국문초록

해안지역 지하수 개발 및 관리를 위한
최적모델에 관한 연구

Optimization Model for Development and Management of Groundwater in Coastal Areas

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최적모델은 흡수 모델과 최적화 기법으로 구성한다. 밀도류나 해수첨
투와 같은 해안대수층의 특성을 고려한 해안지하수 흡수 모델들은 많다. 하
지만 효율적인 지하수 개발 및 관리와 관련된 최적모델은 선진국에서도 개발
및 초기 적용 단계이다.

이 연구에서는 해안지역의 지하수 개발 및 관리를 위한 4단계 전략을 제
시하고 처음 두 단계에서 의사결정 지원시스템으로 활용될 수 있는 최적모
델을 개발하였다. 4단계, 인공합
양 그리고 지하댐으로 구성된다. 이 전략은 단계가 진행됨에 따라 개발가
능한 지하수량이 증대되며 이에 따라 비용도 증대된다. 이 전략은 지하수
수요에 따라 취해야하는 방법을 제시한다.

최적모델은 지하수 환경과 기존 관개의 관리를 보호하며 최대 양수량과
최적 위치를 선정한다. 또한 해수세기의 제어를 위한 담수의 주입 또는 해
수 양수의 최적해를 구하는데 사용된다. 개발된 모델의 최적해의 주요 매개
변수인 함양율, , .
최수윤은 함양물에 민감하고, 관정의 최적위치는 수리전도도에 민감함을 보았다. (1.대상자) 

연구 결과 최적모델은 수학적 정확성을 확보하였으며, 3[1]월 호흡에서도 우수하게 적용되는 것으로 나타났다. 본 모델은 해안지역의 지하수 개발 및 관리에 유용한 도구가 될 것으로 판단한다.

주요어: , , , , , , 4

감도 분석, , , 검정
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I. Introduction

1. Objectives

The objective of this work is to develop an optimization-simulation model that can be used in decision making processes of development of groundwater resources in coastal areas. Common goals of groundwater developments may be the maximization of net freshwater pumping rates and the minimization of adverse impacts of pumping to groundwater environments. To achieve the goals pumping rates and well locations must be distributed optimally. The optimization-simulation model will be able to provide the optimal solution.

For the model to be applicable to real-world problems consideration must be given to following aspects. Groundwater flow in coastal area is subject to the influence of subsurface saltwater. Therefore, the simulation model must be able to correctly model the flow system. The optimization technique must be able to deal with complex functions which may be either convex or differentiable.

Depending on demand levels different methods may be used to develop groundwater in coastal areas. Logical steps are proposed for cost effective development. The optimization model can be a useful tool in implementing some of the steps.

Optimal solutions depend on a number of parameters, physical and numerical. Sensitivities of the model solutions to major parameters are investigated to guide potential users of the model. When a new model is developed, the model needs to be verified for its mathematical accuracy and
validated for its applicability to real problems. Verification is conducted numerically and validation is performed using experimental data. Sand tanks are used to collect experimental data. Realistic optimization problems involve a distribution of pumping wells in the horizontal plane. Three-dimensional salt water intrusion experiments are conducted to study the movement of freshwater-saltwater interface due to the distributed pumping.

2. Necessities

Recently, nature of water management, development and planning is changing. It is called “the changing water paradigm” (1). And the recognition on water also has changed gradually. After Earth Summit in 1992, the roles of human and water for ecosystem functions have taken on the issue. Although there are many concepts and efforts for the sustainable development to harmonize human with nature, only ecosystem always receives all by-products of human activities.

Excessive groundwater pumping is one of these by-products of human activities. It affects wetlands and other surface water ecological systems by interfering in the natural hydrologic cycle. Also it may cause saltwater intrusion in many coastal areas. It makes portions of these aquifers unusable for human consumption and groundwater quality degradation by salinization. These are two of the most serious threats to fresh groundwater resources in coastal areas.

Korea is confronted with water shortage. The total amount of annual precipitation per capita is just 2,705 m³ which is about 10% of the world average. And the amount of annual renewable water resources per captia is
1,550 m³. Because of this value, Korea is classified as a water-short country (2).

One of major topographic features of Korea is peninsula with sea on three sides. The number of Counties and Sub-counties belong to the coastal area in Korea is 322. The number of Counties and Sub-Counties in coastal areas is 23% of the whole number of these in Korea. A population of 2.5 millions live in coastal areas and that of 0.87 millions live in island areas (Table 1).

<table>
<thead>
<tr>
<th>Coastal Area</th>
<th>Administrative Status</th>
<th>Population</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>21 Cities, 29 Districts, 322 Counties and Sub-counties</td>
<td>2.5 millions</td>
</tr>
<tr>
<td>Island Area</td>
<td>Unit</td>
<td>Area (㎢)</td>
</tr>
<tr>
<td></td>
<td>3,170</td>
<td>3,786</td>
</tr>
</tbody>
</table>

Average water supply rate in these areas is about 40%, which is less than half of the national average (87.1%). The average water supply rate of seven metropolitan city is 98.2% (Figure 1). And water shortage in coastal areas is estimated about 0.15 billion m³/year, most of the coastal areas expected to have the restricted water supply during drought periods (3).
In coastal areas, freshwater is rather scarce and the dependence on groundwater is about four times larger than the national average (Figure 2) due to insufficient surface water and/or inadequate water distribution infrastructure.

The groundwater development and management of coastal area must differ from those of inland area because of the condition exerted by the sea. Therefore an overexploitation or inadequate development of a coastal
aquifer can cause saltwater intrusion. This may not only be a threat for the public and industrial water supply but also for agriculture and horticulture.

Bear (4) indicated that the intensive extraction of groundwater has upset the long established balance between freshwater and saltwater potential, causing encroachment of saltwater into freshwater aquifers. As a large proportion of the world’s population (about 70%) dwells in coastal zones, the optimal exploitation of fresh groundwater and the control of saltwater intrusion are the challenges for the present-day and future water supply to engineers and managers.

To overcome these problems in coastal areas of Korea, the optimization model has been developed (5). This model is enhanced in this work for considering saltwater pumping rate to control saltwater wedge and the optimal location of the wells.
3. Contents and Scope

This dissertation presents an optimization model for groundwater development in coastal areas. In the first part the four-stage strategy and details of the optimization model for sustainable coastal groundwater development are presented. In the second part results of the sensitivity analysis, verification and validation of model are described.

Optimization model generally consists of simulation model and optimization technique. Chapter 2 introduces literature survey about coastal groundwater flow and model, general optimization technique and optimization model. Experimental studies for saltwater intrusion are investigated.

A four-stage strategy for cost-effective groundwater development in coastal areas is introduced in chapter 3. Appropriate groundwater development methods are proposed for the demand level.

Chapter 4 presents the optimization model. The structure and details of this model, the features of the simulation model and the optimization technique are introduced. In this work a two-phase model is used as the simulation model and a parallel genetic algorithm is used as the optimization technique.

Chapter 5 discusses the sensitivity analysis for the optimization model. In this chapter the influence of hydraulic conductivity, recharge rate and aquifer thickness on the ratio of optimal pumping rate and optimal well location is introduced.
Chapter 6 discusses the verification of the optimization model. The optimization model is applied to a hypothetical unconfined aquifer. Example problems that belong to the first two steps of the cost-effective groundwater development strategy are used. Optimal solutions for freshwater pumping rate, saltwater pumping rate and well location are verified.

Chapter 7 discusses the saltwater intrusion experiments and validation of the optimization model. Three measurement methods are discussed for measuring distributed freshwater/saltwater interface. Experiments are conducted in two sand tanks. One is a cross-sectional sand tank and the other is three-dimensional sand tank.
II. Literature Survey

1. Governing Equations for Coastal Groundwater Flow

The governing equation for the fresh groundwater flow is restricted to fluid flow with a constant density or to cases where the differences in density or viscosity are extremely small or absent. A general form of the governing equation for the fresh groundwater flow is presented as below (7).

\[ -\frac{\partial}{\partial x} \left( K_x \frac{\partial h}{\partial x} \right) + \frac{\partial}{\partial y} \left( K_y \frac{\partial h}{\partial y} \right) + \frac{\partial}{\partial z} \left( K_z \frac{\partial h}{\partial z} \right) = S_s \frac{\partial h}{\partial t} - R^* \]  

(1)

where \( K_x, K_y, \) and \( K_z \) are components of the hydraulic conductivity tensor [LT\(^{-1}\)]. \( S_s \) is the specific storage [L\(^{-1}\)]. \( R^* \) is a general sink/source term that defines the volume of inflow to the system per unit volume of aquifer per unit of time [T\(^{-1}\)]. \( h \) is the groundwater head [L]. \( x, y, z \) are the Cartesian coordinates [L] and \( t \) is time [T].

Although the difference in densities between fresh water and saltwater is only 2.5%, it has a significant effect on piezometric heads and thus on the groundwater flow regime. There are three different approaches in dealing with freshwater and saltwater flow.

- sharp interface with stagnant saltwater
- sharp interface with flowing saltwater
- solute transport and flow with concentration dependent density
1) Sharp-Interface with Stagnant Saltwater

This approach is commonly called the Badon Ghyben-Herzberg principle (BGH). In this approach a sharp interface is assumed between freshwater and saltwater. The position of an interface between fresh and saline groundwater is based on difference between fresh and saline groundwater heads (Figure 3).

