

**Two New Methods for Optimal Design
of Subsurface Barrier
to Control Seawater Intrusion**

By

Mundzir Hasan Basri

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Submitted to the Faculty of Graduate Studies

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**TWO NEW METHODS FOR OPTIMAL DESIGN OF SUBSURFACE BARRIER TO CONTROL
SEAWATER INTRUSTION**

BY

MUNDZIR HASAN BASRI

**A Thesis/Practicum submitted to the Faculty of Graduate Studies of The University of
Manitoba in partial fulfillment of the requirement of the degree
of
DOCTOR OF PHILOSOPHY**

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ABSTRACT

This research has provided two new methods to control seawater intrusion using a subsurface barrier through development and application of the implicit and explicit simulation-optimization approaches. The subsurface barrier refers to a semipervious underground grout curtain that is emplaced down to an impermeable layer and constructed parallel to the coast. In the past, the subsurface barrier was perceived as being very costly. Investigators in Japan focused their work on empirical subsurface barrier projects and development of models based on a trial and error approach. They found that the subsurface barrier method is physically and economically feasible; therefore, it has become a viable solution to the problem of seawater intrusion in coastal aquifers. In this study the objective is to develop implicit and explicit simulation-optimization models for design of a subsurface barrier that controls seawater intrusion. No prior work has been done in which a model for optimal design of a barrier for controlling seawater intrusion is developed.

The objective of the seawater intrusion control problem is to minimize the total construction costs while requiring that salt concentrations be held below specified values at two control locations at the end of the design period. The construction costs are associated with three consecutive decision variables chosen in this work; width, hydraulic conductivity, and location of the barrier.

Two control locations, (1) at the bottom boundary just landward of the barrier, and (2) in the freshwater zone, are chosen because they indicate the effectiveness of the barrier in protecting aquifers from seawater intrusion.

An implicit simulation-optimization model is developed for the design of a subsurface barrier to control seawater intrusion. This approach combines a groundwater flow and solute transport simulation model with nonlinear optimization. The simulation model is used to provide a distribution of salt concentration in order to establish two nonlinear salt concentration functions relating to the width, hydraulic conductivity, and location of the barrier. The Artificial Neural Network (ANN) and regression models are used to obtain the two salt concentration functions. These concentration functions are then used as constraints in the optimization method. The implicit approach is applied to a hypothetical cross section of a coastal aquifer system under transient state conditions.

The more significant development has been simulating a groundwater flow and solute transport model within the genetic algorithm (GA) based optimization method. Employing the GA in association with such a simulation model eliminates the requirement for a regression model, while providing accurate results within the same range. This is a significant improvement, since the implicit approach requires work because the simulation model is a separate component from the optimization model. The use of the GA in the explicit approach allows the groundwater flow and solute transport simulation model to be fully integrated into the optimization method. As a result, the GA in the

explicit approach reduces the computational burden encountered in the implicit approach.

Both explicit and implicit simulation-optimization models are developed to solve the same problem, and the results are compared. The results indicate that these two models provide a unique solution that may reduce the high construction cost of a barrier. The conclusion drawn from a series of tests performed in support of this work is that the explicit simulation-optimization model using GA performs as well as, if not better than, the implicit simulation-optimization model employing the gradient-based technique. Three tests indicate that the explicit approach outperforms the implicit approach, while one test shows otherwise.

Development of two methodologies for seawater intrusion control through the implicit and explicit simulation-optimization models is a major achievement of the present study. It marks the first time (based on available literature) that such coupling models are used to design an optimal subsurface barrier. Therefore, the methods developed in the implicit and explicit simulation-optimization models are the main contribution which this study makes to the field of groundwater research. Some components of the two approaches are the practical contributions of this work. For example, in the implicit approach the ANN model is used in a nontraditional fashion to derive the analytical form of the salt concentration - decision variable relationships. Another example is the mesh generator developed to handle change in grid because of the nature of the problem encountered in the optimization of a subsurface barrier for seawater intrusion control.

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Chapter One

INTRODUCTION

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Chapter 1

INTRODUCTION

1.1 Statement of the problem

Water managers are often faced with seawater intrusion problems that threaten to invade groundwater in low-lying coastal aquifers. These coastal aquifers lie within some of the most intensively exploited areas of the world; approximately 60 percent of the world population (~3.6 billion) lives in such low-lying areas. This figure is likely to double over the next century, and therefore the threat from seawater intrusion into freshwater aquifers will most likely become a major problem in the future [Hydrocoast, 1995]. If current levels of industrial development and population growth are not controlled significantly in the near future, the amount of groundwater use will increase dramatically, to the point that the control of seawater intrusion becomes a major challenge to future water supply engineers and managers [Bear and Cheng, 1999].

The threat of seawater intrusion into low-lying coastal aquifers will be exacerbated in the event of sea level rise resulting from global climate change. As atmospheric temperature increases, the sea rises due to thermal expansion of the ocean and melting of ice-caps and glaciers. Globally, sea level has risen 10-25 cm over the past century [National Climatic Data Center, 2000 and U.S. EPA, 2001].

Seawater intrusion problems are even more complex when development expands not only to serve the increasing population, but also to improve the standard of living and to satisfy the advancement of industry. Once an aquifer is contaminated, remediation is difficult and costs related to the implementation of corrective measures are prohibitive. In most cases, the contaminated aquifer is abandoned, which results in the loss of a precious groundwater resource. It is therefore important to develop methods to prevent, or at least control, seawater intrusion. The objective of the present study is to develop two methods for the optimal design of a subsurface barrier that controls seawater intrusion.

1.2 The Importance of Subsurface Barriers

Various control methods that deal with seawater intrusion problems have been implemented. Banks and Richter [1953] propose five approaches to prevention or control of seawater intrusion: (1) rearrangement of pumping pattern, (2) artificial recharge, (3) hydraulic barrier ridge, (4) subsurface barrier, and (5) pumping trough. The first three embody the main research thrust of a number of groundwater modelers and investigators. In these approaches, the seawater “wedge” thrusting inland is hydraulically reversed. Banks and Richter [1953] report the application of these approaches to real-life problems, especially in the United States.

Other investigators, such as Bruington [1969] and Todd [1974], each elaborate on a specific control method. Bruington [1969] reports the application of a hydraulic barrier ridge in the West Coast Basin Barrier Project in Los

Angeles County. Todd [1974] reports the implementation of the first method in Long Island, New York, and in coastal aquifers in Israel. Although initial costs are very high, the subsurface barrier method often a potential permanent solution to the seawater intrusion problem in narrow coastal groundwater basins with relatively shallow aquifers [Banks and Richter, 1953]. Todd [1980] and Bruington [1969] also note that the subsurface barrier is a viable solution to the seawater intrusion problem.

In light of the aforementioned literature review, it may be said that the rearrangement of the pumping pattern, artificial recharge, and application of a hydraulic barrier ridge, have each been successful in mitigating seawater intrusion. However, these methods do not allow for full development and utilization of the available groundwater storage capacity. The present study deals with an approach that permits the ultimate development and utilization of the available groundwater storage capacity. This goal can only be attained via installation of a subsurface barrier [Sugio et al., 1987].

The superiority of the subsurface barrier over other strategies for the control of seawater intrusion is widely recognized. Historically, subsurface barriers were in use in Sardinia during the era of the Roman Empire [Hanson and Nilsson, 1986]. More recently, this method has been elaborated by Professor Kachi [Kawasaki et al., 1993]. Even though Kachi's plan for a subsurface barrier project in the 1940s was not accepted, the essence of the proposed project was attractive. In 1973, The Ministry of Agriculture, Fishery, and Forestry (MAFF) of Japan considered subsurface barriers to solve the shortage of water in Southern

Japan. The Kabashima project was the first such barrier constructed in Japan. The Kabashima barrier is designed to increase freshwater supply to satisfy the town of Nomozaki in Nagasaki Prefecture. Later, the subsurface barrier design was modified for control of seawater intrusion.

Sugio et al. [1987] focus their research on the development of numerical and physical modeling techniques for seawater intrusion control. Nagata and Kawasaki [1997] review construction methods implemented in the subsurface barrier projects, either for storing freshwater or for controlling seawater intrusion. Seven subsurface barriers with a depth range of 11 - 25 m, crest length of 60 - 1,835 m and storage capacity of $0.012 - 9.5 \times 10^6 \text{ m}^3$, have been constructed in Japan. Four other barriers with the depth range of 36 - 81 m, crest length of 1,088 - 2,489 m and storage capacity of $1.58 - 10.50 \times 10^6 \text{ m}^3$ are under construction. Many more are in the design stage.

Subsurface barriers are used not only for the stated purposes, but also for directing, trapping and treating groundwater contaminants in situ. Newman [1995] reports the use of barrier-material zeolites treated with the surfactant hexadecyltrimethylammonium (HDTMA). HDTMA effectively traps many types of organic and inorganic contaminants in soil, while allowing water to pass through the barrier. Burgess [1995] reports the use of montan wax for the horizontal barrier. In examining the pros and cons of various approaches to verifying the subsurface barrier, the Department of Energy (DOE) of the United States continues to pursue the development of technology to detect discontinuities in the barriers [Shannon, 1995]; however, this technology is not yet available.

Discontinuities in subsurface barriers are created by injecting slurries of polymers or montan wax and by inserting interlocking sheet piling. The identification of discontinuities is very important to confining hazardous contamination, such as radioactive waste. Moreover, the use of subsurface barriers is becoming an alternative solution to dealing with unresolved problems of radioactive waste storage in the Yucca Mountains in New Mexico as opposed to the use of a geologic repository, following serious debate on the reliability of the latter [North, 1997].

The most recent work, by Moridis et al. [1999], investigates two alternative designs of a viscous liquid barrier (VLB) for the subsurface isolation of contaminants. The method utilizes lance injection to emplace a surface-modified colloidal silica barrier. The first design is based on maximizing uniformity and minimizing permeability by determining the optimal lance spacing, the injection spacing, injection volume and rate, and the gel time. The second design estimates similar design parameters based on standard empirical practices, more specifically the standard practice of chemical grouting injection. Results of the Moridis investigations indicate that a design based on the optimization approach is significantly better than the standard engineering design.

With respect to other methods, the subsurface barrier method is more competitive in terms of operation-and-maintenance costs. For example, artificial recharge must be operated and maintained for a very long period of time. Such operation-and-maintenance systems require a continuous input of energy as well as periodic maintenance and monitoring. This requirement incurs high costs. The

subsurface barrier method, on the other hand, has no such requirements with respect to its operation and maintenance.

Methods for constructing a barrier to increase groundwater storage capacity, and to control seawater intrusion, have been detailed by Nagata and Kawasaki [1997]. Their report notes that many subsurface barriers have been built in Southern Japan, and many more are planned for the future. The success of subsurface barrier projects in Japan suggests that the subsurface barrier method has advantages over alternative methods in terms of both increasing groundwater storage and controlling seawater intrusion. Furthermore, work done by Sugio et al. [1987] shows that subsurface barriers are a physically and economically feasible solution for increasing groundwater storage capacity and controlling seawater intrusion. In situations where the shape, size, and vertical depth, as well as the geologic structure of a groundwater basin, favor the use of a subsurface barrier, this method can be a permanent solution, as indicated by Banks and Richter [1953] and becomes a viable solution to seawater intrusion problems.

1.3 Limitations of Subsurface Barriers

Theoretically, the subsurface barrier has advantages over other constructs. Practically, however, it is not popular, for several reasons. The main drawback is that a subsurface barrier is considered to be very expensive in terms of construction cost [e.g. Rogoshewski et al., 1983; Hanson and Nilsson, 1986; EPA, 1987; and Todd, 1980]. The U.S. EPA indicates that subsurface barriers present procedural difficulties of a political, social, and economic nature. In addition,

Todd [1980] indicates that resistance to earthquakes and to chemical erosion is major issues confronting implementation of the subsurface barrier method. Furthermore, Rogoshewski et al. [1983] point out other drawbacks, such as the relatively advanced technology involved in constructing a barrier, as well as the limited availability of specialty firms capable of constructing such barriers.

Although some research suggests the limitations of subsurface barriers in controlling seawater intrusion, the disadvantages reported are not evident in the Japanese projects. Initially, a mathematical model and physical laboratory test developed by Sugio et al. [1987] indicated the superiority of the subsurface barrier. Later, the use of new construction methods, construction materials, and sophisticated machinery reported by Nagata and Kawasaki [1997] confirmed those findings. Osuga [1996] notes that these new features have made subsurface barriers viable, and at the same time have overcome the recognized drawbacks to a notable degree. However, high construction costs continue to remain an issue. The purpose of the present study is to develop two methods that can reduce the construction costs addressed by Osuga [1996].

1.4 Objective of the Study

The effectiveness of a subsurface barrier for controlling seawater intrusion has been studied by resort to field studies, laboratory experiments, and numerical simulation models. Although field studies provide valuable information, they are very time-consuming and prohibitively expensive to perform. Whereas laboratory experiments are convenient and less expensive to carry out, they generally fail to

yield information adequate to a field-scale problem and are not practical when dealing with the replication of physical models. On the other hand, numerical simulation models provide a relatively inexpensive means of obtaining relevant information about the configuration and location of the freshwater-saltwater interface. In addition, these simulation models can also be used to test the validity of results from field and laboratory experiments.

It is important to note that these simulation models can act as valuable predictive tools and so aid in the design of cost-effective control measures with respect to seawater intrusion into coastal aquifers. Cost-effective design, specifically as to the minimum effective dimension of a subsurface barrier, is paramount in controlling the cost of constructing a barrier [Osuga, 1996]. To date, no single simulation model can provide the minimum dimensions of an effective barrier. The research question being addressed in this thesis is how to design an optimal subsurface barrier for seawater intrusion control using the available simulation model in order to minimize the construction cost.

The objective of this thesis is to develop two methods for the optimal design of a subsurface barrier that controls seawater intrusion. The first combines a standard groundwater-flow/solute-transport simulation model—SUTRA [Voss, 1984]-- with a nonlinear optimization solver—MINOS [Murtagh and Saunders, 1995]. The second is a fully integrated simulation-optimization model in which the groundwater flow and solute transport simulation model runs within an optimization technique. A global optimization technique, Genetic Algorithm (hereafter referred to as GA), is used as an optimization tool. To test

the two methods developed in this research, the data from work carried out by Voss [1984] are used.

1.5 Implicit and Explicit Simulation-Optimization Models

The technology of constructing subsurface barriers has advanced considerably in recent years. However, due primarily to large initial construction costs, subsurface barriers are not as common as other seawater intrusion control methods. There is a need to design a subsurface barrier with minimum effective dimensions, which reduces the volume of construction materials required, consequently reducing overall construction cost [Osuga, 1996].

To date, no work has been done on reduction of costs in installing a subsurface barrier. In theory, the physical and mathematical models developed by Sugio et al. [1987] can be used to design a thinner barrier. However, determining dimensions for a thinner barrier, using these models, is time-consuming, laborious work because a trial-and-error approach has to be employed. Therefore, some type of optimization scheme should be incorporated in conjunction with a simulation model to determine an optimal design. The main difficulty in dealing with a linked simulation-model/optimization-method is that the dimension of the barrier is not part of the governing equations for the groundwater-flow/solute-transport simulation model. For this reason, the commonly adopted techniques in water resources management, as discussed by Gorelick [1983], may not be employed; therefore the implicit and explicit simulation-optimization models are proposed.

Through the use of the proposed simulation-optimization models, an optimal design of a subsurface barrier for control of seawater intrusion can be developed. By determining the optimal design, the construction cost may be substantially reduced. In this thesis, the reduced construction cost is generated by minimizing the dimension of the barrier, maximizing the value of the hydraulic conductivity of the material used for constructing the barrier, and minimizing the distance of the barrier from the saltwater aquifer. It should be noted, that under certain conditions such as specific geologic formations, geohydrologic conditions, or when important facilities and utilities are in proximity to the shore, the location of the barrier may be predetermined by externalities. The proposed simulation-optimization models promise to cope with the case of fixed location without any difficulty. The implicit simulation-optimization model is detailed in Chapters 3, 4 and 5. The thesis then proceeds with the explicit model.

1.6 Methods

Two strategies for dealing with seawater intrusion control problems are developed in this thesis. The first combines a groundwater simulation model with a general gradient-based optimization technique. This linked simulation-optimization model represents the “implicit” approach. The second couples the same groundwater simulation model with a GA-based optimization technique. This coupled simulation-GA represents the “explicit” approach.

1.6.1 Implicit Simulation-Optimization Approach

The implicit approach is developed to solve an optimization problem using decision variables taken from the simulation model. In the optimization formulation, the decision variables are implicitly expressed. The approach is straightforward in terms of the procedural steps involved. The finite element simulation model SUTRA (Voss, 1984) serves as an independent module to supply information required in establishing equations for the optimization model.

The simulation-optimization modeling procedure begins with the use of SUTRA to determine the initial conditions. The same initial conditions are used for the explicit approach. SUTRA is equipped with a mesh generator in both approaches. A combination of decision variables is generated systematically within SUTRA, and each combination requires a grid supplied by this mesh generator. In the implicit approach, any combination of decision variables is generated for the range of possible solutions.

The simulation model is executed based on combinations of decision variables within the range to obtain specific salt concentrations at each of two control locations. Each concentration is associated with the decision variables, using a regression model. Nonlinear functional relationships are obtained through the use of an artificial neural network model. Given the functions from the neural network, a regression analysis is executed to obtain the equations that relate salt concentrations to decision variables. These equations become part of a set of constraints in the nonlinear programming formulation.

Costs of constructing a barrier are correlated with each of the decision variables, and the summation of associated costs is treated as an objective function for the programming formulation. By including the non-negativity and the lower/upper bounds of the decision variables, the formulation of seawater intrusion control problems is accomplished and the optimization model is solved to obtain an optimal design. Such an approach has been used to identify locally optimal solutions for the clean-up of contaminated aquifers [e.g. Gorelick et al., 1984; Ahlfeld et al., 1986; and Ahlfeld, 1987]. Linked simulation-optimization models have also been used in studies involving the economics of groundwater management [Bredehoeft and Young, 1970; and Daubert and Young, 1982]. In groundwater quality management, Wagner and Gorelick [1987] apply the same approach to identify optimal remediation strategies.

1.6.2 Explicit Simulation-Optimization Approach

The explicit approach is a fully integrated simulation-optimization model, in which the groundwater-flow/solute-transport simulation model runs within optimization technique. The decision variables are explicitly expressed in the formulation of the optimization technique. A simulation model is used to obtain the aquifer-system response required in the optimization model. The simulation model used is essentially the same as that employed in the implicit approach. Unlike the implicit approach, which employs a gradient-based optimization technique, the explicit approach uses a GA-based optimization technique. The

SUTRA model is a subroutine that is repeatedly called by the GA optimization procedure.

The implementation of the explicit approach begins with the generation of an initial population of decision variables. When binary random numbers representing these variables are generated by GA, the simulation model is executed to quantify salt concentrations at two different control locations. Once the GA has these concentrations, the three most commonly adopted operators in GA; selection, crossover, and mutation, are applied.

The fitness of population members is evaluated, based on the objective function and a set of associated constraints inherent to the formulation. In this study, the simulated concentration values which are treated as constraints are added to the objective-function values in order to disqualify the solutions that violate one or more constraints. Selection is based on the tournament selection method. The next step is to apply crossover. Mutation is applied after crossover is performed. To ensure that the most fitted individual from the previous generation is delegated to the next generation, an elitist strategy is implemented. The process of evaluation, selection, crossover, and mutation is repeated for a user-specified number of generations.

An approach similar to the explicit simulation-optimization model just discussed has been applied in groundwater quality management and aquifer management [e.g. Kinney and Lin, 1994; Ritzel et al., 1994; Cieniawski et al., 1995; Cedeno and Vemuri, 1996].

1.7 Outline of the Thesis

This thesis consists of seven chapters:

- **The first is the Introduction.**
- **Chapter 2 presents the methods which are commonly adopted for control of seawater intrusion. Subsurface barriers, including the history of their usage to control seawater intrusion in Japan, are discussed. The latest developments to methods using subsurface barriers are reviewed, and presented in the Appendix. The Appendix should provide an adequate background to understand the motivation for this research, and could be skipped or skimmed by readers who are already familiar with this background material.**
- **Chapter 3 describes the method and main components of the proposed implicit simulation-optimization model.**
- **Chapter 4 presents the theoretical background of the proposed implicit simulation-optimization model.**
- **Chapter 5 describes the application of the implicit simulation-optimization model to a hypothetical confined coastal aquifer.**
- **Chapter 6 provides a review of GA applied to the fields of water resources, including groundwater management. In this chapter the formulation and application of the explicit simulation-optimization model is described.**
- **Chapter 7 discusses the results obtained from the implicit and explicit simulation-optimization models.**
- **Chapter 8 summarizes the contributions of this thesis and contains suggestions for further research.**

Chapter Two

CONTROL METHODS

2.1 Altering existing pumping schedules

2.2 Artificial Recharge

2.3 Hydraulic Ridge

2.4 Pumping Trough

2.5 Subsurface barrier

Chapter 2

CONTROL METHODS

Many methods have been devised for controlling seawater intrusion in coastal aquifer systems. Generally, these can be summarized into three categories: hydrodynamic control, extraction wells, and physical containment. Hydrodynamic control refers to any effort to project the hydraulic gradient of the system seaward. By maintaining the seaward gradient, the freshwater-saltwater interface is repositioned to a desired location or even rendered into an equilibrium condition. The strategies invoked involve (1) reduced pumping, (2) relocation of wells, (3) aquifer recharge, (4) creation of a hydrodynamic barrier, and (5) creation of a pumping trough.

Strategies (1) and (2) diminish the magnitude of the cone of depression in order to terminate the flow toward the production wells. The other three use the simple hydraulic concept of installing a hydraulic barrier to control seawater intrusion.

There are two approaches to utilization of extraction wells. The first is simply to extract seawater before it reaches the production wells. Accordingly, the extraction wells are installed between production wells and the underground

saltwater "front". This method is applicable to small groundwater basins where the extent of saltwater contact is very limited and easily monitored.

The second method combines extraction wells with injection wells, which achieve more effective protection of production wells from seawater intrusion [Todd, 1980]. The extraction wells withdraw salty water and injection wells recharge freshwater with high pressure. This method protects production wells from the invasion of seawater and renders seawater wedge into new equilibrium condition.

The third strategy available for seawater intrusion control involves physical containment. An impermeable or semi-impermeable subsurface barrier is constructed. The construction of the barrier provides two benefits: (i) the invasion of seawater into freshwater inland may be controlled, and (ii) the freshwater table landward of the barrier may rise, increasing the groundwater storage capacity (Sugio et al., 1987).

Theoretically, these methods are commonly adopted to control seawater intrusion. Practically, five methods are described in many groundwater textbooks [e.g. Todd, 1980]. These methods are surveyed in the following sections.

2.1 Altering existing pumping schedules

The method of altering the existing pumping schedule is also often called modification of the pumping pattern, and is widely used to limit seawater intrusion. The method involves dispersing the location of production wells so that

spreading the wells throughout the groundwater basin may reduce concentrated drawdown in localized pumping zones [Bear, 1979].

This method can be illustrated as the effect of the concentration of pumping wells on the drawdown of the water table. High concentration in pumping tends to create groundwater overdraft. Lowering of the water table due to dispersed individual pumping is much less than that associated with group pumping (composite drawdown).

The main objective of this method is to establish a groundwater level that creates a seaward hydraulic gradient. This objective can be achieved by relocating the wells, followed by altering the pumping schedule. Although the methods of relocating and scheduling have been widely implemented, sometimes these methods are not sufficient to reestablish the water table as desired. The additional action of reducing pumping rates is then required. If the reduction of the amount of groundwater extracted becomes more important than the relocation and scheduling changes, the method is no longer called "altering the existing pumping schedule", but "reduction of groundwater extraction".

Essential factors in the implementation of the "reduction of groundwater extraction" method are determination of allowable volume extracted, the schedule of pumping over the entire basin, and the dispersal-pattern of production wells. Hence, this method requires special tools to make the system work properly. A groundwater management model can be used to handle these factors. This management model is not a simple system because it involves the recharge system

as well. It becomes more complicated when the source of water is groundwater combined with surface water and/or imported water.

