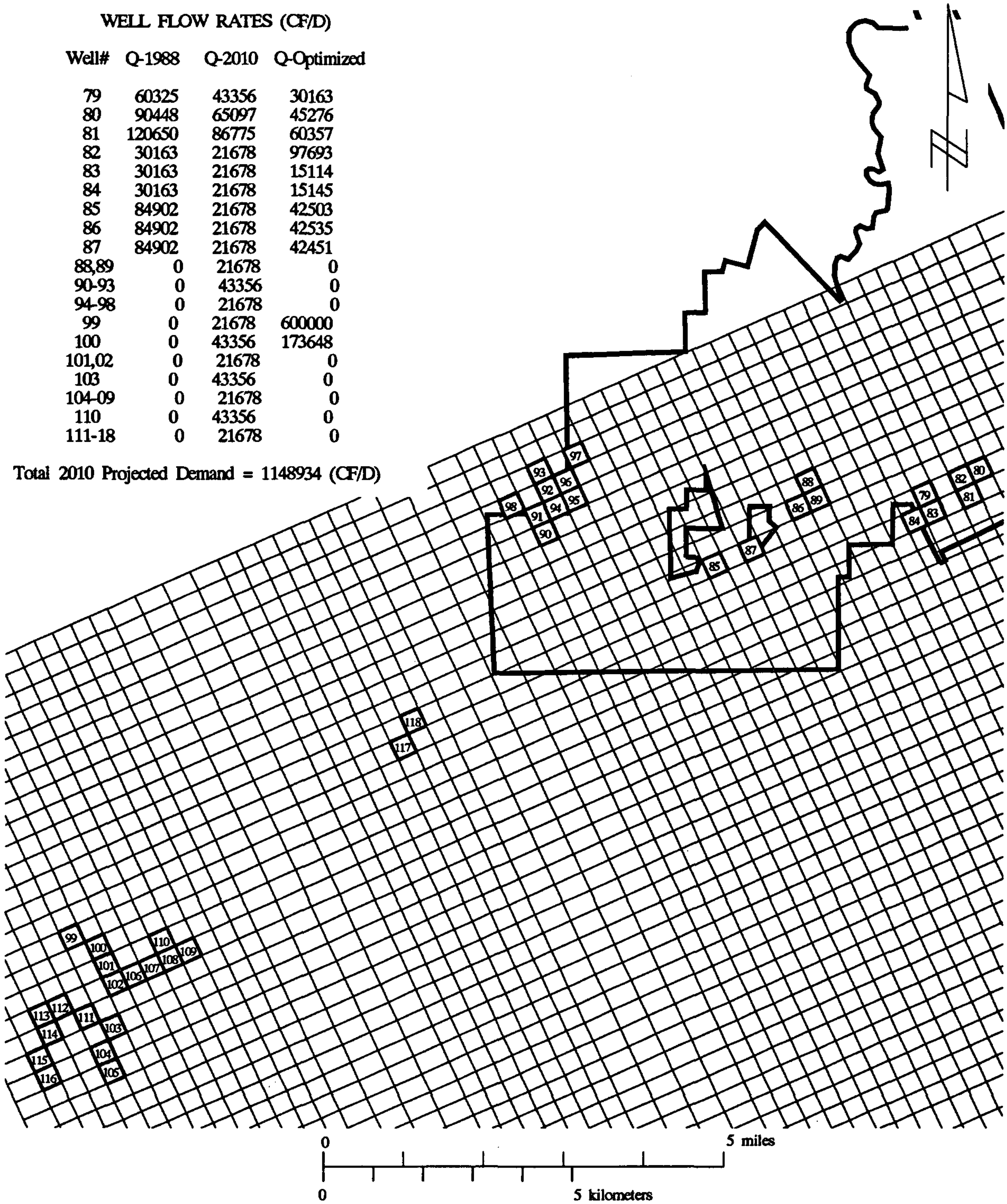


Figure 5.10

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for Ormond Beach water service area.

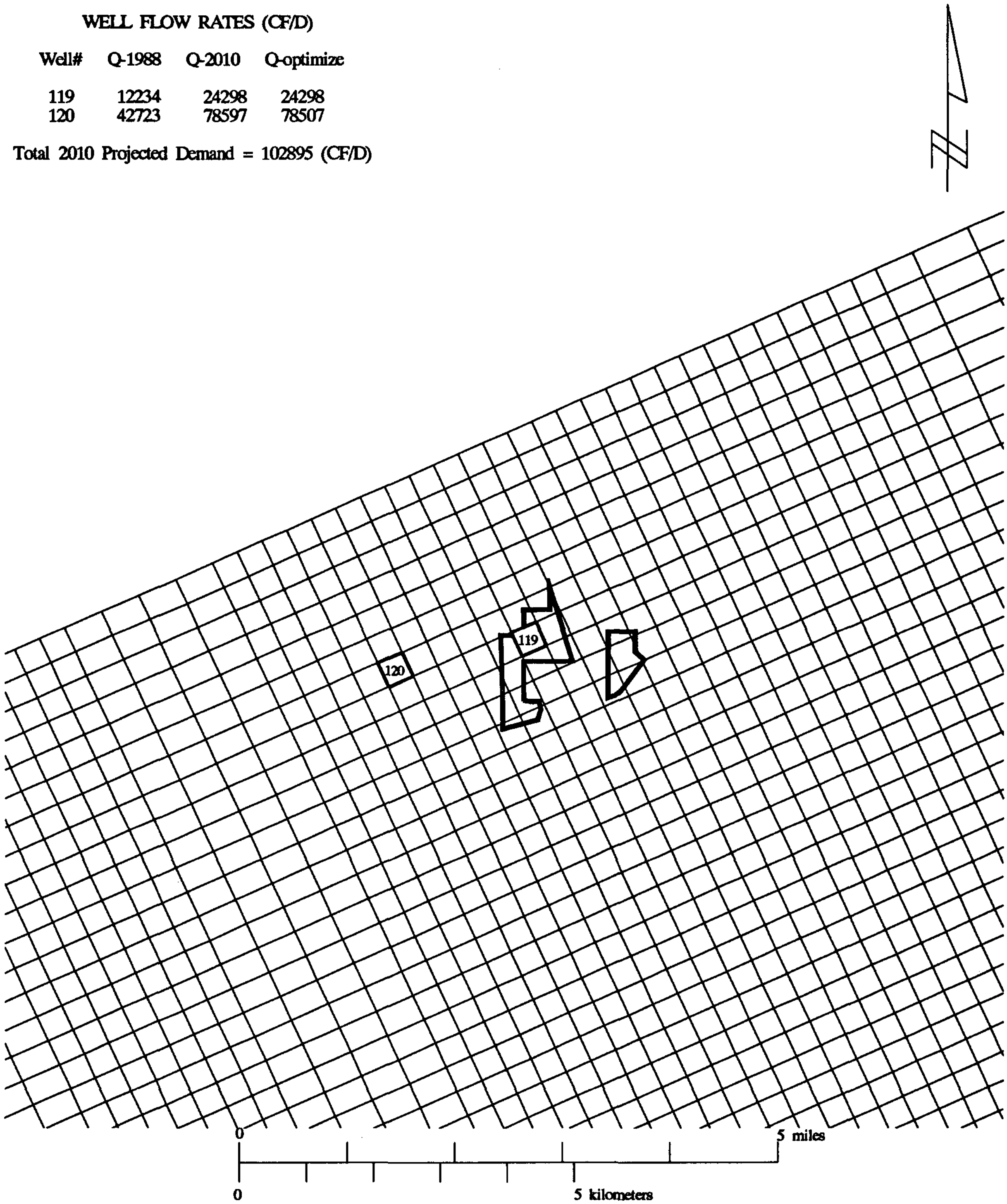


Figure 5.11

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal strategies for Tymber Creek & the Trails water service areas.

WELL FLOW RATES (CF/D)

Well#	Q-1988	Q-2010	Q-optimize
1	3640	3640	0
2	5460	5460	0
6	5624	5624	0
7	6093	6093	0
26	806	806	0
27	806	806	0
28	4033	4033	0

WTP#	Q-1988	Q-2010	Q-optimize
1	0	0	21105
3	0	0	5725

Total 2010 projected Demand = 26462 (CF/D)

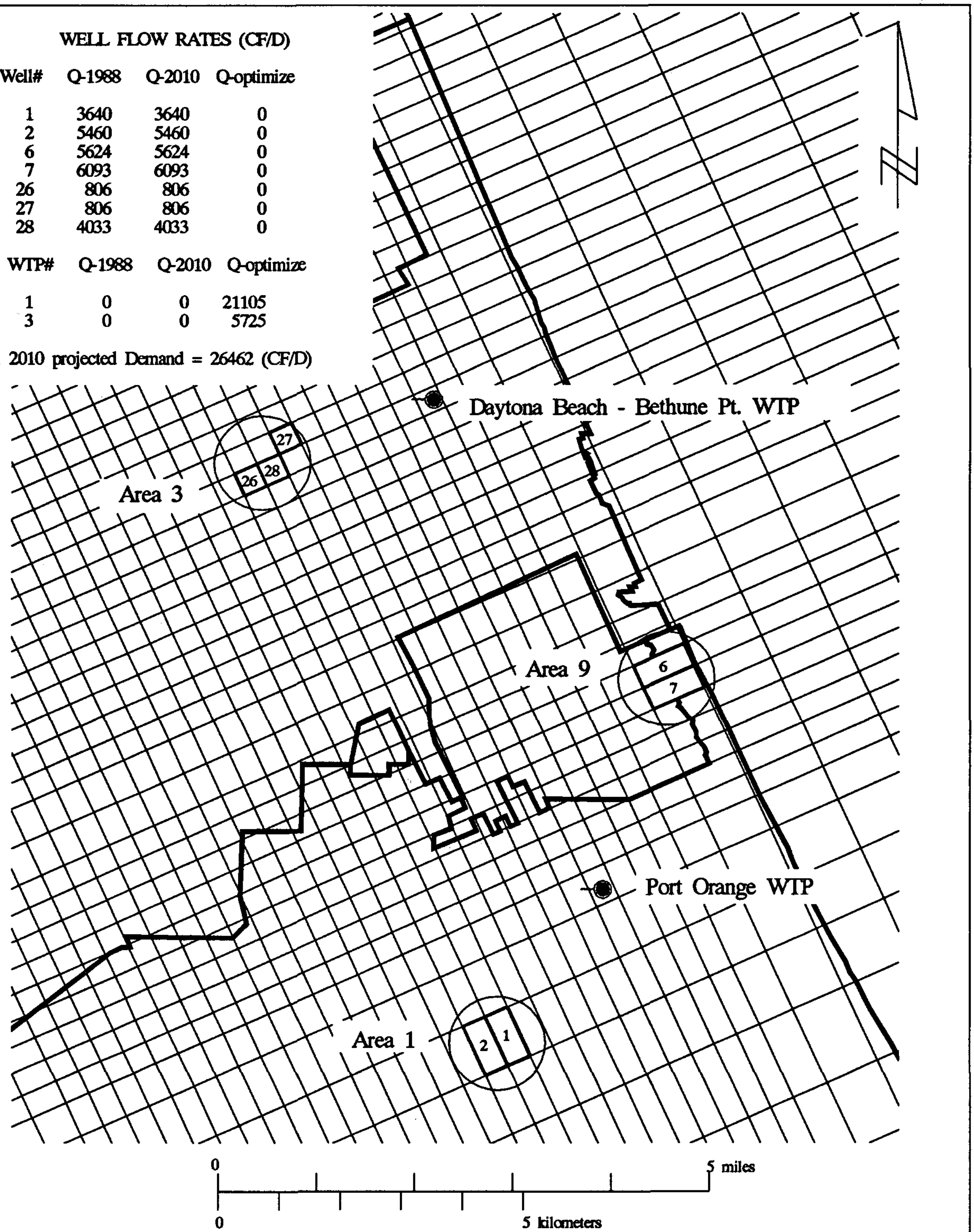


Figure 5.12

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal stratagies for agricultural water service areas 1, 3, and 9.

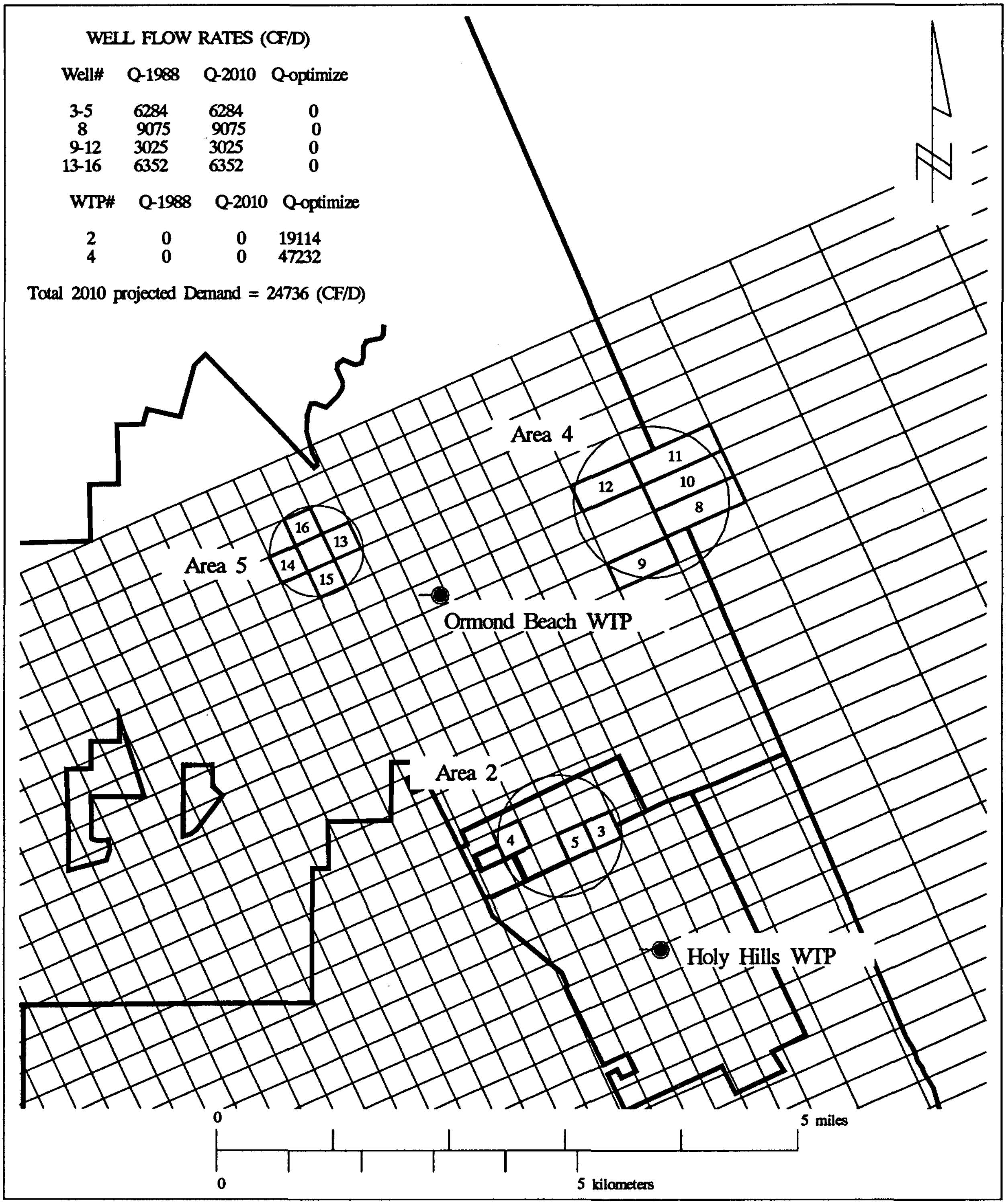


Figure 5.13

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal stratagies for agricultural water service areas 2, 4, and 5.

WELL FLOW RATES (CF/D)

Well#	Q-1988	Q-2010	Q-optimize
17	9952	9952	0
18	29856	29856	0
19	28191	28191	85747
20-21	28191	28191	0
22	4644	4644	0
23	4644	4644	97155
24	27865	27865	0
25	65017	65017	0

WTP#	Q-1988	Q-2010	Q-optimize
5	0	0	46795

Total 2010 Project Demand = 226551 (CF/D)

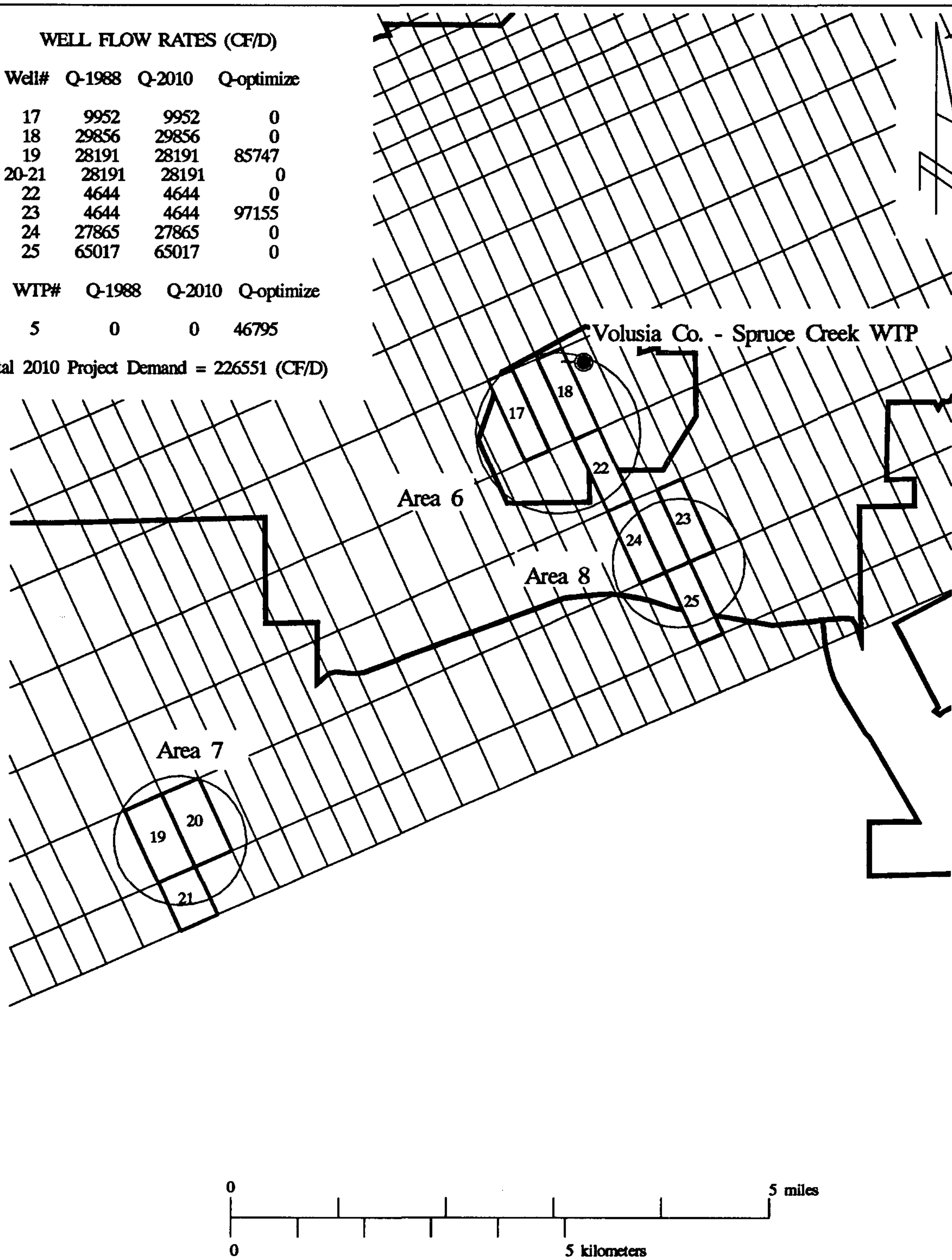


Figure 5.14

Year 1988, Projected year 2010, and Model 1 Optimized year 2010 withdrawal stratagies for agricultural water service areas 6, 7, and 8.

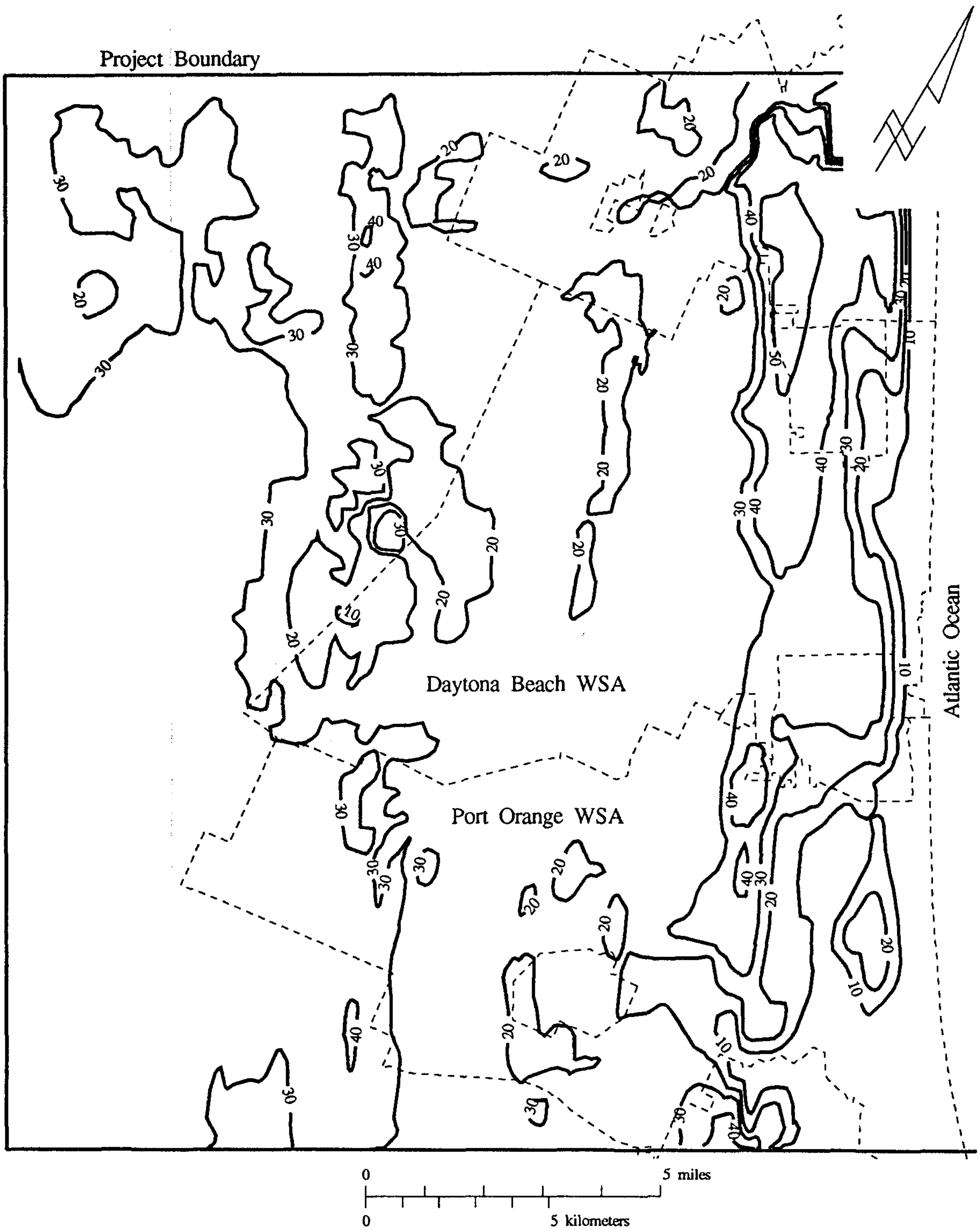


Figure 5.15

Flow simulation model pressure head results in the surficial aquifer due to Optimization Model 1 withdrawal strategy for year 2010.

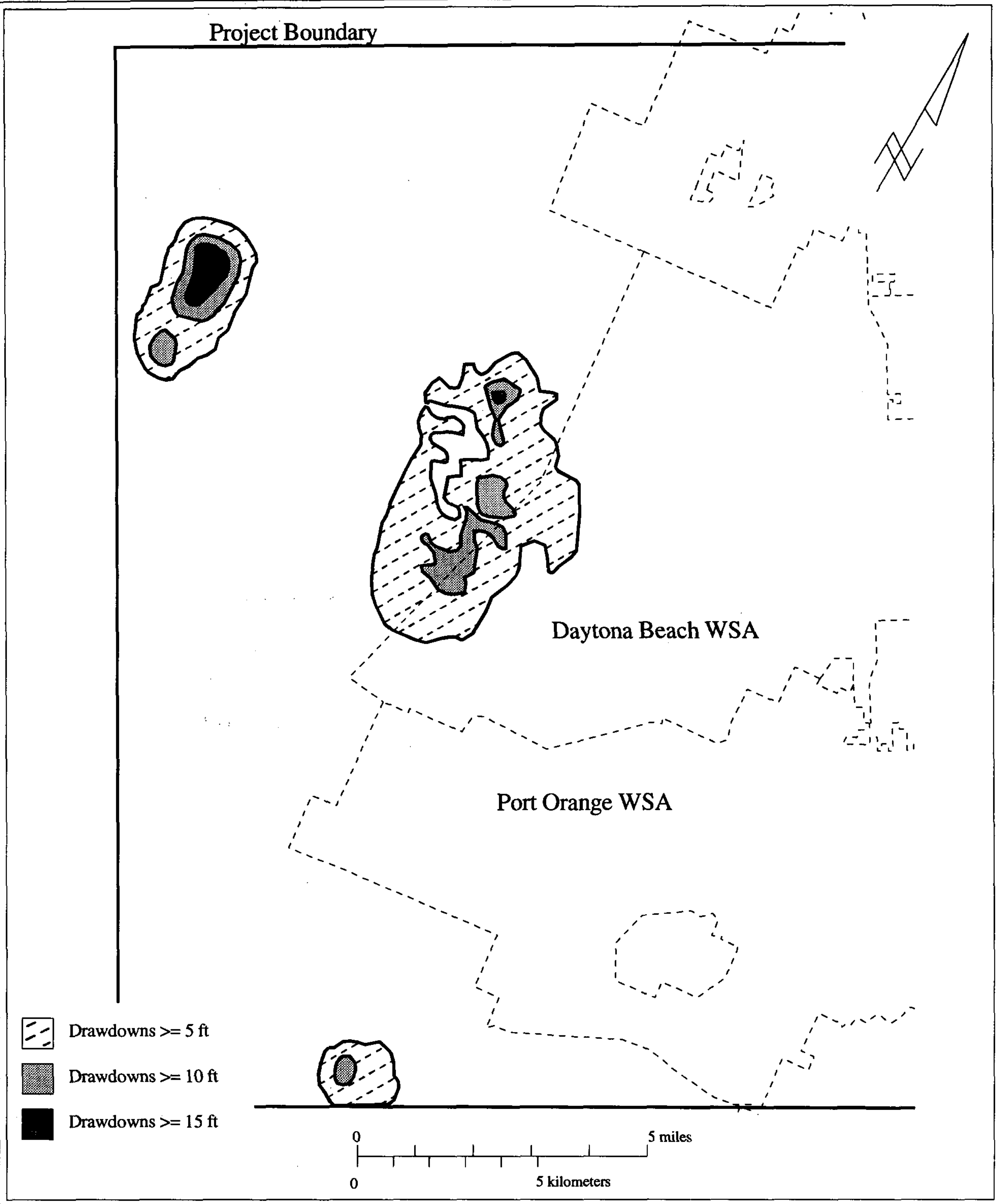


Figure 5.16

Flow simulation model drawdown results in the surficial aquifer due to Optimization Model 1 withdrawal strategy for year 2010.

deviations of water table elevations. The objective is to determine a water allocation strategy which produces a water table with a low point that is high as possible.

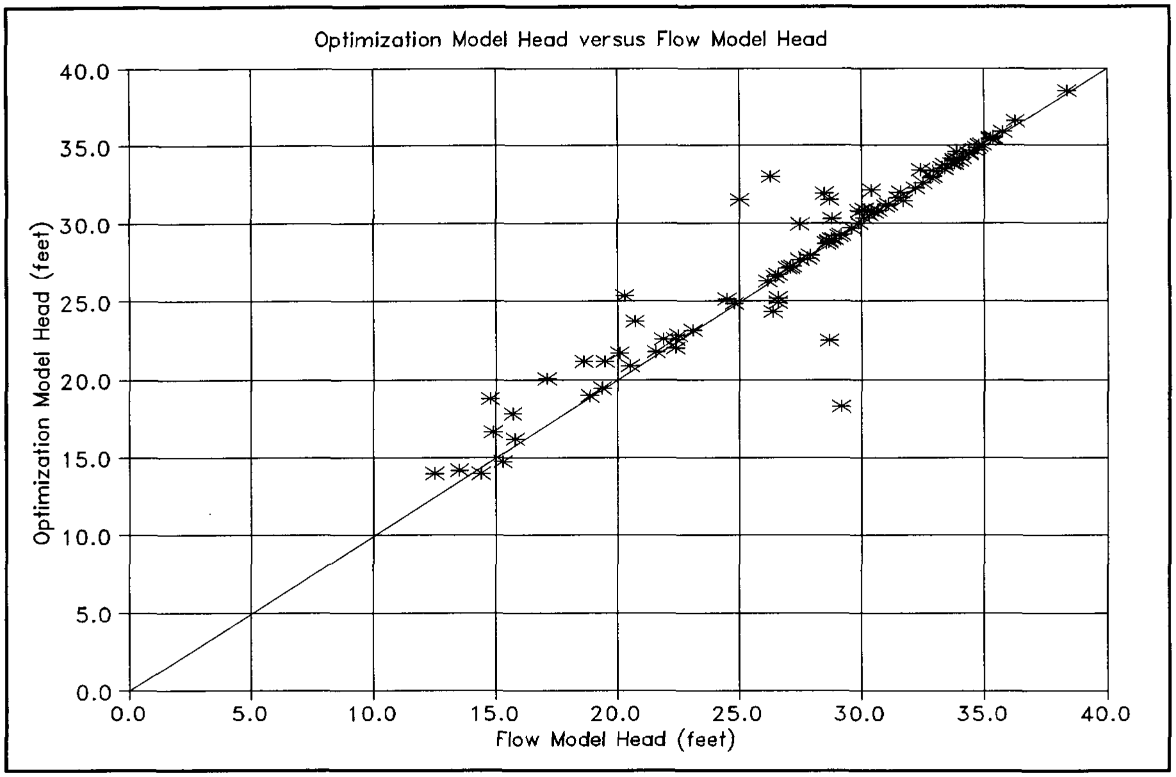
The optimum water allocation strategy as predicted by the model is shown in Tables 5.2 and 5.3. Table 5.6 displays the pressure heads and drawdowns determined by both the optimization and simulation models. The optimization model predicted a minimum pressure head of 14.0 feet while the simulation model predicted a value of 12.5 feet. Pressure heads and drawdowns predicted by the optimization model were plotted against those of the simulation model as shown in Figures 5.17a and 5.17b, respectively. The coefficients of determination for these plots of optimization model 2 versus simulation model were 0.960 for pressure head and 0.847 for drawdown. A mean difference of 0.41 feet with a standard deviation of 1.88 feet was determined to exist between predicted head and drawdown responses. Figures 5.18 and 5.19 display the predicted pressure head elevations and drawdown values, respectively, when the optimum strategy is incorporated in the ground water flow simulation model.

5.6.3 Model 3

The objective function of optimization model 3 was to minimize the average drawdown at all specified control points designated as having a high potential for vegetative harm. The average drawdown equals the sum of all drawdown values over all control points divided by the number of control points. This is the same as determining a water allocation strategy while minimizing total drawdown or maximizing total or average head at specified control points. Any of these objectives produces an identical water allocation strategy and therefore identical aquifer response. This objective is different than the previous two objectives in that all control points are given equal

Table 5.6 - Calculated pressure head and drawdown values while maximizing minimum head at sensitive wetland control points (Results from revised Model 2).

	Numerical Grid		Simulation Model		Optimization Model 2		Difference	
Control	Cell Location		head	drawdown	head	drawdown	head	drawdown
Point	row	column	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
1	5	1	28.7	1.00	28.76	0.94	-0.06	0.06
2	8	1	28.6	1.25	28.69	1.21	-0.09	0.04
3	15	1	27.8	2.05	27.76	2.04	0.04	0.01
4	19	1	31.7	1.94	31.45	2.15	0.25	-0.21
5	22	1	33.4	1.26	33.46	1.24	-0.06	0.02
6	25	1	35.8	0.81	35.85	0.75	-0.05	0.06
7	28	1	33.8	0.57	33.79	0.51	0.01	0.06
8	35	1	34.0	0.31	34.05	0.25	-0.05	0.06
9	41	1	34.1	0.28	34.19	0.21	-0.09	0.07
10	47	1	34.1	0.31	34.18	0.22	-0.08	0.09
11	53	1	34.4	0.38	34.52	0.28	-0.12	0.10
12	18	2	28.7	3.34	22.47	9.63	6.23	-6.29
13	56	2	35.4	0.49	35.53	0.37	-0.13	0.12
14	16	3	26.4	3.56	24.31	5.59	2.09	-2.03
15	30	3	34.6	0.66	34.72	0.58	-0.12	0.08
16	32	3	34.7	0.56	34.82	0.48	-0.12	0.08
17	37	3	35.4	0.41	35.47	0.33	-0.07	0.08
18	54	3	33.9	0.46	34.09	0.31	-0.19	0.15
19	17	4	29.2	3.98	18.29	14.81	10.91	-10.83
20	41	5	34.4	0.47	34.47	0.34	-0.07	0.13
21	44	5	33.8	0.45	34.00	0.30	-0.20	0.15
22	47	5	35.3	0.43	35.43	0.27	-0.13	0.16
23	53	5	34.9	0.44	35.03	0.27	-0.13	0.17
24	30	6	34.8	0.70	35.00	0.60	-0.20	0.10
25	32	6	35.3	0.61	35.39	0.51	-0.09	0.10
26	50	6	36.3	0.45	36.54	0.26	-0.24	0.19
27	59	7	33.7	0.78	33.79	0.71	-0.09	0.07
28	58	8	33.8	0.76	33.84	0.66	-0.04	0.10
29	39	9	32.5	0.57	32.57	0.43	-0.07	0.14
30	60	9	32.8	1.22	32.91	1.19	-0.11	0.03
31	16	10	28.9	2.37	28.98	2.22	-0.08	0.15
32	32	10	33.8	0.72	33.99	0.61	-0.19	0.11
33	14	11	28.7	2.00	28.82	1.88	-0.12	0.12
34	34	11	31.0	0.74	31.10	0.60	-0.10	0.14
35	52	11	33.3	0.58	33.57	0.23	-0.27	0.35
36	54	11	33.4	0.56	33.74	0.26	-0.34	0.30
37	59	11	32.2	1.86	32.26	1.84	-0.06	0.02
38	17	12	31.5	1.70	31.66	1.54	-0.16	0.16
39	31	12	30.7	0.84	30.79	0.71	-0.09	0.13
40	37	12	30.5	0.81	30.66	0.64	-0.16	0.17
41	39	12	32.9	0.76	33.03	0.57	-0.13	0.19
42	49	12	31.6	0.60	31.98	0.22	-0.38	0.38
43	43	13	30.3	0.73	30.58	0.42	-0.28	0.31
44	46	13	30.4	0.65	30.81	0.29	-0.41	0.36
45	16	14	28.8	1.38	28.95	1.25	-0.15	0.13
46	33	14	30.1	1.04	30.32	0.88	-0.22	0.16
47	54	14	32.9	0.78	33.34	0.26	-0.44	0.52
48	59	14	26.6	7.07	25.20	8.50	1.40	-1.43
49	14	15	26.6	1.17	26.73	1.07	-0.13	0.10
50	41	15	28.8	1.99	30.24	0.56	-1.44	1.43
51	5	16	27.2	0.71	27.27	0.63	-0.07	0.08
52	12	16	29.9	0.99	30.00	0.90	-0.10	0.09
53	38	16	24.5	5.39	25.17	4.73	-0.67	0.66



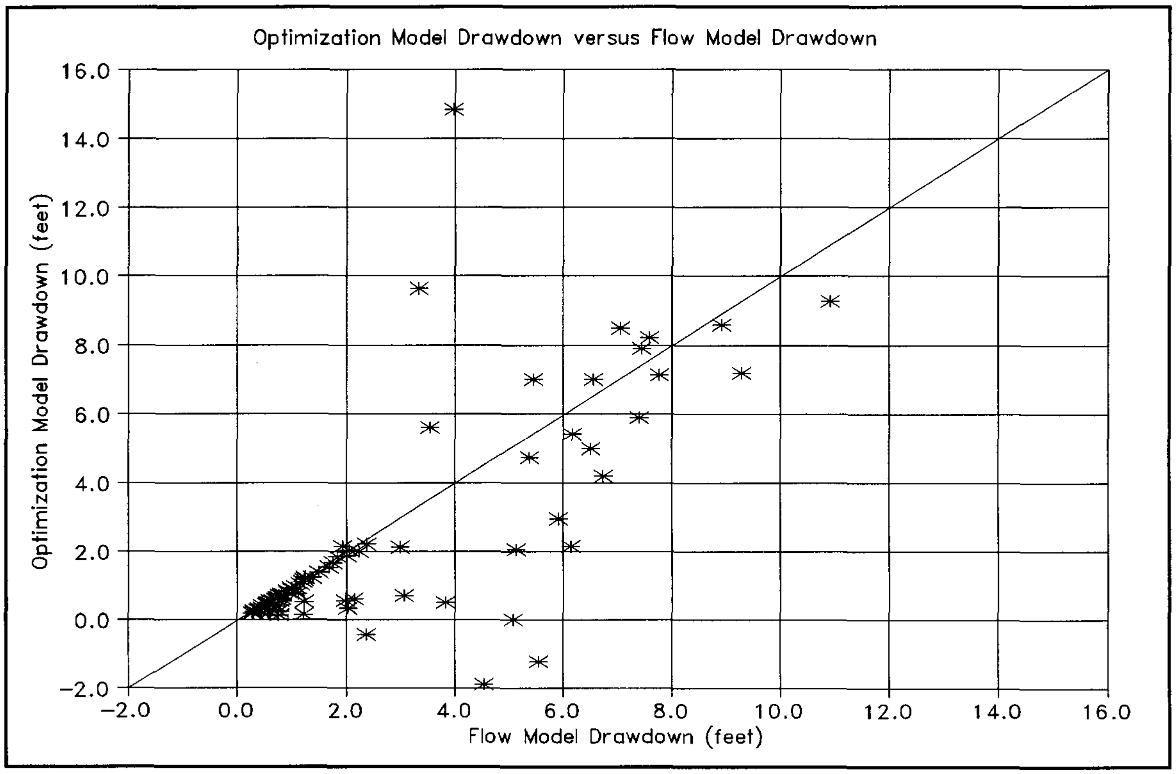


Figure 5.17 - Correlation between simulation model results and revised optimization model 2 results.
a) Pressure heads for year 2010; b) Drawdowns from year 1988 to 2010.