![Figure 3. The Badon Ghyben-Herzberg principle: a fresh-salt interface in an unconfined coastal aquifer](image)

BGH principle can be expressed as follows:

At the interface,

\[
\text{Saline groundwater pressure} = \text{Fresh groundwater pressure}
\]

\[
\rho_s H g = \rho_f (H + h) g \iff \rho_s H = \rho_f H + \rho_f h
\]  

(2)
\[ h = \frac{\rho_s - \rho_f}{\rho_f} H \] (3)

The equation is correct if there is only horizontal flow in the freshwater zone and the saline water is stagnant. Though the position of the interface is not correct at the outflow zone, the equation closely approximates the real situation (8).

2) Sharp Interface with Flowing Saltwater

This approach is also called a two-phase flow approach. A sharp interface is assumed between freshwater and saltwater. However, movement of saltwater can be described. Vertical average within the respective flow regime is generally used. This integration implies a hydrostatic condition, i.e., horizontal flow only. With this the governing equations for freshwater flow and saltwater flow become (9)

\[ \nabla \cdot (b_f K_f \cdot \nabla h_f) = b_f S_{f} \frac{\partial h_f}{\partial t} - \sigma \frac{\partial \xi}{\partial t} - Q_f \] (4)

and

\[ \nabla \cdot (b_s K_s \cdot \nabla h_s) = b_s S_{s} \frac{\partial h_s}{\partial t} + \sigma \frac{\partial \xi}{\partial t} - Q_s \] (5)

where \( \nabla \) is the gradient operator, \( \cdot \) inner vector product, Superscripts \( f \) and \( s \) refer to freshwater and saltwater, \( b_f \) and \( b_s \) are the thickness of the freshwater and saltwater zones in aquifer [L], \( K_f \) and \( K_s \) are hydraulic
conductivities with respect to freshwater and saltwater \([LT^{-1}]\), \(h_f\) and \(h_s\) are the hydraulic heads, \(S_s\) and \(S_f\) are aquifer specific storage coefficients in the freshwater and saltwater zones \([L^{-1}]\), \(\theta\) is the effective porosity, \(\xi\) is the height of saltwater-freshwater interface above the datum \([L]\), \(Q_f\) and \(Q_s\) are volumetric fluxes of the freshwater and saltwater due to pumping (or recharge etc). Thicknesses of the freshwater and saltwater zones in aquifer \([L]\) are defined as

\[
b_f + b_s = B
\]

(6)

where \(B\) is saturated thickness which is dependent on the aquifer condition (confined or unconfined)

The interface position is determined from

\[
\xi = \frac{\rho_f}{\rho_s - \rho_f} \left[ \frac{\rho_s}{\rho_f} h_s - h_f \right] = \frac{\rho_f h_s - \rho_s h_f}{\rho_s - \rho_f}
\]

(7)

where \(\rho_f\) and \(\rho_s\) are the freshwater and saltwater densities. Equation (7) means that the pressures of freshwater and saltwater are equal at the fresh/saltwater interface.

3) Solute Transport and Flow with Concentration Dependent Density

In reality there is a smooth transition zone between freshwater and saltwater. The saltwater is characterized with high concentration of total dissolved solids (TDS). The average concentration of saltwater is
approximately 40,000 mg l \( \ell \) (10). On the other hand, concentration of freshwater is generally less than 100 mg l \( \ell \). This concentration affects density of fluid. In this approach freshwater and saltwater are treated as the same liquid, but with varying concentration.

In the transition zone the concentration changes smoothly. The thickness of the transition zone depends on the number of variables such as dispersivity, flow velocity and porosity. When the thickness of a transition zone is small, the flow field may be approximated with the previous sharp interface approach.

The governing equations are composed of one flow equation for groundwater and one transport equation for dissolved solute.

The governing equation for groundwater flow is following:

\[
-\frac{\partial}{\partial x_i} \left[ K_{ij} \left( \frac{\partial h'}{\partial x_j} + \eta \epsilon_i \right) \right] = S_s \frac{\partial h'}{\partial t} + \partial_t \frac{\partial c}{\partial t}, \quad i,j=1,2,3 \quad (8)
\]

where \( K_{ij} \) is the hydraulic conductivity tensor [LT\(^{-1}\)], \( c \) is the solute concentration [ML\(^{-5}\)], \( \epsilon_j \) is the unit vector in the upward vertical direction, \( S_s \) is the specific storage of the aquifer [L\(^{-1}\)], \( \theta \) is the porosity of the porous medium, and \( \eta \) is defined as

\[
\eta = \frac{\rho_f}{c_s (\rho_s - \rho)} \quad (9)
\]

where \( c_s \) is the solute concentration that corresponds to a maximum
density ($\rho_s$), and $h'$ is a reference hydraulic head [L] defined as

$$h' = \frac{-p}{\rho_0 g} + z$$ (10)

where $p$ is fluid pressure [FL$^{-2}$], $\rho_0$ is a reference (freshwater) density [ML$^{-3}$], $g$ is the gravitational acceleration [LT$^{-2}$], $z$ is the elevation above a reference datum plane and the reference hydraulic head is directly related to the true hydraulic head, $h$ [L]. The relationship may be shown to take the form

$$h = \frac{-p}{\rho g} + z = \frac{-\rho_0 h' + z}{\rho} + z = \frac{h' + z \eta_c}{1 + \eta_c}$$ (11)

where $\rho$ is the fluid density [ML$^{-3}$].

The governing equation for solute transport is following:

$$-\frac{\partial}{\partial x_i} (D_{ij} \frac{\partial c}{\partial x_j}) - V_i \frac{\partial c}{\partial x_i} = 6R(\frac{\partial c}{\partial t} + \lambda c), \quad i,j = 1,2,3$$ (12)

where $D_{ij}$ is the apparent hydrodynamic dispersion [L$^2$T$^{-1}$], $V_i$ is the Darcy velocity of fluid [LT$^{-1}$], $R$ is the retardation coefficient, and $\lambda$ is the decay or degradation constant of the solute. For a conservative solute species, $R=1$ and $\lambda=0$.  

13
2. Numerical Models for Coastal Groundwater Flow

Saltwater intrusion problems have been solved by using different methods, ranging from the basic Badon Ghyben-Herzberg principle with the sharp interface models to the more sophisticated theories with the solute transport models which take into account variable densities. The groundwater flow model is always part of any model concerned with the movement of salt-fresh water interface and/or solute transport (11).

There are also many models and studies for coastal groundwater flow (Table 2). The coastal groundwater flow models are classified into two categories. One is sharp interface (two-phase) model and the other is density-dependent flow and solute transport model.

Sharp interface models (two-phase) are based on the assumption that the interface between fresh and saline groundwater is abrupt. In this model coupled freshwater and saltwater flow equations are solved simultaneously. Most coupled two-fluid sharp interface models have been limited to a quasi-three-dimensional model in single layer or a two-dimensional vertical section. But, SHARP (12) and DUSWIM (6) were developed as a quasi-three-dimensional model that allows multiple aquifer layers.

When dispersion is important, it is necessary to solve coupled flow and solute transport equations. Models that simulate density-dependent flow require initial pressure and density distribution. At the beginning of a time step, these initial values are used to generate the first approximation of the flow field. The resulting head values are input to the transport models, which redistribute solute. A new density distribution is calculated from the transport results, ending the first iteration of the first time step. The
second iteration begins with the substitution of the newly calculated densities into the flow model. Iteration is continued until closure is attained. This process is repeated for all time steps (7).

Table 2. The list of coastal groundwater flow and solute transport models

<table>
<thead>
<tr>
<th>Name</th>
<th>Features</th>
<th>Author</th>
<th>Remark</th>
</tr>
</thead>
<tbody>
<tr>
<td>DSTRAM</td>
<td>It is based upon a finite-element discretization of the three-dimensional advective-dispersive transport equations</td>
<td>Peter Huyakorn and Sorab Panday</td>
<td>DDFST</td>
</tr>
<tr>
<td>FEFLOW</td>
<td>It is a finite-element package for simulating 3D and 2D fluid density-coupled flow, contaminant mass(salinity) and heat transport in the subsurface</td>
<td>Hans J. G. Diersch</td>
<td>DDFST</td>
</tr>
<tr>
<td>HST3D</td>
<td>The Heat- and Solute-Transport Program (HST3D) simulates ground-water flow and associated heat and solute-transport in three dimensions</td>
<td>Kenneth L. Kipp</td>
<td>DDFST</td>
</tr>
<tr>
<td>MOCDENSE</td>
<td>Two-dimensional, cross-sectional model for the analysis of saltwater intrusion. It simulates conservative solute transport and dispersion of one or two constituents in a ground-water system with density-dependent flow</td>
<td>W.E. Sanford and L.F. Konikow</td>
<td>DDFST</td>
</tr>
<tr>
<td>SHARP</td>
<td>Quasi-three dimensional finite difference model for simulating freshwater and saltwater flow, separated by a sharp interface in layered coastal aquifers</td>
<td>H.I. Essaid</td>
<td>SI</td>
</tr>
<tr>
<td>SUTRA2D3</td>
<td>Model for 2D or 3D saturated-unsaturated, variable-density ground-water flow with solute or energy transport</td>
<td>Voss, C. I., and Provost, A.M</td>
<td>DDFST</td>
</tr>
<tr>
<td>SWICHA</td>
<td>It is a three-dimensional finite element code for analyzing saltwater intrusion in coastal aquifers. The model simulates variable density fluid flow and solute transport processes in fully-saturated porous media</td>
<td>B. Lester</td>
<td>DDFST</td>
</tr>
<tr>
<td>SWIFT</td>
<td>It is a fully transient, three-dimensional model which simulates the flow and transport of fluid, heat (energy), brine, and radio-nuclide chains in porous and fractured geologic media</td>
<td>GeoTrans, Inc</td>
<td>DDFST</td>
</tr>
<tr>
<td>DUSWIM</td>
<td>It is developed to simulate simultaneously, freshwater and saltwater flow, separated by a sharp interface. And it can handle both single and multiple aquifers separated by confining layers or aquitards</td>
<td>Namsik, Park</td>
<td>SI</td>
</tr>
</tbody>
</table>
Although solute transport models are most rigorous in dealing with groundwater flow in coastal aquifers, practical applications are limited to simple and small scale problems due to the refined data requirements and computing time. Therefore a two-phase model may be more appropriate for analyzing coastal groundwater flow problems.