The groundwater management model, without considering the water surface distribution system as an integral part, simply simulates the rate of withdrawal and the pumping schedule. Then the response can be monitored in observation wells. The levels in observation wells should not be lower than the acceptable level, which indicates the safe yield of the aquifer. When integrating the surface and groundwater systems, it is necessary to deal with the capacity of both systems. An integrated surface groundwater system becomes a dual-capacity situation, which depends upon the availability of surface water. The quantity and quality of the water should be considered in this integrated system. Development of a model of a coupled surface groundwater system is more difficult [Todd, 1980].

The cost of production in the integrated system is an important factor to be considered. In most cases, pricing of surface and groundwater supplies is quite different. Based on quality, accessibility, and exploitation cost considerations, the use of surface water is not more attractive than the use of groundwater. The groundwater source is actually more attractive than the surface water source. It is important to note that the choice does exist, but is very subjective. People who have been served by fairly reliable piped surface-sourced water often consider exploitation of groundwater as the secondary option.

Another important point is that, in recent years, economic growth has become a major concern, especially in developing countries. As a consequence,

the demand for water increases dramatically. Groundwater users tend to discourage the reduction of pumping rates, the scheduling of pumping times or any type of control. Therefore, the practicality of this strategy is questionable.

2.2 Artificial Recharge

Todd [1980] defines artificial recharge as a method which augments the natural movement of surface water into underground formations by several techniques. The objectives of an artificial recharge project are to increase water supply, to improve groundwater quality or to augment flow. One of the primary purposes for using artificial recharge basins in coastal areas is to prevent seawater intrusion. The construction method depends on several factors, such as topography, geology, soil conditions, and the availability of water surrounding the area of interest. The artificial recharge methods include water spreading, recharge wells and induced recharge wells [Todd, 1980].

In the water spreading method, groundwater is recharged by infiltration into unsaturated media before it percolates to the water table.. Structures such as stream channels, ditches, and furrows, as well as flooding and irrigation are often used in the water spreading method of artificial recharge. The surface spreading method works effectively if there is no impervious layer between the water table to be raised and the bottom layer of flooded areas. Furthermore, it suits only unconfined aquifers. In the case of confined aquifers, an impervious layer is too difficult to percolate through without additional effort. Construction of recharge wells is effective for recharging a confined aquifer.

The choice of the surface spreading technique is dependent upon several factors. The first factor is cost and availability of land. The cost of land is an important factor, particularly in urban areas. The availability of land for flooding is a necessary condition. Another factor to be considered is the type of soil [Todd, 1980]. Gravel, or gravel and sand are strongly recommended. One basic concern is the infiltration rate, which may over time become the bottleneck in the application of this technique. A third factor to be considered is evaporation. The loss of water by evaporation can be a major constraint, considering the ratio of depth to surface area.

After assessing these three main factors, additional consideration should be given to: benefit from recreation, environmental impact and, in some cases, the distance between the recharge areas and the areas of groundwater exploitation.

The effectiveness of the surface spreading technique is questionable when clogging problems are encountered [Bear, 1979]. The surface water spreading technique relies on the rate of infiltration to transfer water from the surface into porous formations. The rate of infiltration is high only at the beginning of the operation and decreases considerably after reaching the peak. The decrease is caused mainly by the filling of the soil pores by water. The saturated soil causes soil particle swelling, at the same time as soil dispersion occurs; hence surface tension becomes a more prominent factor. Soil responses due to a saturated condition may reduce the pore space available for, and the rate of, water infiltration.

Furthermore, Bear [1979] points out a number of causes of clogging when soil is saturated. They are as follows: (i) the retention of suspended solids; (ii) growth of algae and bacteria; (iii) the release of entrained or dissolved gases from water; and (iv) chemical reactions between dissolved solids and the soil particles and/or the native water present in the void space. As a result of any combination of these factors, the spreading operation method works effectively only at the outset and the recharge rate almost invariably declines with time [Freeze and Cherry, 1979].

Another type of artificial recharge involves recharge wells. A well is used to transfer water from the surface into the aquifer. The well used may be an ordinary pumping well or one specially designed for this purpose. An attractive mechanism is a dual-purpose well that has two functions-- to discharge and recharge water from/to the aquifer. Use of a dual-purpose well is economically preferable to construction of a special recharge well. The purpose of recharge wells is to overcome the high cost of the water spreading technique in areas where suitable land is scarce and/or expensive.

The design of a recharge well makes it appear as if it reverses the function of a pumping well but this is not the case. Correct design of a recharge well involves complicated roles, and may successfully address several problems. It is acknowledged that pumping water from an aquifer withdraws not only freshwater, but also fine material, which can go through the pores of water-bearing formations on the approaches to the well. This may cause clogging of the well screen. Conversely, recharge water from the surface quite often carries fine

material such as silt. Again, clogging of the screen and the aquifer itself may occur [Bear, 1979].

In addition to clogging problems, several other difficulties are related to the recharge well technique. For instance, a large amount of dissolved air is carried together with recharge water. The existence of dissolved air in aquifers may lessen their hydraulic conductivity. Research on water quality indicates that various bacteria can also be found in recharge water. Under certain circumstances, bacteria can grow quickly and eventually reduce the filtering area of the well screen. Recharge water contains chemical constituents that induce flocculation. This process is described as a reaction between high sodium-ion content and colloidal soil particles [Bear, 1979]. Despite the disadvantages, however this method remains as the primary option in combatting seawater intrusion.

The third type of artificial recharge method is induced recharge. This is an indirect way of recharging an aquifer. Lakes, ponds, or rivers supply the water. By pumping groundwater surrounding the lakes, ponds, or rivers, the water table in the vicinity of the source is depressed. The water table must be lower than the water level of the lake, pond, or river. In this case, the water percolates from the lake, pond, or river to the areas where the water table needs to be raised. Therefore, these wells induce aquifer recharge.

The effectiveness of this method in terms of the amount of water to be recharged depends upon several factors. The most important factor is the hydraulic conductivity of the aquifer and areas adjacent to the lake, pond, or

river. Higher permeability allows water to enter the aquifers at higher rate. The second factor is the pumping rate that affects the hydraulic gradient. Flow rate from a lake to an aquifer is a function of the hydraulic gradient. Other factors such as type of soil, distance from the stream surface and natural groundwater movement also affect the amount of water transferred to an aquifer. By analyzing the factors above, one can ascertain that this method is comparable to the water spreading method.

2.3 Hydraulic Ridge

The purpose of the hydraulic ridge method is to recharge the aquifer by injection. It requires a line of recharge wells, which are usually located landward from the toe of the interface or seawater wedge. The wells must be located far enough from the interface toe to provide enough space for seaward flow. Thus, the pressure of recharged water can push the interface seaward. By injecting freshwater with pressure, a pressure ridge can be maintained [Todd, 1980].

The advantages and disadvantages of this method are quite similar to those of the artificial recharge method. In addition to the information discussed in the two previous sections, it is important to note that the freshwater to be recharged must be of better quality than the water used in the artificial recharge methods.

2.4 Pumping Trough

In contrast to the injection wells strategy, the pumping trough or extraction barrier approach requires a line of pumping wells which control how much water will be withdrawn along the freshwater-saltwater interface. These pumping wells should be located between the interface toe and the coastline. The exact location should be based on the shape of the interface. Pumping wells are used to withdraw intruding saline water and to drain it to the sea. Pumping wells work if the saltwater hydraulic gradient is not large enough to displace the freshwater/seawater interface. In addition to withdrawing seawater through trough wells, the groundwater level along the line of trough wells will be lowered, which will cause seaward infiltration of freshwater. Eventually a new equilibrium may be achieved. Conversely, if the water table in the freshwater zones is depleted, saltwater infiltrates landward [Todd, 1980].

A major disadvantage to this method is that the amount of freshwater that can be withdrawn must be reduced. Without reducing the pumping rate, it is impossible for the system to achieve equilibrium. Reducing the demand for water is not a workable solution, especially to the users. Other difficulties in implementing this procedure relate to monitoring of the water table and determining the amount of water withdrawn. Monitoring of the water table over the entire groundwater basin is needed in order to predict the location of the interface toe. Water table data can be used to predict the time when a new equilibrium will be achieved. To reveal the shape of the saline wedge and the location of the interface toe, there must be additional monitoring of groundwater

quality (for example, monitoring the concentration of Chlor or salinity). The concentration of Chlor is more accurate in revealing the position of the saline wedge, but it is an expensive monitoring system [Todd, 1980].

After collecting information on the shape and location of the interface, the next task is to determine the amount of saltwater to be pumped. Withdrawing less water than is actually required to control seawater wedged will lead to the following problems: (a) the freshwater region may be intruded by saline water; and (b) a positive seaward hydraulic gradient may not be formed. Conversely, pumping more water than the required amount causes saltwater and freshwater to be withdrawn from the aquifer. Although the amount of freshwater pumped is not large, it should be avoided in areas where there is a lack of freshwater sources. Ideally, the amount of saltwater pumped should be slightly higher than the rate at which seawater is intruding.

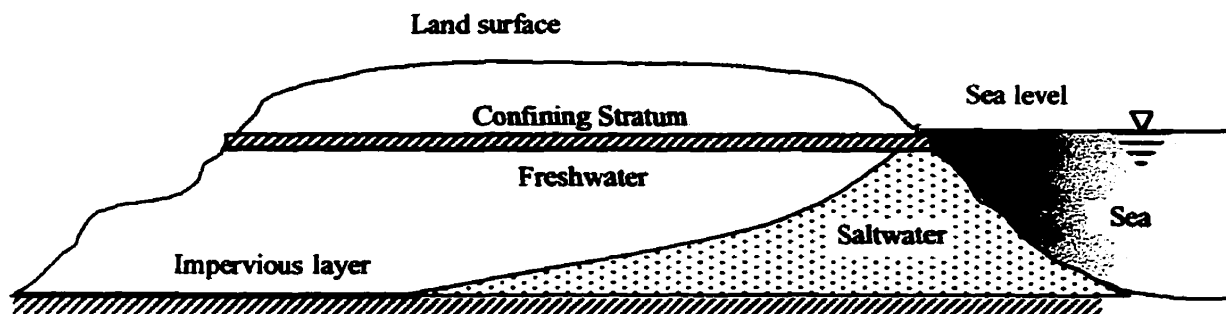
2.5 Subsurface barrier

A subsurface barrier can be defined as an underground semi-impervious or impervious structure constructed in a coastal aquifer [Aiba, 1983]. It is used to impede the infiltration of seawater inland, and at the same time to increase the groundwater storage capacity. In the past, constructing a subsurface barrier to control seawater intrusion was not preferable because the construction costs of a physical barrier were very high [Rogoshewski et al., 1983; Hanson and Nilsson, 1986; U.S. EPA, 1987; and Todd, 1980], and the technology for such a substructure was not readily available [Rogoshewski et al., 1983]. In many

groundwater textbooks, the method of using a subsurface barrier is ranked last as a measure of controlling seawater intrusion [e.g. Todd, 1980].

This control measure is illustrated in Figure 2.1. A slight difference between the piezometric head and sea level may cause seawater intrusion (Figure 2.1a). An increase in water extraction from the aquifer lowers the water table. Hence, seawater contaminates freshwater. As illustrated in Figure 2.1b, the barrier can effectively stop the movement of seawater [Aiba, 1983]. The barrier shown in the figure is semi-impermeable so that it allows some saltwater to go through it. This semi-impermeable barrier can maintain a stable freshwater table, potentially at a higher level than without the barrier.

In another situation, in which there is a considerable difference in the elevation of the freshwater table relative to sea level, groundwater flow is seaward [Aiba, 1983]. Thus, a large amount of fresh groundwater cannot be intercepted. To avoid groundwater flow towards the sea, a barrier can be constructed at an appropriate location to intercept it and increase the aquifer freshwater capacity.



(a)

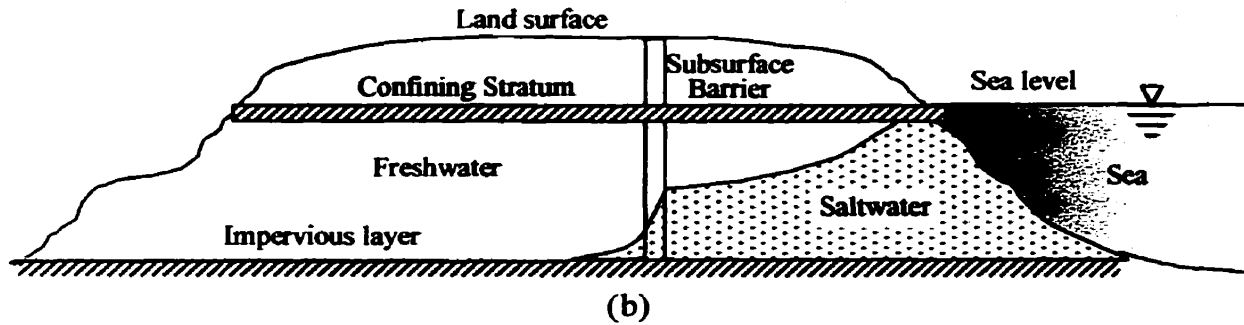


Figure 2.1: An Illustration of Subsurface Barrier [Aiba, 1983].
 (a). Seawater Intrusion Advancing Inland
 (b). Seawater Intrusion Impeded by a Subsurface Barrier

The subsurface barrier is located between the seawater and the production wells and constructed parallel to the coast. It works in the same fashion as a dam across a river, thus the name "underground dam" is given to it by engineers. In the same way as a dam, the barrier should rest on an impervious layer. The method of construction for such a substructure might be an excavated trench backfilled with bentonite clay, or a closely spaced line of wells through which impermeable grout is injected. It is likely that such a barrier could be effective only in relatively shallow formations. The effectiveness of the barrier must be monitored to determine the magnitude of seawater penetration.

A subsurface barrier may be designed to be either impermeable or semi-impermeable. Some investigators indicate that the impermeable barrier is more effective. The barrier may stop the encroachment of seawater completely, while functioning as a dam, collecting water behind it. These two benefits can be achieved simultaneously. However, Sugio et al. [1987] address the weakness of this impermeable system. As human activities that may affect the quality of the

freshwater cannot be entirely controlled, there is no guarantee that contamination does not occur upstream from the barrier. Should this be so, the accumulation of pollutants upstream of the barrier will create new problems for the production wells. For this reason, a semi-impermeable barrier should be constructed, and contaminated groundwater may bleed seaward through this barrier.

Five seawater control strategies have been addressed in this chapter. Their brief review should be useful in providing insights into seawater-intrusion countermeasures. Perhaps all seawater intrusion problems can be rectified through the use of modified pumping patterns and/or artificial recharges such as have been developed and applied for many years in numerous coastal aquifers. However, due to practical constraints, such control methods are not feasible at all sites. In these instances, the subsurface barrier may present a feasible alternate solution. In order that the subsurface barrier be economically viable, it is essential that the dimension of a subsurface barrier be of a minimum size [Osuga, 1996] to minimize construction cost.

Since the focus of this thesis is to develop implicit and explicit simulation-optimization models, the proposed implicit approach will be presented in Chapter 3. In order to have a balanced report between the subject of this thesis, i.e.--subsurface barriers--, and the simulation-optimization models proposed, more detailed technical aspects and the most recent applications of the subsurface barrier will be covered in the Appendix. Knowledge of the history and most recent applications of the subsurface barrier that are essential to the advancement of this method will be presented.

Chapter Three

IMPLICIT APPROACH

3.1 Groundwater Simulation Models

3.2 Groundwater Management Models

3.3 Proposed Implicit Simulation-Optimization Model

3.4 Components of the Proposed Model

3.5 Link Between Simulation and Optimization Models

Chapter 3

IMPLICIT APPROACH

3.1 Groundwater Simulation Models

During the past latter few decades of the 1900s, simulation models were applied in the field of hydrology with varying degrees of success. In the field of groundwater hydrology, numerical simulation models were applied to the management of groundwater resources. Groundwater simulation models are essential to addressing the issues of depletion and contamination in groundwater.

In the past, these problems were addressed with lumped-parameter models, in which the groundwater domain was represented by only one parameter [Buras, 1967; Burt, 1967; and Domenico et al., 1968]. Such models are sufficient when the major concern is related only to the temporal allocation of water. Where both temporal and spatial aspects are to be addressed, constant-parameter approaches have limited applications. In those situations, distributed-parameter approaches should be employed.

Distributed-parameter approaches require the division of the groundwater system under examination into subsystems in which each subsystem is represented by a constant parameter. The system, which consists of many subsystems, is then applied in the manner of numerical simulation models. These simulation models are based on distributed groundwater flow and solute transport processes approximated by finite difference schemes [e.g. Aguado and Remson,

1974; and Alley et al., 1976], or finite element schemes [e.g. Willis and Newman, 1977, and Elango and Rouve, 1980]. Groundwater simulation models discussed throughout this thesis refer to the finite element-based approximations.

3.2 Groundwater Management Models

If used in isolation, groundwater simulation models will not underpin the management of groundwater resources in an efficient manner. For example, problems involving groundwater management alternatives require repeated executions of selected simulation models to render different management scenarios. In other words, seeking an optimal management strategy requires a trial-and-error approach, which promises to be time-consuming and laborious. In addition, the results may not be optimal. The best possible explanation for this situation is the inability of these models to consider important physical and operational restrictions [Gorelick, 1983]. To accommodate these restrictions, linking the simulation model with a management model is the generally adopted procedure.

For a groundwater system with objectives, and constraints, imposed by water managers, combined simulation and management models may adequately predict the behavior of the system and provide the best solution to the problems. Examples of such problems include containing a plume of contaminated groundwater, obtaining a long-term planned water supply, or preventing seawater intrusion. Though substantial research has been published on the use of simulation management models for the first two problems, only a relatively small

number of studies has concentrated on the problems of controlling seawater intrusion. Specifically, no research has investigated the subsurface barrier for control of seawater intrusion using the simulation-optimization models. The author of this thesis is unaware of any research work relating to the development of a simulation-optimization model for optimal design of a subsurface barrier.

The simulation-optimization models cover a broad range of groundwater situations. In this thesis, the term 'simulation-optimization model' refers to the use of simulation models in conjunction with optimization techniques. This term falls under the second category of groundwater management models classified by Gorelick [1983]: (1) hydraulic management models, and (2) policy evaluation and allocation models. In the first category, groundwater management models are used to study management decisions that are primarily concerned with groundwater hydraulics. Policy evaluation and allocation models are developed to solve complex problems where hydraulic management is not the sole concern of the water planner. There are two techniques appropriate to hydraulic management models. The first approach is referred to as the "embedding method", which includes discretized finite difference or finite element approximation equations as part of the constraint set of a linear or nonlinear programming model.

The second technique refers to the "response matrix approach" which uses a group of the unit responses represented as a response matrix in the management model. Each unit response describes the relationship between system responses and management decision variables. Unit responses are developed, based on an

external groundwater simulation model. This simulation model does not necessarily involve numerical approximation equations such as discretized equations derived from finite difference or finite element techniques, but uses any function that can relate state variables of an aquifer system to management decisions.

Gorelick [1983], in his review divided groundwater policy evaluation and allocation models into three groups. The first group refers to hydraulic-economic response models. These models are an improved stage of the response-matrix approach, in which agricultural-economic and/or surface water allocation is included in the formulation. Hence, these models are valuable for addressing more complex problems than those for which the response matrix approach is appropriate.

The second group consists of linked simulation-optimization models. These models use the results of an external aquifer simulation model as input to a series of economic optimization models. In these approaches, the simulation models are separated from the optimization methods. For this reason, more complex problems related to social, political, and economic influences can be included in the formulation. The implicit simulation-optimization model proposed is grouped with this category.

The third group refers to hierarchical models. Large-scale optimization problems can be handled using these models, as they can decompose large and complex systems into a series of independent subsystems and optimize them individually. To optimize a complete model representing the overall problem, the

response matrix approach, again, is applied. Employing the decomposition optimization techniques and response matrix approach results in a multiple-level management model.

Gorelick's review suggests that the linked simulation-optimization models use the results from simulation models as input to the optimization models, while the other simulation-management models treat the discretized flow equations as part of the constraint set of a linear or nonlinear programming formulation. By including management decisions with simulations of groundwater behavior, a complete management model can be solved simultaneously. A simultaneous solution is possible if the decision variables of optimization formulation are explicitly expressed in the governing equations of simulation models. This explicit expression results in management decisions being included in the approximated equations. For example, the objective of a contaminated groundwater management scheme is to minimize the pumping rate, which is one of the factors governing the groundwater flow and contaminant plume.

In some cases, the decision variables are absent from the governing equations or are not directly described. The linked simulation-optimization models reviewed by Gorelick [1983] are examples of such cases. These models were originally used to evaluate the effect of institutional changes on groundwater systems. It should be noted that the institutional parameters such as tax or quota [Bredehoeft and Young, 1970] are not part of the groundwater flow system and should be included separately in an economic model. In these cases, a groundwater simulation model is run first, and then the results are used as input

to the economic optimization model. Outputs of the optimization model, such as the recommended number of production wells, are then compared with those from the simulation model. If a difference is found, the simulation model has to be rerun until an agreement is reached. This procedure is performed for each time interval and has to be repeated over the time horizon of interest.

Parameter estimation or inverse problem, using a distributed finite element scheme [Yeh and Yoon, 1981], is another example of a simulation model in which decision variables are absent from the governing equations. The objective of the inverse-problem approach is to identify the parameters of the groundwater system based on observed values collected in the spatial and temporal domains.

Usually the number of applicable historical observations is very limited and finite, while aquifer parameters vary with space; therefore, the dimension of the parameter over the spatial domain is infinite. The problem of interest in parameter identification is to reduce the number of parameters from the infinite to the finite dimension. This finite dimension has to provide a system that balances the system modeling error and the error associated with parameter uncertainty. In the case of a very fine grid system, the modeling error generated decreases, but parameter uncertainty increases. An increase in parameter uncertainty implies that the reliability of the estimation is reduced.

On the other hand, using a very coarse grid system and adjusting the number of observed data produces large modeling errors. However, the error associated with parameter uncertainty is reduced, thus increasing the reliability level. The best compromise to this problem is to obtain a grid system that

accommodates the trade off between the modeling error and the error associated with parameter uncertainty. It means that the grid system comprising the number of subdomains (or elements) is determined. In other words, the dimension of an element in the horizontal and vertical axes as a representation of a subdomain is optimized.

The problem of determining the dimension of the parameter space in a grid system is referred to as optimizing dimension in parameterization. Since the dimension of the parameter is not explicitly described in the governing equations, the available techniques such as embedding and response matrix techniques cannot be employed. To obtain an optimum dimension in parameterization, a least square criterion representing the system modeling error and a norm of the covariance matrix representing the system reliability are minimized. It should be noted that the work of Yeh and Yoon [1981] deals with a homogenous grid size for the entire domain, whereas the present study considers three different grid sizes.

The present study is similar to the work of Yeh and Yoon [1981] in terms of optimizing the parameter (subdomain) dimension and to the work of Bredehoeft and Young [1970] in terms of using the results of the simulation model as input to an optimization model.

3.3 Implicit Simulation-Optimization Model

The main objective of this thesis is to design an optimal subsurface barrier for seawater intrusion control. This can be achieved by minimizing the total cost of

construction of a subsurface barrier, which has been found to be one of the effective methods for seawater intrusion control. Three factors contributing to the construction costs are considered, and are designated as decision variables in this study. Details of these variables are provided:

1) *The dimension of the barrier.* The barrier is considered to be of one unit length and constructed parallel to the seashore. The design width of the barrier can be varied, depending upon the salt concentration level to be allowed to pass through and mix with the water in the freshwater zone. Since the length and height of the barrier are fixed, the width of the barrier is considered as a decision variable. This consideration is supported by one of the recommendations by MAFF to reduce the construction costs [Osuga, 1996].

2) *The material property.* The hydraulic conductivity of a barrier controls the volume of material used for the construction. The material discussed here is cement for the cement grouting method, as the most effective technique tested in limestone terrain, or montan wax for the cutoff wall method. These create a semi-impermeable barrier, while other materials such as sheet piling, emulsified asphalt and plastics create an impermeable barrier. If low hydraulic conductivity material is used, the volume of material required to construct a barrier is high. For example, as shown in Figure 3.1, the cement grouting method that uses a zigzag pattern for the layout of grouting holes needs a larger diameter of grouting holes, and larger numbers of rows to construct a barrier with lower hydraulic conductivity. In contrast, a smaller diameter and a smaller number of rows are required for a barrier with high hydraulic conductivity. In other words, the

diameter of grouting holes and number of rows in that zigzag system dictate the construction cost. Therefore, the optimum hydraulic conductivity has to be determined to obtain the minimum cost.