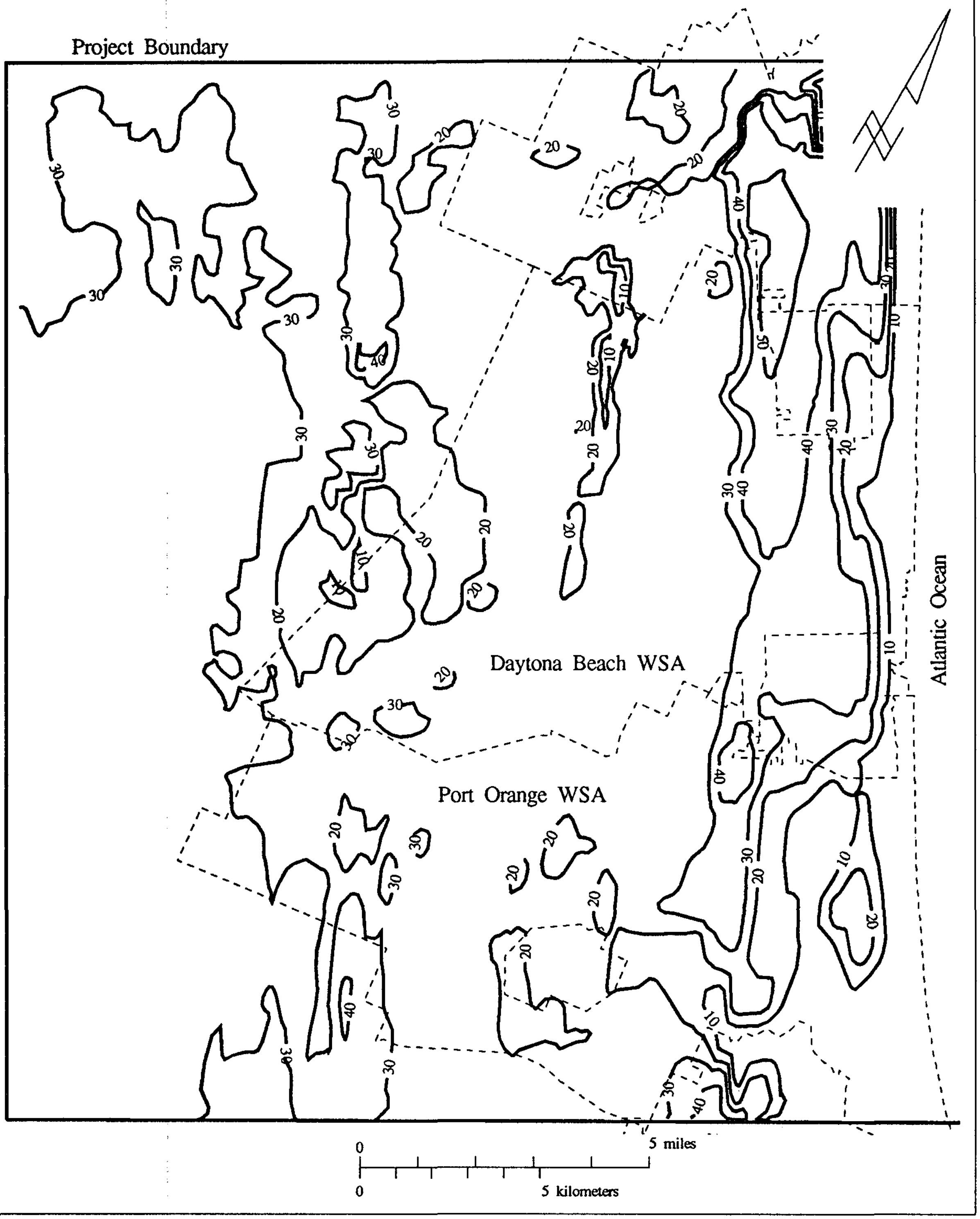


Figure 5.18
Flow simulation model pressure head results in the surficial aquifer due to Optimization Model 2 withdrawal strategy for year 2010.

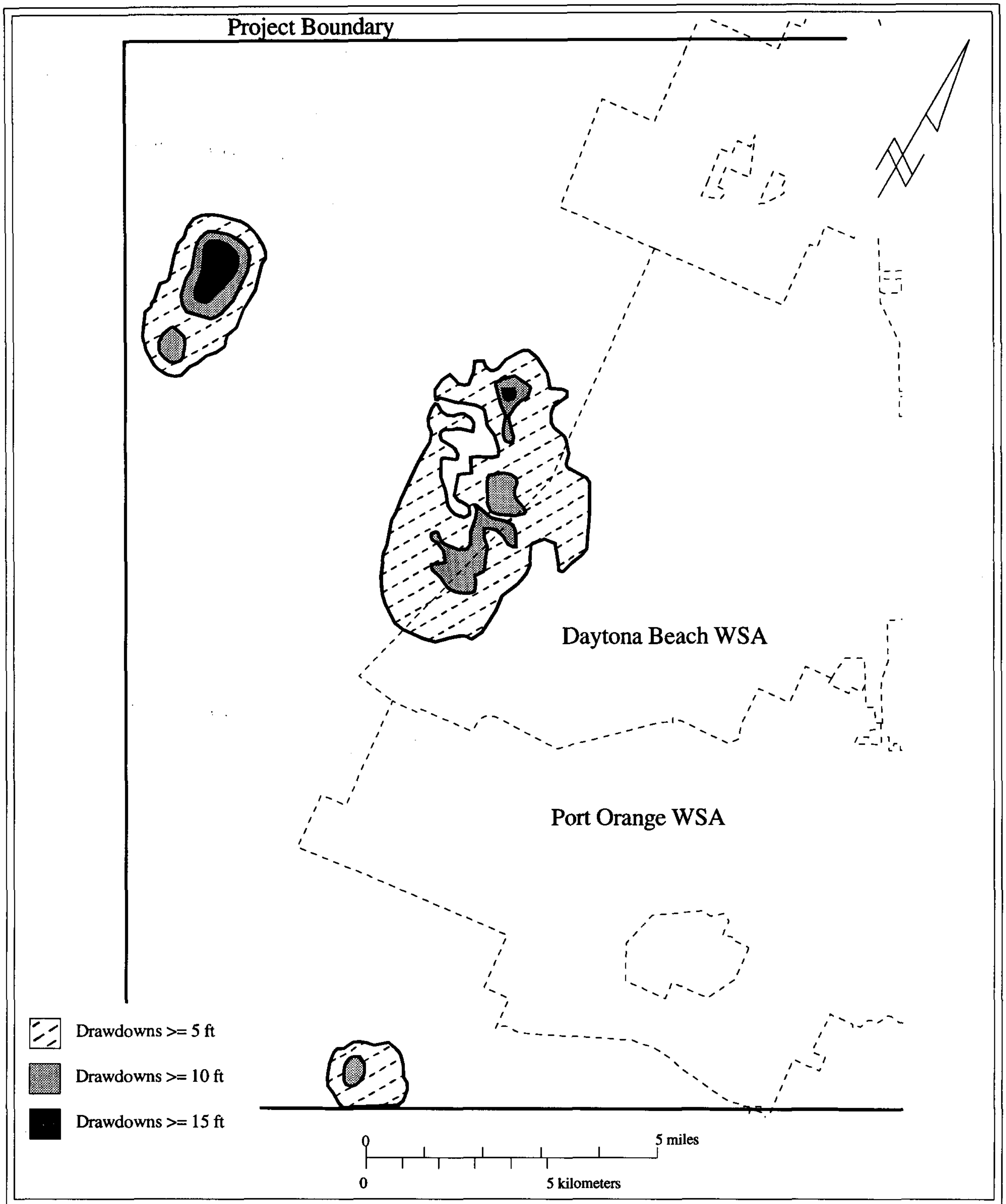


Figure 5.19

Flow simulation model drawdown results in the surficial aquifer due to Optimization Model 2 withdrawal strategy for year 2010.

weight thus, it does not reflect a concern for individual and extreme aquifer response values, but rather the regional system response as a whole.

Tables 5.2 and 5.3 depict the final optimum water allocation strategy as predicted by the model. Table 5.7 displays the pressure head and drawdown values determined by both the optimization and simulation models. The optimization model predicted a minimum average drawdown of 1.36 feet while the simulation model predicted a value of 1.77 feet. This corresponds to an average pressure head of 29.0 feet from the optimization model and 28.5 feet from the simulation model. Pressure head and drawdown values predicted by the optimization model were plotted against those of the simulation model as shown in Figures 5.20a and 5.20b, respectively. The coefficients of determination for model 3 prediction are 0.973 for pressure head and 0.932 for drawdown. A mean difference of 0.42 feet with a standard deviation of 0.61 feet was determined to exist between predicted responses. Figures 5.21 and 5.22 display the predicted pressure head elevations and drawdown values, respectively, when the optimum strategy is incorporated in the ground water flow simulation model.

5.6.4 Model 4

The objective of this model is to minimize maximum drawdown at all sensitive wetland control points. This model differs from model 1 because the optimum allocation scenario satisfies both water quantity as well as water quality constraints. Due to these water quality constraints, this model identified a larger minimum drawdown than found with model 1. The optimum allocation strategy identified is shown in Tables 5.2 and 5.3. Table 5.8 displays the pressure head and drawdown values determined at control points by the optimization model and by MODFLOW when the optimum allocation is

Table 5.7 - Calculated pressure head and drawdown values while minimizing average drawdown at sensitive wetland control points (Results from revised Model 3).

	Numerical Grid		Simulation Model		Optimization Model 3		Difference	
Control	Cell Location		head	drawdown	head	drawdown	head	drawdown
Point	row	column	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
1	5	1	29.2	0.51	29.32	0.38	-0.12	0.13
2	8	1	29.4	0.52	29.49	0.41	-0.09	0.11
3	15	1	29.1	0.67	29.22	0.58	-0.12	0.09
4	19	1	32.8	0.80	32.89	0.71	-0.09	0.09
5	22	1	33.9	0.77	34.02	0.68	-0.12	0.09
6	25	1	35.9	0.64	36.05	0.55	-0.15	0.09
7	28	1	33.8	0.51	33.89	0.41	-0.09	0.10
8	35	1	34.0	0.29	34.11	0.19	-0.11	0.10
9	41	1	34.2	0.24	34.26	0.14	-0.06	0.10
10	47	1	34.2	0.25	34.26	0.15	-0.06	0.10
11	53	1	34.4	0.31	34.59	0.21	-0.19	0.10
12	18	2	30.9	1.12	31.11	0.99	-0.21	0.13
13	56	2	35.5	0.36	35.67	0.23	-0.17	0.13
14	16	3	28.8	1.09	28.94	0.96	-0.14	0.13
15	30	3	34.6	0.63	34.80	0.50	-0.20	0.13
16	32	3	34.7	0.54	34.90	0.40	-0.20	0.14
17	37	3	35.4	0.37	35.56	0.24	-0.16	0.13
18	54	3	34.0	0.35	34.20	0.20	-0.20	0.15
19	17	4	31.8	1.33	31.90	1.20	-0.10	0.13
20	41	5	34.5	0.39	34.59	0.22	-0.09	0.17
21	44	5	33.9	0.35	34.12	0.18	-0.22	0.17
22	47	5	35.4	0.32	35.54	0.16	-0.14	0.16
23	53	5	35.0	0.31	35.14	0.16	-0.14	0.15
24	30	6	34.8	0.73	35.03	0.57	-0.23	0.16
25	32	6	35.3	0.61	35.45	0.45	-0.15	0.16
26	50	6	36.4	0.31	36.65	0.15	-0.25	0.16
27	59	7	34.2	0.33	34.29	0.21	-0.09	0.12
28	58	8	34.2	0.33	34.30	0.20	-0.10	0.13
29	39	9	32.5	0.49	32.69	0.31	-0.19	0.18
30	60	9	33.6	0.43	33.80	0.30	-0.20	0.13
31	16	10	29.3	1.90	29.45	1.75	-0.15	0.15
32	32	10	33.8	0.74	34.03	0.57	-0.23	0.17
33	14	11	29.3	1.39	29.47	1.23	-0.17	0.16
34	34	11	31.0	0.71	31.18	0.52	-0.18	0.19
35	52	11	33.5	0.31	33.69	0.11	-0.19	0.20
36	54	11	33.7	0.29	33.88	0.13	-0.18	0.16
37	59	11	33.5	0.54	33.70	0.40	-0.20	0.14
38	17	12	31.2	1.94	31.43	1.77	-0.23	0.17
39	31	12	30.7	0.88	30.81	0.69	-0.11	0.19
40	37	12	30.6	0.71	30.80	0.51	-0.20	0.20
41	39	12	33.0	0.64	33.17	0.43	-0.17	0.21
42	49	12	31.9	0.33	32.08	0.12	-0.18	0.21
43	43	13	30.5	0.51	30.72	0.28	-0.22	0.23
44	46	13	30.7	0.39	30.93	0.17	-0.23	0.22
45	16	14	28.7	1.48	28.91	1.29	-0.21	0.19
46	33	14	30.2	1.00	30.43	0.78	-0.23	0.22
47	54	14	33.3	0.32	33.47	0.13	-0.17	0.19
48	59	14	32.8	0.92	32.93	0.77	-0.13	0.15
49	14	15	26.6	1.16	26.82	0.98	-0.22	0.18
50	41	15	29.9	0.94	30.25	0.55	-0.35	0.39
51	5	16	27.1	0.80	27.29	0.61	-0.19	0.19
52	12	16	29.8	1.03	30.08	0.83	-0.28	0.20
53	38	16	25.7	4.22	26.65	3.25	-0.95	0.97

Table 5.7 - continued

	Numerical Grid		Simulation Model		Optimization Model 3		Difference	
Control	Cell Location		head	drawdown	head	drawdown	head	drawdown
Point	row	column	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
54	47	16	34.4	0.33	34.65	0.05	-0.25	0.28
55	50	16	30.8	0.32	31.19	-0.09	-0.39	0.41
56	53	16	33.2	0.34	33.49	0.11	-0.29	0.23
57	60	16	29.9	2.08	29.89	2.12	0.01	-0.04
58	3	17	22.8	0.96	23.06	0.74	-0.26	0.22
59	7	17	24.6	0.90	24.81	0.70	-0.21	0.20
60	10	17	26.3	0.91	26.59	0.71	-0.29	0.20
61	14	17	29.5	1.05	29.64	0.86	-0.14	0.19
62	30	17	27.6	3.01	30.15	0.45	-2.55	2.56
63	35	17	22.9	6.77	23.76	5.94	-0.86	0.83
64	44	17	30.7	0.70	31.03	0.37	-0.33	0.33
65	55	17	32.0	0.44	32.29	0.21	-0.29	0.23
66	57	17	31.6	1.29	32.41	0.49	-0.81	0.80
67	27	18	31.1	1.83	31.24	1.66	-0.14	0.17
68	32	18	23.0	6.87	23.02	6.89	-0.02	-0.02
69	37	18	16.7	9.21	18.29	7.61	-1.59	1.60
70	40	18	17.7	7.25	19.45	5.55	-1.75	1.70
71	42	18	20.7	4.61	22.37	2.93	-1.67	1.68
72	4	19	27.3	1.16	27.57	0.93	-0.27	0.23
73	19	19	26.4	1.41	26.61	1.19	-0.21	0.22
74	6	20	28.6	1.14	28.78	0.92	-0.18	0.22
75	9	20	28.8	1.03	28.99	0.81	-0.19	0.22
76	14	20	27.3	1.06	27.55	0.85	-0.25	0.21
77	17	20	26.9	1.29	27.14	1.06	-0.24	0.23
78	12	21	26.7	1.12	26.92	0.88	-0.22	0.24
79	10	22	37.7	1.62	37.98	1.32	-0.28	0.30
80	44	22	27.4	2.15	30.52	-0.92	-3.12	3.07
81	48	22	23.6	1.78	25.73	-0.33	-2.13	2.11
82	51	22	30.8	0.99	32.25	-0.45	-1.45	1.44
83	46	23	30.7	1.67	32.17	0.23	-1.47	1.44
84	43	24	19.3	3.70	21.29	1.71	-1.99	1.99
85	20	25	25.9	1.76	26.21	1.49	-0.31	0.27
86	26	25	19.0	7.03	20.63	5.47	-1.63	1.56
87	30	25	15.2	7.63	14.12	8.78	1.08	-1.15
88	32	25	16.2	8.53	16.98	7.72	-0.78	0.81
89	7	26	18.0	1.83	18.21	1.59	-0.21	0.24
90	10	26	18.0	1.40	18.24	1.16	-0.24	0.24
91	14	26	21.1	1.21	21.43	0.97	-0.33	0.24
92	17	26	22.1	1.25	22.39	1.01	-0.29	0.24
93	23	26	19.8	6.01	20.16	5.65	-0.36	0.36
94	29	26	14.4	6.51	14.81	6.10	-0.41	0.41
95	27	27	22.4	2.41	22.61	2.19	-0.21	0.22
96	31	27	14.1	6.84	14.11	6.89	-0.01	-0.05
97	35	27	13.0	6.94	14.90	5.00	-1.90	1.94
98	33	28	13.3	6.36	14.62	4.98	-1.32	1.38
99	37	28	20.5	1.78	21.32	0.98	-0.82	0.80
100	39	29	20.6	1.01	20.95	0.65	-0.35	0.36
Maximum			37.70	9.21	37.98	8.78	1.08	3.07
Minimum			13.00	0.24	14.11	-0.92	-3.12	-1.15
Average			28.54	1.77	28.95	1.36	-0.42	0.41
Standard Deviation			6.11	2.16	5.91	1.99	0.60	0.61
Variance			37.29	4.66	34.90	3.97	0.36	0.37

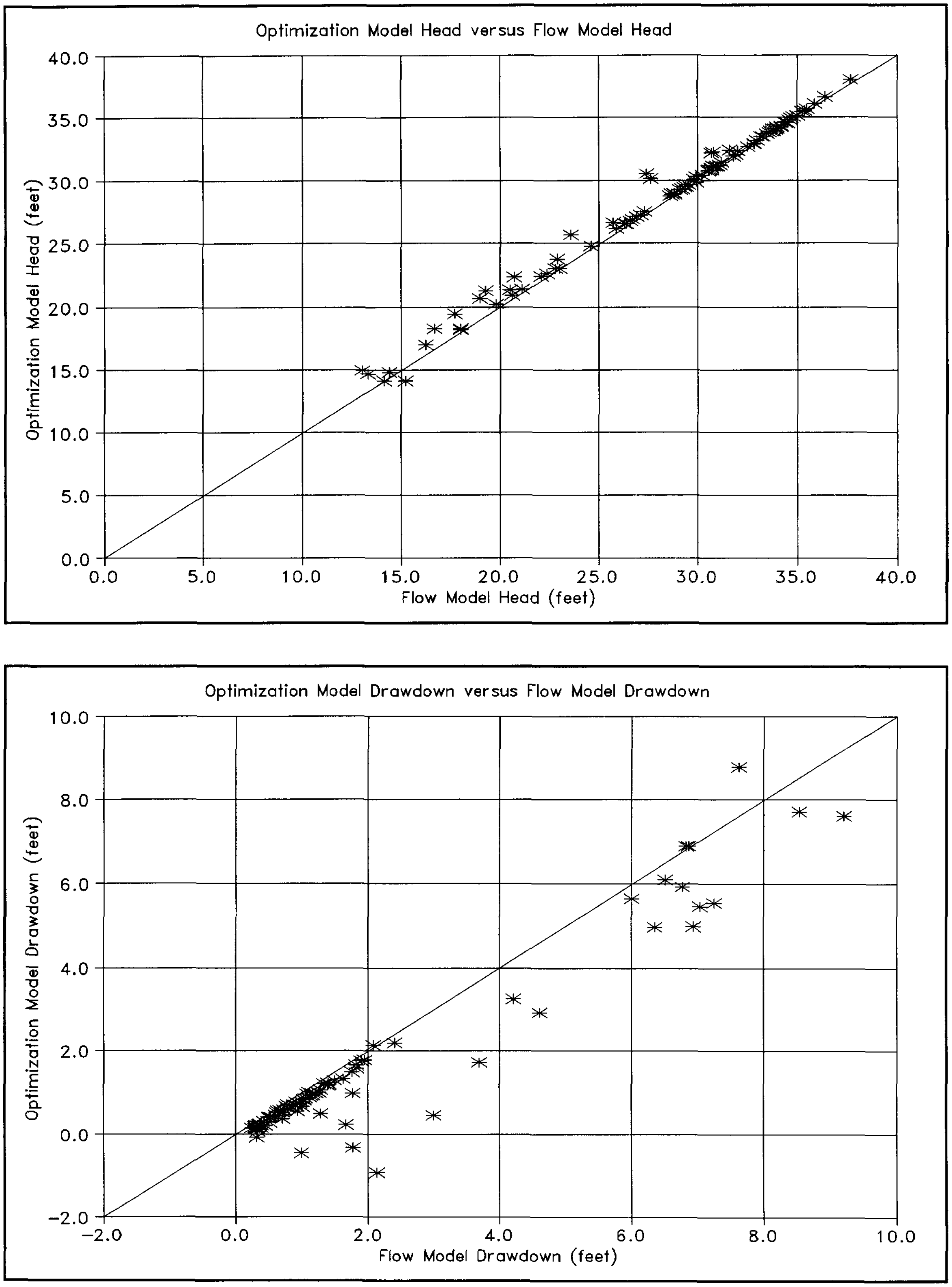


Figure 5.20 - Correlation between simulations model results and revised optimization model 3 results.
a) Pressure heads for year 2010; b) Drawdowns from year 1988 to 2010.

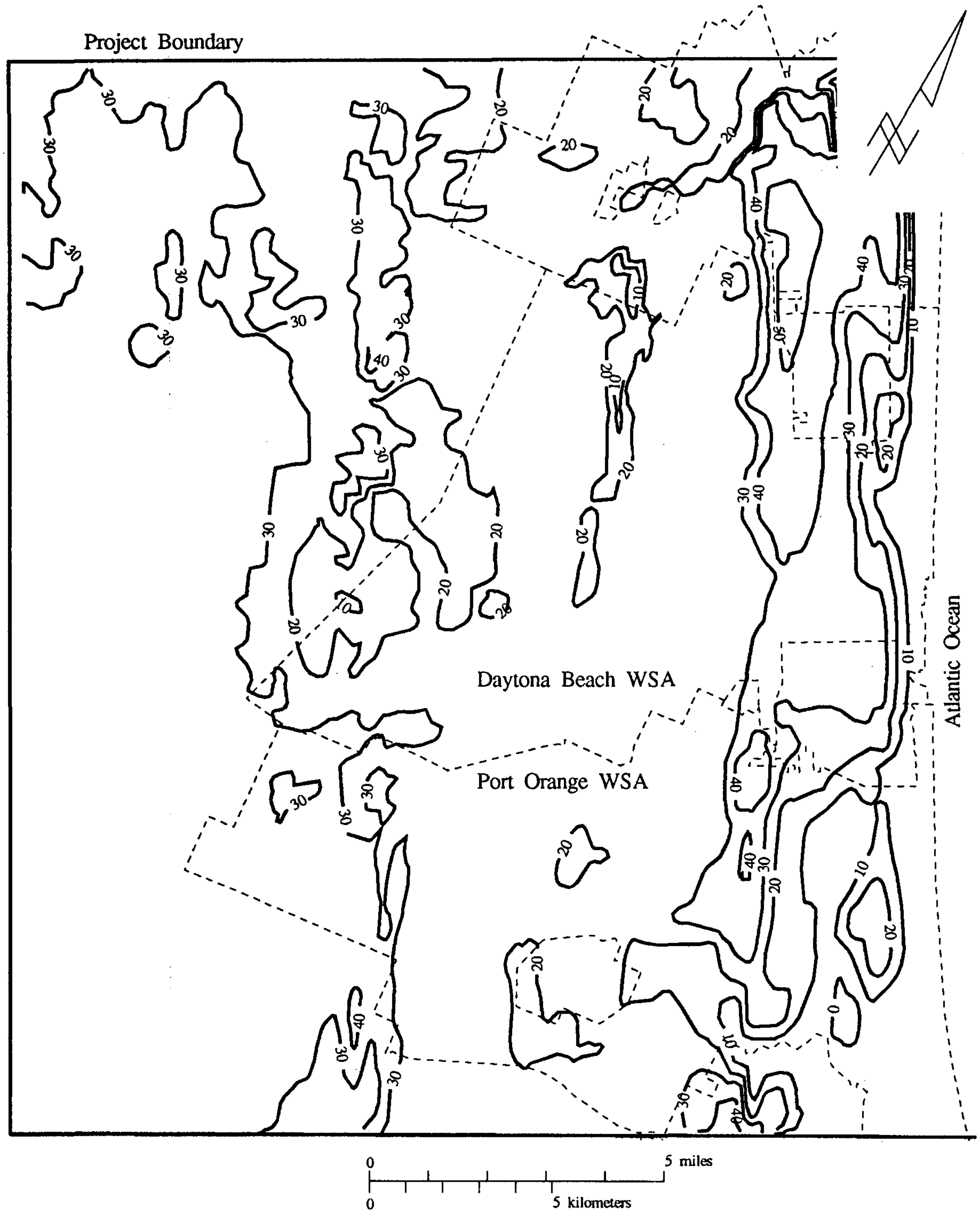


Figure 5.21

Flow simulation model pressure head results in the surficial aquifer due to Optimization Model 3 withdrawal strategy for year 2010.

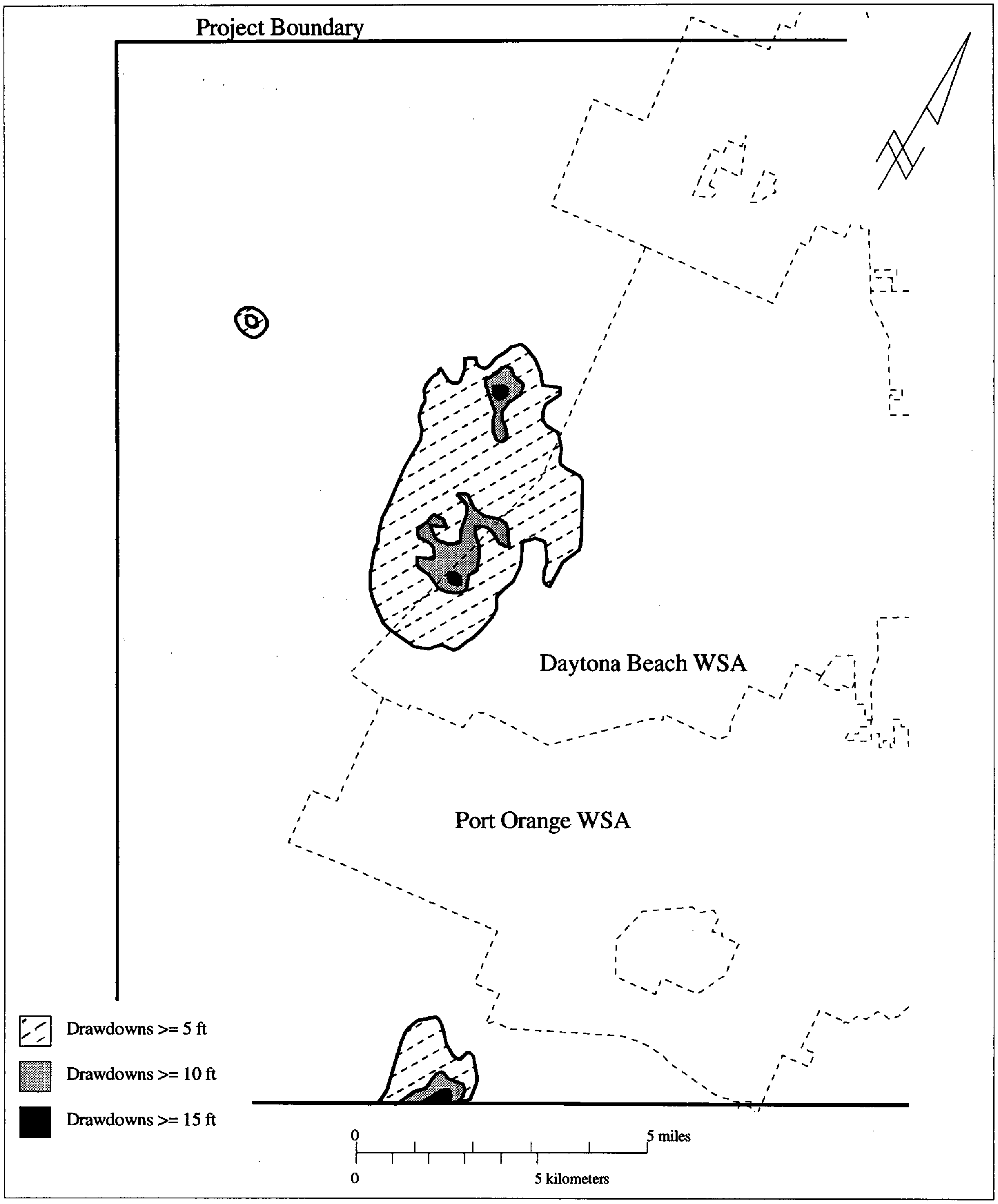


Figure 5.22

Flow simulation model drawdown results in the surficial aquifer due to Optimization Model 3 withdrawal strategy for year 2010.

Table 5.8 - Calculated pressure head and drawdown values while minimizing maximum drawdown at sensitive wetland control points and constraining concentrations at wells (Results for revised model 4).

	Numerical Grid		Simulation Model		Optimization Model 4		Difference	
Control	Cell Location		head	drawdown	head	drawdown	head	drawdown
Point	row	column	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
1	5	1	28.4	1.26	28.4	1.26	-0.04	0.00
2	8	1	28.3	1.62	28.3	1.64	0.04	-0.02
3	15	1	26.8	3.02	26.7	3.09	0.09	-0.07
4	19	1	29.5	4.16	28.6	4.98	0.88	-0.82
5	22	1	30.6	4.15	30.3	4.41	0.31	-0.26
6	25	1	34.2	2.39	34.1	2.48	0.08	-0.09
7	28	1	32.9	1.44	32.8	1.49	0.09	-0.05
8	35	1	33.8	0.48	33.8	0.51	0.01	-0.03
9	41	1	34.1	0.29	34.1	0.31	0.01	-0.02
10	47	1	34.2	0.26	34.1	0.27	0.07	-0.01
11	53	1	34.5	0.29	34.5	0.27	-0.03	0.02
12	18	2	23.8	8.26	23.7	8.45	0.15	-0.19
13	56	2	35.6	0.3	35.6	0.29	-0.01	0.01
14	16	3	24.6	5.37	24.4	5.51	0.21	-0.14
15	30	3	33.9	1.38	33.9	1.45	0.05	-0.07
16	32	3	34.2	1.03	34.2	1.08	-0.02	-0.05
17	37	3	35.2	0.54	35.2	0.58	-0.02	-0.04
18	54	3	34.1	0.3	34.1	0.29	-0.01	0.01
19	17	4	21.7	11.41	21.2	11.93	0.53	-0.52
20	41	5	34.4	0.46	34.3	0.49	0.09	-0.03
21	44	5	33.9	0.37	33.9	0.40	0.00	-0.03
22	47	5	35.4	0.32	35.4	0.33	0.03	-0.01
23	53	5	35.1	0.26	35.0	0.26	0.06	0.00
24	30	6	34.2	1.31	34.2	1.37	-0.03	-0.06
25	32	6	34.9	1.01	34.8	1.06	0.06	-0.05
26	50	6	36.5	0.28	36.5	0.29	-0.01	-0.01
27	59	7	34.3	0.22	34.3	0.23	0.03	-0.01
28	58	8	34.3	0.22	34.3	0.23	0.03	-0.01
29	39	9	32.5	0.58	32.4	0.62	0.12	-0.04
30	60	9	33.8	0.26	33.8	0.29	-0.01	-0.03
31	16	10	28.1	3.09	28.0	3.24	0.14	-0.15
32	32	10	33.6	0.97	33.6	1.03	0.03	-0.06
33	14	11	28.2	2.52	28.1	2.60	0.10	-0.08
34	34	11	30.9	0.85	30.8	0.92	0.12	-0.07
35	52	11	33.6	0.22	33.6	0.23	0.03	-0.01
36	54	11	33.8	0.19	33.8	0.20	0.00	-0.01
37	59	11	33.7	0.32	33.7	0.38	-0.02	-0.06
38	17	12	30.9	2.22	30.9	2.31	0.01	-0.09
39	31	12	30.5	1.03	30.4	1.11	0.11	-0.08
40	37	12	30.5	0.8	30.4	0.88	0.08	-0.08
41	39	12	32.9	0.7	32.8	0.77	0.07	-0.07
42	49	12	32	0.25	31.9	0.27	0.07	-0.02
43	43	13	30.5	0.5	30.5	0.53	0.03	-0.03
44	46	13	30.7	0.34	30.7	0.37	-0.03	-0.03
45	16	14	28.5	1.73	28.4	1.81	0.11	-0.08
46	33	14	30.1	1.09	30.0	1.20	0.10	-0.11
47	54	14	33.5	0.18	33.4	0.19	0.09	-0.01
48	59	14	33.1	0.58	32.4	1.28	0.68	-0.70
49	14	15	26.4	1.42	26.3	1.49	0.09	-0.07
50	41	15	29.8	1	27.1	3.75	2.75	-2.75
51	5	16	27.1	0.83	27.1	0.84	0.04	-0.01
52	12	16	29.7	1.17	29.7	1.22	0.02	-0.05
53	38	16	25.4	4.51	23.4	6.53	2.03	-2.02

Table 5.8 - continued

Control Point	Numerical Grid Cell Location row	column	Simulation Model head (feet)	Simulation Model drawdown (feet)	Optimization Model 4 head (feet)	Optimization Model 4 drawdown (feet)	Difference head (feet)	Difference drawdown (feet)
54	47	16	34.5	0.19	35.28	-0.58	-0.78	0.77
55	50	16	31	0.11	30.63	0.47	0.37	-0.36
56	53	16	33.4	0.14	33.44	0.16	-0.04	-0.02
57	60	16	31.8	0.17	31.71	0.29	0.09	-0.12
58	3	17	23	0.8	23	0.8	0.00	0.00
59	7	17	24.7	0.84	24.64	0.86	0.06	-0.02
60	10	17	26.3	0.91	26.35	0.95	-0.05	-0.04
61	14	17	29.5	1.07	29.37	1.13	0.13	-0.06
62	30	17	27.7	2.87	22.73	7.87	4.97	-5.00
63	35	17	22.5	7.17	21.16	8.54	1.34	-1.37
64	44	17	30.8	0.6	31.09	0.31	-0.29	0.29
65	55	17	32.3	0.18	32.66	-0.16	-0.36	0.34
66	57	17	32.7	0.24	32.09	0.81	0.61	-0.57
67	27	18	31.3	1.65	31.01	1.89	0.29	-0.24
68	32	18	22.9	6.96	20.41	9.49	2.49	-2.53
69	37	18	16.1	9.78	15.53	10.37	0.57	-0.59
70	40	18	17.5	7.49	17.03	7.97	0.47	-0.48
71	42	18	20.8	4.49	20.46	4.84	0.34	-0.35
72	4	19	27.8	0.7	27.8	0.7	0.00	0.00
73	19	19	26.6	1.2	26.47	1.33	0.13	-0.13
74	6	20	29.1	0.64	29.04	0.66	0.06	-0.02
75	9	20	29.1	0.69	29.08	0.72	0.02	-0.03
76	14	20	27.5	0.85	27.48	0.92	0.02	-0.07
77	17	20	27.1	1.07	27.03	1.17	0.07	-0.10
78	12	21	27	0.79	26.95	0.85	0.05	-0.06
79	10	22	38.4	0.9	38.26	1.04	0.14	-0.14
80	44	22	27.8	1.76	26.7	2.9	1.10	-1.14
81	48	22	24.6	0.74	24.49	0.91	0.11	-0.17
82	51	22	31.7	0.11	31.24	0.56	0.46	-0.45
83	46	23	31.2	1.18	30.75	1.65	0.45	-0.47
84	43	24	19.6	3.39	19.27	3.73	0.33	-0.34
85	20	25	26.4	1.32	26.08	1.62	0.32	-0.30
86	26	25	21.8	4.22	16.15	9.95	5.65	-5.73
87	30	25	16.6	6.23	10.97	11.93	5.63	-5.70
88	32	25	16.4	8.32	12.77	11.93	3.63	-3.61
89	7	26	19.3	0.48	19.3	0.5	0.00	-0.02
90	10	26	18.9	0.54	18.82	0.58	0.08	-0.04
91	14	26	21.7	0.68	21.64	0.76	0.06	-0.08
92	17	26	22.5	0.84	22.46	0.94	0.04	-0.10
93	23	26	22.3	3.48	16.4	9.4	5.90	-5.92
94	29	26	16.3	4.64	9.38	11.52	6.92	-6.88
95	27	27	22.9	1.93	22.55	2.25	0.35	-0.32
96	31	27	14.9	6.05	12.07	8.93	2.83	-2.88
97	35	27	12.9	7	12.25	7.65	0.65	-0.65
98	33	28	13.6	6.04	12.4	7.2	1.20	-1.16
99	37	28	20.6	1.67	17.98	4.32	2.62	-2.65
100	39	29	20.7	0.93	20.6	1	0.10	-0.07
	Maximum		38.40	11.41	38.26	11.93	6.92	0.77
	Minimum		12.90	0.11	9.38	-0.58	-0.78	-6.88
	Average		28.37	1.94	27.79	2.53	0.59	-0.59
	Standard Deviation		5.90	2.39	6.68	3.26	1.39	1.39
	Variance		34.82	5.72	44.67	10.62	1.92	1.94

simulated. The optimization model predicted a maximum drawdown of 11.41 feet while the simulation model predicted a value of 11.93 feet.