3. Optimization Techniques

An optimization technique has three key elements: the objective function, the constraints, and the decision variables. Two types of optimization formulations can be constructed with these elements: unconstrained problems, which include an objective function and decision variables, and constrained problems, which contain all three elements (13). Optimization techniques can be classified into two types. One is a local search technique and the other is a global optimization technique.

The optimization technique requires formal definition of the decision variables, the constraints, and the objective function to be optimized. The objective and constraints are translated into mathematical functions of the decision variables to produce the optimization formulation. The formulation is then solved using one of a variety of optimization algorithms such as genetic algorithms, gradient-based nonlinear optimization algorithms, simulated annealing, tabu search, etc. These algorithms are modified and developed for efficient and fast convergence into the optimal solution.

1) Local Search Techniques
Local search techniques are deterministic methods, suitable for the optimization of unimodal (single extreme) functions. These techniques can be classified into two major categories, gradient-based and direct search methods. Gradient methods are applicable only if the analytical expression of the objective function is available and its partial derivatives are easy to compute. Direct search methods are characterized by the fact that they do not compute derivatives; the only information that they need are the values of the objective function.

Local search methods are not appropriate for the optimization of non-convex, multimodal functions. However, because of their efficiency in simple search spaces, their principles are commonly applied in several global optimization algorithms.

2) Global Optimization Techniques

Global optimization techniques aim at identifying the global optimum solution of a function that need not be convex or differentiable. These methods involve the evaluation of the function usually at a random sample of points in the feasible parameter space, followed by subsequent manipulations of the sample using a combination of deterministic and probabilistic scheme. There are a few methods of the global optimization techniques and these methods are briefly introduced in Table 3.
Table 3. Various methods for global optimization technique

<table>
<thead>
<tr>
<th>Method</th>
<th>Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>Deterministic Methods</td>
<td>The uniform grid sampling method is a primitive deterministic approach to the global optimization problem.</td>
</tr>
<tr>
<td>Stochastic Methods</td>
<td>A prespecified number of points at random from the feasible parameter space using any probability distribution function is used.</td>
</tr>
<tr>
<td>Evolutionary and Genetic</td>
<td>The family of evolutionary algorithms is inspired from the mechanics of natural evolution of biological organisms, although these models are crude simplifications of biological reality. Evolutionary programming, genetic algorithms and so on belongs to this method.</td>
</tr>
<tr>
<td>Algorithms</td>
<td></td>
</tr>
<tr>
<td>Simulated Annealing</td>
<td>The method of simulated annealing is based on an analogy with a thermo-dynamical process called annealing.</td>
</tr>
<tr>
<td>The Shuffled Complex Evolution Method</td>
<td>It is a new, heuristic global optimization scheme that combines the strength of the downhill simplex procedure with the concepts of controlled random search, competitive evolution and complex shuffling.</td>
</tr>
<tr>
<td>Tabu Search</td>
<td>The natural system based on human memory process.</td>
</tr>
</tbody>
</table>

4. Optimization Models for Coastal Groundwater

An optimization model generally consists of a simulation model and an optimization technique to obtain an optimal solution. A simulation model is selected according to the characteristics of a given problem, and an optimization technique is selected according to the mathematical characteristics of the optimization problem. Recently, a number of works have been reported on optimization of groundwater development in coastal
areas (Table 4). Objective functions of these optimal models can be grouped into three categories.

1) Optimal withdrawal of groundwater while minimizing the saltwater intrusion [(14),(15),(16),(17),(18),(6), among others].
2) Optimal benefit or cost in groundwater development [(19),(20),(21),(22),(23),(24),(25), among others].
3) Optimal management of groundwater resources (24).

However, most of optimization models concern only one decision variable, either pumping rates or minimum costs. In most real problems, there are two important questions besides cost: where a pumping or injection well should be installed, and how much groundwater can be withdrawn from the well.
### Table 4. Published optimization models

<table>
<thead>
<tr>
<th>Optimization technique</th>
<th>Decision variable</th>
<th>Author</th>
<th>Year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Structured messy GA</td>
<td>Pumping rate</td>
<td>Cheng et al.</td>
<td>2000</td>
</tr>
<tr>
<td>Outer approximation</td>
<td>Pumping rate</td>
<td>Papadopoulou et al.</td>
<td>2000</td>
</tr>
<tr>
<td>method</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bundle trust</td>
<td>Net benefit</td>
<td>Gordon et al.</td>
<td>2000</td>
</tr>
<tr>
<td>method</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GA</td>
<td>Construction cost</td>
<td>Mundzir</td>
<td>2001</td>
</tr>
<tr>
<td>LP</td>
<td>Cost</td>
<td>Hailu</td>
<td>2002</td>
</tr>
<tr>
<td>Simple GA</td>
<td>Benefit &amp; cost</td>
<td>Benhachmi et al.</td>
<td>2003</td>
</tr>
<tr>
<td>Simple GA</td>
<td>Pumping rate</td>
<td>Benhachmi et al.</td>
<td>2003</td>
</tr>
<tr>
<td>Response matrix</td>
<td>Pumping rate</td>
<td>Motz et al.</td>
<td>2003</td>
</tr>
<tr>
<td>method</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>GASA TS</td>
<td>–</td>
<td>Peratta</td>
<td>2003</td>
</tr>
<tr>
<td>Evolutionary Optimization</td>
<td>Cost</td>
<td>Silva et al.</td>
<td>2003</td>
</tr>
<tr>
<td>Response matrix</td>
<td>Pumping rate</td>
<td>Zhou et al.</td>
<td>2003</td>
</tr>
<tr>
<td>method</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>PGA</td>
<td>Pumping rate and</td>
<td>Park and Aral</td>
<td>2004</td>
</tr>
<tr>
<td>well locations</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* GA: Genetic algorithm, LP: Linear programing, SA: Simulated annealing, TS: Tabu search, PGA: Progressive genetic algorithm

Among these optimization models the model developed by Park and Aral (2004) is most flexible in that optimal pumping rates and well locations can be computed. A new model presented in this work differs from those of Guan and Aral (27) and Park and Aral (28) in that it can handle more diverse problems.

### 5. Sand Tank Experiments
Recently several laboratory model studies of saltwater ingress were carried out using sand tanks of suitable size. In most cases, these laboratory models were used to validate numerical models of variable density transport for saltwater intrusion in coastal regions.

Zhang et al., (29) investigated the migration of a contaminant plume in an unconfined aquifer using a sand tank under realistic conditions of tides and beach face. This plume consisted of a salt solution with density varying from 995.2 g/ℓ to 1025.0 g/ℓ. The dimensions of the sand tank are 1650 mm long, 600 mm high and 100 mm wide. The steady state experimental results were used to calibrate numerical model 2DFEMFAT, which was used to compare numerical predictions of contaminant flume transport. This experimental study concluded that seaward boundary conditions can be simplified as a constant freshwater head by neglecting saltwater density and tidal variations to gain great savings in computing effort. However, this has resulted in underestimations of solute mass transport exiting around the shoreline and unrealistic migration paths under the seabed. The experimental results showed that the contaminant moves upward towards the coastline when it approaches the saltwater interface and exits around the coastline.

Panteleit et al., (30) studied geochemical aspects of saltwater intrusion into a porous coastal aquifer system in a two-dimensional sand tank (2m×0.05m×0.5m) experiment. The sand tank was filled with natural aquifer material and, after a conditioning phase with artificial groundwater, a density-driven saltwater intrusion against the hydraulic head was induced. Geochemical processes were recorded by intense sampling at 100 monitoring points at different depths throughout the tank. These studies
concentrated on exchanger reactions during saltwater intrusion. Sodium and Chloride concentrations across the depth were measured using a soil moisture sampler (Porous polymer tube; diameter 2.5mm, pore width 1.1 micrometer) at different depths. It was concluded that the exchanger processes are dominated by a linear rising of the total cation equivalent concentration.

Thorenz (31) used sand tank experiments for validating a numerical model (RockFlow software FEM simulation package). The sand tank (0.96m×0.1m×0.48m) with two perforated walls was used to conduct experiments. Through these perforations, 16 tubes of 1mm inner diameter were inserted for tracer injection and sample extraction. Model accuracy and speed against a standard procedure were evaluated using these experimental results.

A rectangular sand tank (1.7m×0.1m×1m) was used by Jalbert et. al., (32) to verify the validity of non-dimensional solution for the case of a moving interface during the displacement of water by saltwater. The agreement with experimental data showed weak influence of the boundary conditions on the general trend of the density-affected flow. The solution allowed for a simplified general view of the influence of density differences upon values of velocity, permeability, or dispersivity obtained in conservative tracer experiments.