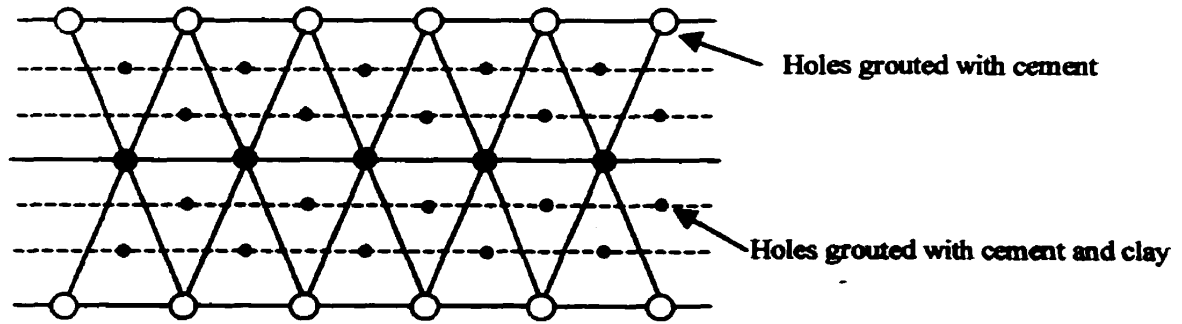


Figure 3.1: An Illustration Showing a Zigzag Pattern of Grouting Holes

3) *The location of the barrier.* The location of a barrier determines the area of the freshwater region landward of the barrier. It is preferable to construct a subsurface barrier near the seashore so that a large freshwater region can be protected against seawater.

The dimensions, material properties, and distance of the barrier from the sea are related to the costs through two pre-specified construction cost functions and a pre-specified damage cost function, respectively. The damage cost function associated with the location of the barrier is distinguished from the two construction cost functions associated with the width and hydraulic conductivity of the barrier because the location does not have a direct effect on the construction cost. Rather, the location of the barrier affects the area of freshwater region to be protected from seawater intrusion. The aquifer seaward of the barrier

is expected to be occupied by saltwater, and therefore potential potable water or freshwater may be lost if the barrier is installed farther landward than necessary.

Another argument can be built from a different perspective, which is economic gain. Economic gain can be expected from protecting the freshwater resource, and inability to protect the resource can be considered as damage loss. This loss is commonly considered in the field of water resources development, specifically in flood control projects. Flood losses or damage reductions have traditionally been computed by estimating the difference in expected annual damage with and without a particular flood measure. A loss function is generally used to estimate flood damages.

A similar loss function is developed for control of seawater intrusion. In this case, the damage function is associated with the location of the barrier. For example, a subsurface barrier constructed near the sea promises a significant economic gain because it protects a larger freshwater region. In other words, the loss of freshwater is relatively small because the infiltration of saltwater is confined to a very small area. Conversely, the economic gain is small when the barrier is located far away from the coastal face. In terms of loss, the damage is big because constructing the barrier further landward allows the seawater to intrude into a larger area of freshwater aquifer. The amount of freshwater sacrificed must be appraised in terms of the cost of either replacement or corrective measures. The costs are represented in terms of monetary units (hereafter referred to as MU).

Considering three factors mentioned above, problems of controlling seawater intrusion involve trade-off among them and can be solved by employing a multi-objective programming approach. The objective functions could be a minimization of the barrier width, maximization of its hydraulic conductivity, and minimization of its distance from the sea subject to salt concentration constraints. However, the use of multi-objective approach is restricted by the peculiarities of selecting an appropriate weight for each objective function in order to obtain good results. Theoretically, the proper weight can be obtained by involving decision makers, but allocation of weights, in practice, might not always be simple. For this reason, minimization of the overall cost of constructing a subsurface barrier is introduced. If the total cost of constructing a barrier is to be minimized, a single objective programming formulation should be considered.

In the simulation model, the dimension of the barrier differs from the rest of the aquifer domain in terms of the grid size and hydraulic conductivity. Sugio et al. [1987] also simulate a barrier using different grid sizes and values of hydraulic conductivity from the rest of the aquifer domain. The area with different hydraulic conductivity, that is, a barrier, has a finer grid than that applied to the rest of the aquifer domain in order to derive more accurate freshwater-saltwater interface within and near the barrier. In addition, the freshwater region landward of the barrier has a coarser grid than the seaward saltwater region. The value of hydraulic conductivity of the barrier is designed to be lower than that of the aquifer system. The number of elements within the

barrier in the horizontal direction that refers to the width of the barrier is to be minimized and its hydraulic conductivity is to be maximized.

The number of elements seaward of the barrier in the horizontal direction that refers to the location of the barrier from the seashore is to be minimized. It should be noted that this thesis deals with three different element sizes within the grid system, whereas the parameter dimension approach of Yeh and Yoon [1981] concerns homogeneous elements for the entire domain.

Time is a very important aspect of every remediation strategy. Typically, the time considered is at the end of a control period or reclamation period. Salt concentrations at the bottom boundary just landward of the barrier, in the freshwater region, are recorded and updated at the end of each simulation. The salt concentrations at every node in the freshwater region are calculated and aggregated. The average value of salt concentrations in the region indicates the area in the freshwater region contaminated by saltwater. This averaged concentration has to be maintained below a specified value. Decision-makers can specify the value, depending on the conditions at the site.

The concentration at the bottom boundary just landward of the barrier provides information on the position of the toe of the freshwater-saltwater interface relative to the intended location. The salt concentration there is to be held below a specified value. By incorporating constraints that limit the salt concentrations in the freshwater region and at the bottom boundary just landward of the barrier, the groundwater quality standard can be met. These two salt concentrations are related to the decision variables: the width, hydraulic

conductivity, and location of the barrier. Once these concentration relationships are established, they are treated as constraints in the nonlinear programming formulation.

3.4 Components of the Proposed Model

The proposed implicit simulation-optimization model consists of two main components. The first is a finite element groundwater flow and solute transport simulation model, and the second represents an optimization model. The groundwater flow and solute transport model is used to simulate the response of a confined coastal aquifer system, in this case the freshwater-saltwater interface, for given initial and boundary conditions. Since the coastal aquifer system is in contact with seawater, identification of the mixing zone between the freshwater and saltwater is very important. The effect of the existence of the subsurface barrier on the shape and location of the interface is observed during the simulations.

Movement within the mixing zone is governed by the density of water. The fact that the densities of freshwater and saltwater are different indicates that the mechanism of freshwater-saltwater interface movement is influenced by this difference. This difference becomes a driving force with respect to groundwater flow in coastal areas. Density as a linear function of solute concentration is commonly employed.

Another important aspect of the phenomena of groundwater flow and solute transport near the sea is that vertical head gradients play an important role

in establishing the freshwater-saltwater interface. Hence, a two-dimensional cross sectional analysis of the transport of salt in a density-dependent fluid flow model is required. In addressing this problem, a simulation model using a finite element scheme is required, and SUTRA [Voss, 1984] is used.

SUTRA (Saturated-Unsaturated TRANsport) simulates fluid movement and the transport of dissolved substances in a subsurface environment. This groundwater flow and solute transport simulation model is chosen for two major reasons. The first reason is that the SUTRA code is developed, based on the Galerkin finite element technique, one of the powerful techniques available for solution of partial differential equations in irregular domains. The second reason is that SUTRA is a standard model for groundwater flow and solute transport simulations because of its accuracy and flexibility [e.g. Voss, 1984, and Gorelick, 1984].

The second component in the implicit simulation-optimization model is a nonlinear gradient-based optimization strategy. As mentioned before, the objective of the optimization model is to minimize the cost involved in constructing a subsurface barrier. The construction cost is considered to replace an original objective of minimizing the width of the barrier suggested by Osuga [1996]. Taking the width of the barrier as the objective may cause difficulty in deciding proper weights. This is because the minimization of the width may be influenced by the material property of the barrier and its location relative to the seashore.

Very low hydraulic conductivity and/or a barrier location some distance from the seashore may result in a minimum width of the barrier. However, the costs involved are very high, because: (1) more material is required for a barrier with very low hydraulic conductivity, and (2) a smaller area of the freshwater aquifer is protected. It means that additional cost is incurred in transferring freshwater from other sources. Therefore, the decision variables for the optimization model are the width, hydraulic conductivity and location of the barrier.

These decision variables should be related to the salt concentrations in the freshwater region and at the bottom boundary just landward of the barrier, which represent the control points to enforce groundwater quality standards. These relationships are characterized by nonlinearity, and their approximation functions are treated as part of a constraint set in the formulation. Since the formulation of the optimization problem is characterized by nonlinear relationships between aquifer system responses and decision variables, as well as between the costs and each decision variable, a nonlinear optimization tool is needed. MINOS [Murtagh and Saunders, 1995] is chosen to solve nonlinear constrained problems investigated in this thesis.

MINOS is a FORTRAN-based optimization solver developed to solve large-scale optimization problems for linear objective functions with nonlinear constraints, nonlinear objective functions with linear constraints or nonlinear objective functions with nonlinear constraints. It is widely recognized as an efficient solver for solving nonlinearly constrained problems, and is numerically

reliable, robust, and transportable for future improvement. It is important to note that the solution technique used in MINOS can only guarantee to find optimal solutions that are local. Hence, any particular solution obtained in the present study may not represent a global optimum.

Theoretically, the physical and numerical models developed by Sugio et al. [1987] can be used to solve the problem that is addressed in the present study. However, a trial and error approach, which is subjective, must be employed to find the optimal solution. Furthermore, the trial-and-error approach typically may not account for all possible solutions. It is computationally difficult to explore all the possible solutions using this approach. Therefore, some type of optimization scheme must be incorporated in conjunction with the simulation model to reach the "optimal" solution.

3.5 Link Between Simulation and Optimization Models

A groundwater-flow and solute-transport model is used to simulate the salt concentrations for the entire domain. The optimization model is used to solve the formulation that leads to the optimum dimensions, material properties, and location of the barrier, which in turn indicates the minimum-cost construction. The link between the two models, simulation and optimization, is made through two salt concentration equations.

The first equation relates the concentration in the freshwater region to the hydraulic conductivity, width, and location of the barrier, and the second equation relates the concentration at the bottom boundary just landward of the barrier to

the width, hydraulic conductivity, and location of the barrier. The salt concentration in the freshwater region can be approximated by an average of the concentrations at all nodes in the region. The salt concentration at the bottom boundary just landward of the barrier indicates the appropriate location of the interface toe. The salt concentration at this interface toe can be calculated at the joint node between the landward side of the barrier and the impervious-bottom boundary. This concentration is essential, to ensure that it is below the acceptable level.

These salt concentrations, which represent the response of the aquifer system as a function of the width, hydraulic conductivity and location of the barrier, are obtained from the simulation model. This simulation model is executed in an exhaustive way so that salt concentrations at two control locations corresponding to all possible ranges of decision variables are obtained.

A regression analysis is carried out to obtain the salt concentration functions within the freshwater region and at the bottom boundary just landward of the barrier. These two concentration functions help to link the simulation model with the optimization technique. Numerous functions that relate salt concentration with the decision variables are possible, and the nature of the relationships is nonlinear. Gorelick [1983] suggests that nonlinearities arise in seawater intrusion control problems. Nonlinear functions may take many forms, and relationships between their operational terms may be both multiplicative and additive. In order to narrow down this possibility, the function approximation capability of an artificial neural network (ANN) model is used to obtain terms and

coefficients of the salt concentration functions. This approach is implemented to identify the analytical form of functions to be used in the nonlinear regression analysis. The internal mathematical relationship between neurons of the artificial neural network is used as an approximation of the salt concentration functions. This function is used as a predefined equation, and the regression model is run to obtain the coefficients.

The relationship between the cost of barrier construction and the decision variables has to be established, and can serve as a performance measure in a complete optimization model. Two construction cost functions and one damage cost function are established. The two construction cost functions are associated with two decision variables: (1) the width of the barrier, and (2) hydraulic conductivity of the barrier. Thus, the associated construction costs per unit width of the barrier and per unit hydraulic conductivity of the barrier are generated. The damage cost function is a representation of the size of the freshwater aquifer obtained as a result of locating the barrier at a calculated distance from the seashore.

Construction cost and damage cost functions have not been explored in depth because the present study focuses on the development of models to obtain an optimal design for a subsurface barrier. In practice, associated costs will vary considerably from one application to another. It is important to note that any agency applying these models would have to substitute the cost figures that are appropriate for the given location. Furthermore, the aquifer system considered is a hypothetical model and thus the costs inserted to develop these functions are not

derived from any specific project or location. For the sake of simplicity, the term ‘cost function’ is used interchangeably for both construction cost and damage cost for the rest of this thesis.

For example, the construction cost for a 0.02 m wide subsurface barrier is 100 MU and for a 0.1 m width is 1000 MU. The 0.02 – 0.1 m indicates the range of possible widths of the barrier to be built in this hypothetical aquifer. This range also means that it is not possible to construct a barrier of less than 0.02 m width and it is economically not feasible to go wider than 0.1 m. Assuming the construction cost is not linearly proportional to the width of the barrier, a quadratic function shown in Figure 3.2 is considered to describe the relationship.

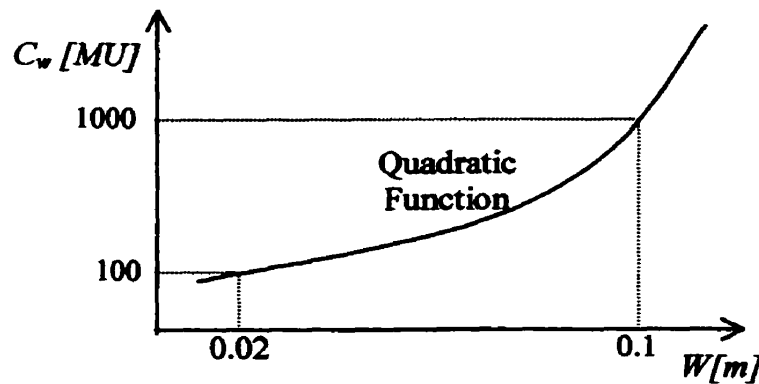


Figure 3.2: A Quadratic Function Describing the Cost Associated with the Width of the Barrier.

Similarly, the construction cost function associated with the hydraulic conductivity is determined based on a possible range of the barrier characteristics. For example, a barrier with a value of hydraulic conductivity of 0.01 m/s which is rather porous, costs 100 MU, while the cost for constructing a barrier with a value

of hydraulic conductivity of 6.5×10^{-4} m/s is 1000 MU. From this example, it appears that the function is not quadratic, but exponential. An exponential function in Figure 3.3 is used to establish the relationship between the construction cost and hydraulic conductivity of the barrier.

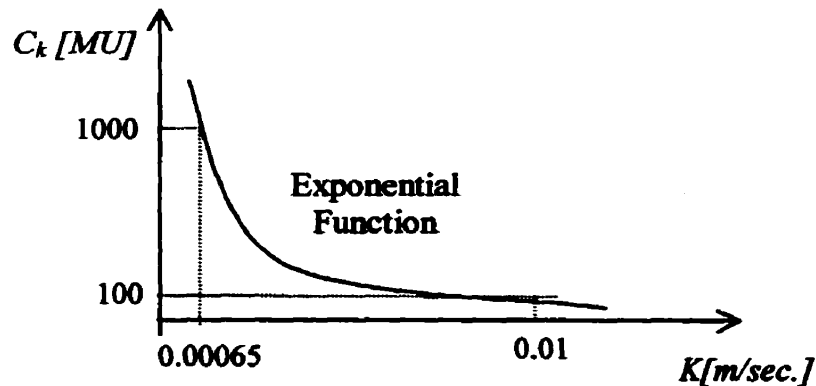


Figure 3.3: An Exponential Function Describing the Cost Associated with the Hydraulic Conductivity of the Barrier.

For the cost function associated with the location of the barrier, the following example is given. A subsurface barrier constructed 0.4 m from the seaward boundary costs 100 MU to supply the water deficit for contaminated area by saltwater and 1.0 m landward of the coastal face costs 1000 MU to supply the deficit for a larger contaminated area. If such costs are nonlinearly proportional to the location of the barrier, the quadratic function in Figure 3.4 may be appropriate to express the relationship.

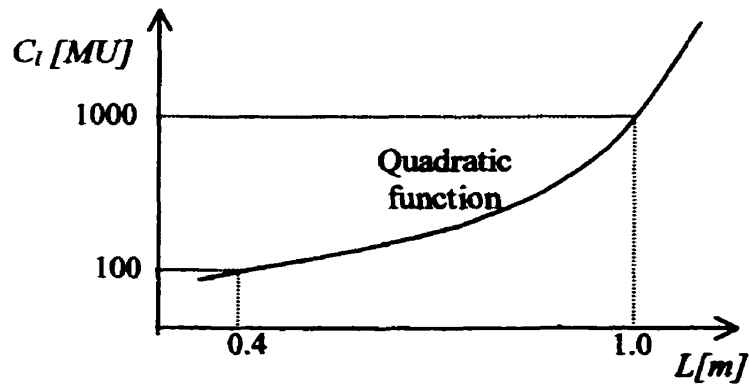


Figure 3.4: A Quadratic Function Describing the Cost Associated with the Location of the Barrier.

Although the three functions presented are crude representations of the mathematical relationships, they adequately illustrate the estimated relative construction costs related to each decision variable. By considering the cost functions, the nonlinear programming formulation becomes a single-objective problem. The objective of this seawater intrusion control problem is to minimize the prescribed cost functions associated with the width, hydraulic conductivity, and location of the barrier. The two nonlinear concentration equations discussed in the earlier section are treated as constraints. Therefore, the formulation of the optimization model is complete and an optimal design of a subsurface barrier can be obtained. The overall procedure is described by the flowchart shown in Figure 3.5.

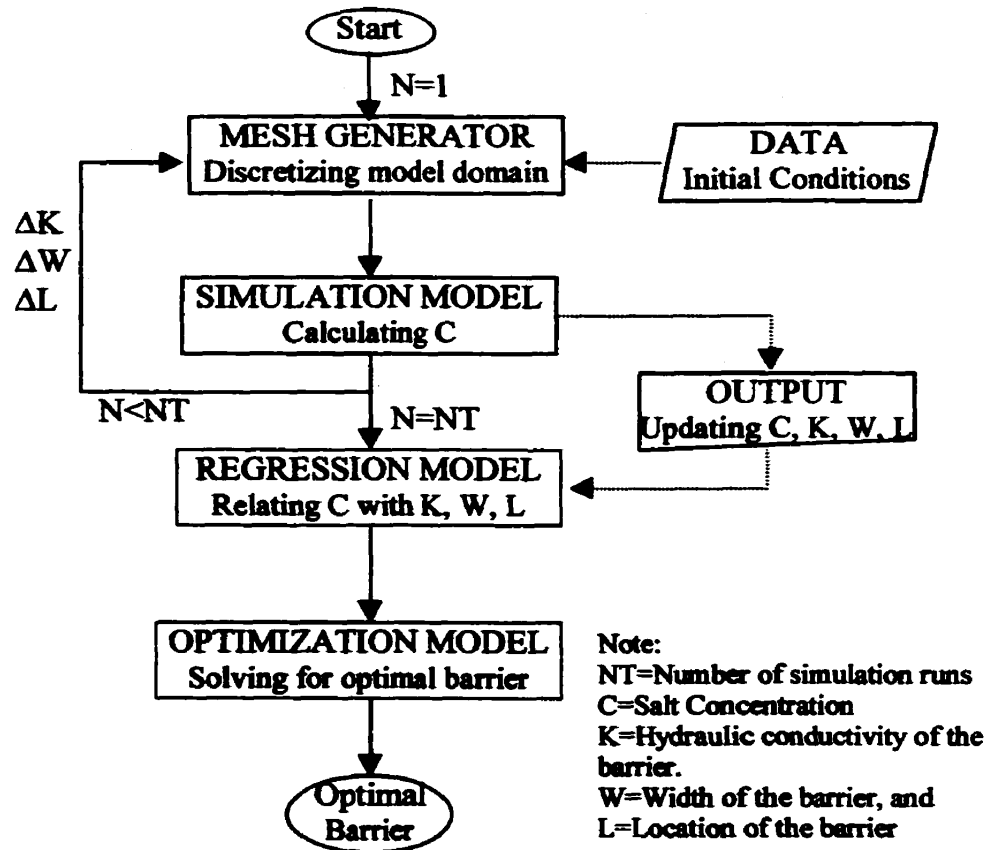


Figure 3.5: Flowchart of the Implicit Simulation-Optimization Model.

The proposed implicit simulation-optimization model has been described. The following chapter will present the theoretical background of two components of the proposed model.

Chapter Four

THEORETICAL BACKGROUND

4.1 Formulation of the Optimization Model

4.2 Groundwater Flow and Solute Transport

Chapter 4

THEORETICAL BACKGROUND

This chapter consists of two main sections that present mathematical formulations. The first describes the formulation of the optimization model, and the second, the equations that govern the groundwater flow and solute transport simulation model.

4.1 Formulation of the Optimization Model

The main objective of the optimization method is to minimize the construction cost by identifying optimal dimension and location of the barrier. Osuga [1996] indicates that if the dimension or, specifically, the width of a subsurface barrier can be minimized, the construction cost may be reduced. Hanson and Nilsson [1986] report that the cost of the construction material represents a large portion of the total cost of a subsurface barrier project.

In general, the less permeable a barrier is, the more primary materials (e.g. cement grout, montan wax or silica gel) are required. The total construction cost is high because the cost is dependent on the volume of material used. Conversely, the total construction cost is reduced if less material is required for the construction of a more permeable barrier. The location of the barrier is

considered in this study because it affects the size of the freshwater aquifer from which groundwater is withdrawn. The maximum freshwater volume can be achieved by locating a barrier at the minimum practical distance from the seashore. When the barrier is located away from the sea, it will result in the abandonment of a portion of the freshwater aquifer. Location is of particular concern when the groundwater basin being studied is relatively uniform in width and depth. Therefore, the objective of the optimization model is to find a solution that involves minimum total cost by calculating the appropriate width, hydraulic conductivity, and location of the barrier.

Considering the appropriate width, hydraulic conductivity, and location of the barrier, the problems of controlling seawater intrusion involve trade-offs among them, which can be arrived at by employing a multi-objective programming approach. The objective functions could be a minimization of the barrier width, maximization of its hydraulic conductivity, and minimization of its distance from the sea, subject to salt concentration constraints. However, the effectiveness of the multi-objective approach is diluted by the uncertainties inherent in selecting an appropriate weight for each objective function in order to obtain valid results. Theoretically, the proper weight can be obtained by involving decision makers, but allocation of weights, in practice, might not always be simple. For this reason, minimization of the overall cost of constructing a subsurface barrier is introduced. If the total cost of constructing a barrier is to be minimized, a single objective programming formulation should be considered.

Therefore, the objective function of the simulation-optimization formulation is to minimize the total construction cost by identifying the optimal width, hydraulic conductivity, and location of the barrier. The total construction cost is represented by two construction cost functions that are associated with the width and hydraulic conductivity of the barrier, and a loss function that is associated with the location of the barrier. The loss function describes cost imputed to loss of that portion of the freshwater aquifer sacrificed to contamination by saltwater. The width, hydraulic conductivity, and location of the barrier are determined in such a way that permissible levels of salt concentrations are not exceeded at the bottom boundary, just landward of the barrier, and the average concentration within the freshwater aquifer. The former is referred to as control location #1 and the latter as control location #2 (see Figure 4.1).