Pressure head and drawdown predicted by the optimization model were plotted against those of the simulation model as shown in Figures 5.23a and 5.23b respectively. The coefficients of determination for model 4 were 0.967 for pressure head and 0.853 for drawdown. For head and drawdown a mean difference of 0.6 feet with a standard deviation of 1.4 feet was determined to exist between predicted responses. As before, points which deviate from the "ideal" line having a slope of one fall on the "conservative" side as experienced for the three models previously demonstrated.

Since this particular water resource optimization model predicts water quality, the chloride concentrations were predicted. The optimum allocation strategy was simulated using DSTRAM, the solute transport model. The predicted chloride concentrations for the optimum allocation strategy are show in Table 5.9 for both optimization and simulation models. Chloride concentrations are displayed in mg/l for every municipal and agricultural well grid cell. As with the pressure head and drawdown values, simulation and optimization model concentrations were compared to determine the level of correlation. This comparison is displayed in Figure 5.24. The coefficient of determination for the chloride concentrations was found to be 0.969. The optimization model was found to produce generally conservative values compared to those of the simulation model (i.e. the maximum and average values are all greater for the optimization model than for the simulation model). A mean difference of 21.0 mg/l with a standard deviation of 113.5 mg/l was determined to exist between predicted responses.

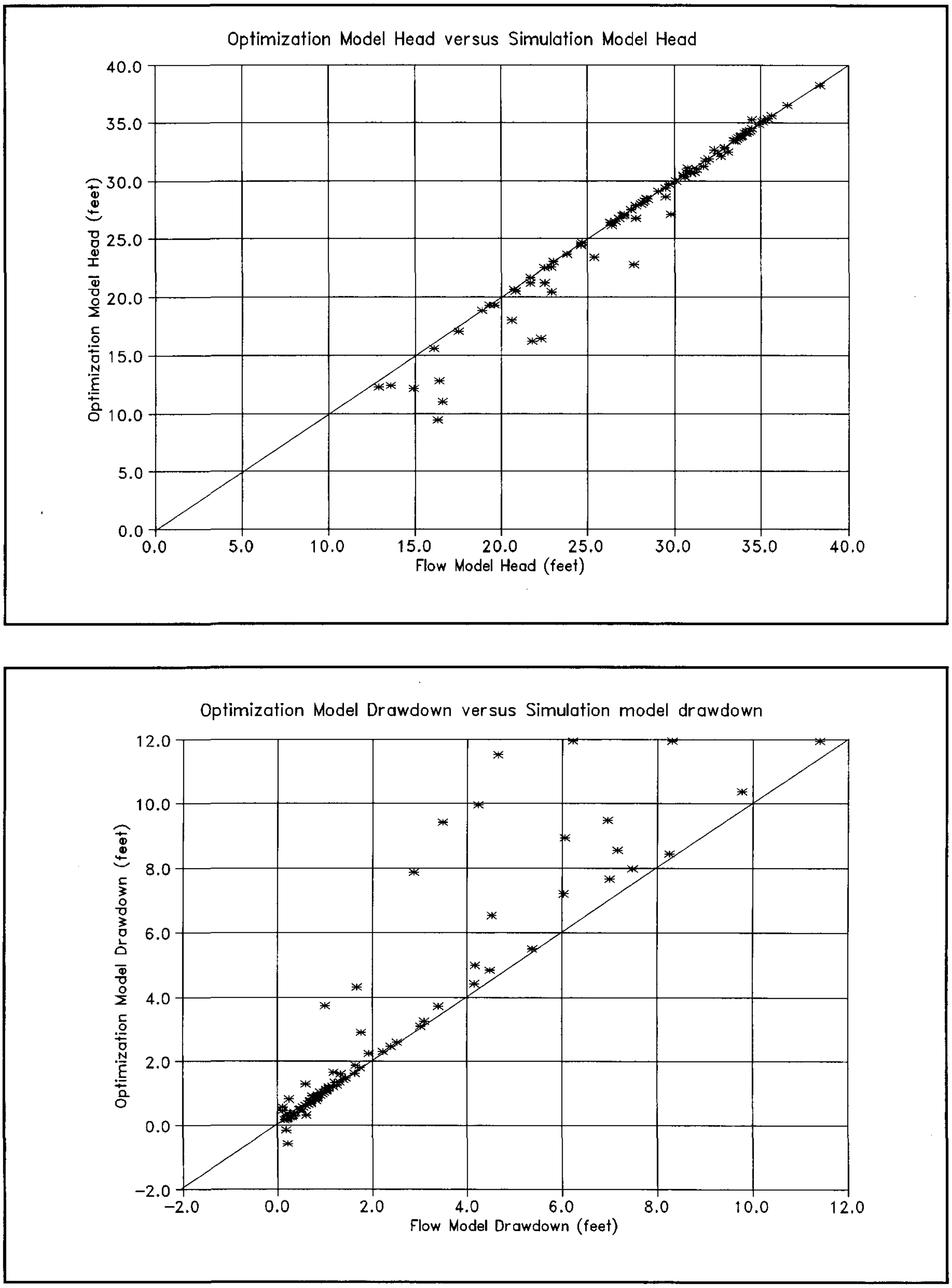


Figure 5.23 - Correlation between simulation model results and revised optimization model 4 results.
a) Pressure heads for year 2010. b) Drawdowns from year 1988 to 2010.

Table 5.9 - Calculated chloride concentrations at muncipal well grid cells while minimizing maximum drawdown at wetland control points (Model 4).

Well Grid Cell ID	Numerical Grid Cell Location Row	Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimal Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 4 Optimal Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL1	45	22	2.11	16.01	26.81	-10.8	21.75	5.1
MWELL2	48	18	1.69	17.62	12.58	5.04	11.65	0.9
MWELL3	46	20	1.31	16.45	24.05	-7.6	19.84	4.2
MWELL4	50	19	0.58	11.9	7.63	4.27	6.51	1.1
MWELL5	49	20	0.81	14.06	14.17	-0.11	11.42	2.8
MWELL6	47	19	1.69	17.62	12.58	5.04	11.65	0.9
MWELL7	48	19	1.69	17.62	12.58	5.04	11.65	0.9
MWELL8	47	18	1.69	17.62	12.58	5.04	11.65	0.9
MWELL9	48	21	2.48	16.28	19.85	-3.57	18.14	1.7
MWELL10	51	19	0.04	10.45	4.45	6	0	4.5
MWELL11	49	19	0.58	11.9	7.63	4.27	6.51	1.1
MWELL12	46	18	0.52	16.63	16.33	0.3	11.55	4.8
MWELL13	50	20	0.81	14.06	14.17	-0.11	11.42	2.8
MWELL14	45	19	0.52	16.63	16.33	0.3	11.55	4.8
MWELL15	50	18	0.58	11.9	7.63	4.27	6.51	1.1
MWELL16	21	6	0	0	0	0	0	0.0
MWELL17	20	4	0	0	0	0	0	0.0
MWELL18	22	8	0	0	0	0	0	0.0
MWELL19	19	7	0	0	0	0	0	0.0
MWELL20	18	5	0	0	0.01	-0.01	1.43	-1.4
MWELL21	20	9	0	0	0	0	0	0.0
MWELL22	23	5	0	0	0	0	0	0.0
MWELL23	22	3	0	0	0	0	0.05	-0.1
MWELL24	24	7	0	0	0	0	0	0.0
MWELL25	51	20	0.04	10.45	4.45	6	0	4.5
MWELL26	51	21	0.04	10.45	4.45	6	0	4.5
MWELL27	52	19	0.04	10.45	4.45	6	0	4.5
MWELL28	52	20	0.04	10.45	4.45	6	0	4.5
MWELL29	52	21	0.04	10.45	4.45	6	0	4.5
MWELL30	53	19	0.04	10.45	4.45	6	0	4.5
MWELL31	53	20	0.04	10.45	4.45	6	0	4.5
MWELL32	53	21	0.04	10.45	4.45	6	0	4.5
MWELL33	46	46	145.67	145.19	160.16	-14.97	115.18	45.0
MWELL34	49	46	160	146.66	172.21	-25.550	137.16	35.1
MWELL35	49	45	123.91	111.4	136.61	-25.21	105.22	31.4
MWELL36	47	47	164.76	156.71	180.26	-23.55	140.67	39.6
MWELL37	46	45	118.7	117.01	134.62	-17.61	94.99	39.6
MWELL38	47	46	164.76	156.71	180.26	-23.55	140.67	39.6
MWELL39	32	38	55.73	52.54	81.4	-28.86	30.2	51.2

Table 5.9 - Continued

Well Grid Cell ID	Numerical Grid Cell Location Row	Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimized Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 4 Optimized Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL40	32	36	55.64	60.76	90.43	-29.67	34.37	56.1
MWELL41	31	32	40.3	51.03	79.3	-28.27	29.23	50.1
MWELL42	32	37	55.64	60.76	90.43	-29.67	34.37	56.1
MWELL43	39	24	0.67	23.41	42.08	-18.67	13.43	28.7
MWELL44	39	26	2.58	26.07	53.93	-27.86	8.12	45.8
MWELL45	39	23	0.1	19.33	35.2	-15.87	12.11	23.1
MWELL46	31	31	34.52	48	86.33	-38.33	41.59	44.7
MWELL47	30	30	34.52	48	86.33	-38.33	41.59	44.7
MWELL48	39	22	0.1	19.33	35.2	-15.87	12.11	23.1
MWELL49	38	23	0.1	19.33	35.2	-15.87	12.11	23.1
MWELL50	37	23	0.04	30.64	45.59	-14.95	0	45.6
MWELL51	36	22	0.04	30.64	45.59	-14.95	0	45.6
MWELL52	39	25	0.67	23.41	42.08	-18.67	13.43	28.7
MWELL53	34	22	0	32.98	39.62	-6.64	35.29	4.3
MWELL54	33	22	0	25.98	32.5	-6.52	27.25	5.3
MWELL55	32	23	0	25.98	32.5	-6.52	27.25	5.3
MWELL56	31	23	0	10.63	16.81	-6.18	7.22	9.6
MWELL57	30	23	0	10.63	16.81	-6.18	7.22	9.6
MWELL58	38	21	0.01	12.72	32.33	-19.61	16.32	16.0
MWELL59	35	23	0	32.98	39.62	-6.64	35.29	4.3
MWELL60	34	23	0	32.98	39.62	-6.64	35.29	4.3
MWELL61	29	24	0	9.34	96.39	-87.05	0	96.4
MWELL62	28	24	0	9.34	96.39	-87.05	0	96.4
MWELL63	27	24	0	5.62	46.55	-40.93	35.57	11.0
MWELL64	26	24	0	5.62	46.55	-40.93	35.57	11.0
MWELL65	56	35	0	0	0	0	0	0.0
MWELL66	56	34	0	0	0	0	0	0.0
MWELL67	22	54	215.26	209.85	223.5	-13.65	176.55	46.9
MWELL68	22	53	180.11	174.92	188.89	-13.97	148.09	40.8
MWELL69	21	54	221.17	212.01	227.47	-15.46	177.79	49.7
MWELL70	21	53	185.44	175.88	191.66	-15.78	150.87	40.8
MWELL71	21	52	185.44	175.88	191.66	-15.78	150.87	40.8
MWELL72	22	34	48.31	40.37	71.33	-30.96	39.07	32.3
MWELL73	22	35	48.31	40.37	71.33	-30.96	39.07	32.3
MWELL74	21	35	51.04	39.37	63.36	-23.99	58	5.4
MWELL75	60	20	0	0	0	0	0	0.0
MWELL76	59	16	0	0	0	0	0	0.0
MWELL77	59	15	0	0	0	0	0	0.0
MWELL78	58	16	0	0	0	0	0	0.0

Table 5.9 - Continued

Well Grid Cell ID	Numerical Grid Cell Location Row	Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimal Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 4 Optimal Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL79	12	50	328.83	272.33	315.6	-43.27	290.09	25.5
MWELL80	12	53	382.03	333.9	383.19	-49.29	335.48	47.7
MWELL81	13	52	382.03	333.9	383.19	-49.29	335.48	47.7
MWELL82	12	52	382.03	333.9	383.19	-49.29	335.48	47.7
MWELL83	13	50	328.83	272.33	315.6	-43.27	290.09	25.5
MWELL84	13	49	253.47	182.69	222.15	-39.46	206.24	15.9
MWELL85	11	39	152.68	81.77	115.71	-33.94	129.88	-14.2
MWELL86	10	44	198.93	97.9	157.84	-59.94	133.59	24.3
MWELL87	11	41	182.32	91.36	130.89	-39.53	141.16	-10.3
MWELL88	9	45	218.89	104.95	166.25	-61.3	147.57	18.7
MWELL89	10	45	198.93	97.9	157.84	-59.94	133.59	24.3
MWELL90	6	32	46.25	47.07	52.82	-5.75	76.64	-23.8
MWELL91	5	32	43.19	55.79	53.52	2.27	61.69	-8.2
MWELL92	4	33	43.19	55.79	53.52	2.27	61.69	-8.2
MWELL93	3	33	38.63	64.97	53.26	11.71	45.7	7.6
MWELL94	5	33	43.19	55.79	53.52	2.27	61.69	-8.2
MWELL95	5	34	67.42	51.92	72.92	-21	78.29	-5.4
MWELL96	4	34	67.42	51.92	72.92	-21	78.29	-5.4
MWELL97	3	35	58.17	38.39	69.28	-30.89	60.32	9.0
MWELL98	4	31	25.62	24.18	40.25	-16.07	37.58	2.7
MWELL99	15	6	218.89	104.95	166.25	-61.3	147.57	18.7
MWELL100	16	7	0	0	0	0	0	0.0
MWELL101	17	7	0	0	0	0	0	0.0
MWELL102	18	7	0	0	0	0	0	0.0
MWELL103	20	6	0	0	0	0	0	0.0
MWELL104	21	5	0	0	0	0	0	0.0
MWELL105	22	5	0	0	0	0	0	0.0
MWELL106	18	8	0	0	0	0	0	0.0
MWELL107	18	9	0	0	0	0	0	0.0
MWELL108	18	10	0	0	0	0	0	0.0
MWELL109	18	11	0	0	0	0	0	0.0
MWELL110	17	10	0	0	0	0	0	0.0
MWELL111	19	5	0	0	0.01	-0.01	1.43	-1.4
MWELL112	18	4	0	0	0.01	-0.01	1.43	-1.4
MWELL113	18	3	0	0	0.01	-0.01	1.77	-1.8
MWELL114	19	3	0	0	0.01	-0.01	1.77	-1.8
MWELL115	20	2	0	0	0	0	0.4	-0.4
MWELL116	21	2	0	0	0	0	0.4	-0.4
MWELL117	13	23	0	0	0	0	0	0.0

Table 5.9 - Continued

Well Grid Cell ID	Numerical Grid Cell Location Row	Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimal Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 4 Optimal Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL118	12	24	0	0.01	0.02	-0.01	0.28	-0.3
MWELL119	8	39	157.36	95.59	115.72	-20.13	156.06	-40.3
MWELL120	7	34	76.84	73.26	77.85	-4.59	112.16	-34.3
AGWELL1	54	47	167.15	150.81	173.77	-22.96	167.15	6.6
AGWELL2	54	46	167.15	150.81	173.77	-22.96	167.15	6.6
AGWELL3	17	54	157.36	95.59	115.72	-20.13	156.06	-40.3
AGWELL4	16	51	182.32	91.36	130.89	-39.53	141.16	-10.3
AGWELL5	17	53	198.93	97.9	157.84	-59.94	133.59	24.3
AGWELL6	45	58	415.88	407.73	420.24	-12.51	333.81	86.4
AGWELL7	46	58	415.88	407.73	420.24	-12.51	333.81	86.4
AGWELL8	8	59	1618.05	1550.58	1728.79	-178.21	1602.61	126.2
AGWELL9	9	58	941.15	822.43	1055.1	-232.67	861.13	194.0
AGWELL10	7	59	1702.42	1598.14	1865.91	-267.77	1702.42	163.5
AGWELL11	6	59	1702.42	1598.14	1865.91	-267.77	1702.42	163.5
AGWELL12	6	58	964.86	813.79	1120.73	-306.94	933.45	187.3
AGWELL13	4	50	312.28	173.23	175.06	-1.83	275.94	-100.9
AGWELL14	4	48	249.74	127.36	134.95	-7.59	250	-115.1
AGWELL15	5	49	249.74	127.36	134.95	-7.59	250	-115.1
AGWELL16	3	49	307.63	163.63	133.31	30.32	288.05	-154.7
AGWELL17	56	33	0	0	0	0	0	0.0
AGWELL18	56	35	0	0	0	0	0	0.0
AGWELL19	59	17	0	0	0	0	0	0.0
AGWELL20	59	18	0	0	0	0	0	0.0
AGWELL21	60	17	0	0	0	0	0	0.0
AGWELL22	57	35	0	0	0	0	0	0.0
AGWELL23	58	37	0	0	0	0	0	0.0
AGWELL24	58	35	0	0	0	0	0	0.0
AGWELL25	59	36	0	0	0	0	0	0.0
AGWELL26	32	47	107.68	114.04	129.4	-15.36	82.72	46.7
AGWELL27	31	49	125.73	130.31	144.38	-14.07	103.9	40.5
AGWELL28	32	48	130.52	137.18	149.66	-12.48	101.14	48.5
Maximum			1702.4	1598.1	1865.9	30.3	1702.4	194.0
Minimum			0.0	0.0	0.0	-306.9	0.0	-154.7
Average			108.1	96.8	117.1	-20.2	102.2	14.8
Standard Deviation			299.8	276.0	322.7	48.3	293.1	45.5
Variance			91180.4	77354.4	105791.5	2946.6	87290.1	2289.1

Figures 5.25 and 5.26 display the predicted pressure head elevations and drawdown values, respectively, for the entire Volusia County subregion when the optimum strategy is simulated with the ground water flow model MODFLOW. Figure 5.27 displays the predicted chloride concentrations in the upper Floridan aquifer when the optimal strategy is simulated with the solute transport model DSTRAM.

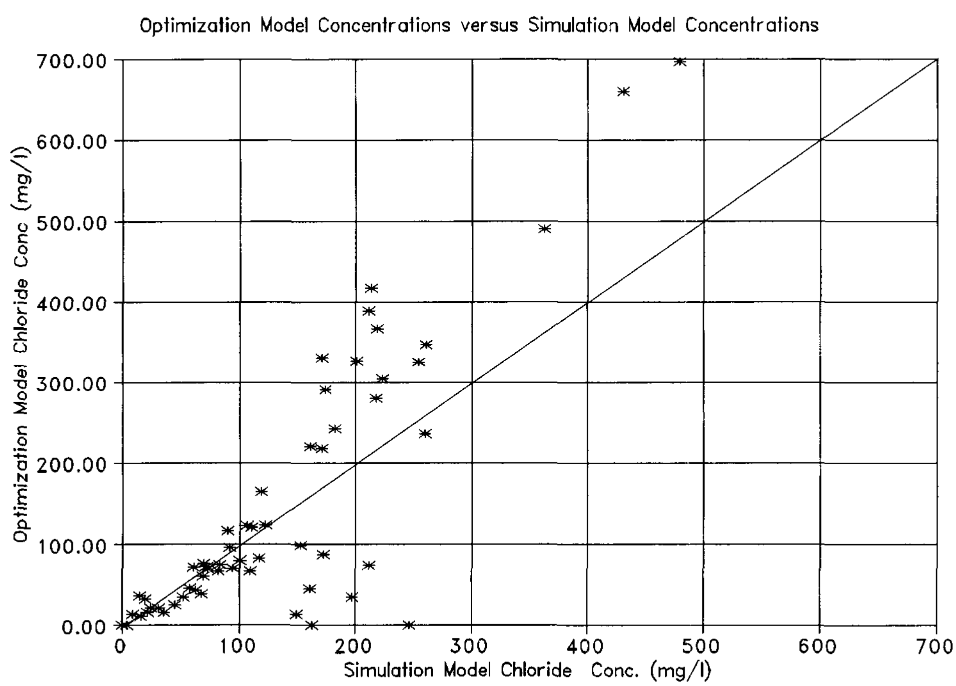


Figure 5.24 - Chloride Concentration correlation between simulation model and optimization model 4.

5.6.4 Model 5

The objective of this model is to minimize maximum relative chloride concentration increase. The optimum allocation strategy identified is shown in Tables 5.2 and 5.3. Table 5.10 displays the pressure head and drawdown values determined at control points by the optimization model and by MODFLOW when the optimum

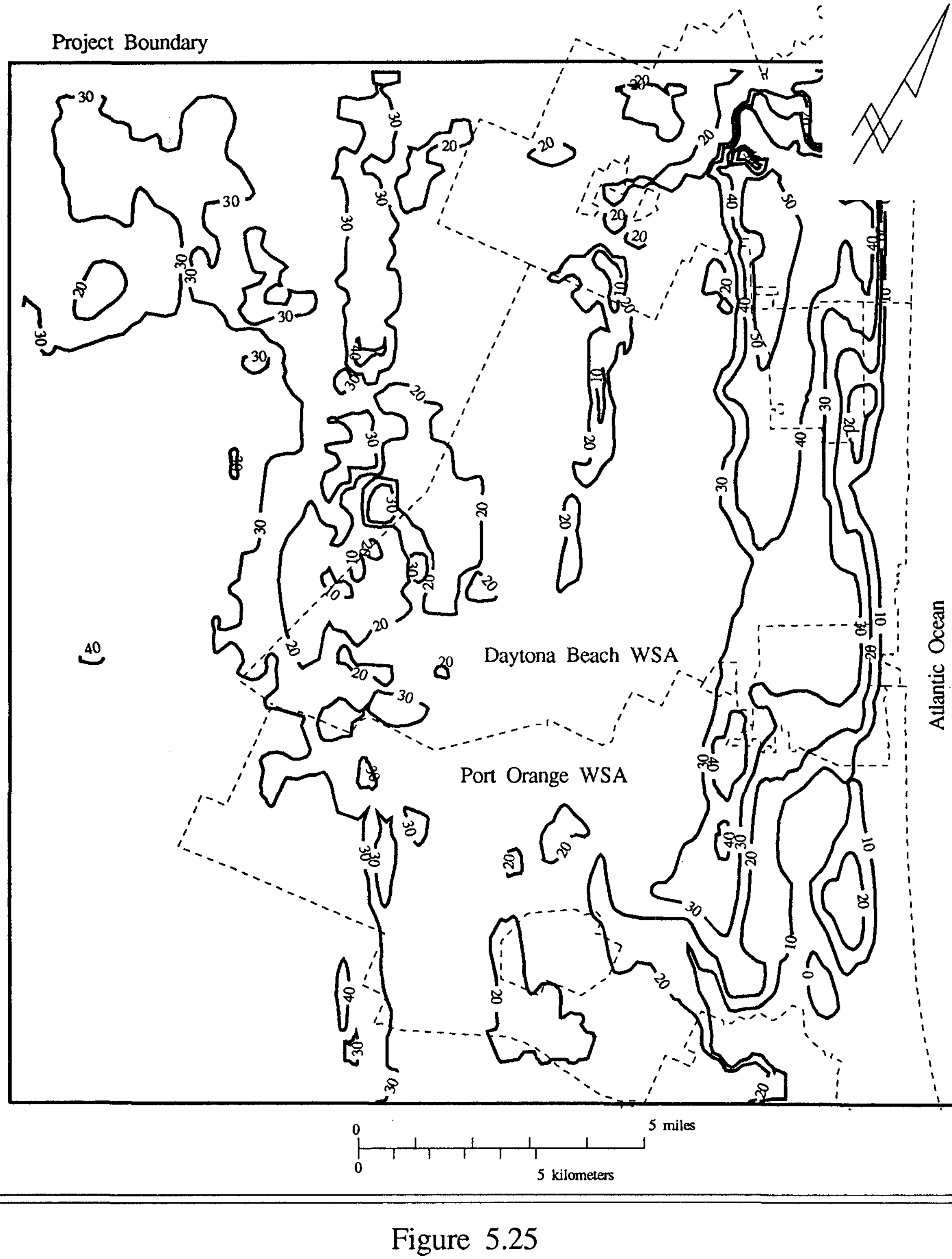


Figure 5.25

Flow simulation model pressure head results in the surficial aquifer due to Optimization Model 4 withdrawal strategy for year 2010.

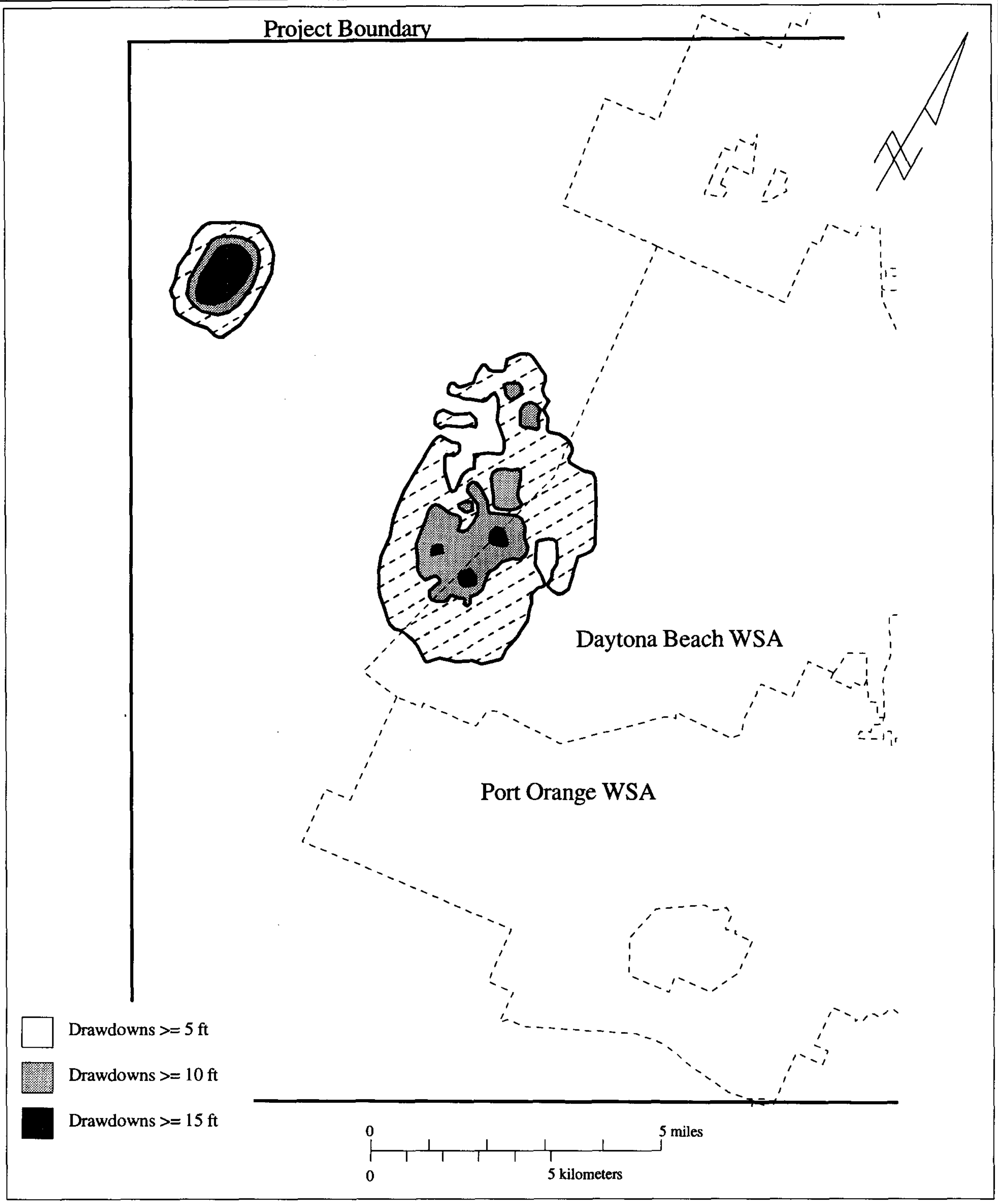


Figure 5.26

Flow simulation model drawdown results in the surficial aquifer due to Optimization Model 4 withdrawal strategy for year 2010.

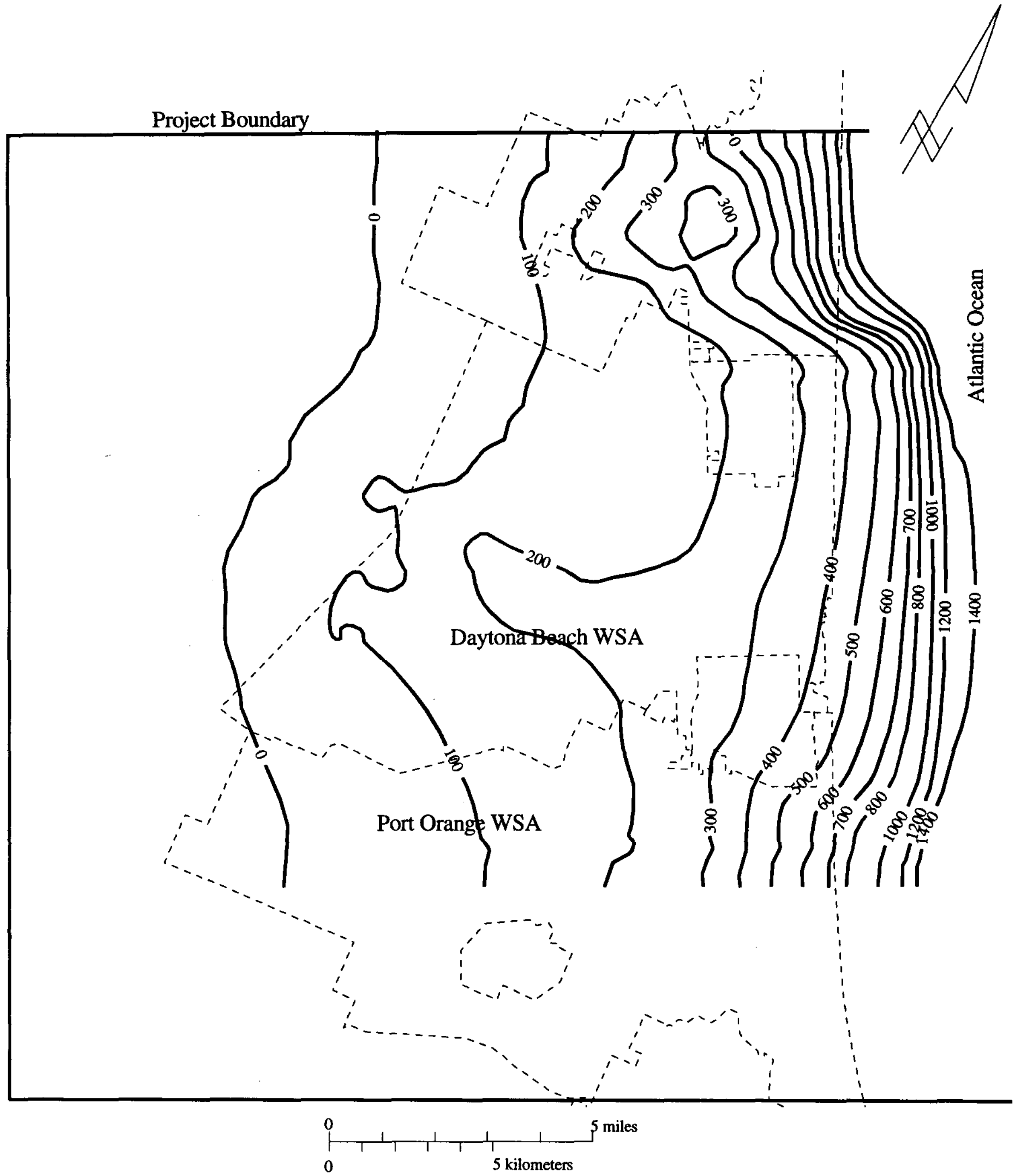


Figure 5.27

Simulation model chloride concentration results in the upper floridan aquifer due to Optimization Model 4 withdrawal strategy for year 2010.

Table 5.10 - Calculated pressure head and drawdown values at sensitive wetland control points for minimizing the maximum relative concentration increase at all well grid cells (Results for revised model 5).