Using a plexiglass sand tank (10m×0.1m×1.2m), Koch and Starke (33) analyzed the effect of density stratification and stochastic properties of medium on steady state macro-dispersion. Experiments were conducted with saltwater concentrations varying from 250 ppm (freshwater) to 35,000 ppm with two inflow velocities of 1 and 4 m/day. For calibration and
validation purposes, the experiments were accompanied by numerical simulation using the SUTRA density-dependent flow and transport model. These studies found for the same concentration contrast a larger sinking of the mixing layer with decreasing inflow velocity and, at the same time, an apparent increase of the lateral dispersion coefficient.

Inouchi et al. (34) compared the results from sand tank (0.9m×0.1m×0.45 m) experiments with the predictions of interface of salt and freshwater by the Boundary Element Method (BEM). Further, Inouchi et al., (34) observed that experimental visual interfaces are below the interface predicted by the analytical solution.
III. Four-Stage Strategy for Groundwater Development in Coastal Areas

Amount of groundwater development depends firstly on the demand. When the demand is small groundwater can be developed without limitations. As the demand grows the dependence shifts to the hydrogeological constraints. In coastal areas one of the major hydrogeological constraints is the saltwater wedge. When groundwater development must consider the constraints, a strategy is needed to attain the maximum benefit, i.e., the maximum groundwater pumping.

A four-stage strategy (Figure 4) is proposed as a guideline for optimal development of groundwater in coastal areas. Later stages guarantee more groundwater production compared to earlier stages. Cost of development increases as stage advances.

![Figure 4. Four-stage strategy for sustainable groundwater development in coastal areas](image)

The most economical groundwater development can be achieved at Stage 1. At this stage maximum groundwater development is realized via optimal
distribution of pumping wells. Both pumping rates and well locations are subject to optimal distribution. It is clear that if wells are placed far away from the coastal line, groundwater production becomes at its maximum. However, real situations may get more complicated. Existing wells whose locations and withdrawal rates are not optimal need be protected from saltwater contamination. Concerns on groundwater environment also affect optimal distribution of pumping wells. If lowered groundwater piezometric head and extended saltwater intrusion into the aquifer are undesirable, pumping wells must be placed near the coast, not far away from it. Therefore optimal distribution of pumping wells must be determined considering all possible constraints.

Once pumping wells are distributed optimally and groundwater production has reached the maximum, additional groundwater development by placing an extra pumping well is impossible. In this situation additional groundwater development may be possible by actively controlling the saltwater wedge. Two methods can be used: freshwater injection and saltwater pumping. The freshwater injection is used to form a hydraulic barrier which prevents the saltwater wedge to intrude inland. Then inland wells can withdraw more groundwater without worrying about saltwater intrusion. This method has been applied to, for example, California, USA. Consideration must be given as to if additional production exceeds the amount of freshwater injection. A saltwater pumping well is generally used to protect a single well from contamination due to upconing. Saltwater pumping tends to create downconing just as the freshwater pumping tends to create upconing. Therefore addition of a saltwater pumping well may reduce saltwater upconing. Development of additional groundwater by the means of saltwater wedge controlling is termed the Stage 2.
In Stage 3 augmentation of fresh groundwater is realized via artificial recharge (AR) and aquifer storage and recovery (ASR). AR is recharge that occurs when the natural pattern of recharge is deliberately modified to increase recharge (35) and ASR is defined as the storage of water in a suitable aquifer through a well during times when water is available, and recovery of the water from the same well during times when it is needed (36). The two schemes are similar in that freshwater groundwater is supplemented artificially for later use. The main difference lies in the locations of injection and withdrawal. Water is generally recharge in the upland area and is withdrawn somewhat downstream for AR, while water is injected and withdrawn practically at the same location for ASR.

The basic question regarding AR and ASR is the ratio of production and injection. This ratio clearly depends on the hydrogeological setting. Optimization may be used to maximize the recovery rate.

Subsurface dams are the last and the most expensive measure for groundwater development. The subsurface dams if constructed properly, collect groundwater, which would otherwise discharge to the sea, for use. A dam may prevent saltwater intrusion and allow virtually all groundwater to be harvested.
IV. Optimization Model

1. Objective Functions

Development of groundwater incurs both benefit and cost. Therefore, development problem can be posed as an optimization problem. When either of benefit or cost is a dominant factor, the optimization problem involves a single objective function. Otherwise, it becomes a multi objective problem.

For the groundwater development problem, it is both necessary and feasible to address multi objectives. The most fundamental objective function is to maximize the net freshwater pumping, which is expressed as

\[
\text{maximize} \quad \sum_{i=1}^{N_f} Q_{i,i}
\]  \hspace{1cm} (13)

where \(N_{f,\text{opt}}\) is the number of freshwater wells where pumping/injection rates are to be optimized. Pumping rates are considered positive, and injection rates are considered negative. The freshwater injection wells are used for controlling the saltwater wedge. The above expression assumes that the injected freshwater is of equal value as the pumped freshwater. If less valuable water is used for the injection, for example, treated sewage water, a different objective function may be used to distinguish the two waters.
Figure 5. Adverse impacts of pumping in coastal areas

Pumping groundwater from aquifers causes inevitably adverse impacts on groundwater environment. Drawdown in piezometric heads and intruded saltwater wedge are prominent impacts. Depending on the condition saltwater intrusion may result in two forms: later intrusion and upcoming (Figure 5). If a pumping well is located away from the saltwater wedge, saltwater wedge, as a whole, would intrude inland. On the other hand if a pumping well is located above the saltwater wedge, it may induce upcoming without causing too much lateral intrusion. Both lateral intrusion and upcoming are undesirable. Thus, these adverse impacts need to be minimized.

\[ \text{minimize } [D + I + A] \]  \hspace{1cm} (14)

where \(D, I, \) and \(A\) represent drawdown, the upcoming, and lateral
intrusion, respectively. When considering above factors, maximum impact can be minimized. However, it turns out for the problem in hand, use of the maximum impact can not distinguish between inferior and superior alternatives. For example for the same pumping rate, the maximum drawdown in piezometric head is practically the same regardless of the well location whereas well locations are very important because a well placed inland causes more saltwater intrusion than a well placed near the coast. If average impact is used instead of the maximum impact, the above problem can be resolved.

The average drawdown normalized by the aquifer thickness can be defined as

\[ D = \frac{V_D}{A_t H} \]  

(15)

in which \( V_D \) is the volume of drawdown in the piezometric level due to wells to be optimized, \( A_t \) is the area of the domain of interest, and \( H \) is the representative saturated thickness of the aquifer. The average upcoming height over the entire domain normalized by the thickness of the saturated aquifer becomes

\[ \bar{I} = \frac{V_S}{A_t H} \]  

(16)

in which \( V_S \) is the volume of increased saltwater due to pumping. \( \bar{I} \) represents a non-dimensional increase in saltwater thickness. The lateral intrusion is represented by newly intruded area due to pumping normalized by the area of the domain. That is
\[ A' = \frac{A_f}{A_T} \quad (17) \]

in which \( A_f \) is the base area of the aquifer where saltwater intruded due to pumping wells to be optimized.

Although all normalized, above average quantities are of different magnitudes. First, the increase in the saltwater thickness is much smaller than drawdown in piezometric level. Use of the differential density ratio may make the two quantities approximately comparable magnitudes, i.e.,

\[ I = I - \frac{(\rho_s - \rho_f)}{\rho_f} \quad (18) \]

In cases where lateral intrusion is be larger than upcoming a weighting factor can be applied to the \( A' \) and

\[ A = \omega_l A' \quad (19) \]

So far, wells that pump or inject freshwater have been considered. The remaining type of wells is a saltwater pumping well to be used for protecting a freshwater pumping well. Cost for saltwater pumping includes energy and disposal of pumped saltwater. The following type of objective function may be used to minimize the net saltwater pumping:

\[ \text{minimize } [\omega_2 \sum_{i=1}^{N} Q_{s,i}] \quad (20) \]
where \( N_s \) is the number of saltwater pumping wells to be optimized, and \( \omega_2 \) is a weighting factor to relate the cost of saltwater pumping to the benefit of extra freshwater that is available.

2. Constraints

In addition to aforementioned objectives the following constraints are applicable:

\[
C_{w,i} < C_{w,t} \quad \text{for } i = 1, N_p
\]

where \( N_p \) is the number of freshwater pumping wells, \( C_w \) is the concentration of applicable species, such as TDS or Cl in the pumped freshwater, and \( C_{w,t} \) is the allowable concentration. Concentration of Cl is commonly used to investigate saltwater intrusion phenomena. Since two-phase model is used for the simulation of groundwater flow, the following expression is used to estimate the concentration.

\[
C_w = \frac{Q_s}{Q_f}
\]

where \( Q_s \) is the rate of saltwater pumping from a well that is supposed to pump freshwater at the rate \( Q_f = (Q_f + Q_s) \). Above constraints are applied to freshwater pumping wells to be optimized and existing wells that are not subject to optimization.

3. Combined Unconstrained Objective Function
Above set of functions constitutes a constrained multi-objective problem. The optimization process may become simpler by combining objective functions into a single function. The constraints also need to be combined into the objective function so that an unconstrained optimization technique, such as the genetic algorithm, can be used. The final form of the objective function for the groundwater development in coastal aquifer becomes

$$\text{maximize } \phi = \alpha \sum_{i=1}^{N_g} Q_f - \omega_1 \sum_{i=1}^{N_s} \frac{Q_s}{Q_f + Q_s} - \omega_2 \sum_{i=1}^{N_s} Q_s$$

(23)

where $\alpha$ is a groundwater protection index representing the adverse impacts in the groundwater environment caused by pumping. $\alpha$ is inversely proportional to adverse impacts and can be defined as below:

$$\alpha = 1 - (D + I + A) \times \omega_3$$

(24)

where $\omega_3$ is weighting factor.