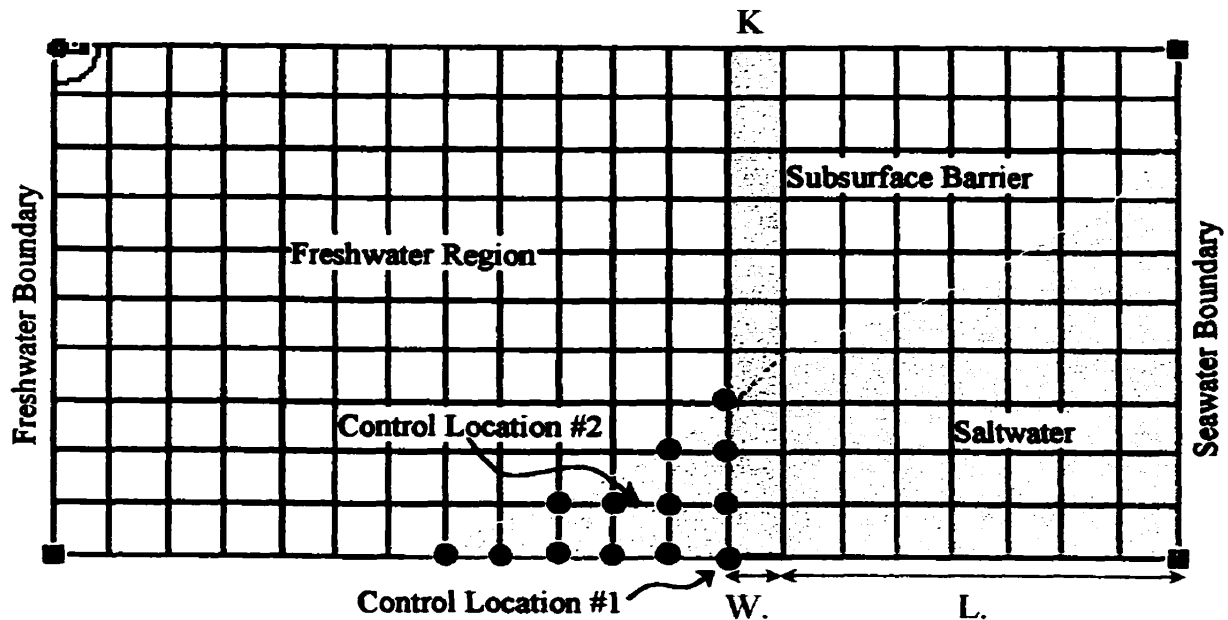


Figure 4.1: An Idealized Coastal Aquifer Showing Two Control Locations

Given the objective of minimizing the cost and the constraints on the salt concentrations as well as the lower and upper bounds of the decision variables, a nonlinear programming formulation is expressed as follows:

Minimize

$$e^T C_i \quad \forall i \quad (4.1)$$

subject to

$$f_1(W, K, L) \leq c_1^* \quad (4.2)$$

$$f_2(W, K, L) \leq c_2^* \quad (4.3)$$

$$W_u \geq W \geq W_l \quad (4.4)$$

$$K_u \geq K \geq K_l \quad (4.5)$$

$$L_u \geq L \geq L_l \quad (4.6)$$

where

e^T row vector of ones and $e^T C_i$ implies summation, where T denotes the transpose operator

C_i construction cost function associated with each decision variables i . For $i=1$, $C_1=C(W)$ is the construction cost function associated with the width of the barrier; for $i=2$, $C_2=C(K)$ is the construction cost function associated with the hydraulic conductivity of the barrier, and for $i=3$, $C_3=C(L)$ is the cost function associated with the location of the barrier;

c_i^* predetermined salt concentrations for case i , where $i=1$ is salt concentration at the node where the landward side of the barrier intersects the impervious boundary and $i=2$ is for the average salt concentration in the freshwater aquifer;

K hydraulic conductivity of the barrier;

W width of the barrier;

L location of the barrier;

Subscripts l and u denote the lower and upper limits.

In Section 4.2, the governing equations that describe the groundwater flow and solute transport simulation model, SUTRA, will be presented.

4.2 Groundwater Flow and Solute Transport

The system describing the phenomenon of seawater intrusion in coastal aquifers represents the main part of the simulation model with respect to the proposed implicit method. This method requires a system that governs groundwater flow and solute transport phenomena. The groundwater flow is explored using the continuity equation (mass conservation) of the fluid in the porous medium. The solute transport is handled by the continuity equation with respect to the dissolved salt. Theoretically, these two equations are sufficient to solve the movement and circulation of saltwater in coastal aquifers. However, two additional equations are required to solve the differential equations using a numerical technique. The required equations are the Darcy equation and a constitutive equation relating fluid density to salt concentration. Through the Darcy velocity and density of the constitutive equations, groundwater flow and solute transport equations can be coupled and solved.

The first governing equation addresses the mass balance of fluid per unit aquifer volume at any point in the aquifer [Bear, 1979]

$$\rho S_{op} \frac{\partial p}{\partial t} + \varepsilon \frac{\partial \rho}{\partial C} \frac{\partial C}{\partial t} + \nabla \cdot (\varepsilon \rho v) = Q_p \quad (4.7)$$

where

- $\rho(x, z, t)$ fluid density, M/L_f^3 , where L_f^3 is fluid volume;
- S_{op} the specific pressure storativity for a rigid solid aquifer matrix, $(M/LT^2)^{-1}$, where storativity is defined as the volume of water released from storage per unit horizontal area of an aquifer and per unit decline of piezometric head, and where L are length units and T are time units;
- $p(x, z, t)$ fluid pressure, M/LT^2 ;
- $C(x, z, t)$ solute concentration as a mass fraction, (mass solute/mass fluid) in units M_s/M , where M_s are units of solute mass and M are units of fluids mass;
- $\varepsilon(x, z)$ aquifer volumetric porosity, 1;
- $v(x, z, t)$ fluid velocity, in units L/T ;
- $Q_p(x, z, t)$ fluid mass source (mass fluid/aquifer volume/time) M/L^2T .

The specific pressure storativity is given by the relationship

$$S_{op} = (1 - \varepsilon)\alpha + \varepsilon\beta \quad (4.8)$$

where

- α porous matrix compressibility, $(M/LT^2)^{-1}$;
- β fluid compressibility, $(M/LT^2)^{-1}$;

It is important to note that the contribution of solute dispersion to the mass average flux of fluid is negligible.

The second governing equation is that which describes the solute mass balance per unit aquifer volume at a point in a cross-sectional aquifer with variable density fluids [Bear, 1979].

$$\varepsilon \rho \frac{\partial C}{\partial t} + \varepsilon \rho v \cdot \nabla C - \nabla \cdot [\varepsilon \rho (D_m I + D) \cdot \nabla C] = Q_p (C^* - C) \quad (4.9)$$

where

D_m the molecular diffusion coefficient of solute in pure fluid including aquifer material tortuosity effects, L^2/T , where tortuosity describes the effect of the configuration of the water occupied portion of the representative elementary volume (REV). REV refers to continuum approach adopted in subsurface flow modeling in order to pass from a molecular level of description to a macroscopic description of the flow. Thus, velocities and pressures can be averaged and assigned to its centroid for a certain range of averaging volume;

I identity tensor, 1;

$D(x, z, t)$ mechanical dispersion tensor, L^2/T ;

The mechanical dispersion tensor for an isotropic porous medium in two spatial dimensions is given by

$$D = \begin{bmatrix} D_x & D_z \\ D_z & D_x \end{bmatrix} \quad (4.10)$$

where

$$D_{xx} = \left(\frac{1}{|v|^2} \right) (d_L v_x^2 + d_T v_z^2) \quad (4.11)$$

$$D_{zz} = \left(\frac{1}{|v|^2} \right) (d_T v_x^2 + d_L v_z^2) \quad (4.12)$$

$$D_{xz} = D_{zx} = \left(\frac{1}{|v|^2} \right) (d_L + d_T) (v_x v_z) \quad (4.13)$$

$$d_L = \alpha_L |v| \quad (4.14)$$

$$d_T = \alpha_T |v| \quad (4.15)$$

where

$|v|$ the magnitude of velocity, L/T;

d_L longitudinal coefficient of mechanical dispersion, L²/T;

d_T transverse coefficient of mechanical dispersion, L²/T;

$\alpha_L(x, z)$ longitudinal dispersivity, L, is defined as a characteristic property of the porous medium in the flow direction;

$\alpha_T(x, z)$ transverse dispersivity, L, is defined as a characteristic property of the porous medium in the direction perpendicular to the flow;

The third equation is Darcy's law, which describes the mass average fluid velocity at any point in a cross section as

$$v = - \left(\frac{k}{\varepsilon \mu} \right) \cdot (\nabla p - \rho g) \quad (4.16)$$

where

$k(x,z)$	permeability tensor, L^2 ;
μ	fluid dynamic viscosity, M/LT ;
g	gravity vector, L/T^2 .

The fourth equation is a linear function that relates density to concentration.

$$\rho = \rho_0 + \frac{\partial \rho}{\partial C}(C - C_0) \quad (4.17)$$

where

ρ_0 fluid density when $C = C_0$; C_0 is a base solute concentration;

$\frac{\partial \rho}{\partial C}$ constant coefficient of density variability.

A general boundary condition for the fluid mass balance that applies at the stationary boundary is

$$\frac{S_y}{|g|} \frac{\partial p}{\partial t} + \varepsilon \rho v \cdot n + Q_{IN}^* = 0 \quad (4.18)$$

where

$S_y(x,z)$ specific yield (volume fluid released/aquifer volume) for unit drop in hydraulic head, 1;

$|g|$ magnitude of gravitational acceleration, L/T^2 ;

Q_{IN}^* fluid mass source due to flow across boundaries (mass fluid recharged per unit area of boundary/time), M/L^2T ;

n unit outward normal to the boundary, 1.

The groundwater flow equation (4.7) and the solute transport equation (4.9) are coupled in three ways. The first coupling is possible when the velocity components, v , are combined through Darcy's law (4.16). The second link is the hydrodynamic dispersion tensor that is a function of molecular diffusion and groundwater velocity (the third term on 4.9). The final connection is the fluid source/sink term on the right side of equations 4.7 and 4.9.

Given the system hydraulic parameters, source/sink and appropriate initial and boundary conditions, equation 4.7 is solved for pressure for a given time interval. The fluid velocities are calculated using equation 4.16. The calculated velocities are then used to define the dispersion coefficients (equations 4.10 – 4.15). Hence, the solute transport equation is solved for concentration at every point in the entire model domain and the solution is calculated simultaneously over time.

In Chapter 3, a brief review of simulation-management models in the field of water resources planning and management was presented, followed by the proposed simulation-optimization model for seawater intrusion control. Chapter 4 was devoted to the mathematical background: of the optimization technique and simulation model. The proposed method and the mathematical representations of the simulation model and optimization scheme lead to its model application to a hypothetical coastal aquifer in Chapter 5.

Chapter Five

MODEL APPLICATION

5.1 Modeling Seawater Intrusion in a confined Coastal Aquifer

5.2 Modeling Procedure

Chapter 5

MODEL APPLICATION

In Chapters 3 and 4, an implicit approach to optimal design of a subsurface barrier for the control of seawater intrusion, and the mathematical background of optimization and simulation models were presented. In Chapter 5, the applicability of the implicit simulation-optimization method is demonstrated by considering a hypothetical confined coastal aquifer subject to a constant seawater level.

5.1 Modeling Seawater Intrusion in a confined Coastal Aquifer

This section deals with groundwater flow and solute transport modeling in a confined coastal aquifer. The hypothetical cross-section of a coastal aquifer system is described schematically in Figure 5.1. This figure shows an aquifer system which receives no recharge and is subject to seawater intrusion. Freshwater enters the system on the landward (left) boundary and a constant sea level corresponds to the seaward (right) boundary. Impervious layers establish the top and bottom boundaries. This aquifer is a two-dimensional cross section of a regional system and represents Henry's Problem solved by Voss [1984]. Henry's problem addresses the steady-state condition of a diffused saltwater wedge within a confined aquifer, balanced against a flowing fresh-water field. The problem has

become a classical test for numerical simulators with solute-dependent density capabilities. Since no data relevant to this study were available in the literature, model parameters from Voss [1984] were employed.

Voss [1984] considered seawater intrusion caused by the greater density of the salt water encroaching into a confined aquifer and studied it in cross-section under steady state conditions. Fresh water enters the aquifer at a constant rate across an inland boundary and discharges across a coastal boundary. Saltwater from the coastal boundary advances against and mixes with the discharging freshwater. Because both the inland and coastal boundary conditions are invariant, a steady-state condition is reached, which balances the intruding seawater wedge against the freshwater flow field.

The model domain used by Voss [1984] is sufficiently simple to represent a cross-section of a regional confined coastal aquifer and not too complex to test the proposed method. The advantage of using this hypothetical aquifer system is that the model development and performance can be controlled. In addition, the modeling of this size of Voss' model can provide an insight into the effect of constructing a barrier at the fresh-saltwater interface relatively easily.

The model developed in this study is two-dimensional, like two previous models by Voss, [1984] and Sugio et al. [1987]. The reason for using a two-dimensional model is that the regional scale problem exhibits a nearly uniform flow field in which lateral convergence is negligible. Therefore, a two-dimensional, cross-sectional model is found to be appropriate for the representation of system behavior.

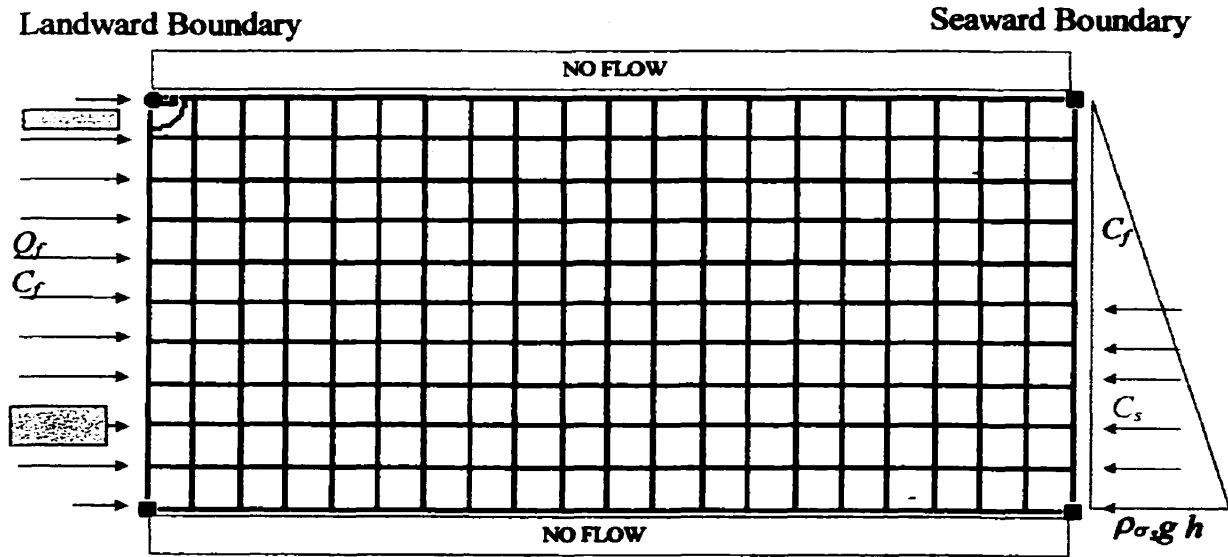


Figure 5.1: An Idealized Coastal Aquifer with Boundary Conditions Imposed.

The assumption is made that the vertical flow component does not play an important role in a regional coastal aquifer system. It implies that the approximation of essentially horizontal flow is employed. In practice, this approach is common in groundwater modeling [Bear and Verruijt, 1987]. Following these assumptions, a constant sea level is considered in this thesis since the fluctuation of the sea is expected to be very small compared to the depth of the sea. Similar kinds of assumptions have been made in earlier works [e.g. Huyakorn et al., 1987, Souza and Voss, 1987, and Voss and Souza, 1987]. Therefore, the hypothetical confined coastal aquifer system can be simplified as a two-dimensional rectangular domain, as shown in Figure 5.1. The length and height of the model domain are 2 m and 1 m, respectively. This domain is discretized into 231 nodes and 200 elements. Time steps are of length 30.0 [seconds], and 360 time steps are taken in the simulations.

At the landward boundary, specific discharge (Q_f) of freshwater and its concentration of dissolved solids (C_f) are specified. Q_f and C_f are specified at the nodes (Figure 5.1) along the left boundary. Each arrow in the boundary nodes represents the flow occupied by a shaded unit area. At the top and bottom of the boundary, nodes receive one half of Q_f . Any freshwater source at this freshwater boundary has a concentration of zero ($C_f = 0.0$ kg dissolved solid/ kg of saltwater).

Leakage and saltwater concentration values have to be specified along the top boundary. In the present study, the effect of rainfall or infiltration on the movement of the freshwater-saltwater interface is not considered, so that no flow is specified at all nodes along the top boundary.

The base of the aquifer system is impervious and therefore it is treated as a no-flow boundary. Accordingly, no flow is specified at the nodes along the bottom boundary. At the seawater boundary, hydrostatic pressure with salt density equal to ρ_s is specified. As shown in Figure 5.1, the triangular hydrostatic seawater pressure of $\rho_s gh$ is specified. The pressure at each node along this boundary is a function of the weight of water above it. Thus, the pressure is determined by the height of the water column.

In a confined aquifer system, the saltwater concentration is not fully specified along the seawater boundary. The freshwater outflow must be set at the top portion of the boundary to allow convective mass transport out of the system (Huyakorn et al., 1987). The height of this top portion is determined by the trial and error procedure. Several runs using SUTRA are performed to obtain a

reasonable initial freshwater-saltwater interface. The saltwater concentration of C_s is set at the nodes in the top portion and zero concentration is set at the nodes in the lower portion of the boundary.

Work related to the modeling of seawater intrusion in an aquifer system, i.e. the finite difference model of Sugio et al. [1987], and Henry's problem solved by Voss [1984], can be found in the literature. The model developed by Sugio et al. [1987] solves the governing equations using a finite difference technique, whereas this work employs a finite element approximation. The finite element method is usually more powerful and more flexible than the finite difference technique. Additionally, the sharp interface assumption is accepted to predict the change in solute concentration from freshwater to saltwater in the work of Sugio et al. [1987]. In this thesis, the salt concentration changes gradually so that it forms the transition zone.

The model applied in this thesis is very similar to Voss's hypothetical aquifer system. This includes all parameters, with the exception of inflow specified at the freshwater boundary. Work presented in this thesis differs from that of Voss [1984] in the existence of a subsurface barrier. Voss's model does not include the barrier in the analysis, while this work does. Table 5.1 presents the parameters used in the simulation.

Table 5.1 Summary of Parameter Values Used in the Simulation Model

Datum		Value
ε	dimensionless	0.35
C_s	kg dissolved solids / kg seawater	0.0357
ρ_s	kg/m ³	1025
$\frac{\partial \phi}{\partial C}$	kg ² of seawater / (kg dissolved solids x m ³)	700
ρ_0	kg/m ³	1000
k	m ² , based on $K = 1.0 \times 10^{-2}$ m/s	1.02×10^{-9}
$ g $	m/s ²	9.8
α, α_T	dimensionless	0.0
D	m ² /s	6.6×10^{-6}
C_f	dimensionless	0.0
Q_f	kg/s	3.2×10^{-2}

After Voss [1984]

5.2 Modeling Procedure

The application of the implicit simulation-optimization model developed in the present study to a regional cross-section of confined coastal aquifer systems can be described as follows:

- (I). The initial conditions are determined. These conditions are obtained through a trial simulation, which calculates the steady-state pressure values at different nodes. For the first run, natural steady pressures are set everywhere in the aquifer based on the freshwater inflow, zero solute concentration everywhere, and the specified pressures at the seaward boundary. This step is carried out together with the trial and error to determine the extent of the top portion at the seaward boundary. The boundary condition for a regional cross-sectional simulation of coastal aquifer systems allows convective mass transport out of the system over a certain height of the top portion at the

seaward boundary. The extent of the outlet portion is unknown a priori, but can be determined by trial and error. The simulation model is rerun and the coastal boundary conditions are updated until the velocities at nodes along the seaward side produce the expected velocity fields of a confined aquifer system. The initial conditions discussed in this step are calculated using the mesh in Figure 5.1. The initial conditions, the shape and location of the interface, and the portion of outlet at the seaward boundary must be correct prior to the simulation. Both are determined by a trial-and-error approach.

The simulation model provides the pressures and concentrations everywhere in the domain of the model. The distribution of salt concentration can be used to check the transition zone between saltwater and freshwater called, the freshwater-saltwater interface. Generally, the interface line refers to the 0.50 isochlor contour that describes contour lines dividing the mixing zone between freshwater and saltwater in half. The isochlor contour for a steady state simulation is shown in Figure 5.2. This figure shows the freshwater-saltwater initial-condition interface before the introduction of a subsurface barrier.

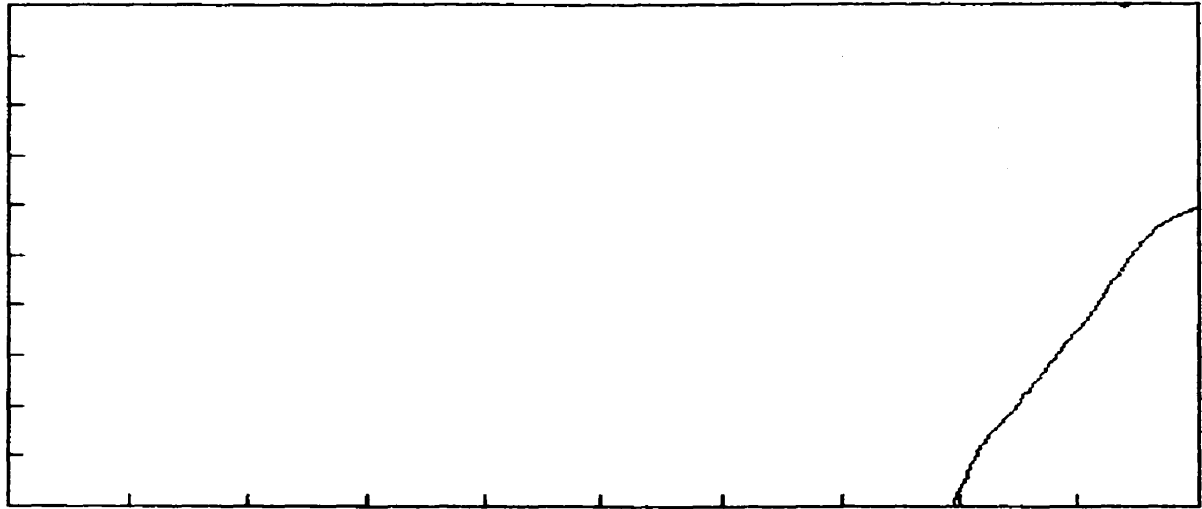


Figure 5.2: The Freshwater-Saltwater Interface at the Initial Condition Without the Barrier

Once the desired interface and the outlet portion are derived, the simulation model is run for the transient-state condition, based on the initial position of the freshwater-saltwater interface. The response of the interface is observed within 360 minutes under a condition of rapid piezometric head drop. This drop is simulated by reducing discharge from 6.0×10^{-2} kg/s to 3.2×10^{-2} kg/s at the freshwater boundary. The isochlor contour at the end of the simulation can be found in Figure 5.3.

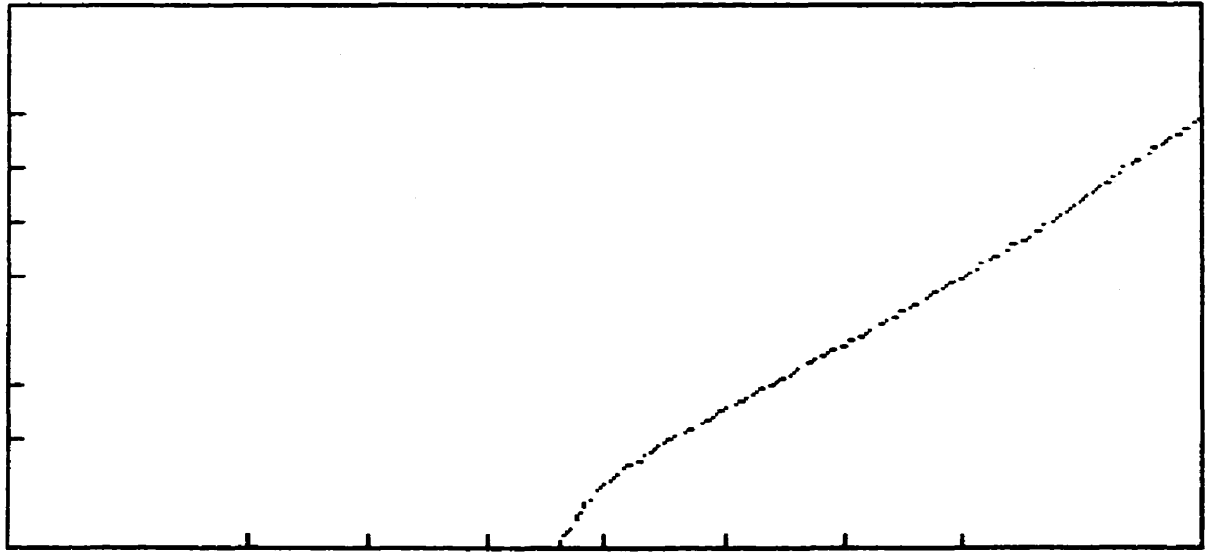


Figure 5.3: The Interface at the End of Simulation ($t=360$ minutes) Without the Barrier.