	Numerical Grid		Simulation Model		Optimization Model 5		Difference	
Control	Cell Location		head	drawdown	head	drawdown	head	drawdown
Point	row	column	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)
1	5	1	28.5	1.16	28.5	1.18	-0.02	-0.02
2	8	1	28.4	1.46	28.4	1.50	0.00	-0.04
3	15	1	27.4	2.43	27.3	2.53	0.13	-0.10
4	19	1	30.5	3.13	30.2	3.36	0.26	-0.23
5	22	1	30.8	3.86	30.5	4.23	0.33	-0.37
6	25	1	34.2	2.35	34.1	2.47	0.07	-0.12
7	28	1	32.9	1.42	32.8	1.49	0.09	-0.07
8	35	1	33.8	0.51	33.8	0.55	0.05	-0.04
9	41	1	34.1	0.34	34.0	0.37	0.07	-0.03
10	47	1	34.1	0.3	34.1	0.33	0.03	-0.03
11	53	1	34.4	0.32	34.5	0.32	-0.08	0.00
12	18	2	28	4.02	27.6	4.52	0.42	-0.50
13	56	2	35.6	0.34	35.6	0.34	0.04	0.00
14	16	3	25.9	4.05	25.7	4.25	0.25	-0.20
15	30	3	33.9	1.39	33.8	1.48	0.08	-0.09
16	32	3	34.2	1.05	34.2	1.13	0.03	-0.08
17	37	3	35.2	0.59	35.2	0.65	0.05	-0.06
18	54	3	34	0.35	34.1	0.35	-0.05	0.00
19	17	4	27.9	5.29	25.9	7.20	2.00	-1.91
20	41	5	34.3	0.54	34.2	0.60	0.10	-0.06
21	44	5	33.8	0.46	33.8	0.50	0.00	-0.04
22	47	5	35.3	0.39	35.3	0.43	0.03	-0.04
23	53	5	35	0.33	35.0	0.34	0.04	-0.01
24	30	6	34.2	1.32	34.2	1.40	0.00	-0.08
25	32	6	34.9	1.05	34.8	1.12	0.12	-0.07
26	50	6	36.4	0.36	36.4	0.39	-0.01	-0.03
27	59	7	34.3	0.23	34.3	0.25	0.05	-0.02
28	58	8	34.3	0.23	34.3	0.25	0.05	-0.02
29	39	9	32.3	0.69	32.2	0.76	0.06	-0.07
30	60	9	33.8	0.25	33.8	0.28	-0.02	-0.03
31	16	10	28.3	2.96	28.0	3.19	0.29	-0.23
32	32	10	33.5	1.05	33.5	1.14	0.04	-0.09
33	14	11	28.2	2.45	28.1	2.56	0.06	-0.11
34	34	11	30.7	0.98	30.6	1.07	0.07	-0.09
35	52	11	33.5	0.35	33.4	0.39	0.09	-0.04
36	54	11	33.7	0.28	33.7	0.32	0.02	-0.04
37	59	11	33.8	0.26	33.8	0.30	0.00	-0.04
38	17	12	31	2.14	30.9	2.27	0.07	-0.13
39	31	12	30.4	1.15	30.2	1.26	0.16	-0.11
40	37	12	30.4	0.97	30.2	1.08	0.18	-0.11
41	39	12	32.8	0.87	32.6	0.97	0.17	-0.10
42	49	12	31.8	0.41	31.7	0.47	0.07	-0.06
43	43	13	30.3	0.7	30.2	0.78	0.08	-0.08
44	46	13	30.5	0.53	30.5	0.61	0.01	-0.08
45	16	14	28.5	1.73	28.4	1.84	0.14	-0.11
46	33	14	29.9	1.31	29.8	1.44	0.14	-0.13
47	54	14	33.3	0.31	33.2	0.36	0.06	-0.05
48	59	14	33.4	0.32	33.3	0.38	0.08	-0.06
49	14	15	26.3	1.45	26.3	1.54	0.04	-0.09
50	41	15	28.6	2.21	25.7	5.13	2.93	-2.92
51	5	16	27.1	0.87	27.0	0.89	0.09	-0.02
52	12	16	29.6	1.22	29.6	1.30	0.00	-0.08
53	38	16	23.7	6.21	21.5	8.39	2.19	-2.18

Table 5.10 - continued

	Numerical Grid			Simulation Model			Optimization Model 5			Difference	
Control	Cell Location			head	drawdown		head	drawdown		head	drawdown
Point	row	column		(feet)	(feet)		(feet)	(feet)		(feet)	(feet)
54	47	16		34.2	0.53		34.67	0.03		-0.47	0.50
55	50	16		30.7	0.48		28.94	2.16		1.76	-1.68
56	53	16		33.2	0.38		33.14	0.46		0.06	-0.08
57	60	16		31.8	0.18		31.93	0.07		-0.13	0.11
58	3	17		22.9	0.85		22.93	0.87		-0.03	-0.02
59	7	17		24.6	0.89		24.56	0.94		0.04	-0.05
60	10	17		26.3	0.97		26.26	1.04		0.04	-0.07
61	14	17		29.4	1.15		29.26	1.24		0.14	-0.09
62	30	17		26	4.65		20.78	9.82		5.22	-5.17
63	35	17		20.4	9.25		18.91	10.79		1.49	-1.54
64	44	17		30.3	1.09		29.86	1.54		0.44	-0.45
65	55	17		32.1	0.35		32.27	0.23		-0.17	0.12
66	57	17		32.6	0.35		31.62	1.28		0.98	-0.93
67	27	18		30.8	2.1		30.53	2.37		0.27	-0.27
68	32	18		20.5	9.35		17.83	12.07		2.67	-2.72
69	37	18		13.6	12.31		12.79	13.11		0.81	-0.80
70	40	18		15.1	9.85		14.47	10.53		0.63	-0.68
71	42	18		18.6	6.71		17.98	7.32		0.62	-0.61
72	4	19		27.7	0.78		27.69	0.81		0.01	-0.03
73	19	19		26.4	1.46		26.19	1.61		0.21	-0.15
74	6	20		29	0.74		28.92	0.78		0.08	-0.04
75	9	20		29	0.79		28.95	0.85		0.05	-0.06
76	14	20		27.4	1		27.3	1.1		0.10	-0.10
77	17	20		26.9	1.29		26.78	1.42		0.12	-0.13
78	12	21		26.8	0.94		26.78	1.02		0.02	-0.08
79	10	22		38.2	1.1		38.02	1.28		0.18	-0.18
80	44	22		25.5	4.12		24.03	5.57		1.47	-1.45
81	48	22		22.1	3.29		21.63	3.77		0.47	-0.48
82	51	22		29.9	1.94		28.76	3.04		1.14	-1.10
83	46	23		29.4	3		28.33	4.07		1.07	-1.07
84	43	24		16.9	6.07		16.34	6.66		0.56	-0.59
85	20	25		25.9	1.8		25.55	2.15		0.35	-0.35
86	26	25		17.7	8.36		12.75	13.35		4.95	-4.99
87	30	25		12.7	10.22		6.77	16.13		5.93	-5.91
88	32	25		12.5	12.19		8.43	16.27		4.07	-4.08
89	7	26		19.2	0.64		19.11	0.69		0.09	-0.05
90	10	26		18.7	0.71		18.63	0.77		0.07	-0.06
91	14	26		21.4	0.91		21.39	1.01		0.01	-0.10
92	17	26		22.2	1.14		22.14	1.26		0.06	-0.12
93	23	26		19.2	6.63		13.14	12.66		6.06	-6.03
94	29	26		12.3	8.64		5.2	15.7		7.10	-7.06
95	27	27		22	2.75		21.71	3.09		0.29	-0.34
96	31	27		11.6	9.38		8.58	12.42		3.02	-3.04
97	35	27		9.5	10.41		8.68	11.22		0.82	-0.81
98	33	28		10.8	8.87		9.44	10.16		1.36	-1.29
99	37	28		18.1	4.21		15.38	6.92		2.72	-2.71
100	39	29		20.1	1.55		19.94	1.66		0.16	-0.11
Maximum				38.20	12.31		38.02	16.27		7.10	0.50
Minimum				9.50	0.18		5.20	0.03		-0.47	-7.06
Average				27.84	2.47		27.16	3.16		0.68	-0.68
Standard Deviation				6.71	3.02		7.61	4.11		1.42	1.42
Variance				44.96	9.13		57.88	16.90		2.02	2.00

allocation is simulated. The optimization model predicted a maximum drawdown of 16.27 feet while the simulation model predicted a value of 12.31 feet.

Pressure head and drawdown predicted by the optimization model was plotted against those of the simulation model as shown in Figures 5.28a and 5.28b respectively. The coefficients of determination for model 5 were 0.972 for pressure head and 0.862 for drawdown. For head and drawdown a mean difference of 0.7 feet with a standard deviation of 1.4 feet was determined to exist between predicted responses. As before, points which deviate from the "ideal" line having a slope of one fall on the "conservative" side as experienced for the three models previously demonstrated.

The optimum allocation strategy was simulated using DSTRAM, the solute transport model. The predicted chloride concentrations induced by the optimum allocation strategy are show in Table 5.11 for both optimization and simulation models. Chloride concentrations are displayed in mg/l for every municipal and agricultural well grid cell. As with the pressure head and drawdown values, simulation and optimization model concentrations were compared to determine the level of correlation. This comparison in displayed in Figure 5.29. The coefficient of determination for the chloride concentrations was found to be 0.954. The optimization was found to produce generally conservative values compared to those of the simulation model (i.e. the maximum and average values are all greater for the optimization model than for the simulation model). A mean difference of 0.6 mg/l with a standard deviation of 28.2 mg/l was determined to exist between predicted responses.

Figures 5.30 and 5.31 display the predicted pressure head elevations and

Table 5.11 - Calculated chloride concentrations at muncipal well grid cells while minimizing maximum relative chloride concentration increase at well grid cells (Model 5).

Well Grid Cell ID	Numerical Grid Cell Location Row	Numerical Grid Cell Location Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimal Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 5 Optimal Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL1	45	22	2.11	16.01	21.09	-5.08	0.64	20.5
MWELL2	48	18	1.69	17.62	13.09	4.53	1.69	11.4
MWELL3	46	20	1.31	16.45	21.26	-4.81	0.9	20.4
MWELL4	50	19	0.58	11.9	10.97	0.93	0.58	10.4
MWELL5	49	20	0.81	14.06	15.85	-1.79	0.81	15.0
MWELL6	47	19	1.69	17.62	13.09	4.53	1.69	11.4
MWELL7	48	19	1.69	17.62	13.09	4.53	1.69	11.4
MWELL8	47	18	1.69	17.62	13.09	4.53	1.69	11.4
MWELL9	48	21	2.48	16.28	19.7	-3.42	2.34	17.4
MWELL10	51	19	0.04	10.45	7.64	2.81	0.04	7.6
MWELL11	49	19	0.58	11.9	10.97	0.93	0.58	10.4
MWELL12	46	18	0.52	16.63	14.09	2.54	0.5	13.6
MWELL13	50	20	0.81	14.06	15.85	-1.79	0.81	15.0
MWELL14	45	19	0.52	16.63	14.09	2.54	0.5	13.6
MWELL15	50	18	0.58	11.9	10.97	0.93	0.58	10.4
MWELL16	21	6	0	0	0	0	0	0.0
MWELL17	20	4	0	0	0	0	0	0.0
MWELL18	22	8	0	0	0	0	0	0.0
MWELL19	19	7	0	0	0	0	0	0.0
MWELL20	18	5	0	0	0	0	0	0.0
MWELL21	20	9	0	0	0	0	0	0.0
MWELL22	23	5	0	0	0	0	0	0.0
MWELL23	22	3	0	0	0	0	0	0.0
MWELL24	24	7	0	0	0	0	0	0.0
MWELL25	51	20	0.04	10.45	7.64	2.81	0.04	7.6
MWELL26	51	21	0.04	10.45	7.64	2.81	0.04	7.6
MWELL27	52	19	0.04	10.45	7.64	2.81	0.04	7.6
MWELL28	52	20	0.04	10.45	7.64	2.81	0.04	7.6
MWELL29	52	21	0.04	10.45	7.64	2.81	0.04	7.6
MWELL30	53	19	0.04	10.45	7.64	2.81	0.04	7.6
MWELL31	53	20	0.04	10.45	7.64	2.81	0.04	7.6
MWELL32	53	21	0.04	10.45	7.64	2.81	0.04	7.6
MWELL33	46	46	145.67	145.19	151.42	-6.23	144.7	6.7
MWELL34	49	46	160	146.66	152.49	-5.83	157.26	-4.8
MWELL35	49	45	123.91	111.4	117.59	-6.19	117.23	0.4
MWELL36	47	47	164.76	156.71	162.45	-5.74	161.23	1.2
MWELL37	46	45	118.7	117.01	123.3	-6.29	116.77	6.5
MWELL38	47	46	164.76	156.71	162.45	-5.74	161.23	1.2
MWELL39	32	38	55.73	52.54	65.64	-13.1	53.38	12.3

Table 5.11 - Continued

Well Grid Cell ID	Numerical Grid Cell Location Row	Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimized Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 5 Optimized Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL40	32	36	55.64	60.76	71.56	-10.8	52.41	19.2
MWELL41	31	32	40.3	51.03	61.99	-10.96	40.31	21.7
MWELL42	32	37	55.64	60.76	71.56	-10.8	52.41	19.2
MWELL43	39	24	0.67	23.41	25.77	-2.36	0.22	25.6
MWELL44	39	26	2.58	26.07	33.86	-7.79	1.85	32.0
MWELL45	39	23	0.1	19.33	23.39	-4.06	0.05	23.3
MWELL46	31	31	34.52	48	61.3	-13.3	34.42	26.9
MWELL47	30	30	34.52	48	61.3	-13.3	34.42	26.9
MWELL48	39	22	0.1	19.33	23.39	-4.06	0.05	23.3
MWELL49	38	23	0.1	19.33	23.39	-4.06	0.05	23.3
MWELL50	37	23	0.04	30.64	31.91	-1.27	0.03	31.9
MWELL51	36	22	0.04	30.64	31.91	-1.27	0.03	31.9
MWELL52	39	25	0.67	23.41	25.77	-2.36	0.22	25.6
MWELL53	34	22	0	32.98	25.51	7.47	0	25.5
MWELL54	33	22	0	25.98	21.5	4.48	0	21.5
MWELL55	32	23	0	25.98	21.5	4.48	0	21.5
MWELL56	31	23	0	10.63	11.25	-0.62	0	11.3
MWELL57	30	23	0	10.63	11.25	-0.62	0	11.3
MWELL58	38	21	0.01	12.72	21.06	-8.34	0.01	21.1
MWELL59	35	23	0	32.98	25.51	7.47	0	25.5
MWELL60	34	23	0	32.98	25.51	7.47	0	25.5
MWELL61	29	24	0	9.34	60.88	-51.54	0	60.9
MWELL62	28	24	0	9.34	60.88	-51.54	0	60.9
MWELL63	27	24	0	5.62	29.91	-24.29	0	29.9
MWELL64	26	24	0	5.62	29.91	-24.29	0	29.9
MWELL65	56	35	0	0	0	0	0	0.0
MWELL66	56	34	0	0	0	0	0	0.0
MWELL67	22	54	215.26	209.85	215.38	-5.53	213.4	2.0
MWELL68	22	53	180.11	174.92	181	-6.08	177.3	3.7
MWELL69	21	54	221.17	212.01	218.48	-6.47	220.52	-2.0
MWELL70	21	53	185.44	175.88	183.08	-7.2	183.83	-0.8
MWELL71	21	52	185.44	175.88	183.08	-7.2	183.83	-0.8
MWELL72	22	34	48.31	40.37	56.66	-16.29	45.34	11.3
MWELL73	22	35	48.31	40.37	56.66	-16.29	45.34	11.3
MWELL74	21	35	51.04	39.37	48.69	-9.32	49.03	-0.3
MWELL75	60	20	0	0	0	0	0	0.0
MWELL76	59	16	0	0	0	0	0	0.0
MWELL77	59	15	0	0	0	0	0	0.0
MWELL78	58	16	0	0	0	0	0	0.0

Table 5.11 - Continued

Well Grid Cell ID	Numerical Grid Cell Location Row	Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimal Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 5 Optimal Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL79	12	50	328.83	272.33	288.64	-16.31	282.72	5.9
MWELL80	12	53	382.03	333.9	349.33	-15.43	342.32	7.0
MWELL81	13	52	382.03	333.9	349.33	-15.43	342.32	7.0
MWELL82	12	52	382.03	333.9	349.33	-15.43	342.32	7.0
MWELL83	13	50	328.83	272.33	288.64	-16.31	282.72	5.9
MWELL84	13	49	253.47	182.69	197.03	-14.34	207.56	-10.5
MWELL85	11	39	152.68	81.77	87.57	-5.8	143.44	-55.9
MWELL86	10	44	198.93	97.9	114.54	-16.64	150.11	-35.6
MWELL87	11	41	182.32	91.36	99.77	-8.41	134.04	-34.3
MWELL88	9	45	218.89	104.95	114.59	-9.64	162.47	-47.9
MWELL89	10	45	198.93	97.9	114.54	-16.64	150.11	-35.6
MWELL90	6	32	46.25	47.07	35.16	11.91	43.07	-7.9
MWELL91	5	32	43.19	55.79	35.38	20.41	39.84	-4.5
MWELL92	4	33	43.19	55.79	35.38	20.41	39.84	-4.5
MWELL93	3	33	38.63	64.97	35.79	29.18	35.59	0.2
MWELL94	5	33	43.19	55.79	35.38	20.41	39.84	-4.5
MWELL95	5	34	67.42	51.92	47.15	4.77	65.97	-18.8
MWELL96	4	34	67.42	51.92	47.15	4.77	65.97	-18.8
MWELL97	3	35	58.17	38.39	45.69	-7.3	53.46	-7.8
MWELL98	4	31	25.62	24.18	27.43	-3.25	23.3	4.1
MWELL99	15	6	218.89	104.95	114.59	-9.64	162.47	-47.9
MWELL100	16	7	0	0	0	0	0	0.0
MWELL101	17	7	0	0	0	0	0	0.0
MWELL102	18	7	0	0	0	0	0	0.0
MWELL103	20	6	0	0	0	0	0	0.0
MWELL104	21	5	0	0	0	0	0	0.0
MWELL105	22	5	0	0	0	0	0	0.0
MWELL106	18	8	0	0	0	0	0	0.0
MWELL107	18	9	0	0	0	0	0	0.0
MWELL108	18	10	0	0	0	0	0	0.0
MWELL109	18	11	0	0	0	0	0	0.0
MWELL110	17	10	0	0	0	0	0	0.0
MWELL111	19	5	0	0	0	0	0	0.0
MWELL112	18	4	0	0	0	0	0	0.0
MWELL113	18	3	0	0	0	0	0	0.0
MWELL114	19	3	0	0	0	0	0	0.0
MWELL115	20	2	0	0	0	0	0	0.0
MWELL116	21	2	0	0	0	0	0	0.0
MWELL117	13	23	0	0	0	0	0	0.0

Table 5.11 - Continued

Well Grid Cell ID	Numerical Grid Cell Location Row	Column	Year 1988 Chloride Concentration (mg/l)	Projected Year 2010 Chloride Concentration (mg/l)	Simulation Model Optimal Scenario Cl concentration (mg/l)	Difference (Proj-Sim model) Cl Concentration (mg/l)	Optimization Model 5 Optimal Scenario Cl concentration (mg/l)	Difference (Sim-Opt model) Cl Concentration (mg/l)
MWELL118	12	24	0	0.01	0.14	-0.13	0	0.1
MWELL119	8	39	157.36	95.59	81.38	14.21	149	-67.6
MWELL120	7	34	76.84	73.26	50.12	23.14	73.73	-23.6
AGWELL1	54	47	167.15	150.81	156.89	-6.08	165.29	-8.4
AGWELL2	54	46	167.15	150.81	156.89	-6.08	165.29	-8.4
AGWELL3	17	54	157.36	95.59	81.38	14.21	149	-67.6
AGWELL4	16	51	182.32	91.36	99.77	-8.41	134.04	-34.3
AGWELL5	17	53	198.93	97.9	114.54	-16.64	150.11	-35.6
AGWELL6	45	58	415.88	407.73	412.12	-4.39	415.88	-3.8
AGWELL7	46	58	415.88	407.73	412.12	-4.39	415.88	-3.8
AGWELL8	8	59	1618.05	1550.58	1590.6	-40.02	1613.89	-23.3
AGWELL9	9	58	941.15	822.43	922.69	-100.26	931.1	-8.4
AGWELL10	7	59	1702.42	1598.14	1694.44	-96.3	1702.42	-8.0
AGWELL11	6	59	1702.42	1598.14	1694.44	-96.3	1702.42	-8.0
AGWELL12	6	58	964.86	813.79	959.38	-145.59	959.6	-0.2
AGWELL13	4	50	312.28	173.23	146.1	27.13	281.57	-135.5
AGWELL14	4	48	249.74	127.36	111.33	16.03	212.93	-101.6
AGWELL15	5	49	249.74	127.36	111.33	16.03	212.93	-101.6
AGWELL16	3	49	307.63	163.63	110.27	53.36	271.27	-161.0
AGWELL17	56	33	0	0	0	0	0	0.0
AGWELL18	56	35	0	0	0	0	0	0.0
AGWELL19	59	17	0	0	0	0	0	0.0
AGWELL20	59	18	0	0	0	0	0	0.0
AGWELL21	60	17	0	0	0	0	0	0.0
AGWELL22	57	35	0	0	0	0	0	0.0
AGWELL23	58	37	0	0	0	0	0	0.0
AGWELL24	58	35	0	0	0	0	0	0.0
AGWELL25	59	36	0	0	0	0	0	0.0
AGWELL26	32	47	107.68	114.04	119	-4.96	104.31	14.7
AGWELL27	31	49	125.73	130.31	135.31	-5	120.16	15.2
AGWELL28	32	48	130.52	137.18	141.54	-4.36	125.85	15.7
Maximum			1702.4	1598.1	1694.4	53.4	1702.4	60.9
Minimum			0.0	0.0	0.0	-145.6	0.0	-161.0
Average			108.1	96.8	101.0	-4.8	101.6	-0.6
Standard Deviation			299.8	276.0	292.1	21.9	296.8	28.2
Variance			91180.4	77354.4	86623.1	618.9	89458.2	984.8

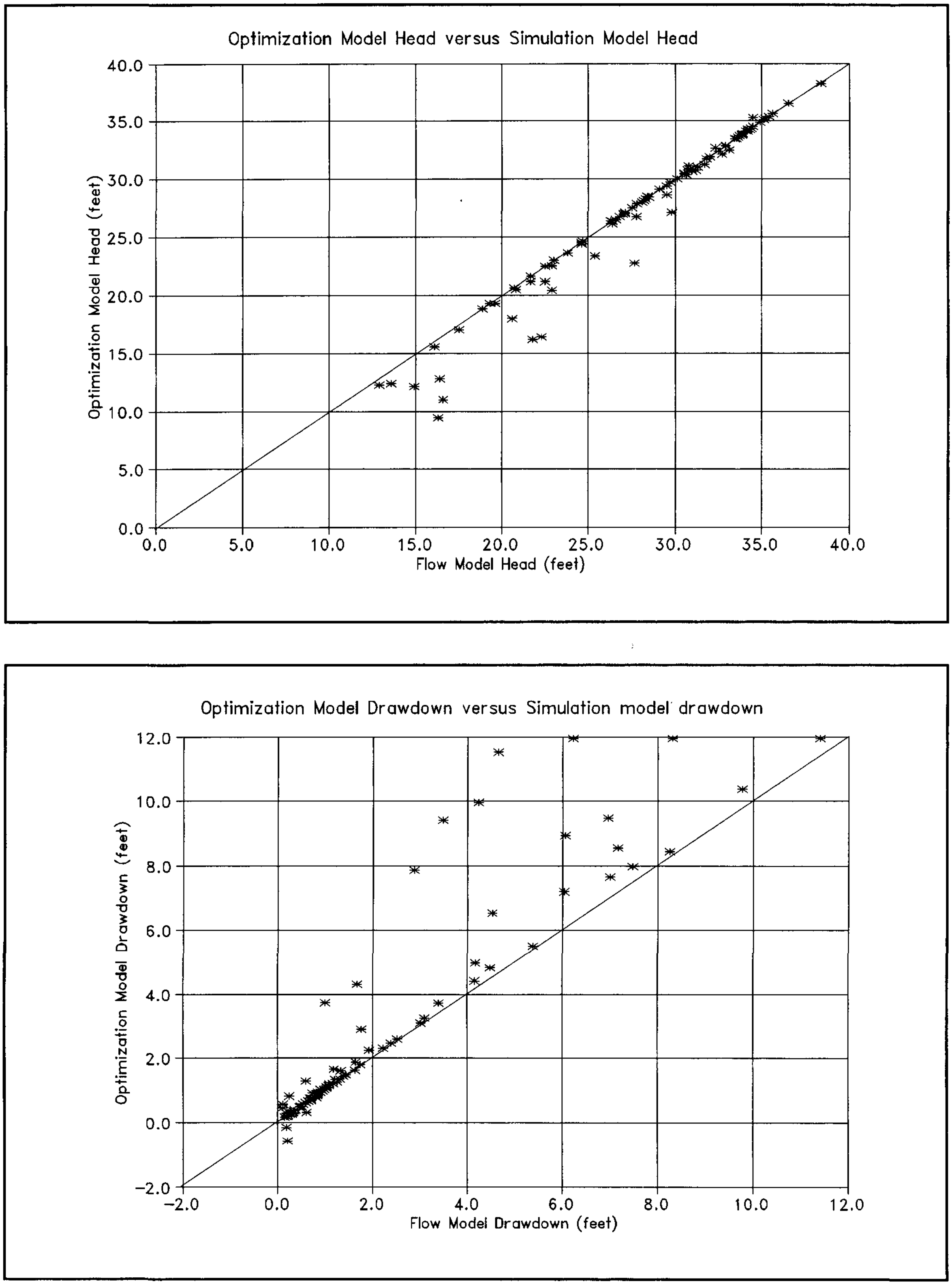


Figure 5.28 - Correlation between simulation model results and revised optimization model 5 results.
a) Pressure heads for year 2010. b) Drawdowns from year 1988 to 2010.

drawdown values, respectively, for the entire Volusia County subregion when the optimum strategy is simulated with the ground water flow model MODFLOW. Figure 5.32 displays the predicted chloride concentrations in the upper Floridan aquifer when the optimal strategy is simulated with the solute transport model DSTRAM.

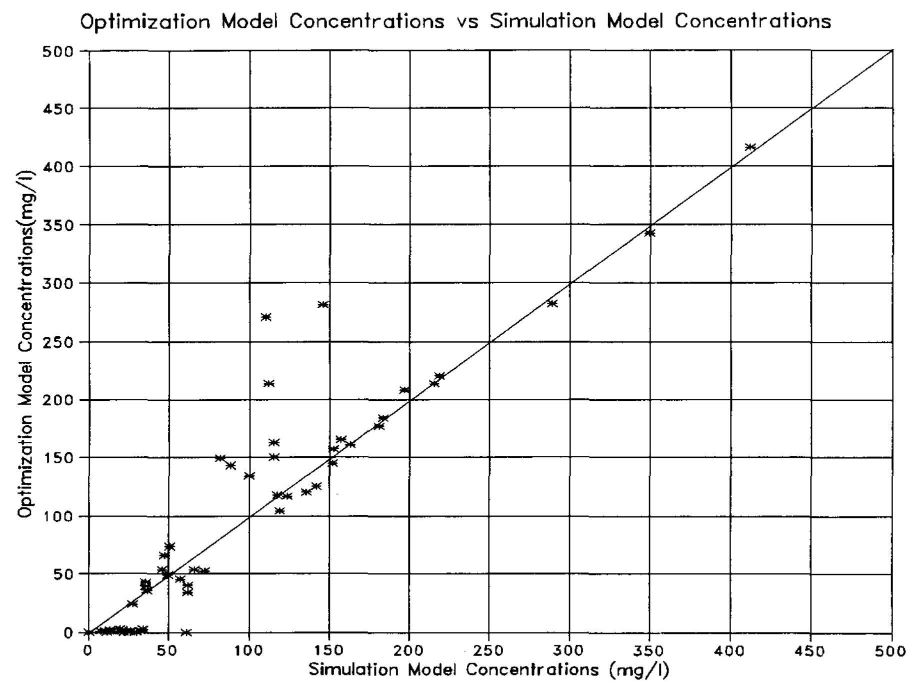


Figure 5.29 - Chloride Concentration correlation between simulation model and optimization model 5.

5.6.5 General Observations

To evaluate the results of the optimization models, the prescribed water resource allocation strategy produced by each model was compared to the current projected year 2010 strategy. Tables 5.2 and 5.3 display the various water allocation strategies including the withdrawal rates experienced in year 1988. Table 5.12 displays the predicted pressure head and drawdown values at the 100 sensitive wetland control points under the projected year 2010 allocation strategy and the optimized allocation strategies.

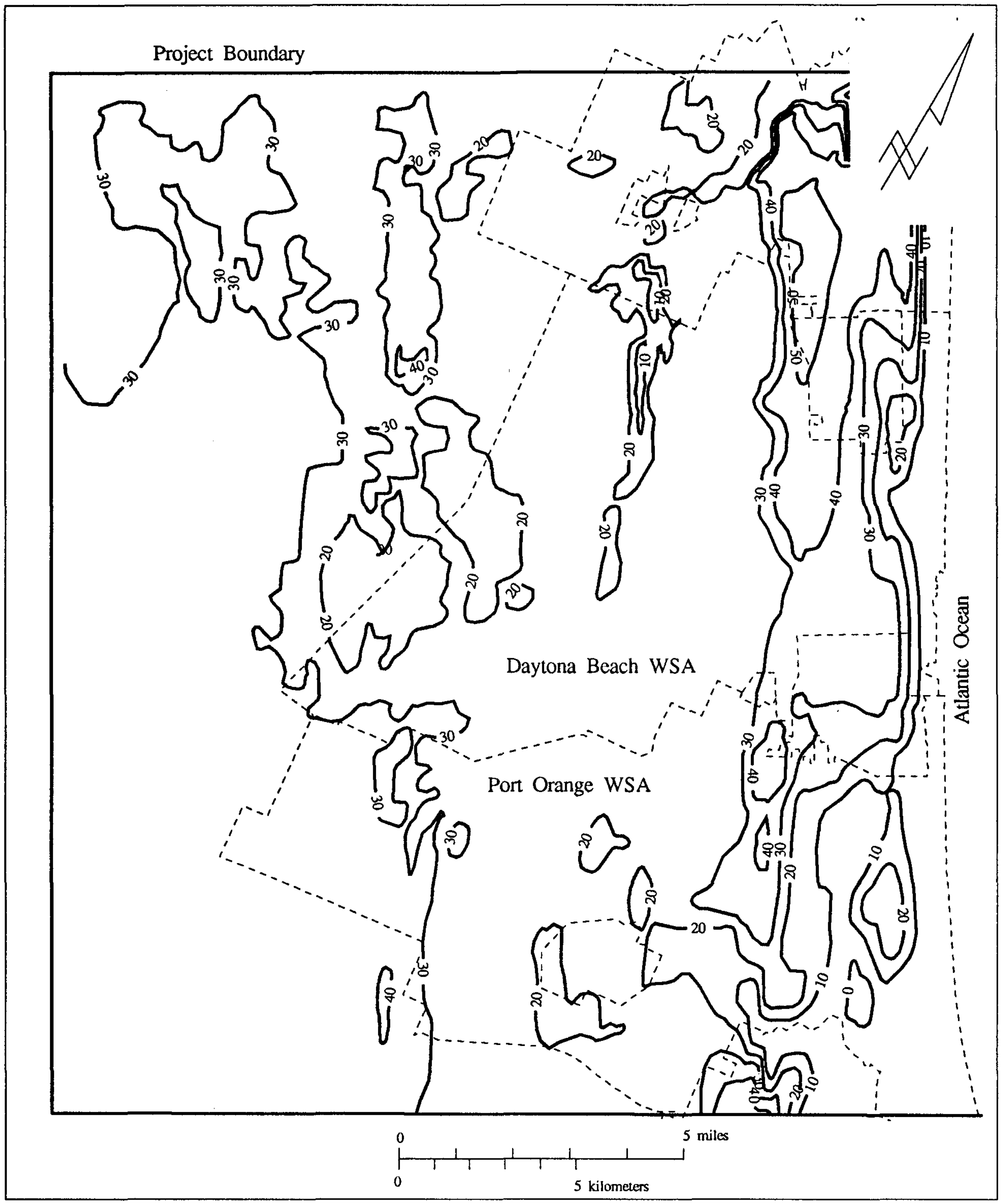


Figure 5.30

Flow simulation model pressure head results in the surficial aquifer due to Optimization Model 5 withdrawal strategy for year 2010.

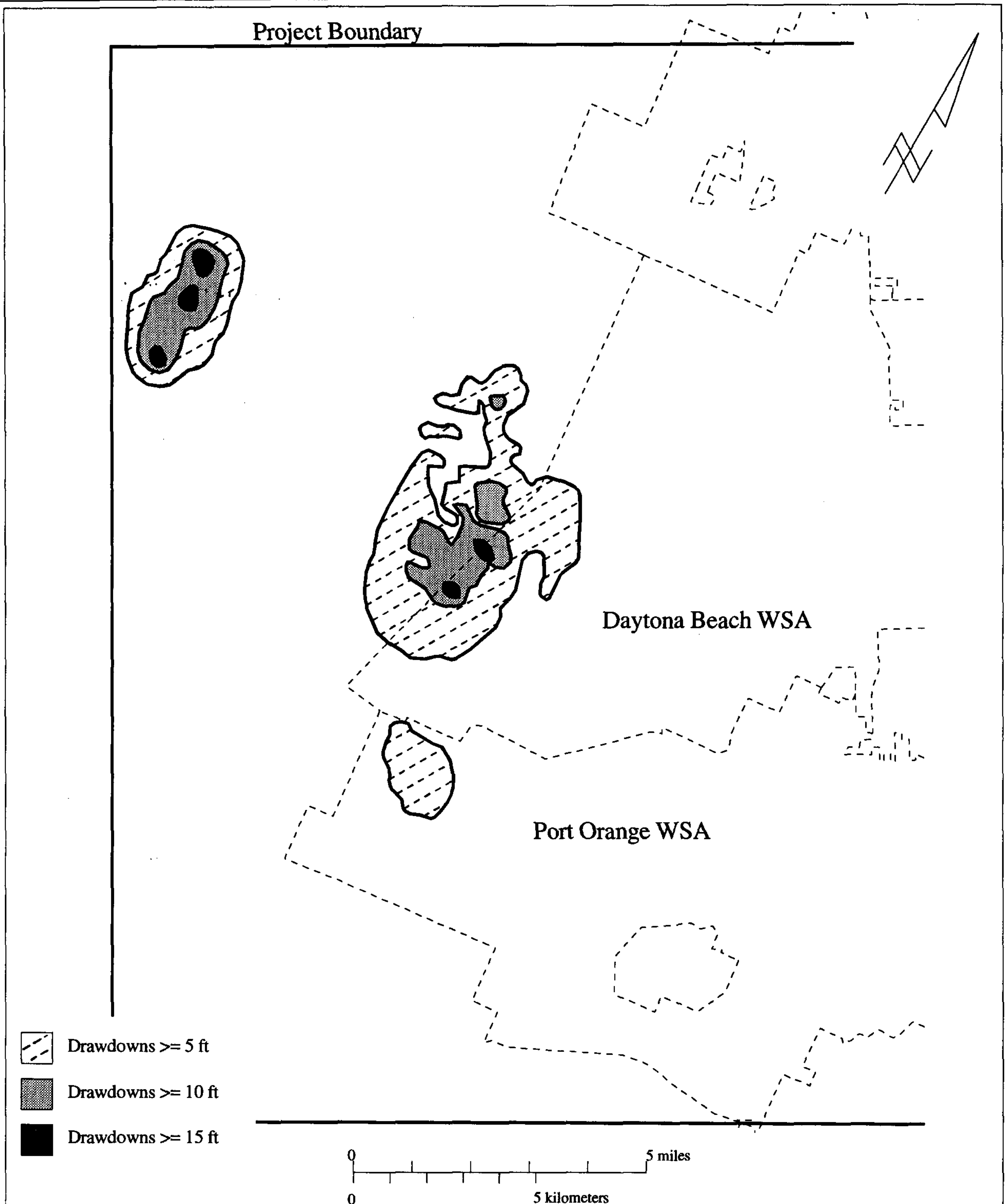


Figure 5.31

Flow simulation model drawdown results in the surficial aquifer due to Optimization Model 5 withdrawal strategy for year 2010.

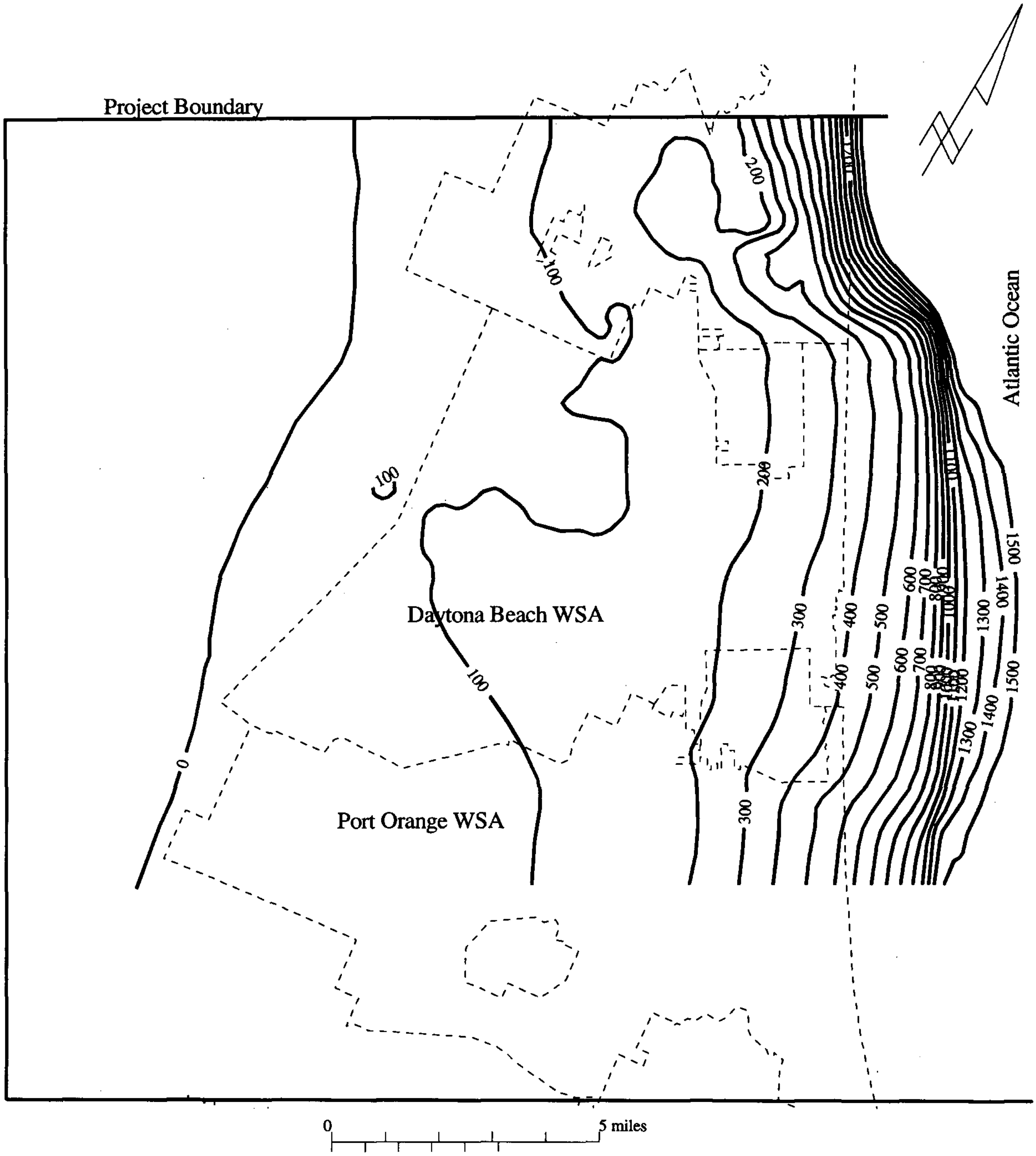


Figure 5.32

Simulation model chloride concentration results in the upper floridan aquifer due to Optimization Model 5 withdrawal strategy for year 2010.

Table 5.12 - Pressure head, drawdown, and difference between projected and optimized values at sensitive wetland control points.