The first term can be considered a benefit, the second item is a penalty function which decrease the value of the objective function when the saltwater pumping is pumped from the wells designed to pump freshwater only and the third item is the cost incurred in saltwater pumping to control the saltwater wedge.

4. Applicability of the Objective Function

Modelling is generally performed under given conditions to assess the influences due to pumping for the different pumping locations and pumping rates. However, optimal well locations and pumping rates can be
determined by an optimization model without using the above procedure. Groundwater development is commonly classified into one of these types. The types of problems which will be discussed in this study is detailed in Table 5.

**Table 5. Types of problem**

<table>
<thead>
<tr>
<th>A priori Determined Variables</th>
<th>Decision Variables</th>
<th>Optimization</th>
</tr>
</thead>
<tbody>
<tr>
<td>CAT1A</td>
<td>Location</td>
<td>Freshwater Pumping Rate</td>
</tr>
<tr>
<td>CAT1B</td>
<td>Pumping Rate</td>
<td>Location</td>
</tr>
<tr>
<td>CAT2A</td>
<td>-</td>
<td>Location &amp;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Freshwater Pumping Rate</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>A priori Determined Variables</th>
<th>Decision Variables</th>
<th>Optimization</th>
</tr>
</thead>
<tbody>
<tr>
<td>CAT2B</td>
<td>-</td>
<td>Locations &amp;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Freshwater Pumping Rate or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Injecting Rate</td>
</tr>
<tr>
<td>CAT4</td>
<td>-</td>
<td>Locations &amp;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Freshwater Pumping Rate or</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Saltwater Pumping Rate</td>
</tr>
</tbody>
</table>

When the location of wells to be optimized is limited within specified areas, the first type should be considered. If there are no limitations for conserving groundwater environment and constraints for protecting wells from saltwater intrusion, designed pumping rate can make existing wells and/or new wells affected from saltwater intrusion. But suggested objective function find an optimal pumping rate while minimizing adverse impact and protecting wells from saltwater intrusion.

In the second type the pumping rate of wells is restricted. If there are
also no limitations and if pumping rate is less than groundwater recharge, well can be placed at anywhere. However the existing and new wells may be contaminated by saltwater according to the given pumping rate and location. The objective function is also designed to find an optimal location while securing targeted pumping rate and protecting wells from saltwater intrusion.

Both optimal pumping rate and optimal location of new wells can be designed in the third type. One of two optimum values cannot be found in the first type and the second type. The optimal pumping rate at the given location can be increased when an whole groundwater domain is considered. If the given pumping rate is not equal to the optimal pumping rate, additional groundwater can be secured in the third type.

If more additional groundwater than optimal pumping rate designed in the third type is required, it can be possible by controlling the saltwater wedge. However the benefit and cost must be considered in this case.

5. Simulation Model

DUSWIM which is used in this work is sharp-interface (two-phase) model and a quasi-three-dimensional model. It can simulate both single or layered aquifers separated by confining layers or aquitards. The uppermost aquifer may be either confined or unconfined. A variety of transient and steady-state boundary conditions can be treated. These include: prescribed head and flux conditions, areally distributed recharge, well pumping or injection, vertical leakages through confining layers and other head-dependent fluxes (e.g., at coastal boundaries). Mathematical model used in this model is based on two vertically integrated governing
equations, one describing freshwater flow and the other describing saltwater flow in an aquifer layer (9). The formulation of governing equations can be found in equation (4) and (5).

A modified Galerkin finite-element method is used to approximate the vertically integrated governing equations. It is written in a convenient form using standard notation of two-phase flow in porous media. The non-linearity of the flow equations is treated using the Newton-Raphson method. A similarly developed numerical model has been applied in many cases [(37),(38)]. This model is also applied for the investigation of regional or sub-regional saltwater intrusion and the assessment of safe yields of well fields (39).

In this two phase model, the rates of freshwater and saltwater extracted depend on the position of the interface and the screened interval of the well. The production rates of freshwater and satwater are given by

\[ Q_f = \left( \frac{K_f L_f}{K_f L_f + K_s L_s} \right) \times Q_t \]  

\[ (25) \]

\[ Q_s = \left( \frac{K_s L_s}{K_f L_f + K_s L_s} \right) \times Q_t \]  

\[ (26) \]

\[ L = L_f + L_s \]  

\[ (27) \]

where \( L \) is the length of screen in the well, \( L_f \) refers to the distance from the interface position to the top of the screen, \( L_s \) refers to the distance from the interface position to the bottom of the screen, and \( Q_t \) is the total production rate of the well.
DUSWIM is suitable for considering the features of coastal areas.

1) Physical Features

DUSWIM has some of physical features and computational features to simulate coastal groundwater flow. The features are described as follows:

- His model is a quasi-three-dimensional, finite-element code.
- It is based on a sharp interface modeling approach that considers freshwater and saltwater as two immiscible liquids with different densities and separated by an abrupt interface.
- It simulates simultaneously flow of freshwater and saltwater in single and aquifers (single and multi-layer including aquitards) which may have areally variable and anisotropic hydraulic properties.
- Within each aquifer, areal flow is considered (i.e. the Dupuit assumptions are used)
- Hydraulic connections between neighboring layers are taken into account by means of vertical leakages through confining units (aquitards)

2) Computational Features

DUSWIM has some of features to simulate coastal groundwater flow. The features are described as follows:

- Two vertically integrated governing equations are used for the mathematical model.
- Governing equations are discretized using the Galerkin finite
element procedure modified to incorporate storage matrix lumping and upstream weighting of mobility.

- A fully implicit Newton-Raphson procedure is used to obtain a linearized system of algebraic equations at each time level.
- The model performs a transient simulation using an automatic time marching scheme designed to facilitate a steady-state condition.
- It utilizes a right-handed Cartesian coordinate system with the x- and y-axes in an areal plane and the z-axis vertically upward.
- The model can control a variety of boundary conditions including prescribed heads and fluid fluxes, areally distributed recharges, well pumping or injection, vertical leakages through confining layers and other head-dependent flux conditions such as those at coastal boundaries and beneath surface water bodies.
- The uppermost aquifer of the system may be confined, semi-confined or unconfined with an areally distributed recharges.
- Mass balance calculation is provided at the end of each selected time step and over the domain of each aquifer.
6. Optimization Techniques

Optimization value can be obtained by combining a simulation model and an optimization technique. In the simulation model, computed values are verified against the given constraints. The optimization technique evaluates the maximum pumping rate for given constraints with handling fresh and saltwater simultaneously. For these evaluation, genetic algorithm which is one of most popular and traditional optimization techniques is used in optimization model.

1) Genetic Algorithms

The increasing demand for freshwater and the decrease in water resources due to over-exploitation and pollution are calling for more efficient utilization of resources and optimal design of hydro-systems. So advanced numerical tools and optimization schemes are needed for their solution to achieve better design and operation policies. Linear and non-linear programming have been widely used in the last few decades. They are generally based on iterative procedures that find local minima quickly, but do not guarantee a global optimum (8).

One of popular and traditional optimization algorithms is genetic algorithms (GAs). It is created by Holland in 1973 and made famous by Goldberg in 1989. The GAs have stochastic techniques based on evolutionary design. GAs are search procedures based on Darwin’s theory (the survival of the fittest) which includes the mechanics of natural selection and genetics (40). GAs express the possible solution as a specified data structure. They use finite length strings or chromosomes over some user-defined alphabet to describe the parameters for each trial solution.
The exploration of the search space is an iterative process that starts with a randomly generated initial population. In the process the population then undergoes a series of random genetic operations such as a transformation and an evolution to evolve into a new and better-fit population. The process continues until a population of acceptable quality (best fitness) is produced so that GAs finally find an optimal solution of an unknown function.

(1) Procedure of Genetic Algorithms

The schematic diagram of the genetic algorithm is shown in Figure 6.

Figure 6. Schematic diagram of the genetic algorithm
GAs simulate the evolutionary process with an initial population of N randomly generated individuals, each of which represents a search point in the space of potential solutions. The population of individuals is allowed to evolve over a number of generations, and each generation, every individual is evaluated for its fitness with regard to the entire population. The evolutionary process is guided largely by the mechanics of the three main operators such as reproduction, crossover, mutation operator.

The main step required to build up a GA for an optimization process are enumerated below (8).

1) **Step 1: Coding**

In a standard GA, GA operates on a population of decision variable sets called string, every individual is encoded as a string, also called a chromosome. Each string is made up of a series of characters or genes, which represent a coding of the decision variables. A binary coding is most often used, but integer coding or even floating point coding can be employed. The genetic coding of a string is called its genotype, while the decoded decision variable is the string's phenotype.

2) **Step 2: String formation**

Once the variables are coded, they are put together to form a string or chromosome which represents a solution in the search space.

3) **Step 3: Initial population**

An initial population of N strings is generated by a random process, e.g. by randomly changing the bits of the initial string. Each possible value has an equal probability of being chose for each gene in the string.
④ Step 4: Fitness evaluation
The fitness function is used to assign a score to each chromosome, measuring the extent to which that chromosome provides a solution to the problem. The fitness function must be appropriate to the problem under examination and characterizes accurately the function to be optimized. When constraints are involved, fitness values may include some penalty for constraint violation.

⑤ Step 5: Reproduction
The reproduction operator drives the population toward better solutions by duplicating individuals according to their fitness. The population size is unchanged.

⑥ Step 6: Crossover
The crossover operator combines string elements of two chosen individuals (parents) of the current population of form one or two offspring.