Figure 5.4 shows the velocity vectors illustrating the saltwater circulation at the end of the simulation without the subsurface barrier. This saltwater circulation indicates that the seawater intrusion encroaches landward by approximately two-thirds of the model domain, from the seaward boundary. This implies that the freshwater region has been progressively contaminated.

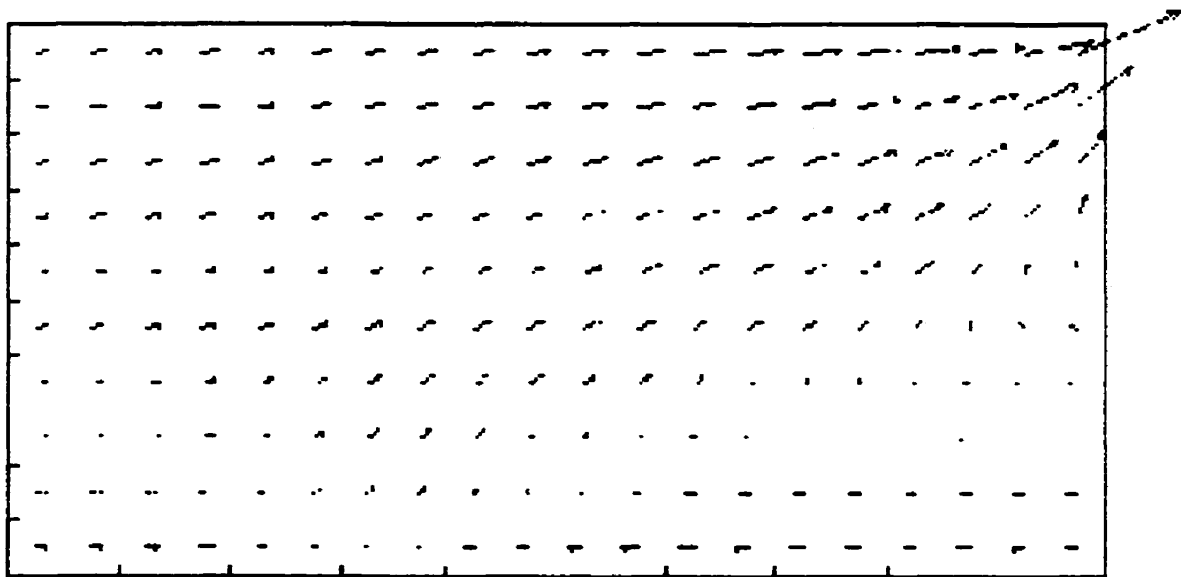


Figure 5.4: The Velocity Vectors Indicating the Flow Pattern at the End of the Simulation Before a Subsurface Barrier is Introduced

(II). After the simulation of the aquifer system without the barrier has been successfully performed, the next step is the simulation of the same system with the subsurface barrier under transient state conditions. At this step, the model domain is discretized for every simulation run. Each simulation run has different values of the three decision variables: width, hydraulic conductivity and location of the barrier, which are varied systematically within the simulation model.

The simulated variation of these decision variables is kept within their possible ranges. For example, the width of the barrier may not be thinner than 0.02 m, nor wider than 1.0 m. It is important to note that the barrier has a finer grid than the grid in the saltwater zone. Since variation in salt concentration is relatively small, the freshwater zone has a coarse grid. The system domain is divided in such a way that the number of nodes and elements remains the same for each the simulation. Figures 5.5 and 5.6 show how the mesh changes from

one simulation to another. The mesh shown in Figure 5.5 indicates that the number of nodes in the saltwater zone, in the barrier, and in the freshwater zone, are 7, 5, and 11, respectively. In the horizontal direction 21 nodes are created. Figure 5.6 shows the change in mesh after a number of simulation runs; for example the number of nodes in the saltwater zone and in the barrier are now seven. To maintain the number of 21 nodes, 9 nodes are introduced in the freshwater zone.

For each combination of width, hydraulic conductivity and location of the barrier, the average salt concentration in the freshwater region (C_{fr}) and salt concentration at the bottom boundary just landward of the barrier (C_{int}) are computed and stored. Numerous simulations (828) are performed to produce a better approximation of solute concentration functions. The stored outputs are split into two sets of data. The first set of data contains the average salt concentration in the freshwater zone and the three decision variables, and the second consists of the salt concentration at the bottom boundary just landward of the barrier and the same three decision variables.

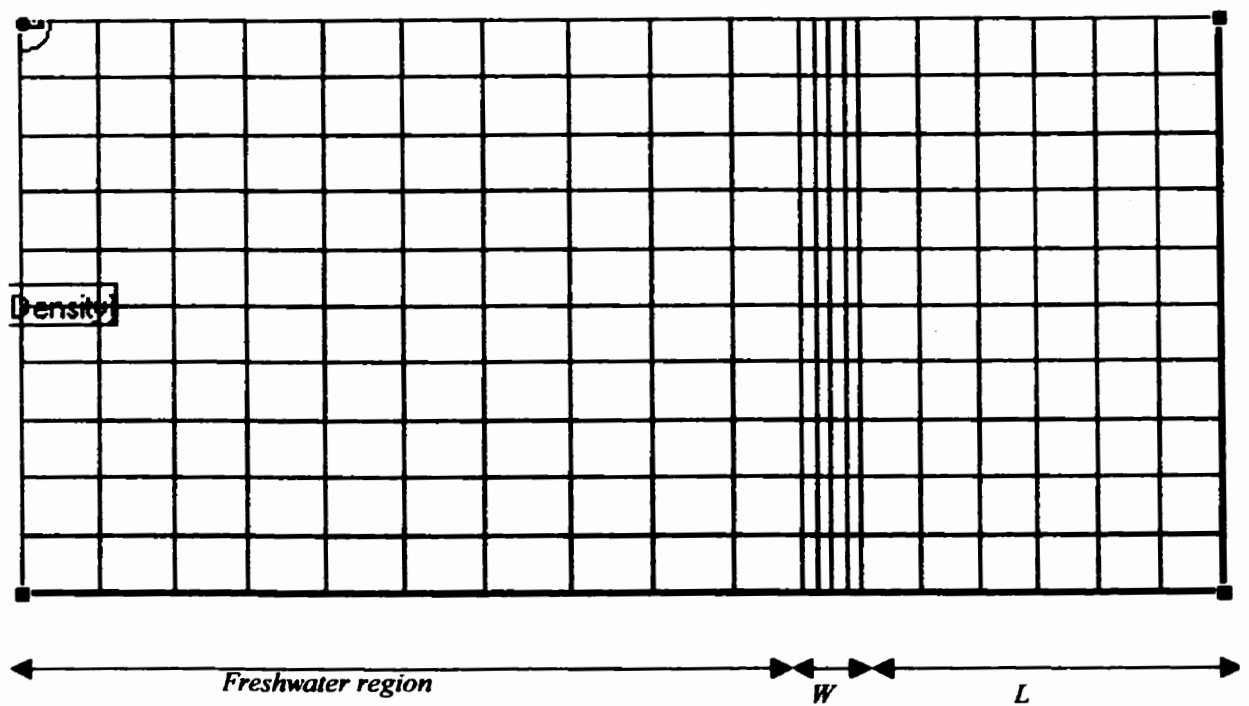


Figure 5.5: Grid with 200 Elements, 231 Nodes, and 4 Elements for the Barrier

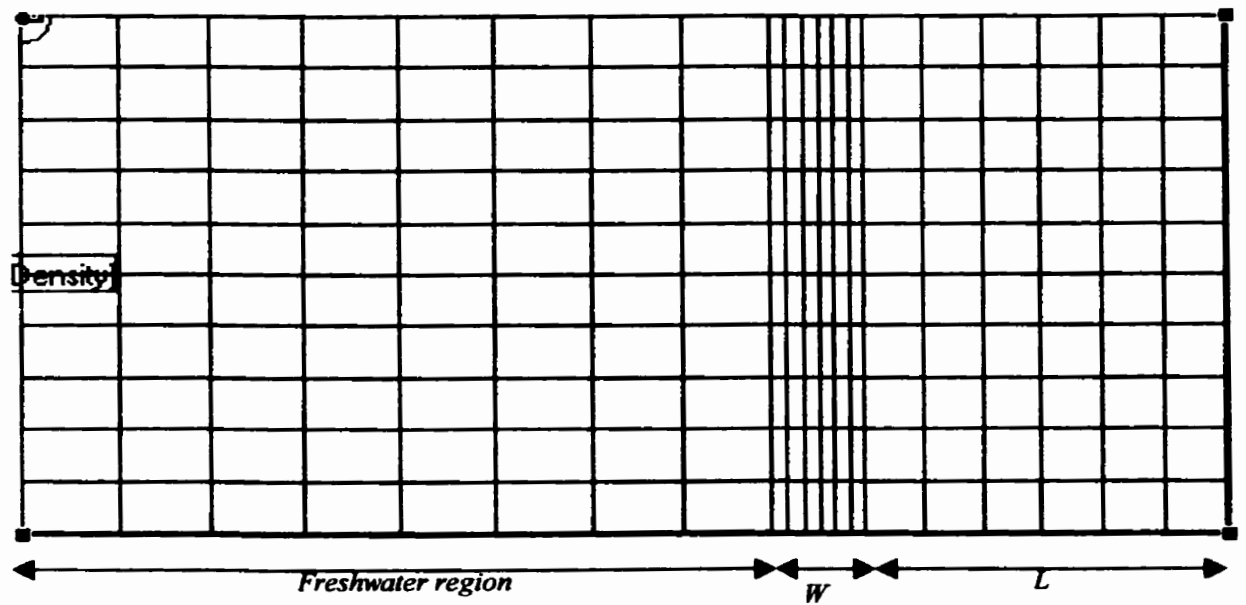


Figure 5.6: Grid With the 200 Elements, 231 Nodes and 6 Elements for the Barrier

(III). A regression analysis is carried out to obtain the salt concentration equations linking the simulation and optimization models. Two predefined nonlinear equations are required to describe the relationship between the salt concentrations at the two control locations and the decision variables for the regression analysis, and are obtained using the functional relationship of nodes of a trained Artificial Neural Network (ANN). An ANN model is employed in this study in a nontraditional fashion to derive the analytical form of these relationships. Neural network structure (number of hidden nodes and layers) is converted into an analytical description of the relationships.

Given the functions from the neural network, the regression model is run to obtain the relationships. The inputs of the neural network model are the decision variables selected in this study: width, hydraulic conductivity, and location of the barrier. The outputs include the salt concentrations at the two control locations. ANN, employed in this study, is limited to two transformation functions: (a) sigmoidal; and (b) hyperbolic tangent [Braincel, 1993]. In this work, the latter is used for further analysis. ANN provides a type of functional relationship (hyperbolic tangent) and the number of hidden nodes as well as the number of hidden layers, which determine the number of terms in the equation as follows:

$$\begin{aligned}
C = & a_0 + a_1 \tanh (b_1 W + c_1 K + d_1 L + e_1) + \\
& a_2 \tanh (b_2 W + c_2 K + d_2 L + e_2) + \dots \\
& + a_n \tanh (b_n W + c_n K + d_n L + e_n)
\end{aligned} \tag{5.1}$$

where C is the salt concentration, W is the width of a barrier, K is the hydraulic conductivity of the barrier, L is the location of a barrier, and $a_0, a_i \dots d_1 \dots e_i \dots d_n$ are constants for the i -th term. Regression is used to determine the values of $a_0, a_i \dots d_1 \dots e_i \dots d_n$.

The transfer functions and the ANN architecture shown in Table 5.2 are obtained from neural network simulations. This table presents information on four important characteristics of ANN; (1) the error, (2) hidden layers, (3) hidden nodes, and (4) transfer function. The results from running the neural network for the first set of concentration-decision variables indicate that the errors are less than 5 % (2.03) in training and 2 % (1.99) in testing, respectively. For the second data set the errors are higher than those of the first data set, but less than 10 % (8.97 %) in training and 5 % (4.88 %) in testing. In both cases, the training errors are higher than the testing errors, which are contradiction with most cases in general. The reason is that the ANN model is run not to memorize the training data. This can be done by interrupting training occasionally and test the ANN model against the unseen testing data. It implies that the unseen testing data are used while training. In this way, the model experiments combinations of initial weights and hidden nodes

and checks several instances of each combination before selecting another. As a result, the error is minimized during testing as shown in Table 5.2.

Training in the neural network refers to a process during which an ANN model passes through input-output data pairs repeatedly, changing the values of the weights to improve performance of the network. Mean squared error (MSE) is used to measure the performance of an ANN model. Testing refers to a process for measuring an ANN's performance during which the network passes through an independent data set, without changing the weights. This error range confirms that the relationship between C_{fr} and W , K , and L is well established.

The hyperbolic tangent appears to be the appropriate transfer function for this data set. The number of hidden layers and nodes that are used to construct the architecture of the network are also presented in Table 5.2. The functional relationship between inputs (W , K , and L) and output (C_{fr}) is captured in the neural network with one hidden layer and four hidden nodes, shown in Figure 5.7. The network describes the relationship as three inputs (nodes 1, 2, and 3), one hidden layer with four nodes (nodes 4, 5, 6, and 7) and one output (node 8).

The same procedure is repeated for the second set of concentration-decision variables, where C_{int} are grouped with W , K , and L . These results are also summarized in Table 5.2 (row #2). The relationship is described, using the same transfer function with different architecture,

i.e. 1 hidden layer and 1 hidden node. It means that the network architecture is much simpler than the one shown in Figure 5.7. Instead of having 4 nodes in the hidden layer, it now has one.

Table 5.2 Network Architecture and Performance Values for Two Data Sets for Establishing $C_{fr}(W,K,L)$ and $C_{int}(W,K,L)$

Concentration Functions	Training Error	Testing Error	Hidden Layers	Hidden Nodes	Transfer Function
$C_{fr}(W,K,L)$	2.03	1.99	1	4	Hyperbolic tangent
$C_{int}(W,K,L)$	8.97	4.88	1	1	Hyperbolic tangent

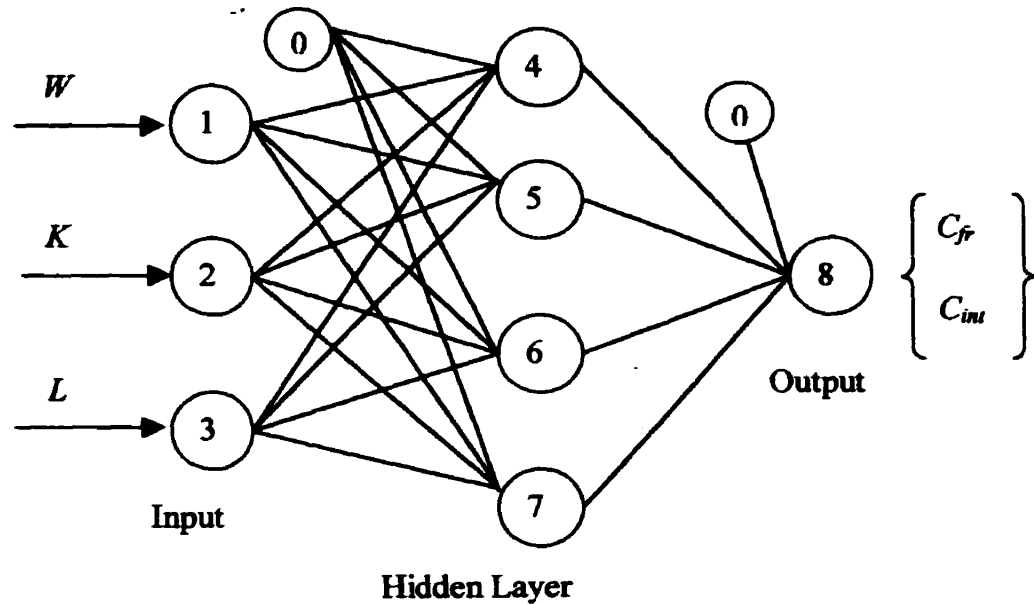


Figure 5.7: Network With 3 Inputs, 1 Output and 1 Hidden Layer With 4 Nodes.

Theoretically, based on the transfer function, architectures (connections and weights - not shown in the table), and processing element function (e.g. $a_1 X_1 + a_2 X_2 + \dots + a_n X_n + b$ where a_i and b are constants and X_i is input variable), the mathematical relationship describing the salt concentration as a function of decision variables can be established. In fact, since most artificial network software does not present the detailed preprocessing formulation (e.g. normalization), it is not possible to derive the relationship directly in the form of an equation from the neural network output. Therefore, a nonlinear regression model is required to supply a complete equation.

The nonlinear regression model is run for two sets of data. These runs are for the equations derived not only from hidden nodes suggested by the neural network, but also for several recognized nodes. For instance, the regression analysis to relate concentrations in the freshwater zone with the decision variables is performed four times, using only one term of the hyperbolic tangent function, indicating one hidden node, to four terms of the hyperbolic tangent function indicating four hidden nodes. Similarly, the regression model is run for the second set of data for the networks that have one to four hidden nodes.

The results indicate that the number of hidden nodes in the neural network does not necessarily become the number of terms in the regression equations, which is clear from Equations 5.2 and 5.3. In Equation 5.2, there are three terms of the hyperbolic tangent function, not

four as suggested by the neural network. Equation 5.3 contains two terms of the hyperbolic tangent function, more than the number of hidden nodes suggested by the neural network. The number of terms of hyperbolic tangent in the regression equation is chosen, based on the highest coefficient of determination, R^2 , value. From regression of $C_{fr} = f(W, K, L)$ the R^2 obtained is 0.95 and for $C_{int} = f(W, K, L)$, R^2 is 0.91.

$$\begin{aligned} C_{fr} = & 1.23 * \tanh(13.30 * K + 1.36 * W + 0.53 * L + 2.03) \\ & + 2.32 * \tanh(0.56 * K - 2.03 * W - 0.22 * L + 2.23) \\ & - 6.18 * \tanh(0.16 * K - 0.40 * W - 0.04 * L + 2.01) + 2.50 \end{aligned} \quad (5.2)$$

$$\begin{aligned} C_{int} = & 0.001 * \tanh(0.61 * K + 4.14 * W + 31.28 * L - 18.21) \\ & + 13.23 * \tanh(48.66 * K - 9.75 * W - 0.42 * L + 4.77) - 13.19 \end{aligned} \quad (5.3)$$

(IV). The relationship between the construction cost and the decision variables is required as a performance measure (objective function) for the optimization model. Three cost functions are developed.

Using the quadratic function and considering the range of possible widths of the barrier and its corresponding construction cost as described in Chapter 4, the following equation is obtained.

$$C(W) = -108333 * W^2 + 24250 * W - 341.67 \quad (5.4)$$

where $C(W)$ is the construction cost as a function of the width of the barrier (W).

The same procedure is carried out to establish the construction cost

associated with the hydraulic conductivity of the barrier. The assumed exponential function and the range of possible hydraulic conductivity values, with its corresponding cost, as described in Chapter 4, are used. The exponential function obtained is as follows:

$$C(K) = 86.30 K^{-0.3461} \quad (5.5)$$

where $C(K)$ is the construction cost as a function of the hydraulic conductivity of the barrier (K).

The cost function representing the size of the freshwater zone, which resulted from locating the barrier at a distance from the seashore, is established, based on the assumed quadratic function and the range of locations, with its relevant damage cost, described in Chapter 4. The cost function associated with the location of the barrier is obtained as follows.

$$C(L) = 1428.60 * L^2 - 500.0 * L + 71.43 \quad (5.6)$$

where $C(L)$ is the damage cost as a function of the location of the barrier (L).

(V). In this step, the optimization problem relative to controlling seawater intrusion is formulated using an objective function with a set of constraints. The cost functions described in step IV are combined into a single objective function. This objective can be expressed as the sum of the construction costs associated with the width, hydraulic conductivity, and location of the barrier. The two nonlinear concentration equations from step III are treated as constraints. These constraints ensure that salt concentrations at the two control locations never exceed the permissible levels.

The concentration at the bottom boundary, just landward of the barrier, is maintained below 0.024 kg of dissolved solids per kg of seawater. This is approximately 67 % of the salt concentration at the sea boundary. At the second control location - the freshwater zone - the concentration is maintained below 0.02 kg of dissolved solids per kg of seawater, which is slightly lower than that at the bottom boundary. In a real-life implementation, these two concentration levels can be determined by decision makers.

The complete formulation for seawater intrusion control problems using a subsurface barrier is as follows:

$$\begin{aligned} \text{Minimize } & -108333 * W^2 + 24250 * W - 341.67 + 86.30 * K^{-0.3461} \\ & + 1428.6 * L^2 - 500.0 * L + 71.43 \end{aligned} \quad (5.7)$$

The design of a subsurface barrier is constrained by:

- The saltwater concentration at the bottom boundary just landward of the barrier.

$$\begin{aligned} & 0.001 * \tanh(0.61 * K + 4.14 * W + 31.28 * L - 18.21) \\ & + 13.23 * \tanh(48.66 * K - 9.75 * W - 0.42 * L + 4.77) - 13.19 \leq 0.024 \end{aligned} \quad (5.8)$$

- The saltwater concentration in the freshwater zone.

$$\begin{aligned} & 1.23 * \tanh(13.30 * K + 1.36 * W + 0.53 * L + 2.03) \\ & + 2.32 * \tanh(0.56 * K - 2.03 * W - 0.22 * L + 2.23) \\ & - 6.18 * \tanh(0.16 * K - 0.40 * W - 0.04 * L + 2.01) + 2.50 \leq 0.02 \end{aligned} \quad (5.9)$$

- Additional constraints of the design model also include the lower and upper bounds of the decision variables.

$$0.01 \geq K \geq 0.00056 \quad (5.10)$$

$$0.1 \geq W \geq 0.01 \quad (5.11)$$

$$1.0 \geq L \geq 0.4 \quad (5.12)$$

The results obtained from the optimization model will be presented in Chapter 7. Presentation of the results is followed by a detailed discussion of the results. At this point the application of the implicit simulation-optimization model has been presented. The proposed explicit model and its application will be described Chapter 6.

Chapter Six

EXPLICIT MODEL

6.1 Genetic Algorithms

6.2 Basic Theory of Genetic Algorithms

6.3 GAs in Water Resources

6.4 GAs in Groundwater

6.5 GA Application to Seawater Intrusion Control

6.6 Model Formulation

6.6 Simulation Tests

Chapter 6

EXPLICIT APPROACH

The essence of the explicit simulation-optimization model is to fully integrate to Genetic Algorithms (GAs) with the groundwater-flow and solute-transport simulation model detailed in Chapters 4 and 5. The GAs are used to search for the optimal design of a subsurface barrier for seawater intrusion control. This chapter begins with an introduction to GAs. Next, section 6.3 discusses of some applications of GAs to water resources as reported in the literature. In sections 6.4 and 6.5 the formulation of the explicit simulation-optimization model is presented. Finally, in section 6.6, several simulation tests are described.

6.1 Genetic Algorithms

GAs are heuristic approaches to searching a solution space for optimal solutions. These heuristic-based approaches, which mimic the mechanisms of natural selection, were first introduced by Holland [1975]. The processes of evolution in nature inspired him to introduce the concept of GAs in science and engineering. Since then, GAs have been fairly successfully applied to in a diverse range of optimization problems in many disciplines. Excellent introductions and reviews of many important applications of GAs can be found in Goldberg [1989], Davis [1991], Michalewicz [1992], and Haupt and Haupt [1998]. The first three authors

present a comprehensive introduction to GAs, and Haupt and Haupt [1998] make a presentation of GAs that is easy to follow and understand. This chapter is not devoted to reviewing all previous work on GAs found in the literature; rather it will highlight some important applications in the field of water resources. Before reporting on their applications to water resources, the basic theory of GAs will be presented.