Control Point	Numerical Grid Cell Location		Projected Year 2010		Model 1 - Min Max Drawdown			Model 2 - Max Min Head			Model 3 - Min Avg Drawdown			Model 4 - Min Max Drawdown			Model 5 - Min Max Relative CC		
	row	column	Head (feet)	DD (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)
1	5	1	29.1	0.53	28.4	1.31	0.78	28.7	1.00	0.47	29.2	0.51	-0.02	28.4	1.26	0.73	28.5	1.18	0.65
2	8	1	29.3	0.57	28.2	1.67	1.10	28.6	1.25	0.68	29.4	0.52	-0.05	28.3	1.64	1.07	28.4	1.5	0.93
3	15	1	28.9	0.89	26.9	2.86	1.97	27.8	2.05	1.16	29.1	0.67	-0.22	26.7	3.09	2.20	27.3	2.53	1.64
4	19	1	32.4	1.21	30.2	3.44	2.23	31.7	1.94	0.73	32.8	0.80	-0.41	28.6	4.98	3.77	30.2	3.36	2.15
5	22	1	33.7	1.03	31.0	3.68	2.65	33.4	1.26	0.23	33.9	0.77	-0.26	30.3	4.41	3.38	30.5	4.23	3.20
6	25	1	35.9	0.71	34.3	2.23	1.52	35.8	0.81	0.10	35.9	0.64	-0.07	34.1	2.48	1.77	34.1	2.47	1.76
7	28	1	33.8	0.53	33.0	1.35	0.82	33.8	0.57	0.04	33.8	0.51	-0.02	32.8	1.49	0.96	32.8	1.49	0.96
8	35	1	34.0	0.32	33.9	0.47	0.15	34.0	0.31	-0.01	34.0	0.29	-0.03	33.8	0.51	0.19	33.8	0.55	0.23
9	41	1	34.1	0.30	34.1	0.30	0.00	34.1	0.28	-0.02	34.2	0.24	-0.06	34.1	0.31	0.01	34.0	0.37	0.07
10	47	1	34.1	0.33	34.2	0.27	-0.06	34.1	0.31	-0.02	34.2	0.25	-0.08	34.1	0.27	-0.06	34.1	0.33	0.00
11	53	1	34.4	0.39	34.4	0.32	-0.07	34.4	0.38	-0.01	34.4	0.31	-0.08	34.5	0.27	-0.12	34.5	0.32	-0.07
12	18	2	30.4	1.70	26.9	5.19	3.49	28.7	3.34	1.64	30.9	1.12	-0.58	23.7	8.45	6.75	27.6	4.52	2.82
13	56	2	35.4	0.47	35.5	0.40	-0.07	35.4	0.49	0.02	35.5	0.36	-0.11	35.6	0.29	-0.18	35.6	0.34	-0.13
14	16	3	28.5	1.48	24.9	5.05	3.57	26.4	3.56	2.08	28.8	1.09	-0.39	24.4	5.51	4.03	25.7	4.25	2.77
15	30	3	34.6	0.63	34.0	1.32	0.69	34.6	0.66	0.03	34.6	0.63	0.00	33.9	1.45	0.82	33.8	1.48	0.85
16	32	3	34.7	0.55	34.3	0.98	0.43	34.7	0.56	0.01	34.7	0.54	-0.01	34.2	1.08	0.53	34.2	1.13	0.58
17	37	3	35.3	0.43	35.2	0.53	0.10	35.4	0.41	-0.02	35.4	0.37	-0.06	35.2	0.58	0.15	35.2	0.65	0.22
18	54	3	33.9	0.47	34.0	0.36	-0.11	33.9	0.46	-0.01	34.0	0.35	-0.12	34.1	0.29	-0.18	34.1	0.35	-0.12
19	17	4	31.4	1.80	22.7	10.42	8.62	29.2	3.98	2.18	31.8	1.33	-0.47	21.2	11.93	10.13	25.9	7.2	5.40
20	41	5	34.3	0.52	34.4	0.47	-0.05	34.4	0.47	-0.05	34.5	0.39	-0.13	34.3	0.49	-0.03	34.2	0.6	0.08
21	44	5	33.8	0.50	33.9	0.39	-0.11	33.8	0.45	-0.05	33.9	0.35	-0.15	33.9	0.4	-0.10	33.8	0.5	0.00
22	47	5	35.2	0.47	35.4	0.34	-0.13	35.3	0.43	-0.04	35.4	0.32	-0.15	35.4	0.33	-0.14	35.3	0.43	-0.04
23	53	5	34.9	0.46	35.0	0.32	-0.14	34.9	0.44	-0.02	35.0	0.31	-0.15	35.0	0.26	-0.20	35.0	0.34	-0.12
24	30	6	34.9	0.70	34.3	1.25	0.55	34.8	0.70	0.00	34.8	0.73	0.03	34.2	1.37	0.67	34.2	1.4	0.70
25	32	6	35.3	0.63	34.9	0.97	0.34	35.3	0.61	-0.02	35.3	0.61	-0.02	34.8	1.06	0.43	34.8	1.12	0.49
26	50	6	36.3	0.49	36.4	0.32	-0.17	36.3	0.45	-0.04	36.4	0.31	-0.18	36.5	0.29	-0.20	36.4	0.39	-0.10
27	59	7	33.9	0.57	33.8	0.67	0.10	33.7	0.78	0.21	34.2	0.33	-0.24	34.3	0.23	-0.34	34.3	0.25	-0.32
28	58	8	33.9	0.61	33.9	0.64	0.03	33.8	0.76	0.15	34.2	0.33	-0.28	34.3	0.23	-0.38	34.3	0.25	-0.36
29	39	9	32.4	0.64	32.5	0.57	-0.07	32.5	0.57	-0.07	32.5	0.49	-0.15	32.4	0.62	-0.02	32.2	0.76	0.12
30	60	9	33.3	0.78	33.0	1.08	0.30	32.8	1.22	0.44	33.6	0.43	-0.35	33.8	0.29	-0.49	33.8	0.28	-0.50
31	16	10	29.5	1.69	27.6	3.58	1.89	28.9	2.37	0.68	29.3	1.90	0.21	28.0	3.24	1.55	28.0	3.19	1.50
32	32	10	33.8	0.79	33.6	0.94	0.15	33.8	0.72	-0.07	33.8	0.74	-0.05	33.6	1.03	0.24	33.5	1.14	0.35
33	14	11	29.4	1.24	27.8	2.84	1.60	28.7	2.00	0.76	29.3	1.39	0.15	28.1	2.6	1.36	28.1	2.56	1.32
34	34	11	30.9	0.83	30.9	0.83	0.00	31.0	0.74	-0.09	31.0	0.71	-0.12	30.8	0.92	0.09	30.6	1.07	0.24
35	52	11	33.2	0.65	33.6	0.28	-0.37	33.3	0.58	-0.07	33.5	0.31	-0.34	33.6	0.23	-0.42	33.4	0.39	-0.26

Table 5.12 - continued

Control	Numerical Grid Cell Location		Projected Year 2010		Model 1 - Min Max Drawdown			Model 2 - Max Min Head			Model 3 - Min Avg Drawdown			Model 4 - Min Max Drawdown			Model 5 - Min Max Relative CC		
Point	row	column	Head (feet)	DD (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)
36	54	11	33.4	0.57	33.7	0.29	-0.28	33.4	0.56	-0.01	33.7	0.29	-0.28	33.8	0.2	-0.37	33.7	0.32	-0.25
37	59	11	33.0	1.10	32.4	1.68	0.58	32.2	1.86	0.76	33.5	0.54	-0.56	33.7	0.38	-0.72	33.8	0.3	-0.80
38	17	12	31.7	1.47	30.7	2.46	0.99	31.5	1.70	0.23	31.2	1.94	0.47	30.9	2.31	0.84	30.9	2.27	0.80
39	31	12	30.6	0.95	30.5	1.02	0.07	30.7	0.84	-0.11	30.7	0.88	-0.07	30.4	1.11	0.16	30.2	1.26	0.31
40	37	12	30.4	0.92	30.6	0.77	-0.15	30.5	0.81	-0.11	30.6	0.71	-0.21	30.4	0.88	-0.04	30.2	1.08	0.16
41	39	12	32.8	0.87	33.0	0.67	-0.20	32.9	0.76	-0.11	33.0	0.64	-0.23	32.8	0.77	-0.10	32.6	0.97	0.10
42	49	12	31.5	0.74	31.9	0.28	-0.46	31.6	0.60	-0.14	31.9	0.33	-0.41	31.9	0.27	-0.47	31.7	0.47	-0.27
43	43	13	30.1	0.90	30.5	0.48	-0.42	30.3	0.73	-0.17	30.5	0.51	-0.39	30.5	0.53	-0.37	30.2	0.78	-0.12
44	46	13	30.2	0.85	30.7	0.34	-0.51	30.4	0.65	-0.20	30.7	0.39	-0.46	30.7	0.37	-0.48	30.5	0.61	-0.24
45	16	14	29.0	1.16	28.3	1.92	0.76	28.8	1.38	0.22	28.7	1.48	0.32	28.4	1.81	0.65	28.4	1.84	0.68
46	33	14	29.9	1.24	30.1	1.07	-0.17	30.1	1.04	-0.20	30.2	1.00	-0.24	30.0	1.2	-0.04	29.8	1.44	0.20
47	54	14	32.9	0.76	33.4	0.28	-0.48	32.9	0.78	0.02	33.3	0.32	-0.44	33.4	0.19	-0.57	33.2	0.36	-0.40
48	59	14	31.6	2.13	27.4	6.35	4.22	26.6	7.07	4.94	32.8	0.92	-1.21	32.4	1.28	-0.85	33.3	0.38	-1.75
49	14	15	26.8	0.97	26.2	1.57	0.60	26.6	1.17	0.20	26.6	1.16	0.19	26.3	1.49	0.52	26.3	1.54	0.57
50	41	15	28.1	2.68	30.0	0.79	-1.89	28.8	1.99	-0.69	29.9	0.94	-1.74	27.1	3.75	1.07	25.7	5.13	2.45
51	5	16	27.3	0.67	27.0	0.89	0.22	27.2	0.71	0.04	27.1	0.80	0.13	27.1	0.84	0.17	27.0	0.89	0.22
52	12	16	30.0	0.87	29.6	1.29	0.42	29.9	0.99	0.12	29.8	1.03	0.16	29.7	1.22	0.35	29.6	1.3	0.43
53	38	16	23.9	6.03	25.9	4.04	-1.99	24.5	5.39	-0.64	25.7	4.22	-1.81	23.4	6.53	0.50	21.5	8.39	2.36
54	47	16	33.3	1.35	34.5	0.18	-1.17	33.9	0.78	-0.57	34.4	0.33	-1.02	35.3	-0.58	-1.93	34.7	0.03	-1.32
55	50	16	26.8	4.33	31.0	0.12	-4.21	28.7	2.39	-1.94	30.8	0.32	-4.01	30.6	0.47	-3.86	28.9	2.16	-2.17
56	53	16	32.4	1.20	33.4	0.21	-0.99	32.4	1.22	0.02	33.2	0.34	-0.86	33.4	0.16	-1.04	33.1	0.46	-0.74
57	60	16	28.9	3.09	27.4	4.64	1.55	26.6	5.45	2.36	29.9	2.08	-1.01	31.7	0.29	-2.80	31.9	0.07	-3.02
58	3	17	23.0	0.77	22.9	0.85	0.08	23.1	0.70	-0.07	22.8	0.96	0.19	23.0	0.8	0.03	22.9	0.87	0.10
59	7	17	24.8	0.74	24.6	0.91	0.17	24.8	0.72	-0.02	24.6	0.90	0.16	24.6	0.86	0.12	24.6	0.94	0.20
60	10	17	26.5	0.77	26.3	1.00	0.23	26.5	0.78	0.01	26.3	0.91	0.14	26.4	0.95	0.18	26.3	1.04	0.27
61	14	17	29.6	0.90	29.4	1.18	0.28	29.6	0.93	0.03	29.5	1.05	0.15	29.4	1.13	0.23	29.3	1.24	0.34
62	30	17	24.7	5.87	27.6	3.02	-2.85	27.5	3.08	-2.79	27.6	3.01	-2.86	22.7	7.87	2.00	20.8	9.82	3.95
63	35	17	20.4	9.32	23.1	6.60	-2.72	21.9	7.79	-1.53	22.9	6.77	-2.55	21.2	8.54	-0.78	18.9	10.79	1.47
64	44	17	29.1	2.28	30.8	0.53	-1.75	30.1	1.22	-1.06	30.7	0.70	-1.58	31.1	0.31	-1.97	29.9	1.54	-0.74
65	55	17	31.3	1.15	32.1	0.36	-0.79	30.4	2.03	0.88	32.0	0.44	-0.71	32.7	-0.16	-1.31	32.3	0.23	-0.92
66	57	17	29.0	3.93	31.5	1.44	-2.49	29.9	3.01	-0.92	31.6	1.29	-2.64	32.1	0.81	-3.12	31.6	1.28	-2.65
67	27	18	30.6	2.30	31.1	1.82	-0.48	31.1	1.76	-0.54	31.1	1.83	-0.47	31.0	1.89	-0.41	30.5	2.37	0.07
68	32	18	18.3	11.59	23.2	6.74	-4.85	22.4	7.46	-4.13	23.0	6.87	-4.72	20.4	9.49	-2.10	17.8	12.07	0.48
69	37	18	14.7	11.20	17.1	8.79	-2.41	14.9	10.93	-0.27	16.7	9.21	-1.99	15.5	10.37	-0.83	12.8	13.11	1.91
70	40	18	15.7	9.34	18.3	6.69	-2.65	15.7	9.31	-0.03	17.7	7.25	-2.09	17.0	7.97	-1.37	14.5	10.53	1.19

Table 5.12 - continued

Control	Numerical Grid Cell Location		Projected Year 2010		Model 1 - Min Max Drawdown			Model 2 - Max Min Head			Model 3 - Min Avg Drawdown			Model 4 - Min Max Drawdown			Model 5 - Min Max Relative CC		
Point	row	column	Head (feet)	DD (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)	Head (feet)	DD (feet)	Diff (feet)
71	42	18	17.3	8.05	21.4	3.97	-4.08	18.6	6.74	-1.31	20.7	4.61	-3.44	20.5	4.84	-3.21	18.0	7.32	-0.73
72	4	19	27.6	0.86	27.7	0.75	-0.11	27.9	0.63	-0.23	27.3	1.16	0.30	27.8	0.7	-0.16	27.7	0.81	-0.05
73	19	19	26.6	1.25	26.5	1.37	0.12	26.6	1.21	-0.04	26.4	1.41	0.16	26.5	1.33	0.08	26.2	1.61	0.36
74	6	20	28.9	0.84	29.0	0.69	-0.15	29.1	0.58	-0.26	28.6	1.14	0.30	29.0	0.66	-0.18	28.9	0.78	-0.06
75	9	20	29.0	0.84	29.1	0.75	-0.09	29.2	0.62	-0.22	28.8	1.03	0.19	29.1	0.72	-0.12	29.0	0.85	0.01
76	14	20	27.4	0.96	27.4	0.95	-0.01	27.5	0.81	-0.15	27.3	1.06	0.10	27.5	0.92	-0.04	27.3	1.1	0.14
77	17	20	27.0	1.14	26.9	1.21	0.07	27.1	1.07	-0.07	26.9	1.29	0.15	27.0	1.17	0.03	26.8	1.42	0.28
78	12	21	26.7	1.04	26.9	0.87	-0.17	27.0	0.74	-0.30	26.7	1.12	0.08	27.0	0.85	-0.19	26.8	1.02	-0.02
79	10	22	37.9	1.41	38.3	0.99	-0.42	38.4	0.85	-0.56	37.7	1.62	0.21	38.3	1.04	-0.37	38.0	1.28	-0.13
80	44	22	23.0	6.59	28.2	1.41	-5.18	25.0	4.53	-2.06	27.4	2.15	-4.44	26.7	2.9	-3.69	24.0	5.57	-1.02
81	48	22	17.9	7.53	24.8	0.58	-6.95	20.3	5.09	-2.44	23.6	1.78	-5.75	24.5	0.91	-6.62	21.6	3.77	-3.76
82	51	22	25.1	6.71	31.7	0.10	-6.61	26.3	5.56	-1.15	30.8	0.99	-5.72	31.2	0.56	-6.15	28.8	3.04	-3.67
83	46	23	26.7	5.67	31.4	0.99	-4.68	28.5	3.82	-1.85	30.7	1.67	-4.00	30.8	1.65	-4.02	28.3	4.07	-1.60
84	43	24	15.5	7.48	20.1	2.93	-4.55	17.1	5.93	-1.55	19.3	3.70	-3.78	19.3	3.73	-3.75	16.3	6.66	-0.82
85	20	25	26.2	1.47	26.1	1.58	0.11	26.2	1.51	0.04	25.9	1.76	0.29	26.1	1.62	0.15	25.6	2.15	0.68
86	26	25	21.8	4.30	19.5	6.56	2.26	19.5	6.52	2.22	19.0	7.03	2.73	16.2	9.95	5.65	12.8	13.35	9.05
87	30	25	3.8	19.07	15.7	7.19	-11.88	15.3	7.60	-11.47	15.2	7.63	-11.44	11.0	11.93	-7.14	6.8	16.13	-2.94
88	32	25	5.6	19.14	16.6	8.10	-11.04	15.8	8.93	-10.21	16.2	8.53	-10.61	12.8	11.93	-7.21	8.4	16.27	-2.87
89	7	26	18.7	1.16	19.3	0.52	-0.64	19.4	0.46	-0.70	18.0	1.83	0.67	19.3	0.5	-0.66	19.1	0.69	-0.47
90	10	26	18.4	1.03	18.8	0.59	-0.44	18.9	0.53	-0.50	18.0	1.40	0.37	18.8	0.58	-0.45	18.6	0.77	-0.26
91	14	26	21.3	1.02	21.6	0.77	-0.25	21.6	0.71	-0.31	21.1	1.21	0.19	21.6	0.76	-0.26	21.4	1.01	-0.01
92	17	26	22.3	1.05	22.4	0.97	-0.08	22.4	0.92	-0.13	22.1	1.25	0.20	22.5	0.94	-0.11	22.1	1.26	0.21
93	23	26	21.6	4.16	20.5	5.32	1.16	20.7	5.13	0.97	19.8	6.01	1.85	16.4	9.4	5.24	13.1	12.66	8.50
94	29	26	8.5	12.40	14.9	6.00	-6.40	14.8	6.16	-6.24	14.4	6.51	-5.89	9.4	11.52	-0.88	5.2	15.7	3.30
95	27	27	22.3	2.54	22.5	2.28	-0.26	22.5	2.25	-0.29	22.4	2.41	-0.13	22.6	2.25	-0.29	21.7	3.09	0.55
96	31	27	10.7	10.28	14.7	6.28	-4.00	14.4	6.56	-3.72	14.1	6.84	-3.44	12.1	8.93	-1.35	8.6	12.42	2.14
97	35	27	10.2	9.73	13.5	6.39	-3.34	12.5	7.41	-2.32	13.0	6.94	-2.79	12.3	7.65	-2.08	8.7	11.22	1.49
98	33	28	11.8	7.79	13.8	5.78	-2.01	13.5	6.17	-1.62	13.3	6.36	-1.43	12.4	7.2	-0.59	9.4	10.16	2.37
99	37	28	19.0	3.32	21.0	1.31	-2.01	20.1	2.15	-1.17	20.5	1.78	-1.54	18.0	4.32	1.00	15.4	6.92	3.60
100	39	29	20.3	1.35	20.7	0.86	-0.49	20.5	1.12	-0.23	20.6	1.01	-0.34	20.6	1	-0.35	19.9	1.66	0.31
Maximum			37.9	19.14	38.3	10.42	-8.72	38.4	10.93	-8.21	37.7	9.21	-9.93	38.26	11.93	1.51	38.02	16.27	-2.87
Minimum			3.8	0.3	13.5	0.1	-0.20	12.5	0.28	-0.02	13	0.24	-0.06	9.38	-0.58	-0.68	5.2	0.03	-0.27
Average			27.60	2.71	28.24	2.07	-0.64	28.03	2.28	-0.43	28.54	1.77	-0.95	27.79	2.53	0.46	27.16	3.16	0.44

This table also includes the difference in aquifer response values between the projected and optimum strategies. For comparison, Figures 5.33 and 5.34 display the predicted pressure head elevations and drawdown values, respectively, when the projected year 2010 strategy is simulated with MODFLOW. In addition, Figure 5.35 displays results when the projected year 2010 withdrawal strategy is simulated with DSTRAM.

Using model 1, maximum drawdown values (from year 1988 to 2010) decreased from 19 feet with the projected strategy to 10 feet with the optimized strategy. For model 2, minimum pressure head increased from approximately 4 feet with year 2010 projected strategy to approximately 13 feet with the optimized strategy. With the use of model 3, minimum average drawdown decreased from 2.7 feet with the projected strategy to 1.8 feet with the optimized strategy. And with model 4, maximum drawdown values (from year 1988 to 2010) decreased from 19 feet with the projected strategy to 11 feet with the optimized strategy. Maximum drawdowns are minimized with model 4; however, to achieve lesser drawdowns in the west, chloride concentrations generally increased along the eastern coastline. Minimizing the relative chloride concentration increase with model 5 caused simular incroachment of the chloride concentration front. Finally in all the models, the optimized withdrawal strategies minimized projected year 2010 impacts in the Daytona Beach water services area by expanding pumpage into neighboring eastern or northwestern areas where there are fewer vegetative harm control points.

The minimum discharge constraint was of considerable importance in determining the optimum allocation strategy. This constraint specifies that withdrawal rates at all active wells must be at least 50 percent of the 1988 flow. Most municipal well grid cells supplying ground water in year 1988, were pumping at the minimum level under 2010

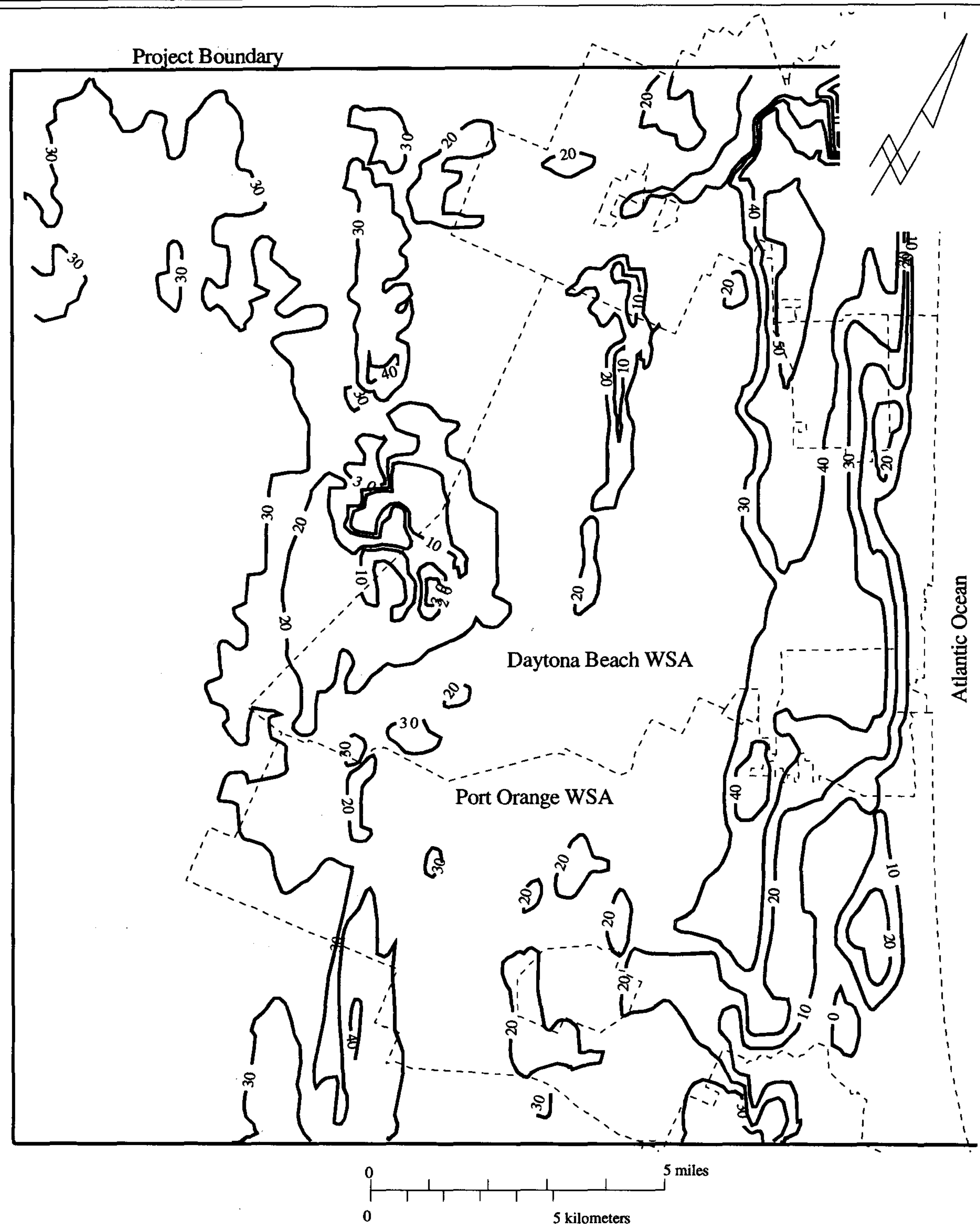


Figure 5.33

Flow simulation model pressure head results in the surficial aquifer due to the projected year 2010 withdrawal strategy

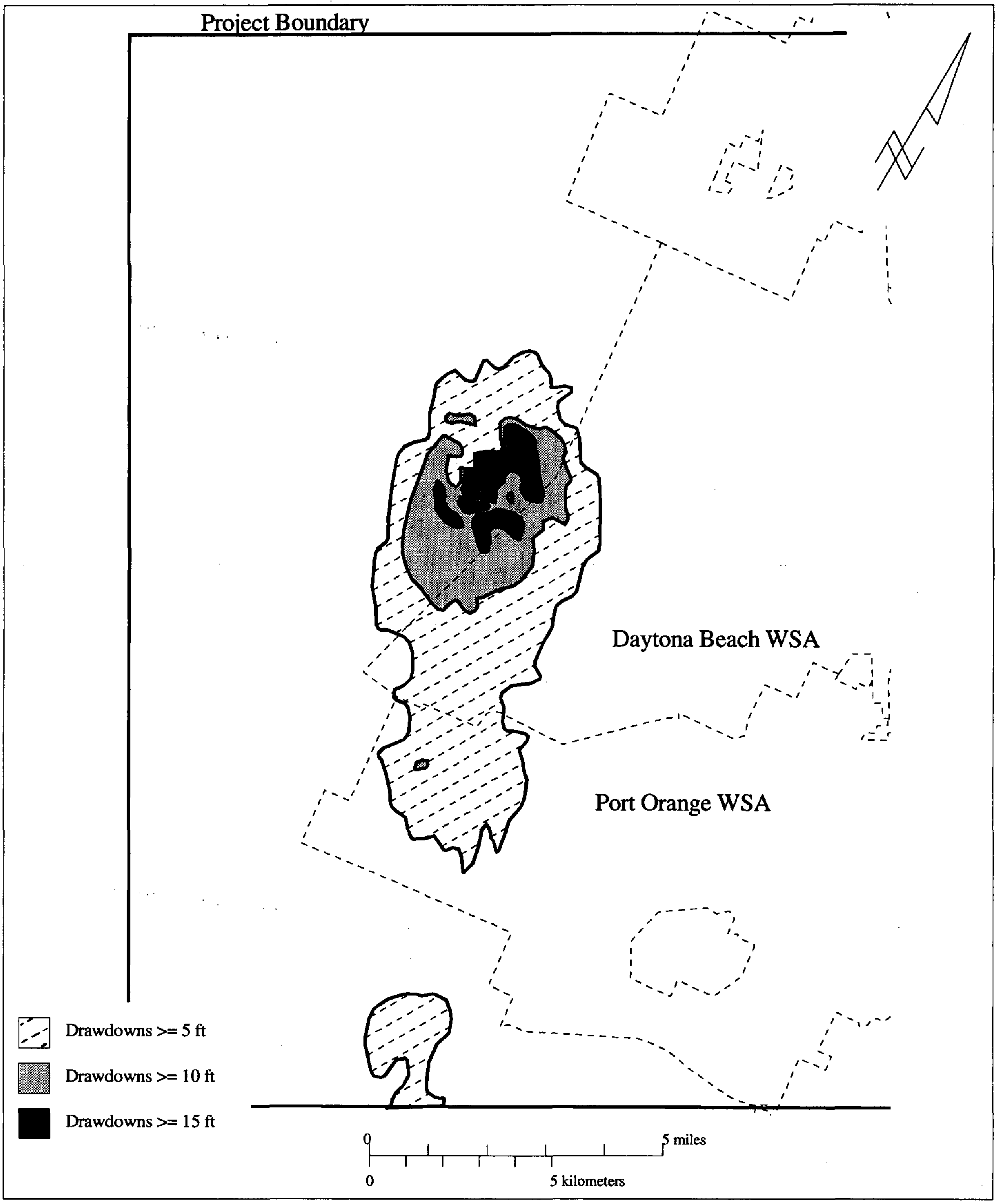


Figure 5.34

Flow simulation model drawdown results in the surficial aquifer due to the projected year 2010 withdrawal strategy.

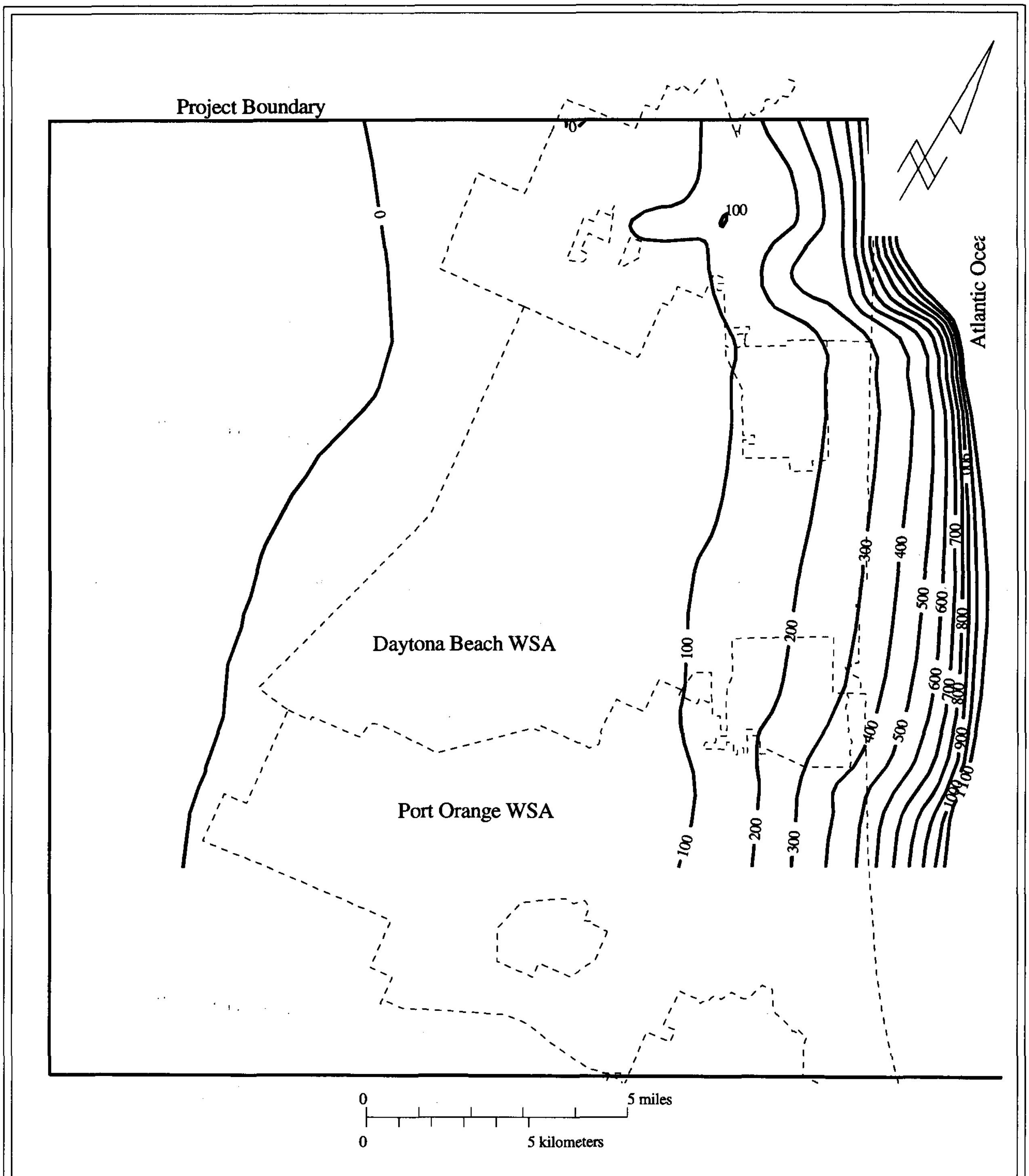


Figure 5.35
Simulation model chloride concentration results in the upper floridan aquifer due to the projected year 2010 withdrawal strategy.

optimum allocation strategy.

For 5 of the 12 municipal water service areas, there exists well grid cell locations which did not supply ground water in year 1988. These include Port Orange West well field, Daytona Beach West - South Daytona water treatment plant, Smyrna Beach/Samsula well field, and Ormond Beach State Road 40 and Hudson well fields. In the remaining water service areas, municipal well grid cells used in year 1988 are the only cells available to meet the demands of year 2010.

The location of the most favorable municipal well grid cells with respect to minimizing adverse environmental effects can be identified from Table 5.2. According to this analysis, future increases in withdrawal should be located at the following municipal well grid cells:

Port Orange:	cells 19,20,21,23,24,32.
Daytona Beach:	cells 46,48,52,58,59,64.
Spruce Creek:	cells 65,66.
Holly Hill:	cell 67,73.
Smyrna Beach:	cells 75,77.
Ormond Beach:	cells 80,82,89,97,99,113.
Tymber Creek:	cell 119
The Trails:	cell 120

To further evaluate the optimization process, pressure head elevations and drawdown values induced by the four optimum year 2010 allocation strategies were all graphically compared to those induced by the projected year 2010 strategy. Differences in induced pressure heads between the projected and optimum strategies are displayed in Figures 5.36 through 5.40 for optimization models 1 through 5, respectively. Negative

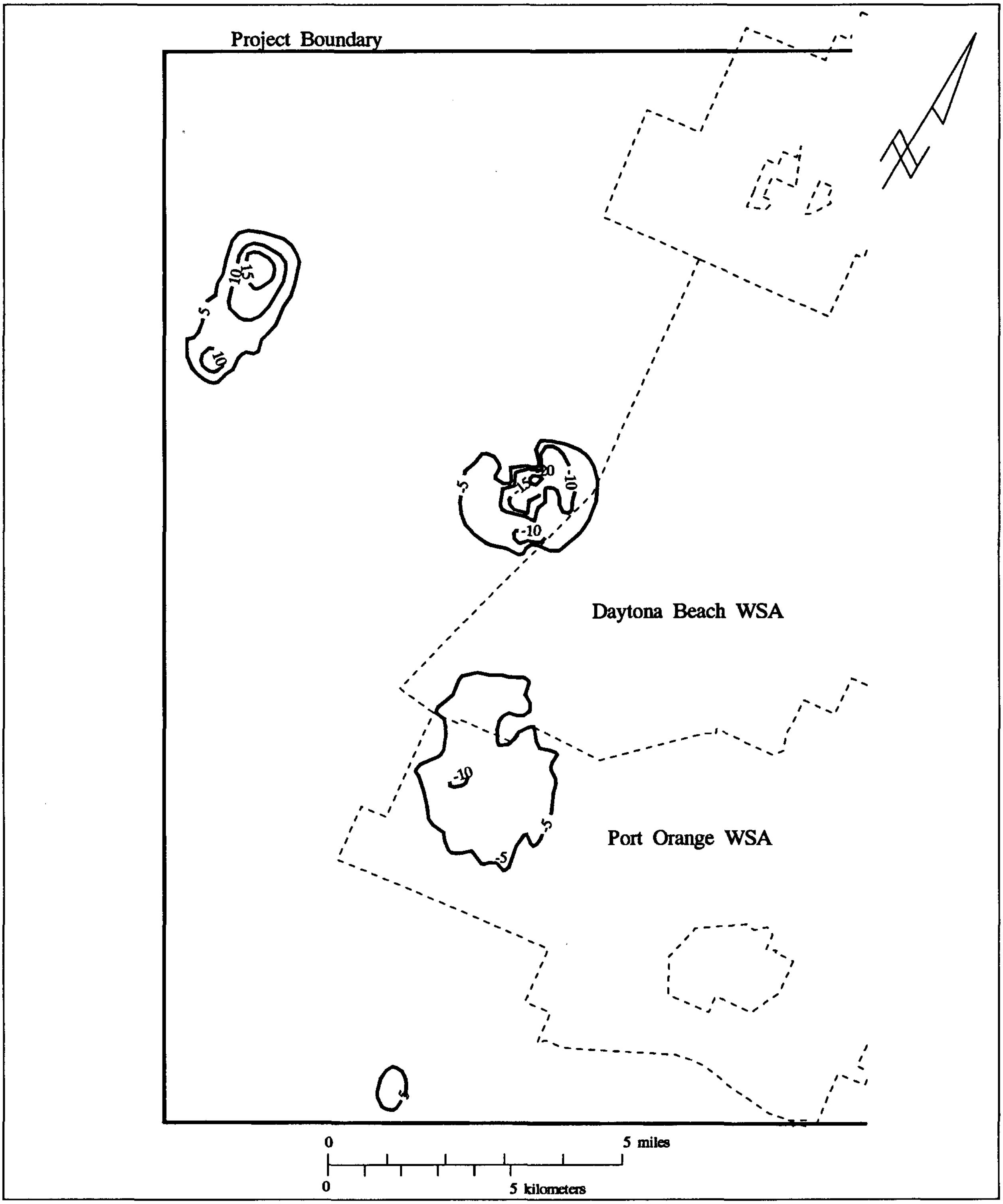


Figure 5.36

Difference in induced pressure head between the projected year 2010 strategy and the strategy identified by Model 1 (Projected - Optimized).