⑦ Step 7: Mutation
The mutation operator causes randomly selected digits of th offspring’s to change in value, but with a small probability, with the purpose of keeping the population diverse and preventing premature convergence.

⑧ Step 8: Iterative process and termination rule
There are two principal ways in which subsequent generations can be produced:

- by creating sufficient children to replace the original population, a procedure called generational replacement.
- by creating only sufficient children to replace the worst m parents
(m>1), a procedure called *steady-state replacement.*

(2) Features of Genetic Algorithms

Genetic algorithms have become a popular tool and are used widely, especially in engineering and industrial applications for analysing optimization problems. There are many applicaton of GAs in hydraulic network and water distribution systems, groundwater management, etc (8).

GAs have advantages, which are summarized as follows:

- Genetic algorithms are well adapted to situations in which a large number of local optimum or global near-optimal solutions exist.
- GAs can handle discontinuous and discrete variable without difficulty.
- There is no derivative computation required
- GAs are highly adapted to parallel processing

But, GAs have a few of disadvantages, which are summarized as follows:

- There is no convergence guarantee
- They require a large number of fitness evaluations
- It is possible to be misled by the so-called GA-deceptive problems

2) Parallel Genetic Algorithms Method

Genetic algorithm has a capability to search for a global optimum (40). However, the method has large computational requirements. In this work, the large computational requirements can be efficiently handled with
parallel processing. The GA is well suited for parallel processing.

MPI process is used for parallel processing. The study of MPI Process had been first discussed at Williamsburg workshop in April, 1992. MPI was developed by government, industry circles and academic circles of USA during 1993~94. And a final version of draft is announced in May, 1994.

The MPI for parallel computational processing is briefly introduced. MPI is the complex of libraries and macros to exchange message with each processes (nodes). It can be used with program language such like C, C++, Fortran etc., and used for parallel computers, clusters and heterogeneous networks.

MPI can be easily started by calling 'mpi_init' subroutine and ended by calling 'mpi_finalize' subroutine. And the header file ('mpif.h') should be called to connect fortran language. This includes the definition of parameters for MPI. And the number of nodes and division of each node should be assigned. The procedure of message passing is performed by point-to-point communication or/and collective communication. By these data communication, an efficient computation can be done.

Figure 7 depicts a schematic diagram of the parallel computation. A PC cluster, composed of 32 processors, is used in this study (Figure 8).
Figure 7. Parallelization on the N-node cluster

Figure 8. Picture of PC cluster
V. Sensitivity Analysis for Optimization Model

The purpose of a sensitivity analysis is to quantify the uncertainty in the calibrated model caused by uncertainty in the estimates of aquifer parameters, stresses, and boundary conditions (7). Therefore sensitivity analysis is important to understanding not only the characteristic of model but the behavior and influence of parameters in the model.

As the major parameters is selected from the sensitivity analysis, these should be severely took consideration into applying the model into a field. Tran (11) studied for the response of saltwater intrusion lengths with respect to the stresses and transmissivities not using an optimization model but a SHARP simulation model (12).

![Diagram](image_url)

Figure 9. The schematic diagram of unconfined aquifer

In this work, the sensitivity analysis of optimization model is performed
for the one-dimensional hypothetical unconfined aquifer (Figure 9). Prior to the sensitivity analysis, major three parameters which may affect the optimal solutions such as optimal pumping rate \( (Q_{opt}) \) and optimal location of the well \( (L_{opt}) \) in coastal aquifer is selected. First parameter is hydraulic conductivity \( (K) \), Second parameter is recharge rate \( (Q_{rach}) \) defined as recharge rate divided by precipitation, and third parameter is aquifer thickness \( (B) \). According to the change of these, the behavior of objective function value \( (B.F) \), optimal solutions and ratio of optimal pumping rate \( (Q_{opt}/Q_{rach}) \) are investigated in accordance with type of problem. The average annual precipitation \( (1,283\text{mm/year}) \) is referred to calculate recharge rate. The range of hydraulic conductivity, recharge rate and aquifer thickness is decided by referring to some investigation reports [(41),(42),(43),(44),(45), Table 6].

<table>
<thead>
<tr>
<th>Area</th>
<th>Depth of alluvium (m)</th>
<th>Hydraulic Conductivity (m/sec)</th>
<th>SGD/Precipitation (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Min.</td>
<td>Max.</td>
<td>Min.</td>
</tr>
<tr>
<td>Nakdong River</td>
<td>5.2</td>
<td>15.2</td>
<td>15.2</td>
</tr>
<tr>
<td></td>
<td>2.93</td>
<td>3.2</td>
<td></td>
</tr>
<tr>
<td>Youngsan-Somejin River</td>
<td>6</td>
<td>20</td>
<td>30.77</td>
</tr>
<tr>
<td></td>
<td>2.11</td>
<td>2.48</td>
<td></td>
</tr>
<tr>
<td>Keum River</td>
<td>5.7</td>
<td>9.9</td>
<td>10.77</td>
</tr>
<tr>
<td></td>
<td>0.57</td>
<td>0.64</td>
<td></td>
</tr>
<tr>
<td>Buan-Gun</td>
<td>3.6</td>
<td>30</td>
<td>16</td>
</tr>
<tr>
<td></td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
</tbody>
</table>

The range of sensitivity analysis is given in Table 7.
Table 7. Range of sensitivity analysis of optimization model

<table>
<thead>
<tr>
<th>Type</th>
<th>Range of parameters</th>
<th>Target value</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( Q_{rech} ) (%)</td>
<td>( K ) (m/day)</td>
</tr>
<tr>
<td>CAT1A</td>
<td>1~10</td>
<td>10~100</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>10~30</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>10~30</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>20</td>
</tr>
<tr>
<td>CAT1B</td>
<td>6</td>
<td>10~30</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>10~30</td>
</tr>
<tr>
<td></td>
<td>6~10</td>
<td>10~30</td>
</tr>
<tr>
<td>CAT2A</td>
<td>1~10</td>
<td>10~30</td>
</tr>
<tr>
<td></td>
<td>1~10</td>
<td>10~30</td>
</tr>
</tbody>
</table>
1. Optimal Pumping Rate from Well with Arbitrary Location

1) Ratio of Optimal Pumping Rate with Recharge Rate and Hydraulic Conductivity

In this section the behavior of optimal pumping rate is investigated. Hydraulic conductivity and recharge rate are changed from 10m/day to 30m/day and from 1% to 10%. Aquifer thickness is chosen as 20m. Figure 10 and 11 show the perturbation of optimal pumping rate with recharge rate and hydraulic conductivity.

![Figure 10. Ratio of optimal pumping rate versus recharge rate](image)

As it is seen from the figure ratio of optimal pumping rate is small in
highly conductive aquifer and in low recharge region. The ratio of optimal pumping rate increases with curvilinear relationships according to the recharge rate in the low conductive aquifer. But the ratio of optimal pumping rate varies nearly linearly with the recharge rate in the highly conductive aquifer.

![Figure 11. Ratio of optimal pumping rate versus hydraulic conductivity](image)

Figure 11 presents the behavior of ratio of optimal pumping rate according to the hydraulic conductivity. The ratio of optimal pumping rate decreases fast in low recharge region. But it has linear relationship with the hydraulic conductivity in high recharge rate region.

2) Ratio of Optimal Pumping Rate and Objective Function
Value According to X-coordinate

The relationships between the ratio of optimal pumping rate and hydraulic conductivity is investigated via enumeration for the specified recharge rate (Figure 12).

![Figure 12. Ratio of optimal pumping rate versus hydraulic conductivity via enumeration](image)

As it is seen from the figure 12 and 13 the ratio of optimal pumping rate is higher in the low conductive aquifer than in the highly conductive aquifer. But the range of ratio of optimal pumping rate is more sensitive to the well location in the low conductive aquifer. Variation of the ratio is
nearly 20% in the low conductive aquifer whereas it is only about 5% in the highly conductive aquifer.

![Diagram showing objective function value versus hydraulic conductivity via enumeration.]

Figure 13. Objective function value versus hydraulic conductivity via enumeration

3) Ratio of Optimal Pumping Rate and Objective Function Value According to Aquifer Thickness

Figure 14 shows the relationship between ratio of optimal pumping rate and objective function value according to the aquifer thickness. Even though aquifer thickness does not affect the optimal pumping rate, it just
affects the objective function value. Therefore the ratio of optimal pumping rate is independent of the aquifer thickness.

![Graph](image)

Figure 14. Ratio of optimal pumping rate versus objective function value with various aquifer thickness

2. Optimal Well Location with Arbitrary Pumping Rate

1) Optimal Well Location and Objective Function Value According to Ratio of Pumping Rate and Hydraulic Conductivity

In this section the behavior of optimal well location according to the ratio of pumping rate. Hydraulic conductivity is changed from 10m/day to 30m/day and aquifer thickness is given as 20m. Pumping rate is
predetermined from 10% to 80% of 6% of recharge rate.

Figure 15 shows the optimal well location of the given pumping rate with hydraulic conductivity. According to the same increase of pumping rate from 10% to 40%, the optimal well locations move approximately 50m, 100m and 280m according to the increase of hydraulic conductivity. Therefore ratio of pumping rate is less sensitive to optimal well location in low conductive aquifer than in highly conductive aquifer.