6.2 Basic Theory of Genetic Algorithms

The first task of a GA is to randomly generate an initial set of solutions, called the initial population. Each member of this population represents a possible solution to the problem, and is encoded as a chromosome. A chromosome is a string of symbols usually, but not necessarily, a binary bit string. The population of chromosomes evolves through a cycle that involves selection, crossover, and mutation. Each cycle is referred to as a generation. After many generations, the population will contain chromosomes that represent near optimal solutions to the problem.

In each generation, selection, crossover, and mutation operations are performed on the chromosomes, the objective being to find better chromosomes for the next generation. The principle behind selection is that of survival of the fittest. A measure of fitness is chosen, usually the objective function. Individuals with poor fitness (high objective function values) are rejected and do not compete in the next generation. To replace these rejected chromosomes, crossover and mutation are used to generate new members of the population. In

crossover, an “offspring” receives a combination of genetic information from two “parent” chromosomes. Suppose that each chromosome is represented by a string of n symbols. A number k is chosen at random between 1 and $(n-1)$. The offspring then receives the first k symbols from one parent and the rest from the other. This can produce fitter offspring by combining the best genetic characteristics of two individuals.

While crossover tends to focus the search on that part of the solution space already represented, mutation allows the search to spread into regions that have not yet been examined. In each generation, several chromosomes are chosen for mutation. For each chromosome, a randomly chosen set of symbols in its string is altered slightly. Thus, the operations of crossover and mutation allow the solution space to be searched, while selection ensures that only the best individuals survive from one generation to the next. If this procedure continues for enough generations, the final population will contain many chromosomes that represent near-optimal solutions to the problem being considered.

The size of the population influences the time required to find solutions near the global optimum. The population has to be sufficiently large to represent the search space. If the population size is too small, the lack of diversity within each population means that the search will be inefficient. On the other hand, if the population size is too large, the same individuals may compete from one generation to another, because of an insufficient number of string combinations.

GAs, like any other algorithm, have their advantages and disadvantages. The advantages of using GAs are:

- (1) They require no knowledge or gradient information about the response surface,
- (2) Discontinuities present on the response surface have little effect on overall optimization performance;
- (3) They are resistant to becoming trapped in local optima;
- (4) They perform very well for large-scale optimization problems; and
- (5) They can be employed for a wide variety of optimization problems.

The disadvantages of GAs are:

- (1) They have trouble finding the exact global optimum;
- (2) A large number of response (fitness) function evaluations are required;
and
- (3) Configuration is not straightforward.

Based on the algorithm described above, a wide variety of problems in science and engineering, including water resources can be formulated and solved. GAs are especially suitable for coping with highly nonlinear and discontinuous objective functions and constraint sets, which are main characteristics of water and/or groundwater systems. The following section presents the previous work on GAs in water resources.

6.3 GAs in Water Resources

In the field of water resources, GAs have been applied to a variety of problems, ranging from a relatively simple application such as calibration of rainfall-runoff (R-R) models to the most complicated problem in groundwater remediation

projects which accounts for parameter uncertainty. Conceptual R-R models are calibrated using GA combined with the local search method [Wang, 1991]. Seven parameters used in the R-R model are determined by minimizing the sum of squares of differences between simulated and observed discharges. Of ten optimization runs, eight results are able to locate the global minimum. The results from the other two are slightly inferior to the global optimum. This indicates that the performance of the GA is considered as robust and efficient.

Similar work done by Franchini [1996] attempts to combine a GA with a local optimization procedure based on sequential quadratic programming (SRQ) to calibrate a conceptual R-R model automatically. Eleven model parameters are calibrated, and the results indicate that several parameters are tightly grouped while others remain distributed within their respective feasible ranges. Therefore, the coupled GA-SQR model is an efficient and robust tool for calibrating the conceptual R-R models.

The use of GA for calibration purposes is also investigated in the study of Mulligan and Brown [1998]. The objective of their work is to estimate water quality model parameters. Two approaches for handling constraints are investigated. The first approach is to incorporate constraints into the objective function, which is referred to as the penalty method. The second approach is to introduce a special decoding operation. The comparison demonstrates that the penalty function-based GA outperforms the special decoder-based GA.

A large number of applications in pipe network systems using GAs can be found in the literature. In his doctoral dissertation at the University of Michigan

in 1983, Goldberg [1989] presents a series of extensive works on pipeline optimization. Besides presenting his major work in pipeline networks, a wide variety of other subjects is also covered. Murphy et al. [1993] developed a methodology for optimizing a water supply network by employing a simple GA. The objective function of the water supply network was to minimize the cost of a water distribution network by finding optimal combinations of pipe sizes.

Simpson et al. [1994], Davidson and Goulter [1995], Halhal et al. [1997], Reis et al. [1997], and Savic et al. [1997] investigate the application of GAs to pipe-network optimization. Simpson et al. [1994] compares the GA-based solutions with those of nonlinear programming (NLP) for pipe network optimization and find that GAs are superior to NLP in terms of their capability of finding acceptable solutions. It is reported, however, that the convergence of GA is somewhat slower than that of NLP.

The layout of a branched rectilinear pipe network is optimized using a new GA approach [Davidson and Goulter, 1995]. Their algorithm employs two new operators, recombination and perturbation, which are suitable for handling the case of the layout design of rectilinear branched networks. This new GA is demonstrated on a small problem and compared with a technique based on a heuristic approach. The results show that the GA optimization model is able to generate better solutions than the heuristic technique.

Dandy et al. [1996] present an improved GA formulation for optimizing the water supply pipe network cost for New York City. They conclude that the GA using variable power scaling of the fitness function and a creeping mutation

operator performs significantly better than the simple GA. Savic and Walters [1997] use GANET (Genetic Algorithm NETWORK, the Integration of GIS, Network Analysis and Genetic Algorithm Optimization Software for Water Network Analysis) to minimize the cost of a water distribution network, and demonstrate that GANET outperforms other optimization techniques.

Similar works on water supply pipe networks using GAs, are also presented. Halhal et al. [1997] apply a structured messy GA for dual objectives to a large optimization problem of network rehabilitation. Reis et al. [1997] address the problem of appropriate location of control valves in a water supply pipe network using GA to obtain maximum leakage reduction for given nodal demands and reservoir levels.

GAs have also been used to optimize the operation of reservoir systems. A four-reservoir system has been formulated and optimized by Esat and Hall [1994], using a GA. The objective is to maximize the benefits from power generation and irrigation water supply subject to constraints on storages and releases from the reservoir. The results of their work concluded that, in reservoir system optimization, standard dynamic programming (DP) techniques are less effective than a GA-based model. This conclusion is supported by a study on a reservoir system, done by Fahmy et al. [1994]. The results demonstrate that the GA-based solutions are better than those of dynamic programming techniques. It is also indicated that GAs can be applied to large river basin systems.

Oliveira and Loucks [1997] explore the use of a GA to optimize the operating rules of multireservoir systems used for water supply and hydropower.

Multireservoir operating policies are defined by rules that specify system release and individual reservoir storage volume targets. These releases and targets are functions of total storage in each of multiple-year periods. Sets of possible operating policies are generated by a GA that employs elitism, arithmetic crossover, mutation, and "en bloc" replacement. A simulation model is used to evaluate each policy generated and to determine a performance index for a given flow series. The most efficient policy is obtained after going through the standard procedure in GA.

Similar work on reservoir operation is presented by Wardlaw and Sharif [1999]. Several alternative formulations of a GA for reservoir systems are evaluated, based upon the four-reservoir, deterministic, finite-horizon problem. This has been done with a view to presenting fundamental guidelines for implementation of the approach to practical problems. Alternative representation, selection, crossover, and mutation schemes are considered. It is concluded that the most promising GA approach for the four-reservoir problem comprises real-value coding, tournament selection, uniform crossover, and modified uniform mutation. Real-value coding operates significantly faster than binary coding and produces better results. The known global optimum for the four-reservoir problem can be achieved with real-value coding.

A nonlinear four-reservoir problem is also considered, along with one with extended time horizons. The results demonstrate that a GA could be used satisfactorily in real time operations with stochastically generated inflows. A more complex ten-reservoir problem is also considered, and results produced by a

GA are compared with previously published results. The GA approach is robust and is easily applied to complex systems. It has potential as an alternative to stochastic dynamic programming approaches.

6.4 GAs in Groundwater

Applications of GAs to groundwater quality problems have been especially effective. The reason is that, in field-scale groundwater remediation systems, typical functions encountered are nonconvex and/or nonlinear. McKinney and Lin [1994] incorporate groundwater simulation models with a GA to solve three groundwater management problems: maximum extraction from an aquifer, minimum-cost water supply development, and minimum-cost aquifer remediation. The results indicate that GAs can effectively and efficiently be used to obtain optimal solutions to these three groundwater management issues. The formulation of the method is straightforward, and provides solutions which are as good as, or better than, those obtained by linear and nonlinear programming.

The application of GA to the multi-objective problem of containing groundwater pollution is the topic of work done by Ritzel and Eheart [1994]. The problem is to seek the set of optimal solutions on the trade-off curve between the reliability and cost of a hydraulic containment system. Two multi-objective optimization problems are formulated, and results are compared with those of mixed integer chance-constrained programming (MICCP). The GA-based solution method, which attempts to avoid the difficulties associated with scaling the criteria of a multi-objective problem (called VEGA), is inferior to the one

that involves finding the set of strings in the population (called Pareto GA) for the problem with the zero fixed cost situation. In addition, the Pareto GA is able to produce a trade-off curve similar to that obtained by MICCP.

Problems of groundwater remediation have also been solved, using GAs, by Rogers and Dowla [1994]. They describe an approach to nonlinear optimization that harnesses the strengths of the genetic algorithm to search for more cost-effective remediation strategies within practical time-frames. Escalating costs of environmental cleanup, together with the conflicting concerns of various stakeholders, motivate the search for improved management methodologies to reduce costs. They present an approach to nonlinear optimization that combines the strengths of artificial neural networks and GAs. Results drawn from a major western Superfund site demonstrate the potential for substantial saving. Superfund is a program established by the U.S. Congress in 1980 to locate, investigate, and clean up the worst site-pollution nationwide. This approach outperforms mathematical programming methods in terms of computational burden and flexibility.

A multiobjective groundwater monitoring problem under conditions of uncertainty is solved using a GA [Cieniawski et al. 1995]. In this study, they evaluate the GA-based solution method by comparing it with the results from simulated annealing. Two aspects are compared: performance and computational intensity. For each iteration, the model based on simulated annealing is superior to the GA-based method. However, a trade-off curve that contains a large number of nondominated points is generated by the GA in a single iteration, and fewer

points by the simulated annealing method. Both require approximately the same amount of computational time.

A similar problem to the one addressed by Rogers and Dowla [1994] is solved by Vemuri and Walter [1996]. They introduce a new multi-niche crowding (MNC) to locate all the peaks of a multi-modal function. By associating these peaks with the utility accrued from different sets of decision variables, it is possible to extend the use of GAs to multi-criteria decision making. The MNC GA is used to decide the optimal location of withdrawal wells, and has the ability to maintain different solutions satisfying multiple conflicting objectives.

Optimal design of aquifer cleanup systems under uncertainty, using a combination of GA and a neural network, is the focus of Aly and Peralta [1999]. The model that accounts for the stochastic nature of hydraulic conductivity is able to find the global optimal solution by producing a trade-off curve between reliability and pump-and-treat system facility size for aquifer cleanup.

One general shortcoming of GAs is in enforcing the given constraints. Hilton and Culver [2000] study two methods for constraint handling within the genetic algorithm framework. The first method, the additive penalty method (APM), is a commonly used penalty function approach, in which a penalty-cost proportional to the total constraints-violation is added to the objective function. The second method, the multiplicative penalty method (MPM), multiplies the objective function by a factor proportional to the total constraints violation. The APM and MPM, using constant and generation-varying constraint weights, are applied to two pump-and-treat design problems.

Overall, the application of the APM resulted in infeasible solutions with small-to-moderate total constraints violations. With the MPM, a set of feasible and near-optimal policies was readily identified for both examples. Additionally, the MPM converges to the solution faster than the APM. These results demonstrate that the MPM is a robust method, capable of finding feasible and optimal or near-optimal solutions while using a range of weights.

Morshed [2000] discusses application of an enhanced GA for optimal groundwater management. Three enhancements are made: (1) fitness reduction method (FRM), (2) search-bound sampling method (SBSM), and (3) optimal resource allocation guideline (ORAG). In order to evaluate them, a nonlinear groundwater problem with fixed and variable costs is selected, in which the corresponding optimal solution using a gradient-based nonlinear programming (NLP) technique is available for comparison. The results of the study show: (1) FRM enhances efficiency of the GA in handling constraints; (2) SBSM enhances accuracy of the GA in solving problems with fixed costs; (3) ORAG enhances reliability of the GA by providing some convergence guarantee for a given computational resource; and (4) when applied with FRM and SBSM, accuracy of the GA is marginally increased from near-optimal to global-optimal by tuning any one of the several parameters.

In the groundwater problems discussed above, GAs are employed to search for optimal solutions. Many other applications of GAs to groundwater problems are found in the literature, but they are more chemical than water-related management problems, such as those exposed in Aksoya and Culvera [2000].

Such an application is not covered here. In the next section, the use of the GA in this thesis will be described.

6.5 GA Application to Seawater Intrusion Control

There have been many applications of GAs in the field of water resources, including groundwater, and they have been used to solve numerous types of groundwater problems. However, there have been no previous applications of formal GA optimization methods to seawater intrusion problems. Moreover, GAs have never been employed to solve seawater intrusion problems that involve subsurface barriers. As aforementioned, the construction costs associated with such barriers are very high. Thus, the optimal design has the potential to reduce costs substantially. The use of GAs to find this optimal design may become an active area of research.

The explicit simulation-optimization model developed here couples a GA with SUTRA [1984]. The optimization technique used in this explicit model is the GA solver developed by Carroll [1996]. It is a simple GA that is developed for unconstrained maximization problems. It can handle only two decision variables, both of which must have the same range. As a consequence, the same number of bits is used to encode each variable.

To solve the problem in designing an optimal subsurface barrier for seawater intrusion control, several modifications are made to the GA [Carroll, 1996]. First, it is modified so that a set of constraints in the formulation can be handled. In addition, modifications are undertaken to manage the difference in

magnitude for each of the decision variables considered in the seawater intrusion control problems. Along with these modifications, the code has to deal with the different range specified for each decision variable. It is also modified to solve a minimization problem.

The SUTRA model used in the implicit simulation-optimization approach also needs several modifications to render it suitable for use in the explicit approach. Within the explicit simulation-optimization framework, the simulation model is a part of the GA model and is repeatedly called by the GA. The simulation model, as in the implicit approach, is used to compute salt concentrations at two different control locations: at the bottom boundary just landward of the barrier, and in the freshwater zone. These salt concentrations are sent back to the GA routine and used to determine the fitness of the solution.

The two control-location-specific salt concentrations are used to calculate the constraints in the explicit simulation-optimization model. If a constraint is violated, a numerical value is added to the objective function. This constraint-handling method is referred to as the additive penalty method (APM). Basically, the APM approach is a simple way of transforming a constrained optimization problem into one that is unconstrained. Such an approach is the most practical solution to problems with implicit constraints, in which a simulation model must be run to check for a violation. This is the case for the current problem. Salt concentrations are required to compute the constraints, but must first be determined by the simulation model.

As the flowchart in Figure 6.1 shows, the simulation model is run after the strings are decoded into the decision variables. Once the salt concentration levels are obtained, the differences between these calculated concentrations and permissible levels are added to the objective function. The final fitness function for the GA would be:

$$F(K,W,L) = e^T C_i + \alpha(x) \quad (6.1)$$

where $\alpha(x)$ = the penalty function associated with the set of M constraints. $\alpha(x)$ can be described as:

$$\alpha(x) = \sum_{m=1}^M \omega_m \nu_{m2} \quad (6.2)$$

where the constraint violation ν_{m2} is defined as

$$\nu_{m2} = [\max(0, f_m(K,W,L) - c_m^*)] \quad (6.3)$$

where ω_m is the weight applied to the violation on constraint $c_m(x)$ and x is the decision variable.

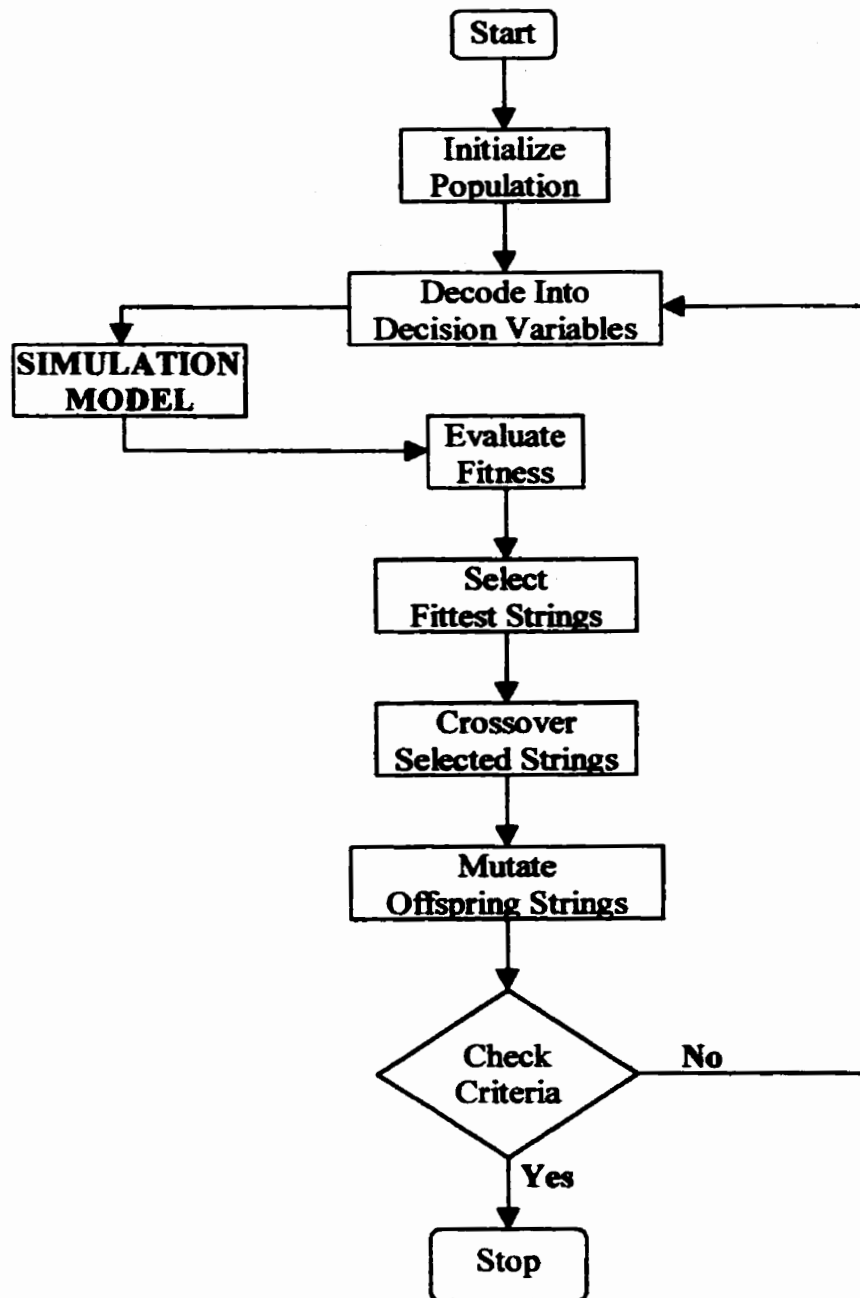


Figure 6.1: Flowchart of the Proposed Explicit Simulation-Optimization Model.

6.6 Model Formulation

The purpose of designing an optimal subsurface barrier for seawater intrusion control is to restrain the encroachment of saltwater moving inland by a physical underground structure within a predetermined time horizon. By formulating this problem as a GA optimization, the number of steps involved can be reduced. As demonstrated in the implicit approach, the involvement of the regression model requires tedious and time-consuming manual work. In the explicit simulation-optimization approach, the regression model is eliminated and replaced by a GA. The steps are performed by the computer, and manual intervention is not required. The problem being solved is described in Chapter 4 (Equations 4.1 – 4.5), and will not be repeated here. Rather, the GA steps will be examined.

There are many versions of GAs, but in general, all GAs have two basic steps: selection, and reproduction. Selection determines which individuals in the population are allowed to produce offspring. Reproduction involves crossover and mutation, as discussed in Section 6.3. Different problems will require different implementations of these operations. The steps involved in finding the optimal design of a subsurface barrier for seawater intrusion control using the GA are as follows:

Step 1: An initial population of decision variables, represented by strings, is randomly generated. A binary coding is used to represent each bit on the string. The binary representation is suggested by Goldberg [1989], who refers to his theoretical work comparing binary with real coding. Holland

[1975] advocates the use of an alphabet with low cardinality in the enhanced schema processing, and also recommends binary representation. Constructing a GA with real or integer numbered coding is often more directly representative of the actual decision variables, can be faster, more stable, and can have lower variability in average fitness. However, binary coding is most often used and makes the performance of a GA optimal [Goldberg, 1989].

Recent works [e.g. Oliveira and Loucks, 1997, and Wardlaw and Sharif, 1999, who apply a GA to reservoir system operation] indicate that the real- numbered representation is better than either the binary or gray representations in terms of proportion of maximum fitness. Also, the variability in average fitness when using real-number representation is much lower than when using binary or gray representations. Michalewicz [1992] concludes that a GA with real representation is faster, more stable and produces better solutions when a problem requires high precision. In general, all investigators agree that the binary coding has some weaknesses when the problem to be solved requires high precision and involves a large number of parameters. Since the problem of controlling seawater intrusion using a subsurface barrier does not require such a high precision, a binary coding GA is suitable for this work.

Step 2: In the explicit simulation-optimization approach, the GA generates the decision variables and passes to the groundwater and solute-transport

simulation model, linked as an independent subroutine. This subroutine controls the input of aquifer data, the domain of interest, and any change in grid system that might be required. The simulation model calculates the distribution of salt concentrations and the concentrations at the two control locations, and returns that data to the GA, which is then used in the evaluation process.

Step3: The fitness of a population member (i.e. each string generated in Step 1) is evaluated. The evaluation is based on the objective function [Equation 5.7] and constraints [Equations 5.8 and 5.9]. There are four main categories of approaches for handling constraints in GAs. The first approach eliminates all infeasible solutions. This significantly reduces the number of searches, but leads to poor results when applied to a problem that has a small ratio between feasible and infeasible solutions.

In the second approach, infeasible solutions are “repaired”. This technique is more applicable to problems that have explicit constraints, in which the simulation model is not required to determine the feasibility of the solution. For implicit-constraint problems, the constraint values are dependent on the simulation model. In this case, it is unclear how infeasible solutions can be repaired. The third approach is to use a modified genetic operator. Usually, the crossover and mutation operations are modified in such a way that only feasible offspring are produced. Again, this is useful for most problems with explicit

constraints. The last approach to the handling of constraints is to add penalty values to the objective function. Any solution that violates one or more constraints is penalized. This is the most appropriate approach to problems with implicit constraints, such as the current problem.

Step 4: After measuring the fitness of each string in the population, selection is performed. In this process, strings from the population are chosen to create a temporary mating pool, from which reproduction will occur. The tournament selection method [Goldberg, 1989] is used because this method closely mimics mating competition in nature. Other selection methods, such as top to bottom pairing, random pairing, and roulette-wheel weighting, do not mimic natural evolution. Tournament selection randomly selects two strings from the population. Their fitness values are compared. The string with the minimum cost is selected to join the temporary mating pool.

Step 5: After the mating pool is created as described in Step 3, crossover is applied. In this process, two parent strings are randomly selected from the mating pool. Then a crossover point (a position in the string) is determined at random. Information following that point is exchanged between the two strings, creating two new individuals. Crossover is performed based on a crossover probability, which determines the number of crossover operations that occur within any generation. This

probability tends to be high, but less than 1. Note that the size of the new population is unchanged from generation to generation.