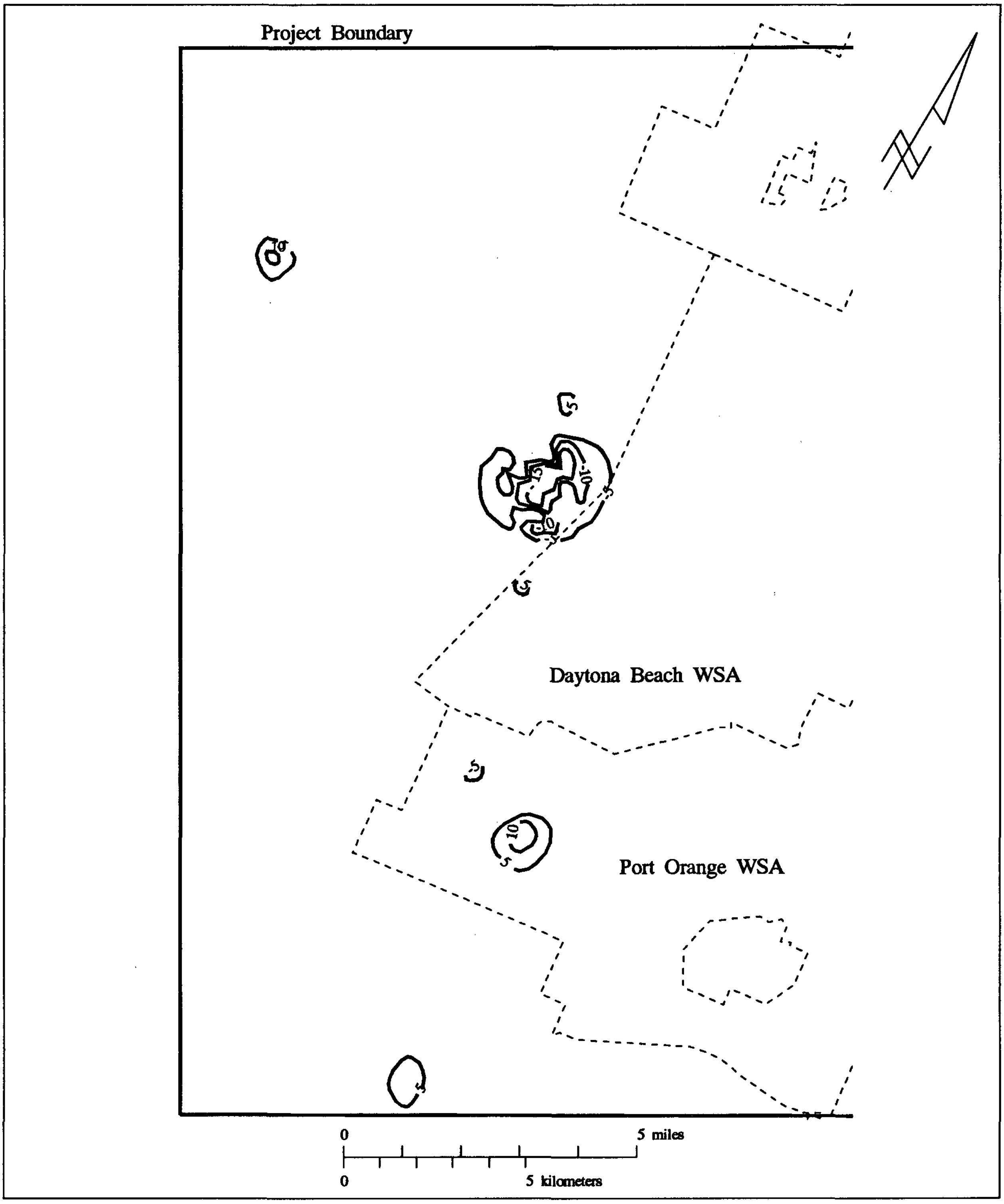


Figure 5.37

Difference in induced pressure head between the projected year 2010 strategy and the strategy identified by Model 2 (Projected - Optimized).

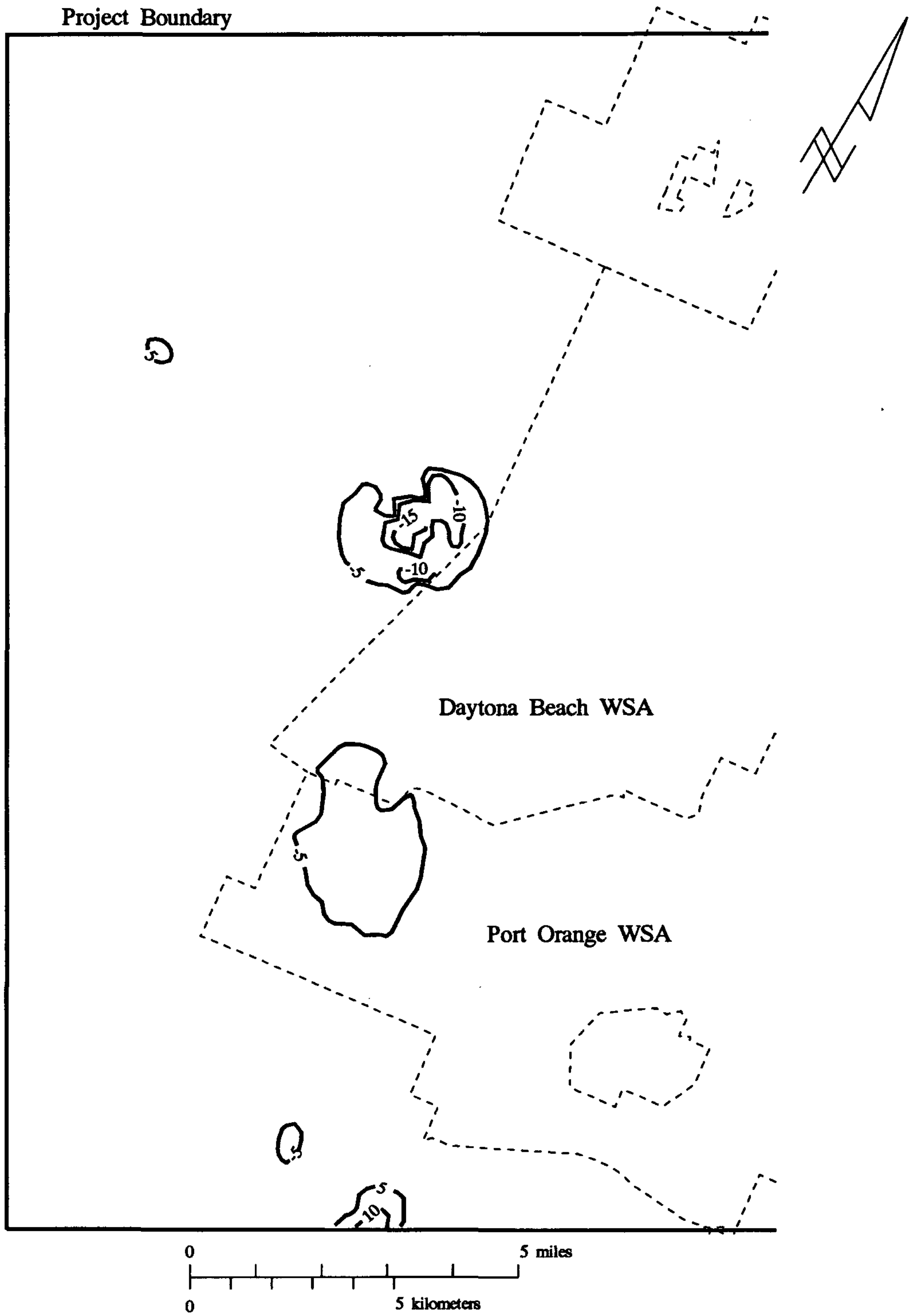


Figure 5.38

Difference in induced pressure head between the projected year 2010 strategy and the strategy identified by Model 3 (Projected - Optimized).

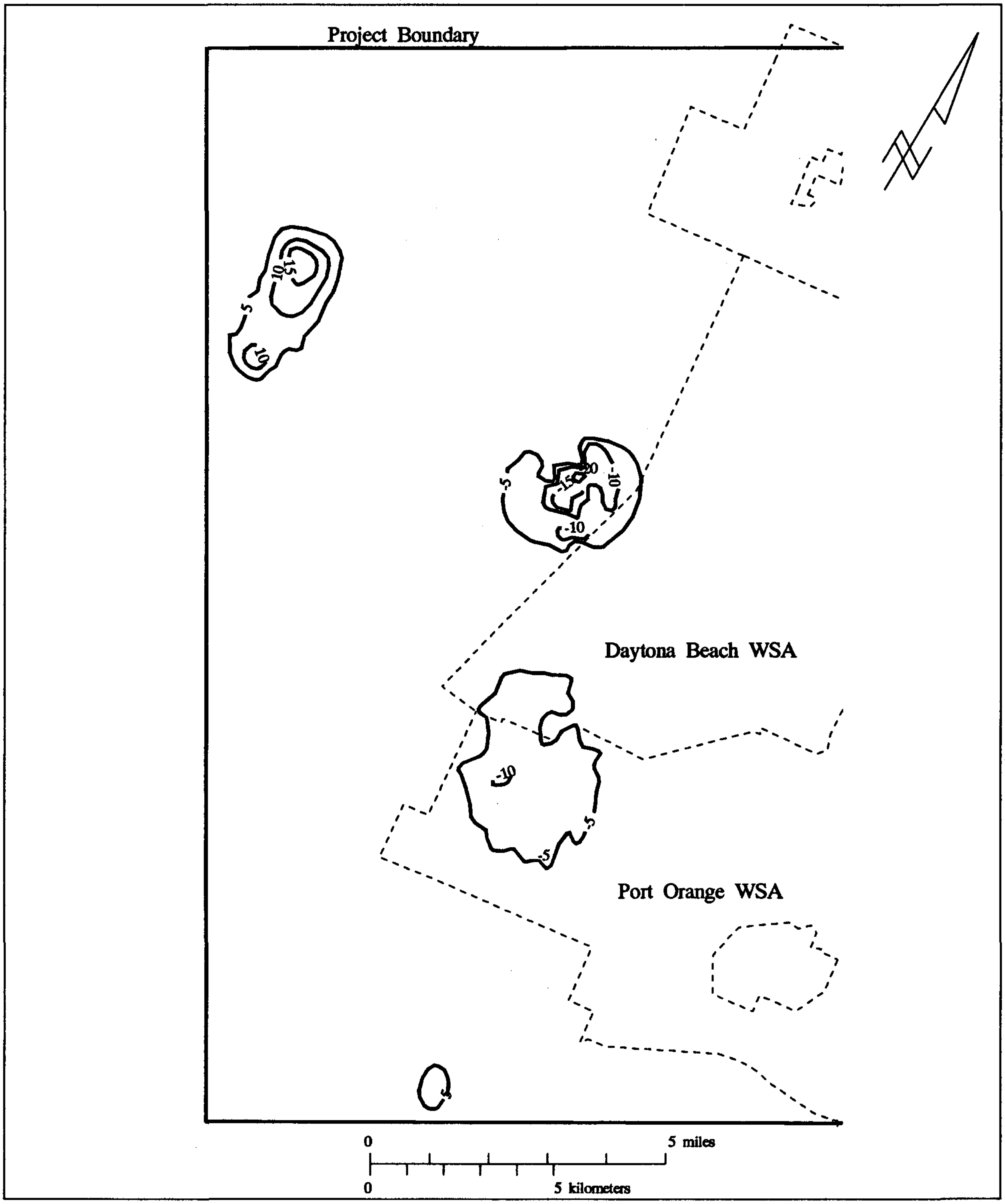


Figure 5.39

Difference in induced pressure head between the projected year 2010 strategy and the strategy identified by model 4 (projected- optimized).

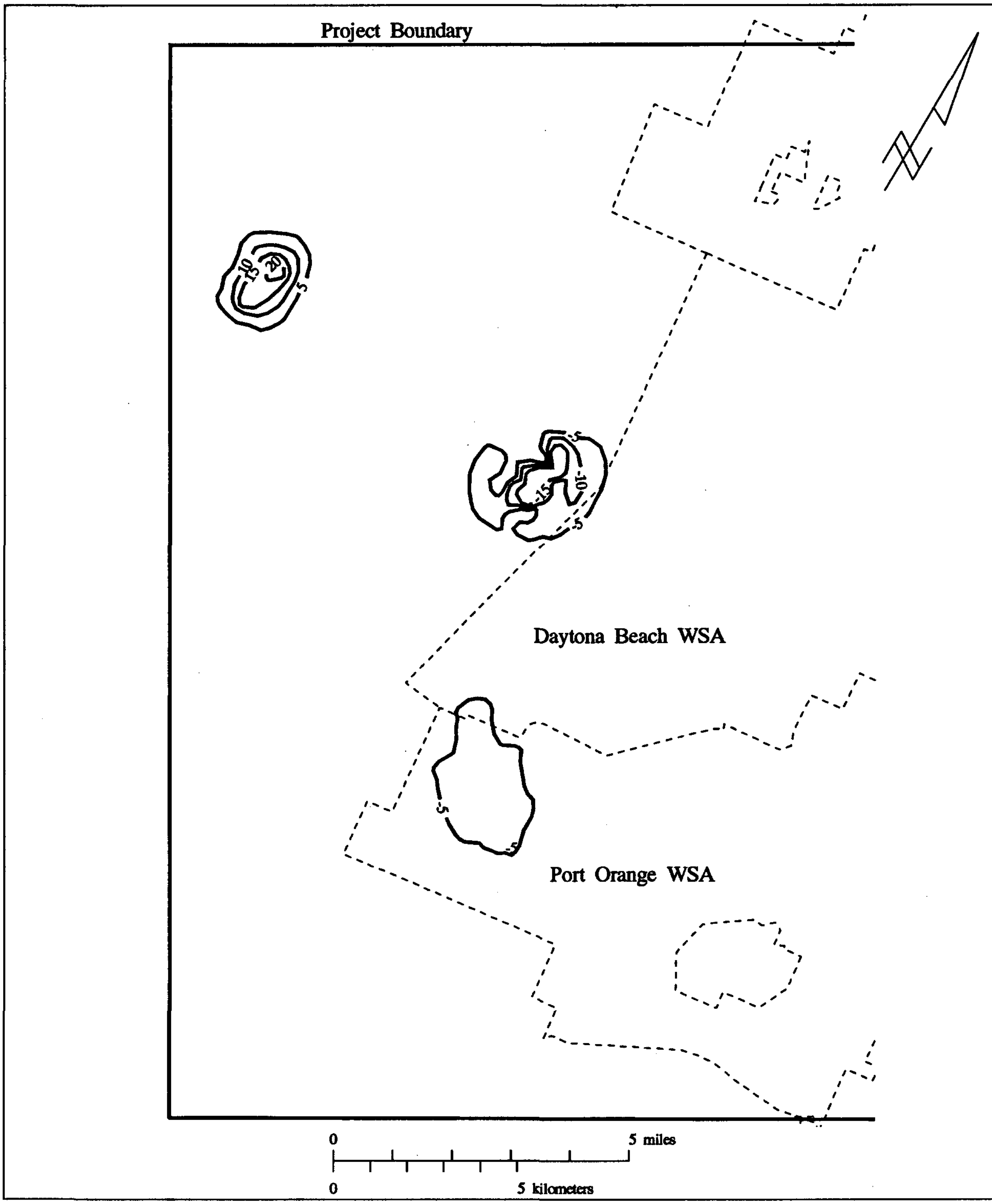


Figure 5.40

Difference in induced pressure head between the projected year 2010 strategy and the strategy identified by model 5 (projected- optimized).

values depict areas of increased water table elevation when the optimized strategy is utilized in place of the projected strategy.

Table 5.3 depicts the optimal water allocation strategies with respect to agricultural well grid cells and wastewater treatment plants. As shown in these results, a majority of the agricultural demand is met by wastewater treatment plant effluent. All five plants included in the optimization model are used to supplement this demand. Only agricultural service areas identified as AG-0236AN and AG-0237AN (areas 7 and 8) continue to utilize ground water to meet demands. The remaining seven service areas satisfy their demand entirely by wastewater treatment plant effluent.

5.6.6 Sensitivity Analysis

For linear programming a simple and useful means of sensitivity analysis can be preformed using shadow (or dual) prices. Shadow prices quantify the change in the objective function value produced from a unit relaxation of binding constraints (i.e., a unit increase in the right-hand-side of a less-than-or-equal-to constraint or a unit decrease in the right-hand-side of a greater-than-or-equal-to constraint). Binding constraints are those that directly effect the optimal solution and are identified as having non-zero shadow prices.

The calculated shadow prices for municipal well grid cells discharge constraint equations are shown in Table 5.13 for minimum discharge constraints (Equation 5.16, the minimum discharge rate for an existing well) and maximum discharge constraints (Equation 5.13, the maximum capacity of a municipal well cell). The modified objective function value obtained from relaxing these constraints is calculated using the following equation:

Table 5.13 - Shadow prices for municipal well grid cells.

shadow price=change in objective value (in feet) per million cfd change in pumpage rate.

		Model 1 - Min Max DD		Model 2 - Max Min Head		Model 3 - Min Avg DD		Model 4 - Min Max DD		Model 5 - Min Max R CC	
Grid Cell ID	Water service area	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint
MWELL1	Port Orange - West Wellfield	3.50	0.00	-3.35	0.00	29.11	0.00	2.77	0.00	0.00	0.00
MWELL2	Port Orange - West Wellfield	2.35	0.00	-2.39	0.00	9.71	0.00	1.94	0.00	0.00	0.00
MWELL3	Port Orange - West Wellfield	3.21	0.00	-3.01	0.00	21.95	0.00	2.53	0.00	0.00	0.00
MWELL4	Port Orange - West Wellfield	2.03	0.00	-2.29	0.00	4.14	0.00	1.76	0.00	0.00	0.00
MWELL5	Port Orange - West Wellfield	2.28	0.00	-2.47	0.00	8.40	0.00	1.93	0.00	0.00	0.00
MWELL6	Port Orange - West Wellfield	2.84	0.00	-2.76	0.00	16.08	0.00	2.33	0.00	0.00	0.00
MWELL7	Port Orange - West Wellfield	2.53	0.00	-2.59	0.00	11.93	0.00	2.12	0.00	0.00	0.00
MWELL8	Port Orange - West Wellfield	2.64	0.00	-2.55	0.00	13.13	0.00	2.14	0.00	0.00	0.00
MWELL9	Port Orange - West Wellfield	2.52	0.00	-2.69	0.00	15.41	0.00	2.06	0.00	0.00	0.00
MWELL10	Port Orange - West Wellfield	1.85	0.00	-2.16	0.00	0.00	0.00	1.47	0.00	0.00	0.00
MWELL11	Port Orange - West Wellfield	2.27	0.00	-2.45	0.00	7.94	0.00	1.92	0.00	0.00	0.00
MWELL12	Port Orange - West Wellfield	3.01	0.00	-2.71	0.00	19.14	0.00	2.32	0.00	0.00	0.00
MWELL13	Port Orange - West Wellfield	2.06	0.00	-2.37	0.00	5.07	0.00	1.80	0.00	0.00	0.00
MWELL14	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL15	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL16	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL17	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL18	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL19	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL20	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL21	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	-49.08	0.00	0.00	0.00	0.00
MWELL22	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL23	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL24	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL25	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL26	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL27	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL28	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL29	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL30	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 5.13 - Continued

		Model 1 - Min Max DD		Model 2 - Max Min Head		Model 3 - Min Avg DD		Model 4 - Min Max DD		Model 5 - Min Max R CC	
Grid Cell ID	Water service area	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint
MWELL31	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL32	Port Orange - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL33	Port Orange - East Wellfield	0.09	0.00	-0.21	0.00	4.12	0.00	0.00	0.00	0.00	0.00
MWELL34	Port Orange - East Wellfield	0.00	0.00	-0.03	0.00	0.64	0.00	0.00	0.00	0.00	0.00
MWELL35	Port Orange - East Wellfield	0.01	0.00	-0.06	0.00	1.51	0.00	0.07	0.00	0.00	0.00
MWELL36	Port Orange - East Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.03	0.00	0.00	0.00
MWELL37	Port Orange - East Wellfield	0.14	0.00	-0.29	0.00	5.32	0.00	0.00	0.00	0.00	0.00
MWELL38	Port Orange - East Wellfield	0.05	0.00	-0.15	0.00	1.97	0.00	0.15	0.00	0.00	0.00
MWELL39	Daytona Beach East - Marion	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL40	Daytona Beach East - Marion	0.19	0.00	-0.38	0.00	3.68	0.00	0.40	0.00	0.00	0.00
MWELL41	Daytona Beach East - Marion	1.26	0.00	-2.61	0.00	23.40	0.00	2.12	0.00	0.00	0.00
MWELL42	Daytona Beach East - Marion	0.08	0.00	-0.17	0.00	1.61	0.00	0.15	0.00	0.00	0.00
MWELL43	Daytona Bch West - Brennan	4.61	0.00	-0.63	0.00	59.43	0.00	1.88	0.00	0.00	0.00
MWELL44	Daytona Bch West - Brennen	3.49	0.00	-1.18	0.00	48.66	0.00	1.70	0.00	0.00	0.00
MWELL45	Daytona Bch West - Brennan	5.10	0.00	-0.31	0.00	64.08	0.00	1.84	0.00	0.00	0.00
MWELL46	Daytona Bch West - Brennan	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL47	Daytona Bch West - Brennan	0.83	0.00	-1.45	0.00	12.13	0.00	0.53	0.00	0.00	0.00
MWELL48	Daytona Bch West - Brennan	5.62	0.00	0.00	0.00	69.12	0.00	1.70	0.00	0.00	0.00
MWELL49	Daytona Bch West - Brennan	5.74	0.00	-0.81	0.00	66.91	0.00	2.59	0.00	0.00	0.00
MWELL50	Daytona Bch West - Brennan	6.34	0.00	-1.29	0.00	70.56	0.00	3.48	0.00	0.00	0.00
MWELL51	Daytona Bch West - Brennan	7.31	0.00	-1.10	0.00	76.70	0.00	3.89	0.00	0.00	0.00
MWELL52	Daytona Bch West - Brennan	4.09	0.00	-0.90	0.00	52.54	0.00	1.82	0.00	0.00	0.00
MWELL53	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL54	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL55	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL56	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL57	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL58	Daytona Beach West - South T	0.00	0.00	0.00	1.87	0.00	-11.85	0.00	-3.46	0.00	0.00
MWELL59	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL60	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 5.13 - Continued

		Model 1 - Min Max DD		Model 2 - Max Min Head		Model 3 - Min Avg DD		Model 4 - Min Max DD		Model 5 - Min Max R CC	
Grid Cell ID	Water service area	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint
MWELL61	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL62	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL63	Daytona Beach West - South T	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL64	Daytona Beach West - South T	0.00	-4.83	0.00	0.61	0.00	-24.93	0.00	0.00	0.00	0.00
MWELL65	Spruce Creek	1.50	0.00	0.00	0.00	1.81	0.00	0.00	0.00	0.00	0.00
MWELL66	Spruce Creek	0.00	0.00	-1.73	0.00	0.00	0.00	0.09	0.00	0.00	0.00
MWELL67	Holly Hill - East Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	1.17	0.00	0.00	0.00
MWELL68	Holly Hill - East Wellfield	0.04	0.00	-0.07	0.00	1.98	0.00	0.00	0.00	0.00	0.00
MWELL69	Holly Hill - East Wellfield	0.02	0.00	-0.02	0.00	0.13	0.00	0.86	0.00	0.00	0.00
MWELL70	Holly Hill - East Wellfield	0.02	0.00	-0.05	0.00	1.81	0.00	0.00	0.00	0.00	0.00
MWELL71	Holly Hill - East Wellfield	0.06	0.00	-0.10	0.00	2.64	0.00	0.05	0.00	0.00	0.00
MWELL72	Holly Hill - West Wellfield	0.09	0.00	-0.16	0.00	2.00	0.00	0.27	0.00	0.00	0.00
MWELL73	Holly Hill - West Wellfield	0.00	0.00	0.00	0.00	0.00	0.00	0.11	0.00	0.00	0.00
MWELL74	Holly Hill - West Wellfield	0.50	0.00	-0.32	0.00	2.61	0.00	0.00	0.00	0.00	0.00
MWELL75	Smyrna Beach / Samsula	0.13	0.00	-0.13	0.00	0.00	0.00	0.08	0.00	0.00	0.00
MWELL76	Smyrna Beach / Samsula	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL77	Smyrna Beach / Samsula	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL78	Smyrna Beach / Samsula	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL79	Ormond Beach - Division Ave	0.07	0.00	-0.08	0.00	2.21	0.00	0.13	0.00	0.00	0.00
MWELL80	Ormond Beach - Division Ave	0.11	0.00	-0.09	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL81	Ormond Beach - Division Ave	0.04	0.00	-0.04	0.00	0.70	0.00	0.08	0.00	0.00	0.00
MWELL82	Ormond Beach - Division Ave	0.00	0.00	0.00	0.00	0.51	0.00	0.01	0.00	0.00	0.00
MWELL83	Ormond Beach - Division Ave	0.09	0.00	-0.13	0.00	1.95	0.00	0.15	0.00	0.00	0.00
MWELL84	Ormond Beach - Division Ave	0.11	0.00	-0.15	0.00	3.19	0.00	0.01	0.00	0.00	0.00
MWELL85	Ormond Beach - State Rd. 40	0.45	0.00	-0.50	0.00	25.38	0.00	0.91	0.00	0.00	0.00
MWELL86	Ormond Beach - State Rd. 40	0.25	0.00	-0.50	0.00	22.53	0.00	0.70	0.00	0.00	0.00
MWELL87	Ormond Beach - State Rd. 40	0.40	0.00	-0.55	0.00	24.41	0.00	0.87	0.00	0.00	0.00
MWELL88	Ormond Beach - State Rd. 40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL89	Ormond Beach - State Rd. 40	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL90	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 5.13 - Continued

		Model 1 - Min Max DD		Model 2 - Max Min Head		Model 3 - Min Avg DD		Model 4 - Min Max DD		Model 5 - Min Max R CC	
Grid Cell ID	Water service area	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint	Minimum Discharge Constraint	Maximum Discharge Constraint
MWELL91	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL92	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL93	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL94	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL95	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL96	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL97	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL98	Ormond Beach - Hudson Well	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL99	Ormond Bch - SR40 &/or Hu	0.00	-0.55	0.00	0.64	0.00	0.00	0.00	-0.08	0.00	0.00
MWELL100	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL101	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL102	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL103	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL104	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL105	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL106	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL107	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL108	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL109	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL110	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL111	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL112	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL113	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL114	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL115	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL116	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL117	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL118	Ormond Bch - SR40 &/or Hu	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL119	Tymber Creek	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
MWELL120	The Trails, Inc.	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

$$S_{new} = S + \Delta RSH \times \frac{SP}{10^{+}6} \tag{5.31}$$

where S_{new} is the new calculated value of the objective function, S is the value of the objective function given from model execution, ΔRSH is the change in the right-hand-side of a specified constraint, and SP is the shadow price associated with that specified constraint. For example, the optimal objective function value for model 1 is 7.92 feet or ($7,915,646.7/10^{+6}$) feet. Using the minimum discharge constraint shadow price of 6.34 ft/cfd for MWELL50 (see Table 5.13) and a decrease in the right-hand-side of 100,000 cfd, the new value of the objective function is 7.3 feet or ($7,281,646.7/10^{+6}$) feet. This suggests that there will be a decrease in the maximum drawdown or improvement of the optimal value of the objective function if pumpage were allowed to decrease at municipal grid cell MWELL50. The maximum discharge constraint for MWELL50 it is a non-binding constraint for model 1 because the shadow price is zero; consequently if the capacity limit were increased the objective function value would not change, because this well grid is already pumping at a rate well below the existion capacity.

A review of Table 5.13 reveals that the maximum drawdown at sensitive wetland areas can be reduced if minimum discharge constraints were relaxed for municipal well pumpage in Port Orange - West (wells MWELL1 through MWELL13) and for Daytona Beach East - Marion and Daytona Beach West - Brennan water treatment plants (wells MWELL40 through MWELL52). The maximum discharge constraint is generally a non-binding constraint except for a few wells including: MWELL21 in model 3; MWELL57 in models 2, 3, and 4; MWELL64 in models 1 and 3; and MWELL99 in model 1, 2, and 4. Increasing the capacity of these wells (and other wells that pump at capacity) would improve the optimal value of the objective functions (i.e., reduce the maximum drawdown).

Similar to that of the municipal supply areas, Tables 5.14 and 5.15 display the shadow prices for the agricultural wells and the wastewater treatment plants, respectively. The shadow prices are for maximum discharge constraints Equations (5.14) and (5.15), which express respectively the capacity of a agricultural well cell and the wastewater supply. New objective function values can be calculated using Equation 5.31. For example, the optimal objective function value for model 1 is 7.92 feet or ($7,915,646.7/10^{+6}$) feet. Using the maximum discharge constraint shadow price of -1.718 ft/cfd for WASTE5 (see Table 5.13) and an increase in the right-hand-side of 100,000 cfd, the new value of the objective function is 7.7 feet or ($7,743,846.7/10^{+6}$) feet. This represents a decrease in the maximum drawdown or improvement of the optimal value of the objective function by allowing an increase in the available supplies of reuse water from wastewater treatment plant WASTE5. In this study no lower limits were placed on pumpage from agricultural well grid cells or on the discharge supplemented by wastewater reuse. Zero-valued shadow prices for the agricultural maximum discharge constraints reveal that these were non-binding constraints for all the models and wells. Shadow prices for maximum discharge constraints for the wastewater treatment plants indicate WASTE5 as the only wastewater treatment plant where increased flows from this plant could be used for agricultural uses that would subsequently improve the optimal solution for all the models (i.e., reduce maximum drawdown or increase minimum pressure head).

Shadow prices for municipal water service areas and agricultural water service areas in regards to service area demand constraints are shown in Table 5.16 and 5.17,

Table 5.14 - Shadow prices for agricultural well grid cells.

shadow price=change in objective value (in feet) per million cfd change in pumpage rate.

Maximum Discharge Constraint					
Grid Cell ID	Model 1 Min Max DD	Model 2 Max Min Head	Model 3 Min Avg DD	Model 4 Min Max DD	Model 5 Min Max RCC Inc
AGWELL1	0	0	0	0	0
.	.	.	.	.	.
.	.	.	.	.	.
.	.	.	.	.	.
AGWELL28	0	0	0	0	0

Table 5.15 - Shadow prices for wastewater treatment plants.

Maximum Discharge Constraint					
Grid Cell ID	Model 1 Min Max DD	Model 2 Max Min Head	Model 3 Min Avg DD	Model 4 Min Max DD	Model 5 Min Max RCC Inc
WASTE 1	0	0	0	0	0
WASTE 2	0	0	0	0	0
WASTE 3	0	0	0	0	0
WASTE 4	0	0	0	0	0
WASTE5	-1.718	2.946	-50.700	-0.835	0.000

Table 5.16 - Shadow prices for municipal water service areas.

Service Demand Constraint					
Area ID	Model 1 Min Max DD	Model 2 Max Min Head	Model 3 Min Avg DD	Model 4 Min Max DD	Model 5 Min Max RCC Inc
POW	0.899	0.000	92.227	0.878	0
POE	1.311	-1.567	23.840	1.36	0
DBM	2.220	-3.252	43.792	2.719	0
DBB	3.069	-5.781	65.655	5.291	0
DBS	9.157	-6.826	118.272	10.816	0
SCK	1.399	-1.101	40.541	1.76	0
HHE	1.101	-1.433	21.396	0.017	0
HHW	2.003	-2.829	44.172	2.108	0
NSB	0.567	-0.536	21.368	0.631	0
OBD	0.971	-1.254	20.911	1.116	0
OB4	1.299	-0.781	2.783	4	0
OBH	1.299	-0.781	24.650	0.794	0
TCU	2.711	-5.641	59.039	24.123	0
TTI	2.453	-3.188	54.647	12.07	0

Table 5.17 - Shadow prices for agricultural water service areas.

Area ID	Service Demand Constraint				
	Model 1 Min Max DD	Model 2 Max Min Head	Model 3 Min Avg DD	Model 4 Min Max DD	Model 5 Min Max RCC Inc
AGAREA1	0	0	0	0	0
AGAREA2	0	0	0	0	0
AGAREA3	0	0	0	0	0
AGAREA4	0	0	0	0	0
AGAREA5	0	0	0	0	0
AGAREA6	1.718	-2.946	50.700	0.835	0
AGAREA7	2.286	-4.609	0	1.336	0
AGAREA8	1.718	-2.946	50.700	0.835	0
AGAREA9	0	0	0	0	0

respectively. These shadow prices represent the change in the objective function produced from the relaxing of water service area demand (i.e., Equations 5.17 and 5.18). Again, the new objective function values can be calculated using Equation 5.31. For example, the optimal objective function value for model 1 is 7.92 feet or ($7{,}915{,}646.7/10^{+6}$) feet. Using the service demand constraint shadow price of 9.157 ft/cfd for Daytona Beach South (DBS) (see Table 5.16) and a decrease in the right-hand-side of 100,000 cfd, the new value of the objective function is found to be 7.0 feet or ($6{,}999{,}946.7/10^{+6}$) feet. This represents a decrease in the maximum drawdown or improvement of the optimal value of the objective function by using less water for DBS water service area. Examination of the results of the municipal water service areas reveals that the objective functions values could be improved (i.e., increase minimum pressure head or reduce maximum drawdown) if year 2010 water service demand for all areas were decreased. The largest improvement for all models could be obtained if Daytona Beach South service demands were decreased. Similarly, for agricultural water service areas the optimal solution could be improved if demands were decreased (i.e., in AREA6 for models 1, 2, 3, 4, and 5; in AREA7 for models 1, 2, 4 and 5; and in AREA8 for models 1, 2, 3, 4 and 5).

CHAPTER 6
6.0 SUMMARY AND CONCLUSIONS

This endeavor demonstrated the use of optimization modeling as a valuable tool for the management of water resources. Five site specific water resource allocation optimization models were developed for Volusia County, Florida and were executed to investigate variety of management objectives. These optimization models incorporate both water quantity and quality aspects of water resource management to determine optimum ground water allocation strategies that satisfying future water service demands and minimize adverse environmental impacts at specified areas. These areas include sensitive wetlands where projected water table declines are predicted to induce a high level of vegetative harm and well fields where excessive withdrawal is predicted to cause a degradation in water quality due to salt-water intrusion or upconing.

The five optimization model were formulated and executed using GAMS (General Algebraic Modeling System). These models were designed to elucidate water resource allocation strategies for water service areas that would: 1) satisfy the water demands of both municipal and agricultural water demands, 2) explore development of both existing and proposed ground water supply areas, and 3) select wastewater effluent as a feasible supply to supplement agricultural demands. Four different objective functions were used to constructed the individual optimization models. These included minimizing the maximum drawdown, maximizing the minimum pressure head, and minimizing the average drawdown at sensitive wetland areas. Water quality aspects were incorporated in two models by constraining chloride concentrations changes at wells while simultaneously minimizing the maximum drawdown at sensitive wetland areas and by minimizing the maximum relative chloride concentration increase at the well grids.

Each water resource allocation optimization model was developed using the "unit response matrix" technique. This method consisted of incorporating a matrix of influence coefficients which represent the response of the aquifer system to a specified change in groundwater pumpage. The influence coefficients were determined via the execution of two ground water simulation models. Coefficients related to pressure head were determined by a flow model (MODFLOW) and those related to chloride concentrations were determined by a solute transport model (DSTRAM). With the unit response matrix concept as the foundation of each optimization model, various constraints and objectives were derived that were specific and common to the five models.

When simulation model predictions were compared to aquifer responses predicted by the optimization model, there were occasions where control points were found to behave in a slightly nonlinear fashion. These non-linear responses were discovered to be due to both a nonadditive affect of drawdown when utilized wells are in close proximity and to an increase in decreasing water table elevation with increasing withdrawal rates. The non-linear affects were reduced however by developing a revised set of response matrices and re-executing the optimization model. This technique achieved only limited success for the chloride transportation responses. This results from the assumption that a linear approximation can accurately simulate the non-linear differential equation describing the transport process.

Year 2010 water resource allocation strategies were examined with the optimization models. Results of each model revealed where future well fields should be developed and to what extent future and existing wells should be utilized. Aquifer responses induced by the optimized allocation strategies were compared to those induced by the projected year 2010 allocation strategy. Adverse environmental impacts due to a depressed water table were found to decrease substantially when the optimized strategies were simulated with the ground water flow and transport models. Simulation results for the projected year 2010 allocation strategy predicted a maximum drawdown at sensitive wetland control points of 19.14 feet. The simulation model results for the optimal strategies predicted by the five optimization models were: 10.42 feet for model 1, an improvement of 8.72 feet; 10.93 feet for model 2, an improvement of 8.21 feet; 9.21 feet for model 3, an improvement of 9.93 feet; 11.93 feet for model 4, an improvement of 7.21 feet; and 16.3 feet for model 5, an improvement of 2.9 feet when compared to the projected year 2010 strategy.