![Figure 15. Optimal well location versus the ratio of pumping rate with various hydraulic conductivities](image)

Objective function value also has similar trends according to the hydraulic conductivities and ratio of pumping rates (Figure 16). As it is seen from the figure 15 and 16 optimal well location is sensitive to the ratio of pumping rate in highly conductive aquifer.
2) Optimal Well Location According to Recharge Rate and Hydraulic Conductivity

Figure 17 presents the response of optimal well location according to the hydraulic conductivity and recharge rate. Pumping rate is specified as 50% of recharge rate. Although the changes of hydraulic conductivity is same from 10m/day to 20m/day, the optimal well location is less sensitive in high recharge rate region than low recharge rate region. As the ratio of pumping rate increases the optimal well locations move toward coastline.
3. Optimal Pumping Rate and Optimal Well Location

1) Ratio of Optimal Pumping Rate and Optimal Well Location According to the Recharge Rate and Hydraulic Conductivity

Optimal pumping rate and optimal well locations are investigated according to the recharge rate with various hydraulic conductivity (Figure 18 and 19). The behavior of ratio of optimal pumping rate is similar to Figure 10. But the ratio of optimal pumping rate is higher than the results presented in Figure 10. And the optimal well locations move toward coastline. This results is same trends presented in Figure 10 and 17.
Figure 18. Ratio of optimal pumping rate versus recharge rate with various hydraulic conductivities

Figure 19. Optimal well location versus recharge rate with various hydraulic conductivities
4. Results

- Ratio of optimal pumping rate and recharge rate have curvilinear relationship in low conductive aquifer, and the radius of curvature is larger in low conductive aquifer than highly conductive aquifer (Figure 10).
- The ratio of optimal pumping rate is more sensitive in low recharge region than high recharge region according to the hydraulic conductivity (Figure 11).
- If the recharge rate and aquifer thickness is same, the ratio of optimal pumping rate is more sensitive according to the hydraulic conductivity near coastline (Figure 12).
- Aquifer thickness does not affects the optimal pumping rate (Figure 14).
- As it is seen from figure 15, the optimal well location is sensitive to the ratio of pumping rate in highly conductive aquifer (Figure 15).
- As recharge rate increases the optimal well locations move toward coastline. The change of optimal well location is smaller in low conductive aquifer (Figure 17).
- When both optimal pumping rate and optimal well location are considered, the optimal pumping rate increase (Figure 10, 18).

As it is seen from the results of sensitivity analysis for the optimal solutions

- **Ratio of optimal pumping rate**
  - decreases according to the increase of hydraulic conductivity
  - increases according to the increases of recharge rate, the relationship is different according to the hydraulic conductivity.
  - is more sensitive to the recharge rate than to the hydraulic
conductivity.
- is independent on the aquifer thickness

- Optimal well location
  - moves toward the coastal line according to the increase of recharge rate
  - moves toward the inland according to the increase of hydraulic conductivity
  - is more sensitive to the hydraulic conductivity than the recharge rate

- Considering both the optimal pumping rate and the optimal well location results in increasing the ratio of optimal pumping rate.

- In summary, it is more important to estimate the recharge rate in deciding the ratio of optimal pumping rate. Although the difference of the ratio of optimal pumping rate according to the hydraulic conductivity ranges from 40 percentages to 20 percentages, it would be better to choose a low conductive aquifer for increasing groundwater pumping rate. But in the low conductive aquifer the fluctuation of the ratio of optimal pumping rate is larger than in highly conductive aquifer according to the distance from coastal line, it should be consider the optimal well location simultaneously.
VI. Verification of the Optimization Model in Cross Section

Verification of a model refers to processes of checking mathematical accuracy of model solutions. Comparing solutions with analytical solutions are commonly used for verification. The optimization model, developed in this work, involves two major numerical components: a two-phase freshwater and saltwater flow simulation model and a genetic algorithm optimization code.

The flow simulation model had been verified independently (39). The genetic algorithm code had also been verified (40). In this chapter the verification of the optimization model is presented. Verification of the model is conducted by inspecting the values of the objective function and adverse impacts of all possible solutions. Therefore the verification is performed via enumeration.

![Conceptual model for verification of the optimization model](image)

Figure 20. Conceptual model for verification of the optimization model
The dimension of a hypothetical cross section is 20m thick and 1km long. Left-hand side boundary of the domain is assigned to be a coastal boundary and the right-hand side boundary is assigned to be a freshwater influx boundary. The density of freshwater and saltwater are 1,000 kg/m³ and 1,025 kg/m³ respectively, and the hydraulic conductivity is 50 m/day. The inflow rate of freshwater is 1.0 m³ 1/m through the inland boundary. The schematic diagram of the hypothetical unconfined aquifer is shown in Figure 14. In this aquifer a partial penetrating arbitrary well is installed at x=250m. A pumping rate of this well is 0.432 m³ day. Saltwater intruded into the inland aquifer due to pumping (Figure 22, middle line). But there is no problem in using this well.

As mentioned in chapter 3, groundwater development is increased according to the increase of stage. In this work optimization problems are analyzed for this aquifer according to the first two of the four-stage strategy (Table 5). It is identified that the optimal pumping rate increases (Table 8)

<table>
<thead>
<tr>
<th>Type of problem</th>
<th>$Q_{opt}$\ (m³ 3day)</th>
<th>Ratio of increase\ (%)</th>
<th>x-coordinate\ (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CAT1A</td>
<td>$Q_f=0.76$</td>
<td>75</td>
<td>$L=250$</td>
</tr>
<tr>
<td>CAT2A</td>
<td>$Q_f=0.864$</td>
<td>13.6</td>
<td>$L_{opt}=400$</td>
</tr>
<tr>
<td>CAT4</td>
<td>$Q_f=0.95$ \ $Q_s=0.106$</td>
<td>10</td>
<td>$L_{opt}=400$</td>
</tr>
</tbody>
</table>

The optimal solutions are verified by examining the history of objective function values and adverse impacts in accordance with locations for
CAT2A and CAT4. The influence of groundwater protection index to the objective function value is investigated for CAT2A.

1. Optimal Pumping Rate from Well with Arbitrary Location

The purpose of this section is to obtain an optima pumping rate in given location \((x=250\text{m})\) on coastal groundwater systems. In this case, the question would be how much we can pump from this well of partial penetration. Since the problem is simple cross-sectional, the answer can be found via simple trial and error. Therefore, the optimal solution can be verified in a straightforward manner.

Execution of the optimization model resulted in the pumping rate of 0.762 \(\text{m}^3/\text{day}\). The freshwater-saltwater interface according to the pumping rate is depicted in Figure 22.

![Figure 21. Optimal pumping rate in the arbitrary well](diagram)

\[Q_{\text{opt}} = 0.76\text{m}^3/\text{day}\]
As it can be seen, the interface is in close proximity of the bottom of the well screen. Distance between the interface and the well can be constrained to ensure more safety.

Table 9 shows the objective function value, groundwater protection index and ratio of saltwater pumping rate in the vicinity of the optimal pumping rate.

<table>
<thead>
<tr>
<th>Q (m³/day)</th>
<th>Objective function</th>
<th>Adverse impact</th>
<th>Ratio of saltwater (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.72</td>
<td>0.7683×10⁻²</td>
<td>0.9146</td>
<td>0</td>
</tr>
<tr>
<td>0.74</td>
<td>0.7844×10⁻²</td>
<td>0.9121</td>
<td>0</td>
</tr>
<tr>
<td>0.76</td>
<td>0.8044×10⁻²</td>
<td>0.9095</td>
<td>0</td>
</tr>
<tr>
<td>0.78</td>
<td>-4.008</td>
<td>0.9072</td>
<td>4</td>
</tr>
<tr>
<td>0.80</td>
<td>-6.153</td>
<td>0.9065</td>
<td>6.1</td>
</tr>
</tbody>
</table>

Although groundwater protection index decreases before 0.76 m³/day of pumping rate, objective function value increases because of augmentation of pumping rate. However the ratio of saltwater in the well as well as groundwater protection index increases Therefore it is clear that 0.76 m³/day of pumping rate is an optimum value.

2. Optimal Pumping Rate and Well Location

Another type of question may be where to place the well and how much groundwater to withdraw. The proposed optimization model can handle this problem. If the objective is maximum pumping, the well would be located at the inland end of the domain. But, if impact on groundwater environment is a concern, the problem becomes a trade-off question and must be
formulated as a multi-objective optimization problem. In this study a simple approach is taken: weights are used to convert multi-objectives into a single objective. For the weight used in the study the optimal well location and pumping rate are 400m and 0.864m³/day, respectively. Pumping rate is about 13% higher than the optimal pumping rate from the well of arbitrary location. The interface position for the optimal pumping rate is depicted in Figure 23.

![Optimal pumping rate and location](image)

**Figure 22. Optimal pumping rate and location**

Figure 23 presents optimal pumping rates from a well of various locations from x=100m to 1000m. As it is seen from the figure the optimal pumping rate increase as the well is placed farther inland. However, the objective function value stops increasing after the optimal location (x=400m) since the groundwater protection index decreases (Figure 24).
Figure 23. Pumping rates and objective function values
Figure 24. Groundwater protection index and adverse impacts

It must be noted that the objective function value depends on the weight $\omega_3$. Effect of $\omega_3$ is depicted in Figure 25. As the value increases, the optimal point moves toward the coastal boundary. That is to say, the optimal solution may vary depending on the importance of groundwater environment or groundwater development.

Groundwater protection index should be carefully selected according to the objective of the groundwater management policy, such as the preservation of circumstances, or the development of water resources (Figure 25).
3. Optimal Control of Saltwater Wedge

There are two common methods to control saltwater wedge. The first method is injection of freshwater to create groundwater mound. The second method is extraction of saline groundwater. In this section, an optimal solution of the second method is studied to control the saltwater wedge.