Step 6: The next operation, after crossover, is mutation. This operator changes a small percentage of the bits of a string by flipping from 1 to 0 or vice versa. The bits that are mutated are chosen at random.

When selection, crossover, and mutation have been performed, chosen parents and offspring join the new population. An elitist strategy is implemented at this step. In this strategy the string with the best fitness is always selected to survive into the new population. To accomplish this, the least fit individual of the current generation is replaced by the most fit individual of the previous generation.

Step 7: The process of evaluation, selection, crossover and mutation (Steps 2 – 6) is repeated until convergence is achieved or the maximum number of expected generations is exceeded.

Having developed the explicit simulation-optimization model, several tests are used to verify its performance. These are discussed in the next section.

6.6 Simulation Tests

Two sets of tests are performed for the explicit approach. Two tests are carried out for each set. In the first set, the cost functions are established arbitrarily.

The second set uses adjusted functions, in which the range of construction costs is modified so that the solutions sought do not indicate a priority for any one decision variable.

Test #1:

The simulation runs in Test #1 are intended to determine the best configuration of GA parameters and penalty weights. For this reason the simulation model is not yet linked to the GA-based optimization method. Instead, Equation 5.7 is used as the objective function, with equations 5.8 and 5.9 as the constraints. The left-hand-side terms of Equations 5.8 and 5.9 are obtained using the simulation model. Constraint violations are treated, using penalty functions. Traditionally, the weighting of a penalty for a particular problem constraint is based on judgment. The model has to be run many times before an appropriate combination of penalties is found. The best combination of penalties allows infeasible solutions to die and feasible solutions to flourish. It is obvious from Test #1 that if a set of penalties is too harsh, the few solutions found that do not violate constraints quickly dominate the mating pool and result in local optimal solutions. A penalty that is too lenient can allow infeasible solutions to flourish as they may have better fitness values than feasible solutions.

Test #2:

Test #2 is designed to determine the effect of different cost functions. The simulation model is still not linked to the GA-based optimization method. In Test

#1, the solutions found were in fact upper and lower bound values for one or two of the decision variables. To explore better trade-off values, the cost functions are adjusted. To find such adjusted functions, a series of runs is carried out, in which the cost associated with the location of the barrier is modified. All runs in this step use the GA parameter values obtained from Test #1. Test #2 is followed by rerunning the implicit model using the same cost functions for comparison.

Test #3:

Test # 3 is aimed at implementing the configuration parameter settings obtained from Test #1 into the explicit simulation-optimization model. In this test, the groundwater flow and solute transport simulation model is integrated into the GA-optimization method.

Test #4:

The aim of Test #4 is to examine the performance of the explicit approach in responding to different cost functions. The range of construction costs associated with the location is modified, balancing its contribution to the trade-off among the three decision variables in the objective.

The explicit simulation-optimization method and simulation tests for verification of the GA-based model have been presented. The results and discussion will be presented in Chapter 7.

Chapter Seven

RESULTS AND DISCUSSION

7.1 Results

7.1.1 Results of Test #1

7.1.2 Results of Test #2

7.1.3 Results of Test #3

7.1.4 Results of Test #4

7.2 Discussion

7.2.1 Selection of GA Parameter Values

7.2.2 Comparison Test #1

7.2.3 Comparison Test #2

7.2.4 Comparison Test #3

7.2.5 Comparison Test #4

7.3 Modeling Features

Chapter 7

RESULTS AND DISCUSSION

The GA is applied to the problem of identifying an optimal design for a subsurface barrier for seawater intrusion control. Results obtained from the application of the explicit simulation-optimization model are presented and discussed in the following sections.

7.1 Results

7.1.1 Results of Test #1

Pursuant to numerous test runs, three GA parameters were identified as affecting the search performance most. They are population size, mutation rate, and crossover rate. In addition, the type of crossover is found to have a significant effect on the search performance. The best GA parameter values for this particular problem are: population size = 100, mutation rate = 0.1, and crossover rate = 0.97. The optimal subsurface barrier obtained using those GA parameter values is located at 0.4 m from the seaward boundary, is 0.1 m wide and has a hydraulic conductivity of 0.007 m/s. Table 7.1 presents the change in fitness with variation of the GA parameter values, including the type of crossover. Only the best configurations are shown. The GA was stopped after 75 generations.

Table 7.1 GA Parameter Settings

Population Size	Mutation Rate	Crossover Rate	Crossover Type	Fitness (MU)
50	0.1	0.97	Uniform	1582.86
75	0.1	0.97	Uniform	1582.42
100	0.001	0.97	Uniform	1599.17
100	0.09	0.97	Uniform	1579.70
100	0.1	0.97	Uniform	1579.70
100	0.2	0.97	Uniform	1585.37
100	0.1	0.97	Single point	1584.18
100	0.1	0.98	Uniform	1580.91
100	0.1	0.96	Uniform	1582.51
110	0.1	0.97	Uniform	1581.51
200	0.1	0.97	Uniform	1581.76
500	0.1	0.97	Uniform	1580.15

A similar test was performed for the implicit simulation-optimization approach.

The result is compared with the GA-based solution in Table 7.2.

Table 7.2: Results Obtained from the Explicit and Implicit Models, Using Arbitrary Cost Functions

Approach	Width (m)	Hydraulic Conductivity (m/s)	Location (m)	Fitness (MU)
Implicit	~0.08	4.13×10^{-3}	0.4	1580.24
Explicit	0.10	7.00×10^{-3}	0.4	1579.70

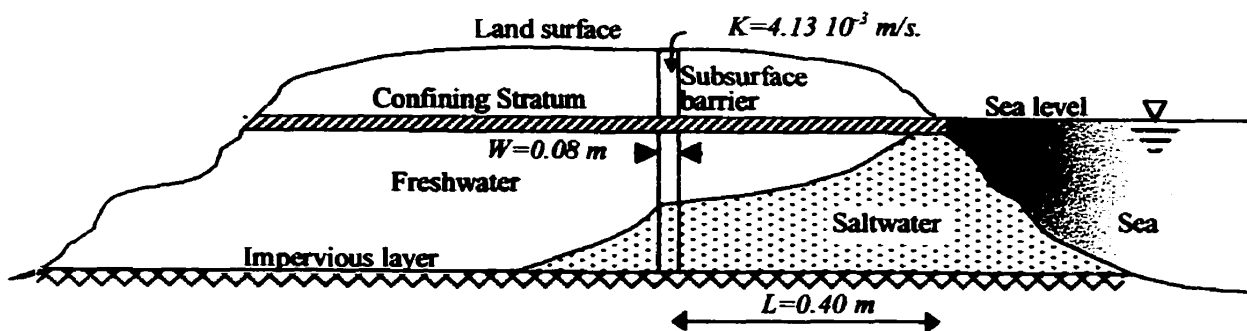


Figure 7.1: A Schematic Representation of the Optimal Design of a Subsurface Barrier

7.1.2 Results of Test #2

In Test #1, the solutions approached the lower or upper limit values for the decision variables. In both implicit and explicit solutions, the optimal barrier is located as close to the sea boundary as is allowed (the lower bound value). In the explicit solution the width of the barrier is the upper bound value, which indicates that the location takes priority over the cost function. This indicates that the cost functions serve as weights in goal programming. That is, some functions place a particular variable a higher priority. In Test #2, such results are not expected, because the adjusted costs are used. The results from both approaches are presented in Table 7.3.

Table 7.3: Results of the Explicit and Implicit Models, Using Adjusted Cost Functions

Approach	Width (m)	Hydraulic Conductivity (m/s)	Location (m)	Fitness (MU)
Implicit	0.06	3.34×10^{-3}	0.77	1182.67
Explicit	0.06	3.10×10^{-3}	0.75	1183.81

The results of the explicit approach are based on 75 GA generations. When the number of generations is increased to 275, the solution improves from 1183.81 to 1182.99 MU. When 1500 generations are used, the solution improves to 1182.58. However, the GA-based solution is still not significantly better than that of the implicit approach.

7.1.3 Results of Test #3

In this test the simulation model is linked to the GA. Table 7.4 compares the optimal solutions obtained from the explicit and implicit approaches.

Table 7.4: Results of the Explicit and Implicit Models, Using Arbitrary Cost Functions (Simulation Model is Linked).

Approach	Width (m)	Hydraulic Conductivity (m/s)	Location (m)	Fitness (MU)
Implicit	~0.08	4.13×10^{-3}	0.40	1580.24
Explicit	0.08	6.40×10^{-3}	0.40	1580.21

7.1.4 Results of Test #4

Table 7.5 shows that the optimal GA-based solution is very close to the one obtained from the gradient-based optimization model. The fitness value does not change by increasing the number of generations from 75 to 275.

Table 7.5: Results of the Explicit and Implicit Models, Using Adjusted Cost Functions (Simulation Model is Linked).

Approach	Width (m)	Hydraulic Conductivity (m/s)	Location (m)	Fitness (MU)
Implicit	0.06	3.34×10^{-3}	0.77	1182.67
Explicit	0.06	3.30×10^{-3}	0.78	1182.90

Figure 7.1 below is the schematic representation of the optimal design of a subsurface barrier, as derived from the implicit simulation-optimization model presented in Table 7.2. The optimal design is a barrier with a width of 0.08 m, a hydraulic conductivity of 4.13×10^{-3} m/s, and a location of 0.4 m from the seashore. This design costs about 1580.24 MU.

7.2 Discussion

7.2.1 Selection of GA Parameter Values

As stated above, the performance of GAs is dependent upon three parameters: population size, mutation probability and crossover probability. It has been suggested by Goldberg [1989] that a good GA performance requires a moderate population size, a low mutation probability and a high crossover probability. These parameters are discussed below.

7.2.1.1 Population Size

The population size dictates the number of chromosomes in the population. A good population of chromosomes contains a diverse selection of potential building blocks, which will result in an effective exploration of the solution space. If a GA loses its diversity, the search converges prematurely and little exploration is accomplished. Table 7.1 demonstrates that a population size of 100 produces better results than 50 or 75.

On the other hand, a larger population size increases diversity at the expense of requiring more fitness evaluations. Also, a larger population size does not guarantee a better solution, as the result in the last row of the same table demonstrates. Although computational time is not included in the table, the effect of more fitness evaluations is obvious. The best population size is dependent on the application and the length of the chromosomes. For this particular problem a

population size of 100 seems to be best. This is at the upper extreme of the range of 50-100 suggested by Goldberg [1989] and of 25-100 suggested by others.

In general, for longer chromosomes and challenging optimization problems, larger population sizes are needed to maintain diversity and thereby improve exploration. This higher diversity can also be achieved through higher mutation rates and uniform crossover rates, as discussed later.

7.2.1.2 Mutation Rate

The mutation probability determines mutational frequency will occur. Mutation is applied to provide new information to the population, and to prevent the population from becoming saturated with similar chromosomes. A higher probability of mutation increases the chance that good individuals will be destroyed, but it also increases population diversity. The result in row #6 of Table 7.1 shows that a fitness of 1585.37 MU is achieved by using a mutation probability of 0.2. This is far from the best value, and represents a solution trapped in a local optimum.

In the third row, the probability of 0.001 is so low that there is no diversity in the population, and a poor fitness value is obtained. The best mutation rate is found to be 0.1, as shown in the fifth row of the same table. This probability is at the upper end of the range of 0.001 - 0.1 suggested in the literature.

7.2.1.3 Crossover Rate

Crossover is performed to explore combinations of bits in existing strings. It is very useful in making the algorithm converge. It is usually suggested that the crossover rate be set at a very high value. During this test, values from 0.5 to 0.1 were used and compared. As presented in Table 7.1 row #5, a crossover probability of 0.97 provides the minimum cost of 1579.70 MU. By decreasing (the same Table, row #9) and increasing (row # 8) this probability, the fitness changes from 1579.70 MU to 1582.51 MU and 1580.91 MU, respectively. Clearly, from Table 7.1 the crossover probability of 0.97 is the best for this particular problem.

7.2.1.4 Type of Crossover

The type of crossover is found to be a weaker influence than the three other parameters. Table 7.1 indicates that the GA with uniform crossover is slightly better than the GA with single crossover. The single crossover as shown in row #7, increases the fitness dramatically from 1579.70 MU (row #5) to 1584.18 MU.

7.2.2 Comparison Test #1

After selection of the GA parameter values, the explicit model was run, to test the parameters obtained. Table 7.2 presents the optimal design of a subsurface barrier, identified through running the implicit and explicit simulation-optimization models.

The optimal solution resulting from application of the explicit model has a total cost of 1579.70 MU, with a barrier width of 0.10 m, a hydraulic conductivity of 7.0×10^{-3} m/s, and a location at 0.4 m from the seaward boundary, with no constraint violations. This solution is slightly better than that of the implicit model, which provides a total cost of 1580.24 MU with a barrier width of about 0.08 m, hydraulic conductivity of 4.13×10^{-3} m/s, and location at 0.4 m from the seashore.

For both approaches, the location of the optimal barrier is at the closest feasible distance from the sea boundary (the lower bound value). In addition, the results of the explicit approach are also pushed to the upper limit value (0.10 m) of the barrier width. This result is rather unexpected, since the width of 0.08 m, obtained from the implicit model, should be nearly optimal.

However, since the implicit solution is the result of interpolation, it is possible that the decision variables generated by the GA-routine fall into an interpolation section in which the true solution is unknown. A possible explanation for the indicated optimal location being at the lower limit value is the high range of costs associated with the location of the barrier. This range is set too high. For the implicit model, the width of about 0.08 m is already close to the upper limit value. This means that the cost function associated with the width of the barrier tends to lead the solution to the end of its range. To support this argument, Test #2 is performed, and the results confirm it.

7.2.3 Comparison Test #2

If the results from Test #1 indicate that the explicit model is superior to the implicit model, Test #2 indicates the opposite. The implicit model results in a minimum cost of 1182.67 MU for a subsurface barrier with the width of 0.06 m, a hydraulic conductivity of 3.34×10^{-3} m/s, and a location 0.77 m from the seaward boundary. This result is better than that of the explicit model, which yields a total cost of 1183.81 MU with the same width and slight difference in the two other decision variables. However, it is clear that a larger number of generations improves the solution. When this GA-based model is run for 1500 generations, the total cost is 1182.58 MU, which is an improvement over the implicit model. From Test #2, it can be concluded that the performance of the GA-based model is also dependent on the number of generations.

Furthermore, the explicit model is superior to the implicit model only for a very large number of generations, such as 1500. When using a reasonable number of generations, say 75, the superiority of the explicit model is not evident. Overall, the results from the two models are comparable thus far.

It is important to note that the results presented in Table 7.3 indicate that the cost functions play a role in arriving at the optimal solution. By changing the cost functions related to the width and location, different optimal designs are identified. Since the constraints are unchanged, they do not draw the solution towards the boundaries, as was suspected in the previous discussion.

In the next sections, the results of integrating the simulation model with the GA are discussed.

7.2.4 Comparison Test #3

The results shown in Table 7.4 are not unexpected, because the interpolation used in Test #1 is absent. In this test, any combination of decision variables should produce concentrations at the two control locations, with no interpolation necessary. The results from the explicit model are similar to those obtained with the implicit model. The optimal solution, with a fitness value of 1580.21 MU, was found at generation 75, in a reasonable computation time (11 hours, not considered high compared to those of the implicit approach). The width and location of the barrier are in agreement with those of the implicit model. The difference is that the barrier with the hydraulic conductivity of 6.40×10^{-3} m/s obtained from the explicit model is more porous than the one obtained from the implicit model (4.13×10^{-3} m/s). This marginal difference makes the GA-based solution slightly better than the gradient-based solution.

Test #3 is repeated to see whether an increased number of generations can improve the solution. The model was run for 275 generations, taking almost 4 days of computational time. However, the fitness value is not improved. For this reason, the explicit model was not run with 1500 generations.

7.2.5 Comparison Test #4

In Test #3, the integration of the simulation model allowed the GA to avoid solutions at the bounded value. Test #4 should provide an improvement of the solution. The location of the barrier is shifted away from the sea boundary, as presented in Table 7.5. The improvement is expected from adjusting the range of cost functions. By lowering the range of total cost associated with the location of the barrier, all cost functions have balanced weight in the objective function. Table 7.5 shows that the solution is no longer trapped in the lower limit value. A barrier with the fitness value of 1182.90 MU, and the three decision variables very close to the implicit solution, was identified after 75 generations with an execution/computation time of 11 hours. This time the explicit model was inferior to the implicit model. After 275 generations (approximately 4 days of computational time), the fitness value remained unchanged.

Overall, the search performance of the explicit simulation-optimization model appears to be case-dependent. Since population size, mutation and crossover rates are problem-specific, so are all GA-based solutions.

The overriding conclusion of the series of tests performed in this work is that the explicit simulation-optimization model (using a GA) performs as well as, if not better than, the implicit simulation-optimization model (using gradient-based techniques). Three tests indicated that the explicit approach outperforms the implicit approach, and one test showed otherwise. In one test, however, it took 1,500 generations for the explicit model to outperform the implicit model.

7.3 Modeling Features

The objective of developing the explicit simulation-optimization model for seawater intrusion control is to evaluate the ability of a GA to deliver a solution comparable to that obtained using the implicit simulation-optimization model. From a modeling point of view, the use of the GA allows the groundwater-flow and solute-transport simulation model to be fully integrated into one single optimization model. The GA replaces the regression model used in the implicit simulation-optimization and its optimization solver, MINOS. Execution of these two processes is tedious and time-consuming, due to the nature of the gradient-based optimization techniques, which require the first derivative of the objective function and a set of constraints. Gradient information has to be derived manually and then inserted into the MINOS code for compilation. In the present study, the functions that relate the decision variables to the response surface are continuous. For extensions of the present study, which might result in discontinuous functions, the advantage of adopting the GA is obvious.

In addition to the issue of GA's superiority in handling discontinuity of the functions, the shape of cost functions is also important. This shape determines the convexity of the functions. The methods developed for controlling seawater intrusion using a subsurface barrier suggests an optimization problem in which the objective function is strictly convex (Hessian matrix of the objective function is positive definite). In such a case the optimum is unique. When the optimization problem solved is nonconvex, the solution is harder to find since a local optimum is not necessarily global. In other words, the problem exhibits

multiple local solutions. If multiple optimal solutions are identified, a decision in selecting the best has to be made based on local condition where the barrier is plan to construct. For example, the solution that locates the barrier close to the sea is more preferable than the one away farther from the sea. The reason is that the selected one benefits more groundwater users, while the other implies extra cost for supplying the users seaward of the barrier.

The more significant development has been the simulation of a groundwater-flow and solute-transport model within the GA-based optimization method. Employing the GA in association with such a simulation model eliminates the requirement for a regression model, while providing accurate results within the same range. This is a significant improvement, since the implicit approach requires much manual work.

Chapter Eight

CONCLUSIONS

8.1 Summary

8.2 Contributions of This Research

8.3 Recommendations

8.3.1 Implementation Improvements

8.3.2 Extensions

Chapter 8

CONCLUSIONS

8.1 Summary

This research has explored two new methods for optimal design of a subsurface barrier to control seawater intrusion through development of the implicit and explicit simulation-optimization models. The application of these models is also demonstrated. The implicit approach refers to a combination of a groundwater-flow and solute-transport model with a gradient-based optimization technique that uses a regression model to supply nonlinear constraints as required in its formulation. The explicit approach refers to an integration of the same simulation model with a GA-based optimization technique. The strength of this thesis lies in the formulation and integration of the aforementioned components into a working model, to provide an optimal design of a subsurface barrier.

The results obtained from the explicit approach are compared to those of the implicit approach. The comparisons, in four tests indicate, that the explicit simulation-optimization model that uses GA performs as well as, if not better than, the implicit simulation-optimization model that uses hill-climbing techniques. Using the GA as an optimization tool in the explicit approach to identifying an optimal design for a subsurface barrier, reduces a number of manual tasks and therefore the computational burden. This reduction cannot be

quantified because of idle time created in the implicit approach. The GA-simulation model, on the other hand, has no inherent idle time.

In Chapters 6 and 7, the method developed, based on the explicit simulation-optimization approach, is presented, and the results are compared with those of the implicit approach presented in Chapters 3, 4, and 5. The following section presents the conclusions drawn from this work.

The implicit approach is found to have certain advantages over the explicit approach in the sense of flexibility and its ability to link with any regression model and optimization solver. However, the implicit approach involves manual work that is tedious and time-consuming. The reason for this is that no available technique can be employed to couple the simulation model and optimization method completely, because the decision variables are not explicitly expressed in the governing equations for the groundwater-flow and solute-transport system. On the other hand, the explicit approach has an advantage in that it requires no manual steps to obtain an optimal solution. Also, the procedure integrates the simulation model within the direct optimization method, so that it can be executed in single run of the model.

The overriding advantage of the explicit approach over the implicit one is anticipated when the results from both models are compared. However, this is not altogether obvious, because the optimal solution of the explicit approach is only marginally better than that of the implicit approach. In addition, this optimal solution is obtained by re-running the explicit model, while narrowing the range

of possible solutions. It is important to note that the optimal solution obtained from the implicit approach cannot be improved by following the same strategy.

Even though the difference between the values of optimal solutions obtained from the two approaches is not of major significance, the explicit approach demonstrates its superiority over the implicit approach in terms of pursuing the optimum solution, yet avoiding the need for derivative information. The application of these two models to a real-world problem might result in different solutions, depending on the cost functions and the equations relating the salt concentration-decision variables used in the formulations. The small proportional difference in the final construction cost arrived at in this work may represent a significant amount of money in an actual design for a real-world problem.

8.2 Contributions of This Research

The main contribution of this research is the unique application of a combined groundwater-flow and solute-transport simulation model with the gradient-based optimization to the control of seawater intrusion into coastal aquifers. Another significant contribution of this research is the introduction of a fully linked simulation-GA model for seawater intrusion control. Many studies have employed coupling simulation and optimization models [e.g. Haimes, 1977; Bisschop et al., 1982; Gorelick et al., 1984; and Wagner and Gorelick, 1987]; however, no work has heretofore been carried out on identifying an optimal design for a subsurface barrier to control seawater intrusion. In the present study,

two simulation-optimization models were developed to identify an optimal design for a subsurface barrier. Therefore, based on available literature, this thesis presents a seminal application of a coupled simulation-optimization model to obtain an optimal design for a subsurface barrier to control seawater intrusion.

Although the GA approach to water resources management problems has undergone rapid developments since the 1980s [Wang, 1991], the validity of the application of GA to the problem of controlling seawater intrusion has yet to be recognized. Integrating the groundwater-flow and solute-transport model, SUTRA, with the GA-based optimization technique eliminates the manual steps involved in the implicit simulation-optimization model.

For seawater intrusion problems in coastal areas, methods such as the distribution of pumping patterns are often considered and applied. Conversely, the control of seawater intrusion, using a subsurface barrier, is often overlooked because of high costs of construction. Although the technology of barrier construction has advanced considerably in recent years, it is still in the process of being developed. This continues to make the subsurface barrier less attractive than other extant methods of controlling seawater intrusion. The use of the explicit simulation-optimization model developed in this thesis should reduce barrier construction costs, making development of the barrier economically viable, and attractive.

8.3 Recommendations

In the following sections, possible improvements in the implementation of implicit and explicit approaches are discussed. In the last section, suggestions for further research, including a brief discussion of other problem areas in which these approaches can be applied, are discussed.

8.3.1 Implementation Improvements

The hypothetical coastal aquifer problem with 231 nodes and 200 elements, is an appropriate simulation grid size for the purpose of developing methods that seek optimal design of a subsurface barrier for seawater intrusion control. A similar scale has been used by a number of investigators from Henry [1959] to Voss [1984]. Computational time for a domain of that size is manageable and the solution is tractable. However, the domain of coastal aquifer for such small size is not practical for solving a larger scale problem.

Two ways in which the implementation of the explicit models developed in this study could be improved are described below. These improvements are intended to increase the execution speed with respect to the explicit model in order to find an optimal solution for this scale, and to anticipate a real case problem, which is always on a larger scale. The priority given to the explicit model is obvious since this model demonstrates its superiority both in searching for the optimal solution and the simplicity of its implementation.