Results from the five optimization models showed that the minimum discharge constraint was one of the most important components determining optimal allocation scenarios. This constraint required that the model select allocation scenarios that use at least 50 percent of the 1988 ground water pumping rate from wells active in 1988. Several of the optimal allocation scenarios reflect the use of new wells in the areas of Port Orange West well field, Daytona Beach West - South Daytona water treatment plant well field, and Ormond Beach State Road 40 / Hudson well fields. In general, each model identified scenarios that decreased ground water pumpage at a majority of wells in the Daytona Beach West - Brennan water treatment plant well field and then increased pumpage from a single well in this area while expanding pumpage in the northwest where native vegetation is less sensitive to ground water withdrawals. Model 4 identified a similar allocation scenario with a few minor differences. This strategy however, caused a decrease in water quality when compared to the projected year 2010 strategy such that the optimum scenario did not improve water quality in eastern well fields. A similar phenomena was observed for model 5 scenario but a larger degradation occurs when the model minimizes the relative chloride concentration increase. Basically this objective minimizes the larger relative increase in chloride concentrations but at the cost of total water quality degradation throughout the area.

Sensitivity analyses were performed with the optimization models to determine where management strategies could be changed to induce improvements in ground water levels. These analyses identified areas were the balance between supplies and demands could be changed to decrease the environmental impacts of ground water withdrawals. For example, it was shown that a ten percent decrease in the demand at the DBS (Daytona Beach West - South TP) water service area could induce a water level improvement of 17 percent. These types of analyses give the water resource manager valuable information on where conservation efforts and new well developments should be pursued. It also lets the manager quickly determine how the resource allocation scenario given by each optimization model responds to changes in projected demands and withdrawal limits.

7.0 APPENDICES

7.1 PROGRAM CODES FOR DETERMINING RESPONSE MATRICES

DOS Batch File (INF-COEF.BAT)

```
ECHO OFF
CLS
E                C                H                O
**************************************************
**************
ECHO  BATCH FILE TO DETERMINE INFLUENCE COEFFICIENTS
USING MODFLOW
E                C                H                O
**************************************************
**************
ECHO  A FILE NAMED "COUNT.FIL" WILL BE WRITTEN TO
DRIVE B AND WILL
ECHO  CONTAIN THE NUMBER OF ITERATIONS REMAINING IN
THE BATCH PROGRAM.
ECHO  PLEASE INSERT A DISK INTO DRIVE B.
E                C                H                O
**************************************************
**************
PAUSE
CLS
BEGIN3
IF NOT EXIST PROBLEM.TMP GOTO ENDA
DEL PROBLEM.TMP
CLS
:LOOP
COPY COUNT.FIL B:
ECHO BEGINNING CALCULATIONS FOR NEXT WELL
INF-WEL3
IF NOT EXIST PROBLEM.TMP GOTO ENDB
DEL PROBLEM.TMP
COPY PUMPX.DAT PUMPY.DAT
DEL PUMPX.DAT
CLS
MODFLOWX <INF-COEF.LST
IF NOT EXIST MODFLOW.OUT GOTO ENDC
DEL INF-COEF.WEL
INF-COF3
IF NOT EXIST PROBLEM.TMP GOTO ENDD
DEL PROBLEM.TMP
DEL MODFLOW.OUT
DEL PUMP-VAL.DAT
IF NOT EXIST COUNT2.FIL GOTO LOOP
REM
DEL WELL.WEL
DEL LIST.LST
DEL PUMP.DAT
DEL INT-PTS.DAT
DEL WEL-PTS.DAT
DEL INF-COEF.DAT
DEL WEL-COEF.DAT
DEL INF-COEF.LST
DEL PUMPX.DAT
DEL PUMPY.DAT
DEL INF-COEF.WEL
DEL COUNT.FIL
DEL COUNT2.FIL
DEL COUNT3.FIL
REM
ECHO BATCH FILE HAS COMPLETED EXECUTION!!!
ECHO
ECHO THE INFLUENCE COEFFICIENTS HAVE BEEN WRITTEN TO
THE FILE SPECIFIED.
GOTO END
REM
:ENDA
ECHO  PROBLEM  HAS  OCCURED  IN  EXECUTION  OF  THE
BEGIN.EXE PROGRAM!!!
ECHO  EDIT  INPUT  AND  OUTPUT  FILES  TO  DETERMINE
PROBLEM.
GOTO END
:ENDB
ECHO  PROBLEM  HAS  OCCURED  IN  EXECUTION  OF  THE
INF-WELL.EXE PROGRAM!!!
ECHO  EDIT  INPUT  AND  OUTPUT  FILES  TO  DETERMINE
PROBLEM.
GOTO END
```

```
:ENDC
ECHO PROBLEM HAS OCCURED IN EXECUTION OF THE MODFLOWX.EXE PROGRAM!!!
ECHO EDIT INPUT AND OUTPUT FILES TO DETERMINE PROBLEM.
GOTO END
:ENDD
ECHO PROBLEM HAS OCCURED IN EXECUTION OF THE INF-COEF.EXE PROGRAM!!!
ECHO EDIT INPUT AND OUTPUT FILES TO DETERMINE PROBLEM.
REM
:END
```

FORTRAN Program Code #1 (BEGIN.FOR)

```
C  *** PROGRAM READS EXISTING INPUT FILE NAMES AND THEN PLACES THESE ***
C  *** FILE NAMES INTO FILES WITH SPECIFIC NAMES. THESE FILES ARE    ***
C  *** THEN ACCESSED DURING THE BATCH RUN TO READ USER SPECIFIED     ***
C  *** FILES.  (PROGRAM INCORPORATES WELL POINTS IN INF COEF CALC)   ***
C
                            CHARACTER*20
WELLOLD,LISTOLD,PUMPOLD,INPTOLD,INCOFOLD,FNAME,LIST
                            CHARACTER*20
WELLNEW,LISTNEW,PUMPNEW,INPTNEW,INCOFNEW,LISTNAME
                            CHARACTER*20
WELPTNEW,WLCOFNEW,WELPTOLD,WLCOFOLD
      CHARACTER*1  IN

      WRITE (6,*)
      WRITE (6,*)
      WRITE (6,*)
      WRITE (6,*)
C
      WRITE (6,*) 'THIS COMBINED FORTRAN PROGRAM AND DOS BATCH FILE'
      WRITE (6,*) 'CALCULATES INFLUENCE COEFFICIENTS BY RUNNING'
      WRITE (6,*) 'MODFLOW AS MANY TIMES AS THERE ARE WELLS TO BE'
      WRITE (6,*) 'OPTIMIZED.'
      WRITE (6,*)
      WRITE (6,*) 'INPUT FILES CAN BE READ FROM THE KEYBOARD OR FROM A'
      WRITE (6,*) 'PREVIOUSLY CREATED "LIST" FILE. IF FIRST TIME'
      WRITE (6,*) 'RUNNING PROGRAM, PLEASE INPUT FILES USING KEYBOARD.'
      WRITE (6,*)
      WRITE (6,*) 'READ INPUT FILE FROM KEYBOARD?? (TYPE Y OR N).'
      READ (6,2000) IN
C
C *** READ INPUT FILES ***
C
      IF (IN.NE.'Y') THEN
        WRITE (6,*) 'ENTER NAME OF INPUT "LIST" FILE:'
        READ (6,1000) LIST
        OPEN(UNIT=16,FILE=LIST,ACCESS='SEQUENTIAL')
        READ (16,1000) WELLOLD
        READ (16,1000) LISTOLD
        READ (16,1000) PUMPOLD
        READ (16,1000) INPTOLD
        READ (16,1000) INCOFOLD
        READ (16,1000) WELPTOLD
        READ (16,1000) WLCOFOLD
        CLOSE (UNIT=16)
      ELSE
C
      WRITE (6,*)
      WRITE (6,*)
      WRITE (6,*) 'ENTER NAME OF EXISTING MODFLOW .WEL FILE:'
      WRITE (6,*) '(FILE CONTAINS ALL WELL LOCATIONS AND PUMPAGE FOR'
      WRITE (6,*) 'PARTICULAR INITIAL CONDITION, i.e. PD,1988 ect..)'
      WRITE (6,*) 'THIS IS 1ST CONDITION TO
```

```
CALCULATE INFLUENCE COEF.'
      READ (6,1000) WELLOLD
C
      WRITE (6,*)
      WRITE (6,*) 'ENTER NAME OF EXISTING MODFLOW
.LST FILE:'
      WRITE (6,*) '(FILE CONTAINS THE DATA FILES
REQUIRED TO RUN'
      WRITE (6,*) 'MODFLOW FOR THE PARTICULAR
CONDITION OF INTEREST,'
      WRITE (6,*) 'i.e. 1988, 2010 ect.. MUST BE IN
SPECIFIC ORDER AND'
      WRITE (6,*) 'INCLUDE BINARY FILE OF INITIAL
HEAD CONDITIONS)'
      READ (6,1000) LISTOLD
C
      WRITE (6,*)
      WRITE (6,*)
      WRITE (6,*) 'ENTER EXISTING FILE NAME OF WELLS
TO BE OPTIMIZED:'
      WRITE (6,*) '[FILE SPECIFIES 1) GRID CELL
LOCATION OF PROPOSED'
      WRITE (6,*) 'AND EXISTING WELLS USED IN THE
OPTIMIZATION PROCESS'
      WRITE (6,*) 'AND, 2) ABSOLUTE (NOT RELATIVE)
VALUES OF INCREASED'
      WRITE (6,*) 'PUMPAGE USED TO DETERMINE
INFLUENCE COEFFICIENTS].'
      READ (6,1000) PUMPOLD
C
      WRITE (6,*)
      WRITE (6,*) 'ENTER NAME OF EXISTING "POINTS OF
INTEREST" FILE:'
      WRITE (6,*) '(FILE SPECIFIES LOCATION OF
SPECIFIC GRID CELLS'
      WRITE (6,*) 'WHERE HEAD AND/OR DRAWDOWN IS TO
BE OPTIMIZED).'
      READ (6,1000) INPTOLD
C
      WRITE (6,*)
      WRITE (6,*) 'ENTER NAME OF NEW INFLUENCE
COEFFICIENT FILE:'
      READ (6,1000) INCOFOLD
C
      WRITE (6,*)
      WRITE (6,*) 'ENTER NAME OF EXISTING "WELL
POINTS" FILE:'
      WRITE (6,*) '(FILE SPECIFIES LOCATION OF
SPECIFIC WELL CELLS'
      WRITE (6,*) 'WHERE HEAD AND/OR DRAWDOWN IS TO
BE CONSTRAINED).'
      READ (6,1000) WELPTOLD
C
      WRITE (6,*)
      WRITE (6,*) 'ENTER NAME OF NEW WELL INFLUENCE
COEFFICIENT FILE:'
      READ (6,1000) WLCOFOLD
C
      ENDIF
C
C *** SET UP COUNTER FOR BATCH LOOP BY READING # OF
WELLS ***
C
                                        O    P    E    N
(UNIT=18,FILE=PUMPOLD,ACCESS='SEQUENTIAL')
      READ (18,3000) ICNT
      CLOSE (UNIT=18)
C
C *** PLACE SPECIFIED FILENAMES INTO PREDETERMINED
FILES ***
C
      WELLNEW='WELL.WEL'
      LISTNEW='LIST.LST'
      PUMPNEW='PUMP.DAT'
      INPTNEW='INT-PTS.DAT'
      INCOFNEW='INF-COEF.DAT'
      WELPTNEW='WEL-PTS.DAT'
      WLCOFNEW='WEL-COEF.DAT'
C
C *** OPEN INPUT AND OUTPUT FILES ***
C
      OPEN (UNIT=7,FILE=WELLNEW,STATUS='NEW')
      OPEN (UNIT=8,FILE=LISTNEW,STATUS='NEW')
      OPEN (UNIT=9,FILE=PUMPNEW,STATUS='NEW')
```

```
      OPEN (UNIT=10,FILE=INPTNEW,STATUS='NEW')
      OPEN (UNIT=11,FILE=INCOFNEW,STATUS='NEW')
      OPEN (UNIT=12,FILE=INCOFOLD,STATUS='NEW')
      OPEN (UNIT=30,FILE=WELPTNEW,STATUS='NEW')
      OPEN (UNIT=31,FILE=WLCOFNEW,STATUS='NEW')
      OPEN (UNIT=32,FILE=WLCOFOLD,STATUS='NEW')
C
      OPEN (UNIT=17,FILE='COUNT.FIL',STATUS='NEW')
      WRITE (17,3000) ICNT
      CLOSE (UNIT=17)
      OPEN (UNIT=19,FILE='COUNT3.FIL',STATUS='NEW')
      WRITE (19,3000) ICNT
      CLOSE (UNIT=19)
C
      WRITE (7,1000) WELLOLD
      WRITE (8,1000) LISTOLD
      WRITE (9,1000) PUMPOLD
      WRITE (10,1000) INPTOLD
      WRITE (11,1000) INCOFOLD
      WRITE (12,*) 'INFLUENCE COEFFICIENT FILE'
      WRITE (30,1000) WELPTOLD
      WRITE (31,1000) WLCOFOLD
      WRITE (32,*) 'WELL INFLUENCE COEFFICIENT FILE'
C
      CLOSE (UNIT=7)
      CLOSE (UNIT=8)
      CLOSE (UNIT=9)
      CLOSE (UNIT=10)
      CLOSE (UNIT=11)
      CLOSE (UNIT=12)
      CLOSE (UNIT=30)
      CLOSE (UNIT=31)
      CLOSE (UNIT=32)
C
                                  O     P     E     N
(UNIT=8,FILE='LIST.LST',ACCESS='SEQUENTIAL')
      READ (8,1000) LISTNAME
                                  O     P     E     N
(UNIT=13,FILE=LISTNAME,ACCESS='SEQUENTIAL')
                                  O     P     E     N
(UNIT=15,FILE='INF-COEF.LST',STATUS='NEW')
C
C *** CREATE .LST FILE TO RUN MODFLOW ***
C
      I=0
 20   READ (13,1000) FNAME
      I=I+1
      IF (I.EQ.1) FNAME='MODFLOW.OUT'
      IF (I.EQ.4) FNAME='INF-COEF.WEL'
      WRITE (15,1000) FNAME
      IF (FNAME.NE.'EXIT                ') GOTO 20
C
      CLOSE (UNIT=8)
      CLOSE (UNIT=13)
      CLOSE (UNIT=15)
C
C *** CREATE TEMP FILE IF A PROBLEM ***
C
      OPEN(UNIT=20,FILE='PROBLEM.TMP',STATUS='NEW')
      WRITE (20,*) 'PROBLEM FILE'
      CLOSE (UNIT=20)
C
C *** FORMAT STATEMENTS
C
1000  FORMAT (A20)
2000  FORMAT (A1)
3000  FORMAT (I10)
C
      END
```

FORTRAN Program Code #2 (INF-COEF.FOR)

```
C *** PROGRAM READS FILE SPECIFYING INITIAL WELL
LOCATION AND PUMP ***
C *** RATE, FILE OF WELL LOCATION AND NEW HIGHER
PUMP RATES, AND     ***
C *** EXISTING .LST FILE.  USING THIS DATA, NEW .WEL
AND  LST FILES ***
C *** ARE CREATED FOR USE IN THE MODFLOW PROGRAM.
***
C
      CHARACTER*20 WELLNAME,PUMPNAME
C
```

```
C *** OPEN INPUT AND OUTPUT FILES ***
C
                                        O    P    E    N
(UNIT=7,FILE='WELL.WEL',ACCESS='SEQUENTIAL')
                                        O    P    E    N
(UNIT=9,FILE='PUMP.DAT',ACCESS='SEQUENTIAL')
C
      READ (7,1000) WELLNAME
                                        O    P    E    N
(UNIT=10,FILE=WELLNAME,ACCESS='SEQUENTIAL')
      READ (9,1000) PUMPNAME
                                        O    P    E    N
(UNIT=12,FILE=PUMPNAME,ACCESS='SEQUENTIAL')
C
      CLOSE (UNIT=7)
      CLOSE (UNIT=9)
C
                                        O    P    E    N
(UNIT=9,FILE='PUMP.DAT',ACCESS='SEQUENTIAL')
      WRITE (9,1000) 'PUMPY.DAT           '
      CLOSE (UNIT=9)
C
      OPEN (UNIT=13,FILE='PUMPX.DAT',STATUS='NEW')
                                        O    P    E    N
(UNIT=14,FILE='INF-COEF.WEL',STATUS='NEW')
                                        O    P    E    N
(UNIT=16,FILE='PUMP-VAL.DAT',STATUS='NEW')
C
C *** READ PUMP DATA FILE, WRITE INCREASED PUMPAGE
OF FIRST WELL TO ***
C *** A FILE, DELETE THE WELL FROM PUMP DATA FILE,
REPLACE DATA IN  ***
C *** .WEL FILE, AND MODIFY .LST FILE.
                ***
C
      READ (12,1500) N
      READ (12,2000) IL,IR,IC,QI
      N=N-1
      WRITE (13,1500) N
      DO 5 J=1,N
        READ (12,2000) JL,JR,JC,QJ
        WRITE (13,2000) JL,JR,JC,QJ
 5    CONTINUE
C
C ***
C
      READ (10,3000) M,MM
      READ (10,3100) M
C
      NN=0
      DO 10 I=1,M
        READ (10,2000) JL,JR,JC,QJ
                                                     I    F
((JL.EQ.IL).AND.(JR.EQ.IR).AND.(JC.EQ.IC)) NN=1
 10   CONTINUE
C
      CLOSE (UNIT=10)
                                        O    P    E    N
(UNIT=10,FILE=WELLNAME,ACCESS='SEQUENTIAL')
C
      READ (10,3000) M,MM
      READ (10,3100) M
      IF (NN.EQ.1) THEN
        WRITE (14,3000) M,MM
        WRITE (14,3100) M
        DO 20 I=1,M
          READ (10,2000) JL,JR,JC,QJ
                                                     I    F
((JL.EQ.IL).AND.(JR.EQ.IR).AND.(JC.EQ.IC)) THEN
            QK=QI-QJ
            WRITE (16,2000) IL,IR,IC,QK
            QJ=QI
          ENDIF
          WRITE (14,2000) JL,JR,JC,QJ
 20     CONTINUE
      ELSE
        N=M+1
        WRITE (14,3000) N,MM
        WRITE (14,3100) N
        WRITE (14,2000) IL,IR,IC,QI
        WRITE (16,2000) IL,IR,IC,QI
        DO 30 I=1,M
          READ (10,2000) JL,JR,JC,QJ
          WRITE (14,2000) JL,JR,JC,QJ
```

```
 30     CONTINUE
      ENDIF
C
C
C *** CLOSE INPUT AND OUTPUT FILES ***
C
      CLOSE (UNIT=10)
      CLOSE (UNIT=12)
      CLOSE (UNIT=13)
      CLOSE (UNIT=14)
      CLOSE (UNIT=16)
C
C *** CREATE TEMP FILE IF A PROBLEM ***
C
      OPEN(UNIT=20,FILE='PROBLEM.TMP',STATUS='NEW')
      WRITE (20,*) 'PROBLEM FILE'
      CLOSE (UNIT=20)
C
C *** FORMAT STATEMENTS
C
1000  FORMAT (A20)
1500  FORMAT (I10)
2000  FORMAT (3I10,F10.0)
3000  FORMAT (2I10)
3100  FORMAT (I10)
C
      END
```

FORTRAN Program Code #3 (INF-WELL.FOR)

```
C *** PROGRAM CALCULATES INFLUENCE COEFFICIENTS.  IT
READS IN     ***
C *** MODFLOW OUTPUT, INTEREST POINT DATA, AND
PUMPAGE RATE DATA. ***
C *** (PROGRAM ALSO INCORPORATES WELL POINTS IN INF
COEF CALC)     ***
C
                  C H A R A C T E R * 2 0
PTSNAME,COEFNAME,WLPTNAME,WLCFNAME
      CHARACTER*20 LAY,A
      DIMENSION DD(100,100),VAL(300),WVAL(300)
C
C *** OPEN INPUT AND OUTPUT FILES ***
C
                                    O     P     E     N
(UNIT=7,FILE='MODFLOW.OUT',ACCESS='SEQUENTIAL')
                                    O     P     E     N
(UNIT=8,FILE='PUMP-VAL.DAT',ACCESS='SEQUENTIAL')
                                    O     P     E     N
(UNIT=9,FILE='INT-PTS.DAT',ACCESS='SEQUENTIAL')
                                    O     P     E     N
(UNIT=10,FILE='INF-COEF.DAT',ACCESS='SEQUENTIAL')
                                    O     P     E     N
(UNIT=29,FILE='WEL-PTS.DAT',ACCESS='SEQUENTIAL')
                                    O     P     E     N
(UNIT=30,FILE='WEL-COEF.DAT',ACCESS='SEQUENTIAL')
C
      READ (9,1000) PTSNAME
                                    O     P     E     N
(UNIT=11,FILE=PTSNAME,ACCESS='SEQUENTIAL')
      READ (10,1000) COEFNAME
      OPEN (UNIT=12,FILE=COEFNAME,ACCESS='APPEND')
      READ (29,1000) WLPTNAME
                                    O     P     E     N
(UNIT=31,FILE=WLPTNAME,ACCESS='SEQUENTIAL')
      READ (30,1000) WLCFNAME
      OPEN (UNIT=32,FILE=WLCFNAME,ACCESS='APPEND')
C
      CLOSE (UNIT=9)
      CLOSE (UNIT=10)
      CLOSE (UNIT=29)
      CLOSE (UNIT=30)
C
C  ***  READ  MODFLOW.OUT  DRAWDOWN  DATA  AT  POINTS
SPECIFIED IN FILE  ***
C *** INT-PTS.DAT.  DIVIDE THESE VALUES BY PUMPAGE
RATE IN        ***
C *** IN THE PUMPING INCREASE FILE
              ***
C
 10   READ (7,2000) LAY
      IF (LAY.NE.'DRAWDOWN IN LAYER  1') GOTO 10
```

```
      READ (7,1000) (A, I=1,9)
      READ (7,4000) ((DD(I,J), J=1,66), I=1,60)
C
      READ (8,3000) VALP
C
      READ (11,1500) N
      DO 20 I=1,N
        READ (11,2500) IL,IR,IC
        IF (VALP.NE.0.0) THEN
          VAL(I)=DD(IR,IC)/(-VALP)
        ELSE
          VAL(I)=0.0
        ENDIF
        IF (DD(IR,IC).GE.1.0E+20) IERR=1
 20   CONTINUE
C
      READ (31,1500) NN
      DO 30 J=1,NN
        READ (31,2500) IL,IR,IC
        IF (VALP.NE.0.0) THEN
          WVAL(J)=DD(IR,IC)/(-VALP)
        ELSE
          WVAL(J)=0.0
        ENDIF
        IF (DD(IR,IC).GE.1.0E+20) IERR=1
 30   CONTINUE
C
C *** COUNTER FILE CREATION FOR LOOPING IN BATCH
FILE ***
C
                                             O     P     E     N
(UNIT=20,FILE='COUNT.FIL',ACCESS='SEQUENTIAL')
      READ (20,1500) ICNT
      ICNT=ICNT-1
      CLOSE (UNIT=20)
C
      IF (ICNT.EQ.0) THEN
                                                O    P    E    N
(UNIT=21,FILE='COUNT2.FIL',STATUS='NEW')
        CLOSE (UNIT=21)
      ELSE
                                                O    P    E    N
(UNIT=20,FILE='COUNT.FIL',ACCESS='SEQUENTIAL')
        WRITE (20,1500) ICNT
        CLOSE (UNIT=20)
      ENDIF
C
C ***    WRITE INFLUENCE COEFFICIENTS TO EXISTING
FILE
C
                                             O     P     E     N
(UNIT=22,FILE='COUNT3.FIL',ACCESS='SEQUENTIAL')
      READ (22,1500) KCNT
      JCNT = -(ICNT - KCNT)
      VALCNT=-VALP
C
      WRITE (12,4500) JCNT,VALCNT
      WRITE (32,4500) JCNT,VALCNT
      IF (IERR.EQ.1) THEN
        WRITE (12,*) ' ERROR! A CELL WENT DRY!
DECREASE PUMPAGE VALUE IN
     + "INCREASED PUMPAGE" WELL FILE'
        WRITE (32,*) ' ERROR! A CELL WENT DRY!
DECREASE PUMPAGE VALUE IN
     + "INCREASED PUMPAGE" WELL FILE'
      ENDIF
      WRITE (12,3500) (VAL(I), I=1,N)
      WRITE (32,3500) (WVAL(J), J=1,NN)
C
C *** CLOSE INPUT AND OUTPUT FILES ***
C
      CLOSE (UNIT=7)
      CLOSE (UNIT=8)
      CLOSE (UNIT=11)
      CLOSE (UNIT=12)
      CLOSE (UNIT=22)
      CLOSE (UNIT=31)
      CLOSE (UNIT=32)
C
C *** ENDING STATEMENTS ON SCREEN FOR EACH RUN ***
C
      WRITE (6,*)
      WRITE (6,*) 'RUN CALCULATIONS COMPLETE FOR
SPECIFIED WELL'
```

```
      WRITE (6,*)
      WRITE (6,5000) ICNT
      WRITE (6,*)
C
C  ***  CREATE  TEMP  FILE  TO  CHECK  IF  SUCCESSFUL
PROGRAM ***
C
      OPEN(UNIT=23,FILE='PROBLEM.TMP',STATUS='NEW')
      WRITE (23,*) 'PROBLEM TEST'
      CLOSE (UNIT=23)
C
C *** FORMAT STATEMENTS
C
1000  FORMAT (A20)
1500  FORMAT (I10)
2000  FORMAT (19X,A20)
2500  FORMAT (3I10)
3000  FORMAT (30X,F10.0)
3500  FORMAT (15E10.4)
4000  FORMAT (5X,11F11.3)
4500        FORMAT     (1X,'WELL#     ',I3,5X,'DELTA
PUMPAGE=',F10.0)
5000  FORMAT (5X,'REMAINING RUNS TO BE COMPLETED =
',I3)
C
      END
```

7.2 OPTIMIZATION MODEL FORMULATION USING GAMS

Before the equations which represent the various constraints and objective functions could actually be written using GAMS, specific variables and parameters have to be defined. GAMS uses a format which incorporates this information into sets, tables, and parameters as input into the model. The variable and equation statements in GAMS define decision variables and equations or inequalities that comprise the optimization model. The following sections explain the use of GAMS programming software.

7.2.1 Sets

Sets are the basic building blocks of a GAMS model, corresponding exactly to the indices in the algebraic representation of models (Brooke, Kendrick, & Meeraus 1988). For example, a set may define an array of well grid cells, sensitive wetland control points, or several other important decision variable ranges. Seven different sets were used to create each optimization model and are listed below defined by letters h through o, not including l:

1) h defines all the well grid points incorporated in the optimization model. This includes a total of 148 well grid cells made up of both municipal and agricultural wells.

2) i defines the 120 municipal well grid cells in the optimization model.

3) j defines the 100 grid cell points in the optimization model where there is a high potential of harm to vegetation in sensitive wetland areas.

4) k defines the 14 Municipal water service areas in the optimization model.

5) m defines the five wastewater treatment plants in optimization model.

6) n defines the 28 agricultural well grid cells in the optimization model.

7) o defines the 9 different agricultural water service areas in the optimization model.

The water supply sets mentioned above are limited to only existing and currently proposed wells and wastewater treatment plants. The water service areas typically refer to individual well fields that supply a distinct part of the project area. These water service areas may or may not be included in the optimization depending on the type of optimization objective. For example, in water quality models five and six the agricultural sets are not included into the optimization objective function ranges. This is due to the high initial chloride concentrations at the agricultural well grid cells.

7.2.2 Tables

GAMS uses items referred to as tables to handle input defined by two-dimensional or higher ordered arrays. GAMS tables are one of several ways for which constants associated with constraints are defined in a GAMS optimization model.

7.2.2.1 Source to Demand Link Tables

Three Tables were created to facilitate the process of constructing water supply and demand constraints which define feasible combinations of sources (well grid cells and wastewater treatment plants) that supply water to the various service areas. These "SERVE" matrices (called tables in the GAMS system) are comprised of ones and zeros, in which a one signifies that the source supplies the particular demand area and a zero means it does not. The following equations were stated in the model to set discharge rates to zero when the coefficient in the matrix was set to zero:

$$if\ \ SERVE1_{i,k} = 0\ \ then\ \ QM_{i,k} = 0.0 \tag{7.1}$$

$$if\ \ SERVE2_{n,o} = 0\ \ then\ \ QA_{n,o} = 0.0 \tag{7.2}$$

$$if\ \ SERVE3_{m,o} = 0\ \ then\ \ QW_{m,o} = 0.0 \tag{7.3}$$

for all i, k, m, n, and o:

where, $SERVE1_{i,k}$ designates which municipal well grid cells i supply which municipal water service areas k, $SERVE2_{n,o}$ designates which agricultural well grid cells n supply which agricultural water service areas o, and $SERVE3_{m,o}$ designates which wastewater treatment plants m can supplement the demand of which agricultural water service areas o.

The tables were developed to allow specific well grid cells to supply more than one water service area. To define which wastewater treatment plants could supplement which agricultural demands, data and maps were reviewed to determine which plants and agricultural areas were in closest proximity.

7.2.2.2 Influence coefficient tables

A large portion of the optimization modeling effort involved determining the influence coefficient tables. These influence coefficient matrices (called TABLES in GAMS) represent the change in head or concentration at various locations with respect to pumping. The target locations on which the constraints were applied, along with well location and corresponding pumping rate, were determined to develop these matrices. The influence coefficient matrices were calculated by performing multiple executions of the simulation models. One simulation was performed for each well grid cell involved in the optimization process, in the flow simulation a total of 148 simulations were made for influence coefficient matrices calculation. Computer programs written in DOS and FORTRAN codes facilitated this process of determining the response matrices (See Appendix A). A similar procedure was used to develop the chloride concentration influence coefficient matrices. SJRWMD executed the necessary simulations to calculate these matrices.

The first step in calculating the influence coefficient matrices involved starting the simulation model with a proper initial condition. It was determined that the closer this initial condition is to the optimized pumping strategy, the better correlation there is between predicted responses of the optimization and simulation models when the optimized pumping strategy is implemented. However, because this optimized strategy is not known, a slightly modified version of the year 2010 projected pumping strategy was used as the initial condition for the flow simulation model. (The flow simulation model the year 2010 pumping strategy given by SJRWMD had to be modified to avoid the drying out of grid cells when the influence coefficient matrices were calculated by increasing discharge rates. This was accomplished by decreasing pumping in problem areas, typically by an order of ten.)

The individual influence coefficients which comprise the influence coefficient matrices were calculated by individually increasing discharge rates of each well grid cell and determining the response at specified control points and all wells incorporated in the optimization process. In the flow simulation model the increased discharge rate was set at 600,000 cubic feet per day (cfd) unless the water service demand rate of the service area which the specific well grid cell supplied was known to have a lower value. The 600,000 cfd maximum value was determined by analyzing the response of the aquifer at a few locations under varying discharge rates. The analysis demonstrated that the response is linear up to approximately 600,000 cfd. For projected year 2010, the maximum discharge rate from any one well grid cell is approximately 200,000 cfd. Therefore, the maximum well grid cell discharge was set at approximately three times the projected maximum discharge. Whenever increased discharge rates drained well grid cells during execution of the simulation model, the maximum allowable discharge was decreased to avoid the drying out condition. When a specific well grid cell supplied more than one service area, the service area demand rates were simply summed.

Through execution of the simulation models, various types of influence coefficients were calculated to develop the six influence coefficient matrices designated as alpha, beta, gamma, theta, gamma, and phi. These matrices are comprised of influence coefficients having units of feet per cubic feet per day (length per volumetric rate) and milligrams per liter per cubic feet per day, for the chloride concentration coefficients. Using these units, the flow simulation influence coefficients originally calculated were generally on the order of magnitude of 10^{-6}. Since coefficients close to one are preferred for matrix inversions by the linear programming solver, the response matrices were multiplied by 10^{+6}. Similarly the concentration influence coefficients were generally on the order of 10^{-4} and these matrices were multiplied by 10^{+4} to make the close to one. Therefore, the resulting units of the influence coefficients are feet per million cubic feet per day and ten thousand milligrams per liter per cubic feet per day. For this study, the optimization model solved the problem using the revised units and then converts the pressure head and drawdown values from one millionth of a foot to feet. This revised units also apply to the chloride concentrations converting them from one ten thousandth of a milligram per liter to milligrams per liter.

7.2.3 Parameters

Once the influence coefficient tables were developed, additional data was gathered consisting of the constant relevant to the model. These one-dimensional arrays of constants (called PARAMETERS in GAMS) included well discharge and wastewater reuse capacity limits, water service demand rates, initial year 1988 withdrawal rates and pressure heads, bottom elevations of the surficial aquifer at well grid cells, initial chloride concentrations, and water quality concentration limits. SJRWMD provided the information for initial pumpage, initial conditions, aquifer characteristic, and model discretization through various reports and simulation model data. Wastewater reuse capacity limits were assumed to be the total volume of effluent discharged. The municipal well capacity limits were taken from the work done in influence coefficient calculation. This value help reduce the problem of cells drying out. The chloride concentration limits were set to reflect initial concentrations in the project area.

7.2.4 Variables

The decision variables of the optimization model are expressed as variables in GAMS and must be defined with the VARIABLES statement. Once the variables were defined in the optimization model, the various constraints can then be stated.

7.2.5 Equations

The constraints define relationships between the various decision variables previously defined and are satisfied for any feasible solution to be identified by the optimization model. GAMS uses the EQUATIONS statement to define the constraint relationships with the decision variable. This part of the model constraints are key to the optimization model formulation within the GAMS environment.

7.2.6 GAMS Optimization Modeling

The GAMS input file used to construct the optimization model is easily revised to optimize a variety of water management scenarios. All the required sets, tables, parameters, variables, equations, and constraints are built into the model to predict strategies under a wide range of management conditions. Using the optimization input file as a skeleton, only the specific values within the GAMS input file need be changed to revise the optimization model and then determine the optimum allocation strategies under revised objective functions and constraints. Once the optimization model is created within GAMS, it is executed using a GAMS linear programming solver. Optimum values of the decision variables are determined when the optimization algorithm identifies an optimum value for the objective function (a maximum or minimum) under a satisfied constraint set. As a result values for decision variables are found consisting of discharge rates from wells and wastewater treatment plants, pressure heads and drawdowns at sensitive wetland areas and well grid cells, and chloride concentrations and concentration changes at well grid cells.

7.3 GAMS OPTIMIZATION MODEL INPUT FILE

```
$TITLE MINIMIZE DRAWDOWN BETWEEN 1988 AND 2010 CONDITIONS WHILE CONSTRAINING QUALITY
$OFFUPPER
*
*****   OPTIMIZATION MODEL FOR THE VOLUSIA COUNTY SUBREGION *****
*
*****   MINIMIZE DRAWDOWN BETWEEN 1988 CONDITIONS AND OPTIMIZED 2010 CONDITIONS   *****
*****   WHILE CONSTRAINING CHLORIDE CONCENTRATIONS AT WELL GRID CELLS            *****
*
*****   SERVICE AREAS ARE WELLFIELDS AND/OR WATER TREATMENT PLANTS
*
 SETS
    H     ALL WELL GRID CELL POINTS   / WELPT1 * WELPT148 /
    I     MUNICIPAL WELL GRID CELLS   / MWELL1 * MWELL120 /
    J     VEGETATION HARM POINTS      / POINT1 * POINT100 /
    K     WATER SERVICE AREAS         / POW, POE, DBM, DBB, DBS, SCK, HHE, HHW, NSB, OBD, OB4, OBH, TCU,
TTI /
    M     WASTEWATER TREATMENT PLANTS / WASTE1 * WASTE5 /
    N     AGRICULTURAL WELL GRID CELL / AGWELL1 * AGWELL28 /
    O     AGRICULTURAL SERVICE AREAS  / AGAREA1 * AGAREA9 /   ;

*****   MWELL# IS THE SUM OF WELLS WITHIN CORRESPONDING GRID CELL *****
*


 TABLE     SERVE1(I,K)     1 MEANS MUN. WELL GRID CELL "I" SERVES WATER SERVICE AREA "K"
            POW  POE  DBM  DBB  DBS  SCK  HHE  HHW  NSB  OBD  OB4  OBH  TCU  TTI
    MWELL1    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL2    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL3    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL4    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL5    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL6    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL7    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL8    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL9    1    0    0    0    0    0    0    0    0    0    0    0    0    0
    MWELL10   1    0    0    0    0    0    0    0    0    0    0    0    0    0
                        .
                        .
                        .
    MWELL115  0    0    0    0    0    0    0    0    0    0    1    1    0    0
    MWELL116  0    0    0    0    0    0    0    0    0    0    1    1    0    0
    MWELL117  0    0    0    0    0    0    0    0    0    0    1    1    0    0
    MWELL118  0    0    0    0    0    0    0    0    0    0    1    1    0    0
    MWELL119  0    0    0    0    0    0    0    0    0    0    0    0    1    0
    MWELL120  0    0    0    0    0    0    0    0    0    0    0    0    0    1   ;


 TABLE     SERVE2(N,O)     1 MEANS AG. WELL GRID CELL "N" SERVES AG. WATER SERVICE AREA "O"
           AGAREA1  AGAREA2  AGAREA3  AGAREA4  AGAREA5  AGAREA6  AGAREA7  AGAREA8  AGAREA9
    AGWELL1    1       0        0        0        0        0        0        0        0
    AGWELL2    1       0        0        0        0        0        0        0        0
    AGWELL3    0       1        0        0        0        0        0        0        0
    AGWELL4    0       1        0        0        0        0        0        0        0
    AGWELL5    0       1        0        0        0        0        0        0        0
    AGWELL6    0       0        1        0        0        0        0        0        0
    AGWELL7    0       0        1        0        0        0        0        0        0
    AGWELL8    0       0        0        1        0        0        0        0        0
    AGWELL9    0       0        0        1        0        0        0        0        0
    AGWELL10   0       0        0        1        0        0        0        0        0
                          .
                          .
                          .
    AGWELL24   0       0        0        0        0        0        0        1        0
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AGWELL25	0	0	0	0	0	0	0	1	0
AGWELL26	0	0	0	0	0	0	0	0	1
AGWELL27	0	0	0	0	0	0	0	0	1
AGWELL28	0	0	0	0	0	0	0	0	1 ;

TABLE SERVE3(M,O) 1 MEANS WASTEWATER TREATMENT PLANT "M" SERVES AG. WATER SERVICE AREA "O"

	AGAREA1	AGAREA2	AGAREA3	AGAREA4	AGAREA5	AGAREA6	AGAREA7	AGAREA8	AGAREA9
WASTE1	1	0	1	0	0	0	0	0	0
WASTE2	0	1	0	0	0	0	0	0	0
WASTE3	0	0	1	0	0	0	0	0	1
WASTE4	0	0	0	1	1	0	0	0	0
WASTE5	0	0	0	0	0	1	0	1	0 ;

TABLE ALPHA(I,J) INFLUENCE COEFFICIENTS FOR WELLS WRT DELTA HEAD

	POINT1	POINT2	POINT3	POINT4	POINT5	POINT6	POINT7	POINT8	POINT9	POINT10
MWELL1	.461E-01	.469E-01	.490E-01	.587E-01	.674E-01	.766E-01	.865E-01	.109E+00	.137E+00	.144E+00
MWELL2	.432E-01	.440E-01	.466E-01	.564E-01	.657E-01	.761E-01	.879E-01	.121E+00	.171E+00	.197E+00
MWELL3	.457E-01	.465E-01	.489E-01	.588E-01	.680E-01	.778E-01	.887E-01	.115E+00	.150E+00	.161E+00
MWELL4	.408E-01	.414E-01	.433E-01	.520E-01	.601E-01	.688E-01	.786E-01	.105E+00	.145E+00	.168E+00
MWELL5	.410E-01	.417E-01	.437E-01	.524E-01	.604E-01	.691E-01	.788E-01	.104E+00	.141E+00	.159E+00
MWELL6	.444E-01	.452E-01	.476E-01	.573E-01	.664E-01	.762E-01	.872E-01	.115E+00	.155E+00	.170E+00
MWELL7	.431E-01	.438E-01	.460E-01	.554E-01	.640E-01	.735E-01	.840E-01	.111E+00	.151E+00	.169E+00
MWELL8	.450E-01	.458E-01	.486E-01	.590E-01	.688E-01	.797E-01	.921E-01	.126E+00	.176E+00	.198E+00
MWELL9	.425E-01	.431E-01	.450E-01	.539E-01	.620E-01	.707E-01	.803E-01	.104E+00	.138E+00	.152E+00
MWELL10	.393E-01	.399E-01	.418E-01	.502E-01	.580E-01	.665E-01	.760E-01	.102E+00	.142E+00	.167E+00

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+	POINT91	POINT92	POINT93	POINT94	POINT95	POINT96	POINT97	POINT98	POINT99	POINT100
MWELL1	.153E+00	.178E+00	.137E+01	.271E+01	.461E+00	.325E+01	.447E+01	.359E+01	.359E+01	.100E+01
MWELL2	.130E+00	.149E+00	.110E+01	.206E+01	.357E+00	.243E+01	.324E+01	.264E+01	.253E+01	.723E+00
MWELL3	.146E+00	.169E+00	.129E+01	.250E+01	.426E+00	.297E+01	.403E+01	.325E+01	.318E+01	.894E+00
MWELL4	.126E+00	.144E+00	.106E+01	.197E+01	.343E+00	.234E+01	.311E+01	.255E+01	.246E+01	.718E+00
MWELL5	.129E+00	.148E+00	.110E+01	.209E+01	.361E+00	.248E+01	.334E+01	.272E+01	.263E+01	.767E+00
MWELL6	.139E+00	.160E+00	.121E+01	.232E+01	.398E+00	.275E+01	.371E+01	.299E+01	.290E+01	.827E+00
MWELL7	.134E+00	.154E+00	.115E+01	.218E+01	.378E+00	.259E+01	.348E+01	.283E+01	.273E+01	.788E+00
MWELL8	.135E+00	.155E+00	.116E+01	.220E+01	.376E+00	.257E+01	.345E+01	.279E+01	.266E+01	.758E+00
MWELL9	.137E+00	.157E+00	.117E+01	.223E+01	.387E+00	.267E+01	.361E+01	.294E+01	.287E+01	.832E+00

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MWELL115	.164E+00	.161E+00	.763E+00	.749E+00	.150E+00	.640E+00	.588E+00	.548E+00	.401E+00	.106E+00
MWELL116	.162E+00	.160E+00	.773E+00	.771E+00	.153E+00	.659E+00	.610E+00	.565E+00	.417E+00	.109E+00
MWELL117	.470E+01	.221E+01	.380E+01	.183E+01	.470E+00	.135E+01	.980E+00	.101E+01	.667E+00	.175E+00
MWELL118	.597E+01	.206E+01	.336E+01	.164E+01	.427E+00	.124E+01	.920E+00	.956E+00	.633E+00	.166E+00
MWELL119	.359E+01	.338E+01	.190E+02	.216E+02	.388E+01	.223E+02	.230E+02	.227E+02	.149E+02	.426E+01
MWELL120	.211E+01	.186E+01	.982E+01	.109E+02	.204E+01	.114E+02	.116E+02	.116E+02	.776E+01	.222E+01

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TABLE BETA(N,J) INFLUENCE COEFFICIENTS FOR AG. WELLS WRT DELTA HEAD

	POINT1	POINT2	POINT3	POINT4	POINT5	POINT6	POINT7	POINT8	POINT9	POINT10
AGWELL1	.308E+00	.298E+00	.267E+00	.281E+00	.293E+00	.301E+00	.308E+00	.301E+00	.326E+00	.325E+00
AGWELL2	.384E+00	.373E+00	.332E+00	.351E+00	.365E+00	.374E+00	.382E+00	.374E+00	.405E+00	.404E+00
AGWELL3	.173E+00	.167E+00	.149E+00	.157E+00	.164E+00	.168E+00	.172E+00	.169E+00	.183E+00	.184E+00
AGWELL4	.219E+00	.211E+00	.188E+00	.199E+00	.207E+00	.211E+00	.217E+00	.213E+00	.231E+00	.231E+00
AGWELL5	.172E+00	.166E+00	.148E+00	.156E+00	.163E+00	.167E+00	.171E+00	.168E+00	.182E+00	.182E+00
AGWELL6	.140E+00	.136E+00	.121E+00	.127E+00	.133E+00	.136E+00	.139E+00	.136E+00	.147E+00	.147E+00
AGWELL7	.147E+00	.143E+00	.128E+00	.134E+00	.140E+00	.143E+00	.146E+00	.143E+00	.154E+00	.154E+00
AGWELL8	.208E+00	.201E+00	.179E+00	.189E+00	.196E+00	.201E+00	.206E+00	.201E+00	.218E+00	.218E+00
AGWELL9	.158E+00	.153E+00	.136E+00	.143E+00	.149E+00	.153E+00	.156E+00	.153E+00	.166E+00	.166E+00
AGWELL10	.148E+00	.143E+00	.127E+00	.134E+00	.140E+00	.143E+00	.146E+00	.143E+00	.155E+00	.155E+00

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+	POINT91	POINT92	POINT93	POINT94	POINT95	POINT96	POINT97	POINT98	POINT99	POINT100
AGWELL1	.748E+00	.731E+00	.438E+01	.507E+01	.928E+00	.532E+01	.552E+01	.546E+01	.374E+01	.109E+01
AGWELL2	.930E+00	.910E+00	.533E+01	.615E+01	.110E+01	.640E+01	.666E+01	.652E+01	.435E+01	.125E+01
AGWELL3	.478E+00	.468E+00	.269E+01	.314E+01	.614E+00	.332E+01	.338E+01	.340E+01	.236E+01	.683E+00
AGWELL4	.624E+00	.612E+00	.344E+01	.399E+01	.784E+00	.420E+01	.427E+01	.428E+01	.297E+01	.858E+00
AGWELL5	.481E+00	.471E+00	.270E+01	.314E+01	.617E+00	.332E+01	.337E+01	.340E+01	.236E+01	.683E+00
AGWELL6	.350E+00	.342E+00	.204E+01	.232E+01	.403E+00	.242E+01	.251E+01	.248E+01	.158E+01	.456E+00
AGWELL7	.367E+00	.358E+00	.214E+01	.243E+01	.421E+00	.254E+01	.263E+01	.261E+01	.166E+01	.479E+00
AGWELL8	.534E+00	.521E+00	.298E+01	.344E+01	.650E+00	.358E+01	.367E+01	.364E+01	.251E+01	.719E+00
AGWELL9	.433E+00	.421E+00	.238E+01	.273E+01	.524E+00	.286E+01	.292E+01	.291E+01	.202E+01	.578E+00

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AGWELL24	.112E+00	.114E+00	.698E+00	.927E+00	.179E+00	.102E+01	.115E+01	.109E+01	.887E+00	.275E+00
AGWELL25	.180E+00	.179E+00	.106E+01	.130E+01	.246E+00	.139E+01	.150E+01	.144E+01	.108E+01	.325E+00
AGWELL26	.424E+00	.414E+00	.243E+01	.275E+01	.494E+00	.288E+01	.297E+01	.296E+01	.189E+01	.553E+00
AGWELL27	.449E+00	.435E+00	.251E+01	.283E+01	.508E+00	.296E+01	.305E+01	.303E+01	.194E+01	.566E+00
AGWELL28	.209E-01	.196E-01	.602E+00	.613E+00	-.621E-01	.653E+00	.747E+00	.723E+00	.535E-01	-.888E-02

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TABLE GAMMA(I,H) WELL INFLUENCE COEFFICIENTS FOR WELLS WRT DELTA HEA

	WELPT1	WELPT2	WELPT3	WELPT4	WELPT5	WELPT6	WELPT7	WELPT8	WELPT9	WELPT10
MWELL1	.243E+02	.952E+01	.132E+02	.911E+01	.101E+02	.117E+02	.108E+02	.102E+02	.122E+02	.829E+01
MWELL2	.103E+02	.378E+02	.127E+02	.153E+02	.152E+02	.178E+02	.201E+02	.257E+02	.136E+02	.136E+02
MWELL3	.146E+02	.130E+02	.215E+02	.115E+02	.130E+02	.172E+02	.149E+02	.144E+02	.148E+02	.103E+02
MWELL4	.982E+01	.152E+02	.112E+02	.267E+02	.186E+02	.139E+02	.167E+02	.132E+02	.147E+02	.218E+02
MWELL5	.109E+02	.146E+02	.125E+02	.183E+02	.235E+02	.150E+02	.176E+02	.133E+02	.173E+02	.160E+02
MWELL6	.126E+02	.170E+02	.167E+02	.139E+02	.153E+02	.245E+02	.197E+02	.188E+02	.157E+02	.121E+02
MWELL7	.116E+02	.188E+02	.145E+02	.167E+02	.180E+02	.197E+02	.248E+02	.171E+02	.166E+02	.143E+02
MWELL8	.110E+02	.255E+02	.139E+02	.131E+02	.136E+02	.198E+02	.179E+02	.378E+02	.132E+02	.116E+02
MWELL9	.124E+02	.127E+02	.137E+02	.139E+02	.161E+02	.147E+02	.154E+02	.124E+02	.253E+02	.125E+02
MWELL10	.910E+01	.136E+02	.101E+02	.225E+02	.167E+02	.124E+02	.146E+02	.119E+02	.135E+02	.283E+02

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+	WELPT141	WELPT142	WELPT143	WELPT144	WELPT145	WELPT146	WELPT147	WELPT148
MWELL1	.815E+00	.885E-01	.530E-01	.561E-01	.489E-01	.406E+00	.435E+00	.422E+00
MWELL2	.977E+00	.822E-01	.493E-01	.526E-01	.461E-01	.331E+00	.354E+00	.343E+00
MWELL3	.869E+00	.867E-01	.519E-01	.551E-01	.481E-01	.378E+00	.405E+00	.392E+00
MWELL4	.107E+01	.910E-01	.542E-01	.579E-01	.505E-01	.340E+00	.362E+00	.352E+00
MWELL5	.986E+00	.921E-01	.548E-01	.584E-01	.509E-01	.350E+00	.376E+00	.365E+00
MWELL6	.915E+00	.866E-01	.518E-01	.551E-01	.481E-01	.362E+00	.387E+00	.377E+00
MWELL7	.961E+00	.884E-01	.528E-01	.562E-01	.491E-01	.354E+00	.378E+00	.368E+00
MWELL8	.924E+00	.809E-01	.486E-01	.518E-01	.454E-01	.339E+00	.363E+00	.352E+00
MWELL9	.938E+00	.937E-01	.557E-01	.593E-01	.516E-01	.371E+00	.398E+00	.387E+00

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MWELL115	.157E+00	.118E-01	.779E-02	.814E-02	.743E-02	.917E-01	.992E-01	.951E-01
MWELL116	.164E+00	.121E-01	.803E-02	.839E-02	.766E-02	.933E-01	.101E+00	.968E-01
MWELL117	.146E+00	.159E-01	.104E-01	.107E-01	.972E-02	.216E+00	.233E+00	.223E+00
MWELL118	.141E+00	.155E-01	.102E-01	.105E-01	.951E-02	.220E+00	.239E+00	.227E+00
MWELL119	.856E+01	.661E+00	.492E+00	.504E+00	.467E+00	.594E+01	.649E+01	.620E+01
MWELL120	.423E+01	.348E+00	.249E+00	.256E+00	.238E+00	.301E+01	.329E+01	.314E+01

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TABLE THETA(N,H) WELL INFLUENCE COEFFICIENTS FOR AG. WELLS WRT DELTA

	WELPT1	WELPT2	WELPT3	WELPT4	WELPT5	WELPT6	WELPT7	WELPT8	WELPT9	WELPT10
AGWELL1	.557E+01	.454E+01	.424E+01	.451E+01	.467E+01	.496E+01	.485E+01	.464E+01	.536E+01	.443E+01
AGWELL2	.676E+01	.551E+01	.505E+01	.542E+01	.561E+01	.599E+01	.586E+01	.564E+01	.646E+01	.533E+01

AGWELL3	.315E+01	.256E+01	.241E+01	.254E+01	.263E+01	.279E+01	.273E+01	.262E+01	.300E+01	.249E+01
AGWELL4	.394E+01	.322E+01	.302E+01	.319E+01	.329E+01	.348E+01	.341E+01	.330E+01	.373E+01	.313E+01
AGWELL5	.313E+01	.255E+01	.240E+01	.252E+01	.261E+01	.277E+01	.271E+01	.261E+01	.298E+01	.247E+01
AGWELL6	.250E+01	.200E+01	.179E+01	.195E+01	.202E+01	.219E+01	.213E+01	.205E+01	.238E+01	.191E+01
AGWELL7	.263E+01	.210E+01	.188E+01	.205E+01	.213E+01	.231E+01	.224E+01	.216E+01	.250E+01	.201E+01
AGWELL8	.359E+01	.295E+01	.275E+01	.291E+01	.301E+01	.319E+01	.313E+01	.301E+01	.343E+01	.286E+01
AGWELL9	.278E+01	.228E+01	.213E+01	.226E+01	.233E+01	.247E+01	.242E+01	.233E+01	.265E+01	.221E+01
AGWELL10	.256E+01	.210E+01	.196E+01	.208E+01	.214E+01	.228E+01	.223E+01	.215E+01	.244E+01	.204E+01

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+	WELPT141	WELPT142	WELPT143	WELPT144	WELPT145	WELPT146	WELPT147	WELPT148
AGWELL1	.210E+01	.258E+00	.175E+00	.178E+00	.164E+00	.133E+01	.145E+01	.139E+01
AGWELL2	.254E+01	.271E+00	.189E+00	.193E+00	.177E+00	.159E+01	.173E+01	.166E+01
AGWELL3	.114E+01	.101E+00	.707E-01	.725E-01	.674E-01	.142E+01	.156E+01	.146E+01
AGWELL4	.143E+01	.125E+00	.892E-01	.914E-01	.847E-01	.184E+01	.203E+01	.190E+01
AGWELL5	.113E+01	.101E+00	.704E-01	.721E-01	.672E-01	.147E+01	.161E+01	.151E+01
AGWELL6	.901E+00	.729E-01	.559E-01	.568E-01	.528E-01	.775E+00	.835E+00	.813E+00
AGWELL7	.947E+00	.762E-01	.586E-01	.595E-01	.554E-01	.778E+00	.839E+00	.816E+00
AGWELL8	.134E+01	.113E+00	.805E-01	.825E-01	.768E-01	.105E+01	.115E+01	.110E+01
AGWELL9	.103E+01	.891E-01	.633E-01	.649E-01	.603E-01	.967E+00	.106E+01	.100E+01
AGWELL10	.955E+00	.808E-01	.579E-01	.593E-01	.551E-01	.826E+00	.904E+00	.859E+00

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AGWELL24	.845E+00	.403E+00	.379E+00	.675E+00	.337E+00	.322E+00	.346E+00	.336E+00
AGWELL25	.924E+00	.301E+00	.322E+00	.347E+00	.750E+00	.410E+00	.442E+00	.428E+00
AGWELL26	.104E+01	.798E-01	.612E-01	.623E-01	.579E-01	.186E+01	.186E+01	.184E+01
AGWELL27	.108E+01	.834E-01	.637E-01	.649E-01	.603E-01	.185E+01	.204E+01	.192E+01
AGWELL28	.185E+00	-.904E-02	.847E-02	.799E-02	.833E-02	-.662E-01	-.479E-01	-.579E-01

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TABLE ZETA(I,H) WELL INFLUENCE COEFFICIENTS FOR WELLS WRT DELTA QUA

	WELPT1	WELPT2	WELPT3	WELPT4	WELPT5	WELPT6	WELPT7	WELPT8	WELPT9	WELPT10
MWELL1	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL2	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL3	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL4	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL5	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL6	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL7	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL8	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL9	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL10	0.0E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00

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+	WELPT141	WELPT142	WELPT143	WELPT144	WELPT145	WELPT146	WELPT147	WELPT148
MWELL1	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL2	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL3	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL4	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL5	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL6	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL7	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL8	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL9	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00
MWELL10	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00

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MWELL115	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.000E+00	0.177E+02	0.170E+02	0.183E+02

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MWELL116  0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.177E+02 0.170E+02 0.183E+02
MWELL117  0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.304E+01 0.264E+01 0.282E+01
MWELL118  0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.676E+01 0.618E+01 0.656E+01
MWELL119  0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.548E+01 0.390E+01 0.444E+01
MWELL120  0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.809E+01 0.696E+01 0.744E+01
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  TABLE    PHI(N,H)   WELL INFLUENCE COEFFICIENTS FOR AG. WELLS WRT DELTA
     WELPT1   WELPT2   WELPT3   WELPT4   WELPT5   WELPT6   WELPT7   WELPT8   WELPT9   WELPT10

AGWELL1 .426E+03 .439E+030.456E+030.271E+030.349E+030.439E+030.439E+030.439E+030.414E+030.188E+03
AGWELL2 .426E+03 .439E+030.456E+030.271E+030.349E+030.439E+030.439E+030.439E+030.414E+030.188E+03
AGWELL3 .198E+02 .177E+020.203E+020.122E+020.161E+020.177E+020.177E+020.177E+020.179E+020.962E+01
AGWELL4 .000E+00 .000E+000.000E+000.000E+000.000E+000.000E+000.000E+000.000E+000.000E+000.000E+00
AGWELL5 .000E+00 .000E+000.000E+000.000E+000.000E+000.000E+000.000E+000.000E+000.000E+000.000E+00
AGWELL6 .488E+02 .475E+020.510E+020.305E+020.397E+020.475E+020.475E+020.475E+020.463E+020.221E+02
AGWELL7 .488E+02 .475E+020.510E+020.305E+020.397E+020.475E+020.475E+020.475E+020.463E+020.221E+02
AGWELL8 .407E+02 .575E+020.501E+020.266E+020.282E+020.575E+020.575E+020.575E+020.413E+020.126E+02
AGWELL9 .701E+02 .732E+020.742E+020.437E+020.558E+020.732E+020.732E+020.732E+020.680E+020.286E+02
AGWELL10.925E+02.929E+020.981E+020.587E+020.759E+020.929E+020.929E+020.929E+020.889E+020.417E+02
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                                  .
                                  .
 +       WELPT141  WELPT142  WELPT143  WELPT144  WELPT145  WELPT146  WELPT147  WELPT148

AGWELL1    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.321E+03 0.279E+03 0.300E+03
AGWELL2    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.321E+03 0.279E+03 0.300E+03
AGWELL3    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.548E+01 0.390E+01 0.444E+01
AGWELL4    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00
AGWELL5    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00
AGWELL6    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.322E+02 0.275E+02 0.296E+02
AGWELL7    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.322E+02 0.275E+02 0.296E+02
AGWELL8    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.154E+03 0.149E+03 0.159E+03
AGWELL9    0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.691E+02 0.615E+02 0.654E+02
AGWELL10   0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.618E+02 0.528E+02 0.570E+02
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                                  .
                                  .
AGWELL24   0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00
AGWELL25   0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00
AGWELL26   0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.213E+04 0.184E+04 0.199E+04
AGWELL27   0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.587E+02 0.521E+02 0.567E+02
AGWELL28   0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.000E+00 0.195E+03 0.177E+03 0.188E+03
                                                                                    ;

  PARAMETERS

    CM(I)    CAPACITY OF EACH MUN. WELL GRID CELL (CUBIC FEET PER DAY)
 /        MWELL1      6.0E+05
          MWELL2      6.0E+05
          MWELL3      6.0E+05
          MWELL4      6.0E+05
          MWELL5      6.0E+05
          MWELL6      6.0E+05
          MWELL7      6.0E+05
          MWELL8      6.0E+05
          MWELL9      6.0E+05
          MWELL10     6.0E+05
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              .
              .
          MWELL115    6.0E+05
          MWELL116    6.0E+05
          MWELL117    6.0E+05
          MWELL118    6.0E+05
          MWELL119    6.0E+05
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MWELL120 6.0E+05 /

DDPRIV(J) DRAWDOWN OF EACH POINT OF INTEREST FOR PRIVATE WELLS NOT OPTIMIZED (FEET)

/	POINT1	.08
	POINT2	.05
	POINT3	.02
	POINT4	.02
	POINT5	.02
	POINT6	.02
	POINT7	.01
	POINT8	.01
	POINT9	.03
	POINT10	.08
	.	
	.	
	.	
	POINT95	.01
	POINT96	.01
	POINT97	.02
	POINT98	.02
	POINT99	.02
	POINT100	.02 /

H0(J) INITIAL HEAD OF EACH POINT OF INTEREST (FEET)

/	POINT1	29.7
	POINT2	29.9
	POINT3	29.8
	POINT4	33.6
	POINT5	34.7
	POINT6	36.6
	POINT7	34.3
	POINT8	34.3
	POINT9	34.4
	POINT10	34.4
	.	
	.	
	.	
	POINT95	24.8
	POINT96	21.0
	POINT97	19.9
	POINT98	19.6
	POINT99	22.3
	POINT100	21.6 /

HW0(H) INITIAL HEAD OF EACH WELL CELL POINT (FEET)

/	WELPT1	25.7
	WELPT2	32.2
	WELPT3	34.1
	WELPT4	32.1
	WELPT5	31.4
	WELPT6	32.0
	WELPT7	31.7
	WELPT8	31.6
	WELPT9	25.7
	WELPT10	32.8
	.	
	.	
	.	
	WELPT144	20.0
	WELPT145	23.7
	WELPT146	46.0
	WELPT147	46.3
	WELPT148	46.4 /

BOTELEV(H) BOTTOM ELEVATION OF SURFICIAL AQUIFER AT WELL CELL POINTS (FEET)

```
/   WELPT1      -10.00
    WELPT2      -10.00
    WELPT3      -10.00
    WELPT4      -10.00
    WELPT5      -10.00
    WELPT6      -10.00
    WELPT7      -10.00
    WELPT8       -9.00
    WELPT9      -10.00
    WELPT10     -10.00
       .
       .
       .
    WELPT144    -30.00
    WELPT145    -30.00
    WELPT146    -40.00
    WELPT147    -40.00
    WELPT148    -40.00   /
```

DM(K) DEMAND OF EACH MUN. WATER SERVICE AREA (CUBIC FEET PER DAY)

```
 /  POW        961230.
    POE         96123.
    DBM        207780.
    DBB       1140334.
    DBS       1555001.
    SCK         43300.
    HHE         15939.
    HHW        159388.
    NSB         80000.
    OBD        263747.
    OB4        351663.
    OBH        549473.
    TCU         24298.
    TTI         78507.   /
```

CW(M) CAPACITY OF EACH WASTEWATER TREATMENT PLANT (CUBIC FEET PER DAY)

```
/       WASTE1    1604400
        WASTE2     320880
        WASTE3    1604400
        WASTE4     802200
        WASTE5      46795   /
```

CA(N) CAPACITY OF EACH AGRICULTURAL WELL GRID CELL (CUBIC FEET PER DAY)

```
/       AGWELL1     16923
        AGWELL2     16923
        AGWELL3     18854
        AGWELL4     18854
        AGWELL5     18854
        AGWELL6     25861
        AGWELL7     25861
        AGWELL8     19414
        AGWELL9     19414
        AGWELL10    19414
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           .
           .
        AGWELL24   281216
        AGWELL25   281216
        AGWELL26    12880
        AGWELL27    12880
        AGWELL28    12880   /
```

DA(O) DEMAND (1990) OF EACH AGRICULTURAL SERVICE AREA (CUBIC FEET PER DAY)

/ AGAREA1	9225.83	
AGAREA2	19113.93	
AGAREA3	11879.47	
AGAREA4	21468.91	
AGAREA5	25762.70	
AGAREA6	40361.56	
AGAREA7	85747.17	
AGAREA8	103588.91	
AGAREA9	5725.04	/

QM0(I) INITIAL PUMPING RATE OF EACH MUN. WELL GRID CELL (CUBIC FEET PER DAY)

/ MWELL1	35475.00	
MWELL2	35539.00	
MWELL3	35475.00	
MWELL4	35475.00	
MWELL5	70951.00	
MWELL6	35475.00	
MWELL7	35475.00	
MWELL8	70951.00	
MWELL9	35475.00	
MWELL10	35475.00	
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MWELL116	0.00	
MWELL117	0.00	
MWELL118	0.00	
MWELL119	12234.00	
MWELL120	42723.00	/

QA0(N) INITIAL PUMPING RATE OF EACH AGRICULTURAL WELL GRID CELL (CUBIC FEET PER DAY)

/ AGWELL1	3704.00	
AGWELL2	5587.00	
AGWELL3	6387.00	
AGWELL4	6514.00	
AGWELL5	6284.00	
AGWELL6	6327.00	
AGWELL7	6604.00	
AGWELL8	9267.00	
AGWELL9	3089.00	
AGWELL10	3153.00	
.		
.		
.		
AGWELL24	27865.00	
AGWELL25	65017.00	
AGWELL26	934.00	
AGWELL27	1062.00	
AGWELL28	4033.00	/

CC0(H) INITIAL CHLORIDE CONCENTRATION AT ALL WELL GRID CELLS "H" (MG PER LITER)

/ WELPT1	2.11
WELPT2	1.69
WELPT3	1.31
WELPT4	0.58
WELPT5	0.81
WELPT6	1.69
WELPT7	1.69
WELPT8	1.69
WELPT9	2.48
WELPT10	0.04
.	
.	
.	

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        WELPT144      0.00
        WELPT145      0.00
        WELPT146    107.68
        WELPT147    125.73
        WELPT148    130.52      /

   CL(H)    WATER QUALITY CONCENTRATION LIMIT OF ALL WELL GRID CELLS (MG PER LITER)

  /     WELPT1      1000.0
        WELPT2      1000.0
        WELPT3      1000.0
        WELPT4      1000.0
        WELPT5      1000.0
        WELPT6      1000.0
        WELPT7      1000.0
        WELPT8      1000.0
        WELPT9      1000.0
        WELPT10     1000.0
              .
              .
              .
        WELPT144    9500.0
        WELPT145    9500.0
        WELPT146    9500.0
        WELPT147    9500.0
        WELPT148    9500.0   /    ;

 VARIABLES

   QM(I,K)       PUMP RATE OF MUNICIPAL WELL GRID CELL "I" FOR SERVICE AREA "K" (CFD)
   QMT(I)        TOTAL PUMP RATE OF MUNICIPAL WELL GRID CELL "I" (CFD)
   QA(N,O)       PUMP RATE OF AGRICULTURAL WELL GRID CELL "N" FOR SERVICE AREA "O" (CFD)
   QAT(N)        TOTAL PUMP RATE OF AGRICULTURAL WELL GRID CELL "N"  (CFD)
   QW(M,O)       REUSE RATE OF WW TREATMENT PLANT EFFLUENT "M" FOR SERVICE AREA "O" (CFD)
   QWT(M)        TOTAL REUSE RATE OF WASTEWATER TREATMENT PLANT EFFLUENT "M"  (CFD)
   DD(J)         DRAWDOWN AT VEGETATION HARM POINTS "J" IN FEET
   TDD(J)        TOTAL DRAWDOWN AT VEGETATION HARM POINTS "J" IN FEET
   DDW(H)        DRAWDOWN AT WELL CELL POINTS "H" IN FEET
   TDDW(H)       TOTAL DRAWDOWN AT WELL CELL POINTS "H" IN FEET
   HD(J)         HEAD  AT VEGETATION HARM POINTS "J" IN FEET
   THD(J)        HEAD TOTAL AT VEGETATION HARM POINTS "J" IN FEET
   HDW(H)        HEAD AT WELL CELL POINTS "H" IN FEET
   THDW(H)       HEAD TOTAL AT WELL CELL POINTS "H" IN FEET
   CI(H)         CHLORIDE CONCENTRATION INCREASE AT ALL WELL GRID CELLS "H"
   TCI(H)        TOTAL CHLORIDE CONCENTRATION INCREASE AT ALL WELL GRID CELLS "H"
   CC(H)         CHLORIDE CONCENTRATION AT ALL WELL GRID CELLS "H"
   TCC(H)        TOTAL CHLORIDE CONCENTRATION AT ALL WELL GRID CELLS "H"
    S            MINIMIZE MAXIMAL DRAWDOWN OVER VEGETATION HARM POINTS

 POSITIVE VARIABLE QM, QA, QW, CC  ;

 EQUATIONS

   MUNTOTAL(I)      CALCULATE TOTAL PUMPING OF MUNICIPAL WELL "I"
   AGTOTAL(N)       CALCULATE TOTAL PUMPING OF AGRICULTURAL WELL "N"
   WTPTOTAL(M)      CALCULATE TOTAL REUSE FROM WASTEWATER TREATMENT PLANTS "M"
   MUNONOFF(I,K)    SET PUMPAGE OF MUNICIPAL WELL GRID CELLS "I" FOR SERVICE AREA "K" TO ZERO
   AGONOFF(N,O)     SET PUMPAGE OF AGRICULTURAL WELL GRID CELLS "N" FOR SERVICE AREA "O" TO
ZERO
   WTPONOFF(M,O)    SET REUSE RATE OF WASTEWATER TREATMENT PLANTS "M" FOR SERVICE AREA "O"
TO ZERO
   MUNDEMND(K)      SATISFY DEMAND AT MUNICIPAL WATER SERVICE AREAS "K"
   AGDEMND(O)       SATISFY DEMAND AT AGRICULTURAL SERVICE AREAS "O"
   MUNCAP(I)        CAPACITY LIMIT AT MUNICIPAL WELL GRID CELLS "I"
   AGCAP(N)         CAPACITY LIMIT AT AGRICULTURAL GRID CELLS "N"
   WWTCAP(M)        CAPACITY LIMIT AT WASTEWATER TREATMENT PLANTS "M"
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      CONCLIM(H)        CONCENTRATION LIMIT AT ALL WELL GRID CELLS "H"
      MUNMIN(I)         MINIMUM LIMIT AT MUNICIPAL WELL GRID CELLS "I"
      DRYCELL(H)        MAINTAIN AT LEAST 1.0 FOOT SATURATED THICKNESS AT WELL CELL POINTS "H"
      HEAD(J)           CALCULATE HEAD AT VEGETATION HARM POINTS "J"
      TOTALHD(J)        CALCULATE TOTAL HEAD AT VEGETATION HARM POINTS "J"
      DRAWDOWN(J)       CALCULATE DRAWDOWN AT VEGETATION HARM POINTS "J"
      TOTALDDN(J)       CALCULATE TOTAL DRAWDOWN AT VEG. HARM POINTS "J"
      WELLHD(H)         CALCULATE HEAD AT ALL WELL CELL POINTS "H"
      TOTWELHD(H)       CALCULATE TOTAL HEAD AT ALL WELL CELL POINTS "H"
      WELLDD(H)         CALCULATE DRAWDOWN AT ALL WELL CELL POINTS "H"
      TOTWELDD(H)       CALCULATE TOTAL DRAWDOWN AT ALL WELL CELL POINTS "H"
      CONCINC(H)        CALCULATE CHLORIDE CONCENTRATION INCREASE AT ALL WELL GRID CELLS "H"
      TOTCNINC(H)       CALCULATE TOTAL CHLORIDE CONCENTRATION INCREASE AT ALL WELL GRID CELLS
"H"
      CLCONC(H)         CALCULATE CHLORIDE CONCENTRATION AT ALL WELL GRID CELLS "H"
      TOTCLCON(H)       CALCULATE TOTAL CHLORIDE CONCENTRATION AT ALL WELL GRID CELLS "H"
      MINDD(J)          MINIMIZE DRAWDOWN AT VEGETATION HARM POINTS "J"  ;

   MUNTOTAL(I)     ..  QMT(I)  =E=  SUM(K,QM(I,K))  ;

   AGTOTAL(N)      ..  QAT(N)  =E=  SUM(O,QA(N,O))  ;

   WTPTOTAL(M)     ..  QWT(M)  =E=  SUM(O,QW(M,O))  ;

   MUNONOFF(I,K)   ..  QM(I,K)$(SERVE1(I,K) EQ 0)  =E=  0.0  ;

   AGONOFF(N,O)    ..  QA(N,O)$(SERVE2(N,O) EQ 0)  =E=  0.0  ;

   WTPONOFF(M,O)   ..  QW(M,O)$(SERVE3(M,O) EQ 0)  =E=  0.0  ;

   MUNDEMND(K)     ..  SUM(I,QM(I,K))  =G=  DM(K)  ;

   AGDEMND(O)      ..  SUM(N,QA(N,O)) + SUM(M,QW(M,O))  =G=  DA(O)  ;

   MUNCAP(I)       ..  QMT(I)   =L=    CM(I)  ;

   AGCAP(N)        ..  QAT(N)   =L=    CA(N)  ;

   WWTCAP(M)       ..  QWT(M)   =L=    CW(M)  ;

   CONCLIM(H)      ..  TCC(H)   =L=    MAX(CL(H), CC0(H) )  ;

   MUNMIN(I)       ..  QMT(I)   =G=    0.5 * QM0(I)  ;

   DRYCELL(H)      ..  HDW(H)   =G=    BOTELEV(H) + 1.0   ;

   DRAWDOWN(J)     ..  DD(J)    =E=    SUM(I,ALPHA(I,J)*(QMT(I)-QM0(I))) + SUM(N,BETA(N,J)*(QAT(N)-QA0(N)))
                                   + (DDPRIV(J) * 1.0E+06) ;

   TOTALDDN(J)     ..  TDD(J)   =E=    DD(J) / 1.0E+06 ;

   HEAD(J)         ..  HD(J)    =E=    (H0(J) * 1.0E+06) - DD(J) ;

   TOTALHD(J)      ..  THD(J)   =E=    HD(J) / 1.0E+06 ;

   WELLDD(H)               ..          DDW(H)          =E=             SUM(I,GAMMA(I,H)*(QMT(I)-QM0(I)))   +
SUM(N,THETA(N,H)*(QAT(N)-QA0(N))) ;

   TOTWELDD(H)     ..  TDDW(H)  =E=    DDW(H) / 1.0E+06 ;

   WELLHD(H)       ..  HDW(H)   =E=    (HW0(H) * 1.0E+06) - DDW(H)  ;

   TOTWELHD(H)     ..  THDW(H)  =E=    HDW(H) / 1.0E+06  ;

   CONCINC(H)      ..  CI(H)    =E=    SUM(I,ZETA(I,H)*(QMT(I)-QM0(I))) + SUM(N,PHI(N,H)*(QAT(N)-QA0(N)))
;
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TOTCNINC(H)    ..  TCI(H)   =E=   CI(H) / 1.0E+04  ;

CLCONC(H)      ..  CC(H)    =E=   (CC0(H) * 1.0E+04) + CI(H)  ;

TOTCLCON(H)    ..  TCC(H)   =E=   CC(H) / 1.0E+04  ;

MINDD(J)       ..  S        =G=   DD(J)  ;

MODEL SR8810Q3 /ALL/  ;

OPTION ITERLIM = 2000  ;

SOLVE SR8810Q3 USING LP MINIMIZING S  ;

DISPLAY QAT.L, QWT.L, QMT.L, TDD.L, THD.L, TCI.L, TCC.L ;
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