A large amount of computation time in model execution is expended in running the simulation model. This simulation run is for a combination of decision variables randomly chosen by the GA as an optimization technique. Without running the simulation model, the fitness function cannot be evaluated. Therefore, it is highly desirable to reduce the number of executions of the simulation model. The reduction in execution time is achieved by eliminating identified infeasible solutions. Some investigators believe that eliminating infeasible solutions in the approach will reduce the chances of finding the optimal solution. This is especially true when the ratio of feasible to infeasible solutions is small. However, this argument becomes weak for the problems requiring a simulation to check for a violation, such as the problem addressed in this thesis. Moreover, the computational time required for simulation is more dominant than that of the GA. Hence, the number of simulation runs has to be reduced in order to develop an efficient explicit simulation-optimization model.

The technique proposed for reducing this number is to not execute the simulation model if a new combination of decision variables generated by the GA is found to be an infeasible solution. The infeasibility of the solution resulting from the new combination can be identified by comparing it with the corresponding value of the closest infeasible solution to the feasible one from the previous generation.

For example, if the subsurface barrier with the width of 0.08 m, hydraulic conductivity of 4.7×10^{-4} m/s, and location of 0.6 m from the seashore is the closest infeasible solution to the feasible region from one generation, a barrier

thinner than 0.08 m, more porous than 4.7×10^{-4} m/s and closer than 0.6 m from the sea, is apparently in the infeasible region. If one of the three values goes in the opposite direction, for instance the width of 0.082 m (wider than the barrier of 0.08 m), the routine in the GA calls the simulation model and it has to be run. Therefore, this suggested improvement is not an elimination of all infeasible solutions, but a reduction of simulation runs that result in a solution further from the optimal solution. Considerable savings in computational time can be obtained if this GA-based optimization method is tailored to this improvement.

The second way to improve the performance of the explicit model is to generate a homogeneous grid for the barrier and the rest of the aquifer system domain. The discretization will then be based on the finest grid within the model domain. In this case, the system has to be discretized, based on the grid in the subsurface barrier. Of course, a finer grid system increases the burden on the computational resources. It may, however, lead to better results because a fine homogeneous grid system allows the proper values of initial conditions to be specified for all simulation runs. The existing models that generate different grids for different simulation runs solve for groundwater-flow and solute-transport, based on approximate values of initial conditions. Approximated initial conditions do not cause a major problem in the final results, but the computation using this type of approximation is less desirable than the one proposed here as being superior.

8.3.2 Extensions

A number of extensions could be made to the implicit and explicit simulation-optimization models proposed in this study. A few of these extensions and other areas of groundwater contamination are discussed.

The current study deals with a control method in a confined coastal aquifer. However, seawater intrusion problems are more prevalent in unconfined coastal aquifers. Therefore, unconfined aquifers are ideal systems to study with the help of these two simulation-optimization models. The difficulties involved with any application of linked simulation-optimization models to an unconfined aquifer system are that: (1) the specific discharge and water table vary from point to point within the aquifer, and (2) the location of the water table, which serves as a boundary in the aquifer is not known a priori. Since all nodes and elements at the boundary change with the time interval, it is referred to as a moving boundary problem. Including this moving boundary into the models creates the problem of updating the solutions at every iteration and time step, and of handling difficulties associated with the change in the grid size in the horizontal direction. Hence, modeling with a moving boundary is not carried out in this research because of the additional time required for computation. Extensive further work would be required to model an unconfined coastal aquifer with a moving boundary.

Further potential scope for future work would be investigating a partially constructed subsurface barrier. The barrier is not constructed to the point where an impervious layer is encountered. Such a structure, of course, allows some salty water to flow underneath the barrier. The idea of allowing salty water to

percolate through a completely constructed barrier was presented in both approaches developed in this work. The idea is to maintain salt concentrations below the permissible levels. It is possible that a partially constructed barrier, which would reduce the construction cost, might provide the same result, and the idea would be worth pursuing.

The scope of this thesis is limited to a single objective optimization problem. An expanded model, one that considers multiple objective optimization problem. The multiple objective optimization entails finding a set of solutions along the trade-off curve/surface between the objectives. For the seawater intrusion control problem discussed in this thesis, three important objectives are to minimize the width of the barrier, to maximize the value of the hydraulic conductivity, and to minimize the distance of the barrier from the seashore. In order to accommodate such a problem, major extensions to the explicit approach developed in this thesis has to be developed.

For many years, pump-and-treat systems seemed to be the only effective method for controlling plumes of contaminated groundwater. The applicability of the systems is questionable when energy for pumping water and operating water treatment systems, as well as maintenance and monitoring costs, are considered. For a long-term remediation project, pump-and-treat systems are expensive; therefore, the funnel-and-gate system was advocated as an alternative method [Starr and Cherry, 1994]. Since then, the concept that combines cutoff walls (flow-directing funnel) and an in situ reactor (located in the gate) has been studied and developed. The details of the latest development of this type of

reactive barrier can be found on the website, <http://www.rtdf.org/public/permbarr/barrdocs.htm>, and in the publication, Ground Water Currents, June 2000, Issue No. 36. This technology is believed to be more competitive than pump-and-treat systems when the explicit model developed in this study is employed. The explicit model, with modifications to account for reactive parts of the system, can be used to reduce the construction cost further, and to obtain an overall optimum design of a subsurface reactive barrier.

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Appendix

SUBSURFACE BARRIER

This appendix provides the following details about subsurface barriers: definition, history, physical aspects, material, and latest developments of the subsurface barrier in Japan.

A.1 Definitions

Subsurface barriers may be used for several purposes, such as controlling saltwater intrusion, storing fresh water in aquifers, and directing groundwater contaminant plumes, as well as simultaneously treating the contaminants in situ. The *subsurface barrier* itself has many other synonyms, such as groundwater dams, underground reservoirs, subsurface dams, sand storage dams, and groundwater storage dams, just to name a few. All terms with the word 'dam' and 'storage' have the connotation of storing fresh water in natural aquifers. The word 'barrier' suggests that the underground structures are installed for the purpose of halting, confining, containing, directing, or even stopping contaminant plumes.

The term *contaminant plumes* are used here to cover a broad field of groundwater contamination. A contaminant is any dissolved solute or nonaqueous liquid that enters groundwater as a consequence of human activity. When the contamination occurs near the sea, another form of groundwater

contamination, i.e., the seawater wedge formed at the freshwater-saltwater interface is encountered. The dissolved solute in this case is salt, and its presence is the reason why seawater intrusion is treated as a special case in groundwater contamination.

The following section describes the definitions of each term. A subsurface barrier is defined as a semi-impermeable underground structure, which rests on an impervious stratum for controlling groundwater contamination, including saltwater intrusion, or for increasing the capacity for storage of usable groundwater within the aquifer. A groundwater dam or subsurface dam or underground storage dam is a dam built across a narrow underground valley for the purpose of storing water in aquifers. These underground structures utilize the damming-of-water technique. Although the subsurface barrier is developed more for controlling freshwater contamination by seawater, ultimately it raises the water table behind the barrier. Raising the water table means also augmenting the storage capacity of the aquifer.

An underground reservoir is constructed for reserving or storing water in aquifers, but is not necessarily an underground dam. The focus of this technique is how to employ artificial recharge to save water in rainy seasons for use in dry seasons.

The final term is “sand storage dam”, which is a dam constructed across a nonperennial stream in order to create a small groundwater basin. The dam itself impounds water in sediments, in this case, sands. The sands behind the dam

accumulate in the rainy season and form a sandy aquifer with high hydraulic conductivity.

Depending upon the location of the dam, there are two major groups of groundwater dams. The first group is the sand storage dam, which is built across the stream below the ground surface. The second group includes all types of groundwater-retaining structures mentioned above, except the underground reservoir. All are constructed below ground level.

A.2 Background

The technique of damming groundwater flow in aquifers to increase availability of water is not new in human civilization and was practiced as far back as the Roman Empire in Sardinia (Hanson and Nilsson, 1986). It is also reported that ancient civilizations in North Africa used the system for conservation purposes. Lately, groundwater dams have been built in Southern and Eastern Africa, India and Japan as well as, most recently in Thailand (at the feasibility study stage).

In developments such as those recorded in Sardinia and North Africa and reported in South and East Africa and India, groundwater dams are constructed on a very small scale and with very simple technology. They serve small communities and are built by members of the community itself. The height of the dam ranges from 3 meters to 10 meters. The structure is quite plain and the material comes from surrounding areas. These groundwater dams are used only for increasing the groundwater storage.

Considerable attention has been given to development subsurface barriers as a strategy for controlling saltwater intrusion. Adoption of the technique for controlling seawater intrusion indicates that the problem exists on a larger, regional scale. Regional coastal aquifer problems deal with tens or even hundreds of kilometers in length and tens of meters in depth. Such a problem scale demands more advanced technology to effectively address it. Todd [1980] includes barrier technique with four other approaches. Inherent weaknesses revealed are the unavailability of adequate technology and the inability of proposed structures to withstand earthquakes.

Recently, the subsurface barrier method has become more attractive. Nagata et al. [1993] reported on 11 subsurface barriers in Japan. Some were built to increase groundwater storage, while others were constructed for seawater intrusion control. The identification of 40 potential subsurface barrier developments in Ryukyu Island is also reported. The use of subsurface barriers is not only for the two stated purposes, but also for directing, trapping and treating groundwater contaminants in situ. Newman [1995] reports on subsurface barrier material zeolites treated with the surfactant hexadecyltrimethylammonium (HDTMA), which may trap many types of organic and inorganic contaminants. This invention is excellent, not only for trapping contaminants, but in allowing water to pass through the barrier. Burgess [1995] reports the use of montan wax for the horizontal barrier. Continuing the pros and cons of the appropriate approach to verify the effectiveness of the subsurface barrier, the Department of Energy (DOE) of the United States attempts to seek verification technology for

detecting discontinuities in the barriers [Shannon, 1995]. This technology is not yet available. Identification of discontinuities in subsurface barriers of injected slurries of polymers or montan wax and interlocking sheet piling is very important for containing hazardous contamination, such as radioactive waste. Moreover, the use of the subsurface barrier becomes a prospective solution to unresolved problems associated with radioactive waste in Yucca Mountain, as an alternative to the proposed geologic repository, pursuant to serious reservations about the reliability of the latter [North, 1997]. Although subsurface barriers have been used for many purposes, the present study attempts to focus on one purpose, namely control of seawater intrusion.

A.3 Physical Setting

In general, a subsurface barrier can be applied in arid and semiarid areas where there is a need to conserve scanty rainfall. Also, areas with a monsoon climate could benefit from installation of a the subsurface barrier designed to retain surplus water during rainy seasons for use during dry periods. The subsurface barrier becomes more valuable where a groundwater basin is near the sea, a condition which may harbor seawater intrusion problems.

The practical value of a subsurface barrier depends upon topographical, geological, hydrological and physical factors of the barrier site and of the aquifer whose capacity is to be enhanced.

Topographical conditions determine the technical aspects of subsurface barrier construction. The first, and one of the most important steps in the investigation

of a subsurface barrier project, is reconnaissance to identify the most favorable site. Hence, the topography should be studied exhaustively. The volume of storage that can be accommodated is also determined by the topographical conditions. For this reason, an area with flat topography is not economically favorable, because of inescapably large extent of the barrier. Conversely, an area with steep topography does not proffer a favorable cost-benefit ratio because even a high barrier will result in only modest storage volume. Hanson and Nilsson [1986] reported that areas with 1-5 % slope are the most feasible.

An ideal geologic condition for the construction of a subsurface barrier occurs when a confined aquifer is underlain by bedrock or unconsolidated formations with low hydraulic conductivity, or some other solid impervious layer. Aquifer aggregates containing a substantial amount of sand and gravel, weathered zones, and alluvial/colloidal layers favor a large storage volume. In addition, high effective porosity and sufficient thickness aquifer are considered highly advantageous. Bedrock consisting of granite, gneiss, and quartzite are positive indications of a water-retention aquifer base.

Groundwater basins confined in long, narrow valleys are hydrogeologically ideal for the construction of a subsurface barrier because of the favorable environment with respect to construction costs. The watershed's yield should be properly assessed to ascertain the prospective benefits of a subsurface barrier project.

For projects in developing countries, involving small-scale barrier, haulage of the bulky construction materials represents a large proportion of the total cost

[Hanson and Nilsson, 1986]. Therefore, the material selected for the main construction with respect to the barrier site should be available within surrounding areas.

Identification and exploitation of the above important characteristics are futile if human activities contributing to groundwater contamination persist. Groundwater must be free of contamination if it is to meet its potential.

A.4 Materials

The material used for the subsurface barrier is one of the most important factors in its construction, not only because the material will be installed underground, but also because it must successfully address three major problems. Although one material rarely possesses the properties needed to fulfill all requirements, cement grouting, sheet piling, puddled clay, emulsified asphalt, silica gel, calcium acrylate, plastics, and montan wax, singly or in combination, may be adequate.

Monitoring of seawater leakage should be considered. Leakage is often used as the measure of barrier effectiveness. Leakage at the bottom of the barrier releases undesirable substances, such as fertilizers, pesticides, and other chemical wastes of agricultural and industrial origin. It is important to note that this leakage flow should have a seaward gradient. In coastal aquifers where the quantity of such dangerous materials is negligible, there is no need to provide for such leakage flow through the barrier.

Todd [1980] reported that there are three major obstacles to implementation of this method of controlling seawater intrusion: construction cost, resistance to earthquakes, and chemical erosion. Since the installation of a physical barrier involves underground work, the cost of construction becomes fairly high compared to other methods. Although many groundwater engineers believe that construction cost is a major constraint to building a barrier, Japanese engineers have succeeded in reducing the cost by utilizing a specialized cement-grouting technique. The effect of earthquakes on such barriers has not been intensively investigated. Chemical erosion is related to the aforementioned leakage problem. Control of subsurface outflow to achieve a balance of salinity is difficult to monitor.

If the subsurface barrier is effective, drawdown is minimized. Thus the barrier permits the use of large amounts of stored fresh water. The steepening of the seaward gradient can be achieved as well. Basically, this method delays or halts saltwater intrusion into the freshwater zones and therefore enhances the groundwater storage capacity of the aquifer.

Subsurface barriers for controlling seawater intrusion are considered the best solution to the problem [Banks and Richter, 1953 and EPA 1987]. They provide permanent control. In the long run, low operation expenses and maintenance costs may balance out the high cost of constructing subsurface barriers.

A.5 Subsurface Barrier Development in Japan

Japan is the most advanced country in the development of subsurface barriers. It was initiated by Professor Dr. K. Kachi in the 1940s. Kachi proposed a subsurface dam in an alluvial fan for an irrigation system in Nasunogahara, Tochigi Prefecture. His proposal was never implemented, but his idea attracted many engineers. However, the Kabashima Subsurface Dam in Nomozaki, Nagasaki Prefecture was completed in 1974. It was designed to meet the demand for water in Nomozaki town. The dam of 25 m height, 60 m crest length and 20,000 m³ storage capacity became the first subsurface barrier in Japan.

Since then the Ministry of Agriculture, Forestry, and Fisheries (MAFF) of Japan has given subsurface barriers more attention. The main problem faced by MAFF is the shortage of water for irrigation purposes in the Southwest of Japan. Areas such as the Ryukyu and Amani Islands, Okinawa and Kagoshima Prefectures suffer from serious drought in summer. Thus, agricultural production from the region has been limited. The average annual rainfall is 2200 mm, an amount considered as abundant. Another important factor is that coral reef limestones called the Ryukyu Limestones dominate the aquifers in the region. The main characteristic of the Ryukyu Limestones is high hydraulic conductivity with large porosity. These properties present a high storage capability. These two factors indicate that water supplies should be adequate, but runoff is excessive.

MAFF studied and investigated potential locations and as a result, 40 potential barrier sites were identified. The majority of the dams were designed

for irrigation. Few are for domestic water supply. The subsurface dams function to increase the storage capacity of unconfined coastal aquifers and/or to prevent seawater intrusion.

In general, subsurface barriers planned by MAFF are groundwater storage barriers and/or seawater intrusion barriers. The former increases the water table of unconfined aquifers. The latter prevents seawater intrusion inland, in addition to augmenting the groundwater storage capacity. Based on the conditions at the 40 sites, MAFF identified two dominant types of aquifers, Ryukyu limestone in Okinawa, and alluvial deposits in other islands. To anticipate the response to subsurface barriers, and to identify the most effective construction method with respect to each to the type of aquifer, two experimental subsurface barriers were built, of which the first was Minafuku Dam on Okinawa, and the second, Nakajima Dam on Ehime.

The Minafuku experimental dam is the first subsurface barrier under this MAFF project. The aquifer is limestone. The aspects to be confirmed by the Minafuku experiment were (1) the hydrogeological conditions created by the design, (2) the selection of construction methods for building the cutoff wall, (3) the analytical water balance model, (4) the design of intake facilities, and (5) storage management and groundwater quality. In the period of 1980-1983 all aspects, such as technical problems and estimation of storage volume were evaluated.

The Minafuku experiment answered the questions raised before initiating the project. The viability of subsurface dams was finally confirmed. The results

indicated that the water table increased even before the construction was completed. After completion of the experimental work on Okinawa, the development of the Nakajima project resumed.

The Nakajima Dam was built to test the cutoff wall technology as applied to alluvial deposits. This dam was constructed prior to the Komesu Dam, which is much larger. Both were constructed for controlling seawater intrusion. The difference is, Nakajima was built in an alluvial aquifer, and Komesu in a limestone aquifer. In situ soil mixing technology called the Soil Mixing Wall (SMW) method was tested at the Nakajima site. The detailed procedure will be covered in the section on construction methods. Two main points had to be confirmed. The first was the extent of fluctuation of the water table before and after construction. The second was to ascertain the ability of the cutoff wall to retard seawater intrusion under conditions of continuous withdrawal in the freshwater zone. The results indicated that the water table rose during the construction work, and that the subsurface barrier had the capability of halting seawater intrusion inland under continuous freshwater withdrawal conditions.

The subsurface barrier developments proposed by MAFF consisted of four major projects: Miyako Irrigation Project, Okinawa Hontoh-Nanbu Irrigation Project, Kikai Irrigation Project and Izena Irrigation Project. These projects are based on their technical feasibility and the magnitude of demand for water. Each project has its own characteristic so that a construction method specific to each is required. The demand for irrigation water varies from project to project. The following is the summary of the MAFF projects.

(1) Miyako Irrigation Project: Miyakojima Island is located in the southern part of the Ryukyu Islands. The geology of the island is dominated by Ryukyu Limestone and Shimajiri Mudstone. Ryukyu Limestone, with a hydraulic conductivity of 0.35 cm/s and porosity of 10% is an excellent aquifer. In contrast, Shimajiri Mudstone with very low hydraulic conductivity, acts as an impermeable layer. Several parallel valleys were formed in Shimajiri Mudstone. This creates channeled underground flow.

Three subsurface dams were constructed. The first was Minafuku Dam on Okinawa. It was completed in 1979. The dam, of 16.5 m height, 500 m crest length and 700,000 m³ storage capacity, is designed to support irrigation of 8,500 hectares of sugar cane on Miyakojima Island. The second is Sunagawa Dam completed in 1993. The dimension of this dam is 49.0 m height, 1,835 m crest length and 9,500,000 m³ storage capacity. It was designed to support to the same irrigation system as Minafuku Dam. The expected groundwater storage was fully realized in 1995. The third is Fukusato Dam, on which construction is under way. By completing Fukusato, with a height of 52.0 m, crest length of 1,720 m and storage capacity of 10,500,000 m³, the water requirement of the 8,500 hectare irrigation system will be fully met. In the original plan of MAFF, Bora Dam, with 22 m height, 2,850 m crest length and 1.6 million m³ storage capacity was in scheme, but, for some reason was not implemented.

(2) Okinawa-Hontoh-Nanbu Irrigation Project: This project is also on Okinawa Island, northwest of the Miyako Irrigation Project. The geology of southernmost area of Okinawa Island is similar to that of Miyakojima Island, with Ryukyu Limestone overlain by Shimajiri Mudstone. This project has three subsurface barriers, creating groundwater interception adequate for the irrigation of 1,400 hectares. Komesu Dam, the largest, has a reservoir capacity of 3.255 million m^3 . Its height of 81.0 m and crest length of 2,489 m make it the highest and longest such installation anywhere. Komesu Dam also controls seawater intrusion. Seawater intrusion is a major problem in Komesu because much of the freshwater aquifer is situated below sea level. The Komesu Dam is designed, not only for protecting freshwater from the invasion of seawater, but also for increasing the storage capacity. The project, commenced in 1992, was not yet (1997) completed. The other two dams, Nashiro and Giiza, have capacities of 0.397 and 0.438 million m^3 , respectively. The height and crest length of the cutoff walls are 25 and 906 m, respectively, for Nashimo Dam, and 51.0 and 955 m, respectively, for Giiza Dam. Both are in the design stage.

(3) Kikai Irrigation Project: Kikai Island is situated 30 kilometers to the east of Amami-Oshima Island, Kagoshima. The geological conditions here are similar to those at the aforementioned project areas. The irrigated area of the Kikai Project is 1,642 hectares, with water to be supplied from the reservoir, whose capacity will be 1.7 million m^3 , enough to provide ~ 100 mm of supplementary moisture. Kikai Dam is the only subsurface barrier within this project. Seawater

intrusion takes place as well in the Kikai valley-shaped groundwater basin. The Kikai Dam is 36 m high and 2,190 m along the crest.

(4) Izena Irrigation Project: Izena Island is 20 kilometers to the north of Okinawa Island. The irrigated area of this project is 660 hectares, fed from the Izena reservoir whose capacity is $0.95 \times 10^6 \text{ m}^3$ (~ 140 mm). The hydrogeological conditions of Izena Island are different from those of the other project areas. The aquifer behind Izena Dam is an alluvial deposit (sand, silt and conglomerate), and its basement is sandstone of Mesozoic or Paleozoic age. The planning and design of this project are still (1997) under way.

As the concept of subsurface barrier is widely accepted in Japan, several other subsurface barriers that are not related to the four irrigation projects above in various stages of development, from feasibility studies to completed construction. Their purpose is to augment agricultural and urban water supplies. The following table (Table A.1) lists these subsurface-barrier projects. Some subsurface barriers, currently at the stage of site selection, are not included.

Table A.1: Summary of subsurface barriers developed by MAFF Japan

No	Name	Location	Use	H (m)	CL (m)	Storage Capacity	Aquifer	Construction Remarks
1	Nasunogahara	Tochigi	Irr.	-	-	-	Alluvial	Not Realized
2	Bora	Okinawa	Irr.	22.0	2850	1600.0	Limestone	Not Realized
3	Kabashima	Nagasaki	WS	25.0	60	20.0	Alluvial	Completed 1974
4	Minafuku	Okinawa	Irr.	16.5	500	700.0	Limestone	Completed 1979
5	Tunegami	Fukui	WS	18.5	202	73.5	Alluvial	Completed 1983
6	Tengakuma	Fukuoka	WS	12.5	129	17.5	Alluvia	Completed 1988
7	Waita	Nagasaki	WS	11.0	105	12.0	Alluvial	Completed 1992
8	Nakajima	Ehime	Irr.	24.8	88	27.0	Alluvial	Completed 1993
9	Sunagawa	Okinawa	Irr.	49.0	1835	9500.0	Limestone	Completed 1993
10	Kikai	Kagoshima	Irr.	36.0	2190	1681.0	Limestone	Under Way
11	Komesu	Okinawa	Irr.	81.0	2489	3255.0	Limestone	Under Way
12	Fukusato	Okinawa	Irr.	52.0	1720	10500.0	Limestone	Under Way
13	Kanjin	Okinawa	Irr.	52.0	1088	1580.0	Limestone	Under Way
14	Nashiro	Okinawa	Irr.	25.0	906	397.0	Limestone	Design Stage
15	Giiza	Okinawa	Irr.	51.0	955	438.0	Limestone	Design Stage
16	Izena	Okinawa	Irr.	-	-	950.0	Alluvial	Design Stage
17	Nakahara	Okinawa	Irr.	45.0	2710	13400.0	Limestone	Design Stage

Note: Storage Capacity in thousand cubic meters.

Irr. = Irrigation purpose.

WS = Water Supply of a town.