Let’s assume that 10% more groundwater than the optimal pumping rate determined in the previous section is required. As the required freshwater pumping rate is higher than the optimal pumping rate, control of saltwater wedge is necessary to protect the well from saltwater intrusion. It turns
out if saltwater is pumped at 0.106 m³/day, the freshwater well can be protected. The interface position is corresponding to increased freshwater pumping and saltwater pumping depicted in Figure 26.

![Figure 26. Optimal saltwater pumping rate and location](image)

The optimal saltwater pumping rate is in accord with the minimal saltwater pumping rate (Figure 27). As the location of saltwater pumping moves toward the coastal line, the saltwater pumping rate should be increased to protect the fresh groundwater pumping well. So, it is a natural result because higher saltwater pumping rate causes less objective function value. Accordingly, the capability of the optimal pumping model that controls the saltwater wedge also is verified.
Figure 27. Saltwater pumping rate and objective function values
VII. Validation of the Optimization Model with Sand–Tank Experiments

In the previous section the optimization model for development of groundwater in coastal area was verified for its mathematical accuracy. In this section experimental validation of the model is presented. Experiments are conducted to investigate the nature and extent of the freshwater–saltwater interface in an unconfined aquifer setup in sand tanks. Two sand tanks, Tank A for cross sectional flow and Tank B for three-dimensional flow, are used. Detailed description of the tanks are given in the following section.

Experiments are conducted according to the results of the optimization model. Pumping rates and well locations determined by the optimization model are applied to experiments. Freshwater–saltwater interfaces and salinities of pumped water are measured to check the validity of the results of the optimization model.

1. Experimental Setup

1) Tank A for Cross Sectional Flow

The schematic diagram of Tank A (0.6m×0.08m×0.3m) for cross sectional flow is depicted in Figure 28. Two constant-level saltwater reservoirs are located at both ends of the Tank A. Saltwater is supplied by the submersible pump. The water-level of saltwater reservoirs is controlled by the valve at five different height so that the overflow through the outlet is fixed. In this work the overflow elevation is 18.2cm from the bottom of
Tank K. Side walls are made of acrylate board so that flow phenomena can be observed when dye is used.

![Diagram of Tank A](image)

Figure 28. Schematic diagram of Tank A for cross-sectional experiment

Freshwater and saltwater is supplied and pumped by variable speed peristaltic pump to maintain constant recharge and pumping rate.

2) Tank B for Three Dimensional Flow

The schematic diagram of Tank B (2.0m×1.6m×0.6m) is prepared to study three-dimensional flow (Figure 29). All frames of the main experimental tank and recycling system facilities such as supply tank, mixing tank and collecting tank are made of stainless steel to keep from rust. And all pump used in this experiment have durability against saltwater. Some pumps are chemical pump and others have plastic or
rubber gear. Both sides of main tank are made of plexiglass to observe the behavior of saltwater intrusion and bottom of Tank B is made of acrylate board. The thickness of acrylate board is 20mm. Tank B is attached with a multi-manometer board. Sixty five manometers are composed of nine manometers and seven manometers in x-direction, y-direction, respectively, and two manometers are connected to the each bottom of reservoirs.

Two constant-level freshwater and/or saltwater reservoirs are located at both ends of the Tank B. Two reservoirs are equipped with an overflow device which can change overflow elevations within pre-specified limits at a constant rate to simulate the tidal fluctuation commonly observed in the sea. Maximum change speed of overflow elevation is 7.5cm/hr.

![Plan View of the Sand Tank](image)

**Figure 29. Shop drawing of Tank B for three-dimensional experiment**

In this work, one of two reservoirs is freshwater reservoir and the other is saltwater reservoir. Although overflow elevation can be changed in Tank
B, the overflow elevation is fixed same as Tank A because this study is focus on not the influence of tidal fluctuation but the behavior of saltwater intrusion. The shop drawing containing the photograph of the sand tank is shown in Figures 30.

![Figure 30. Picture of Tank B for three-dimensional experiment](image)

Tap water is supplied into the freshwater reservoir through faucet. Saltwater is recycled. But the recycling system is essential for recycling because the density of overflow saltwater is changed. Recycled mechanism as follows:

- Overflow saltwater is collected into small container. This container has a sensor. It automatically controls pump to send diluted saltwater into the saltwater collecting tank when the elevation in the container reaches maximum limit.
- Diluted saltwater is pumped into the saltwater mixing tank which has
stirrer to mix diluted saltwater with salt and dye.

- Saltwater is pumped into the constant-head saltwater supply tank. This tank has overflow pipe to maintain a constant-head and five outlets to send saltwater into Tank B.

Experiment and recycling system are controlled by operating box except for adding salt and dye into the saltwater mixing tank.

3) Materials and Methodology

Both tanks are filled with sand whose characteristics are given in table 10 and figure 31. In Tank A the thickness of sand layer is 25cm and in Tank B it is approximately 50cm.

Table 10. Properties of sand

<table>
<thead>
<tr>
<th>Grain size</th>
<th>Intrinsic Permeability</th>
</tr>
</thead>
<tbody>
<tr>
<td>$D_{50} = 0.47 \text{ mm}$</td>
<td>$K = 7.042 \times 10^{-4} \text{ m/sec}$</td>
</tr>
<tr>
<td>$D_{60} = 0.49 \text{ mm}$</td>
<td></td>
</tr>
<tr>
<td>$D_{10} = 0.28 \text{ mm}$</td>
<td></td>
</tr>
<tr>
<td>$C_U = 1.75$</td>
<td></td>
</tr>
<tr>
<td>Well sorted Sand</td>
<td></td>
</tr>
</tbody>
</table>
Figure 31. Grain-size distribution

Tap water of salinity 90ppm is used as the freshwater. Industrial salt is used to make the saltwater. Target specific gravity of the saltwater is set to 1.04. It is measured by hydrometer (Figure 32). At that specific gravity value the salinity is 46.7ppk. This value is measured by WP·84 instrument which can measure conductivity, salinity and temperature of fluid with different two cables (Figure 33, Table 11).
Table 11. Specifications of WP-84

<table>
<thead>
<tr>
<th></th>
<th>Cable type</th>
<th>Ranges</th>
<th>Resolution</th>
<th>Accuracy</th>
</tr>
</thead>
<tbody>
<tr>
<td>Conductivity</td>
<td>k=1.0</td>
<td>0 to 200 mS/cm</td>
<td>0.1 mS/cm</td>
<td>±0.5% of full scale at 25℃</td>
</tr>
<tr>
<td></td>
<td>k=10</td>
<td>0 to 2000 mS/cm</td>
<td>1 mS/cm</td>
<td></td>
</tr>
<tr>
<td>Salinity</td>
<td>k=1.0</td>
<td>0 to 50.0 ppK</td>
<td>0.1 ppK</td>
<td></td>
</tr>
<tr>
<td></td>
<td>k=10</td>
<td>0 to 500 ppK</td>
<td>1 ppK</td>
<td></td>
</tr>
<tr>
<td>Temperature</td>
<td></td>
<td>-10 to 120.0℃</td>
<td>0.1℃</td>
<td>±0.2℃</td>
</tr>
</tbody>
</table>

And, the pumping rate in well is maintained by variable speed peristaltic pump (Figure 34). It consists of three components: pump driver (model: BS-6001, BS-60101), Easy-Load pump head (model: 7518-10), Masterflex L/S silicone tube (#13, #14). The specification of flow rate is presented in table 12.
Table 12. Specification of peristaltic pump

<table>
<thead>
<tr>
<th>Pump driver</th>
<th>Tube</th>
<th>#13</th>
<th>#14</th>
</tr>
</thead>
<tbody>
<tr>
<td>BS-60601</td>
<td></td>
<td>0.36~36</td>
<td>1.3~130</td>
</tr>
<tr>
<td>BS-60101</td>
<td></td>
<td>0.06~6</td>
<td>0.21~21</td>
</tr>
</tbody>
</table>

To identify the size of the salt-wedge, saltwater is mixed with textile dye (Red H-E3B). The streamlines of pore fresh water flow have been visualized by injecting Rodamine B dye. Attempt has been made to visualize the streamlines in the salt-wedge with Uranine (Sodium fluorescein).

1. Measurements

Variables of primary interest are water levels, and the thickness and the elevations of the transition zone. In this study three different methods are tested to identify the transition zone.

1) Dyed Saltwater
First a dye is used to distinguish freshwater and saltwater. The transition zone can be seen through transparent side walls and the bottom plate (Figure 35). In all cases the saltwater zone is clearly visible. However, due to the color of the sand, the transition zone, which will presumably be indicated by dilute color, can not be identified.

![Figure 35. Dye saltwater in sand tank](image)

2) Electrical Resistivity

The second method is electrical resistivity. Two different types of sensor arrays are used: Schlumberger and Lee. Resistivity meter which consists of four components - ED LAB (Model name: ED-330), supply, Power (AC110/200V(60Hz)), IP explorer (Model name: SYSCAL-K2, 1985) Electrical Survey Control Box (Made by Geoexploration Lab in Dong-A university) - used for this experiments. It measures the resistivity and current values.
In measurement of electrical resistivity, apparent electrical resistivity \( \rho_a \) can be obtained using the formula given in Equation (28). Electrical Conductivity (EC) is the reciprocal of the electrical resistivity (Equation 29).

\[
\rho_a = K \frac{V}{I} \tag{28}
\]

\[
C = \frac{1}{\rho_a} \tag{29}
\]

where \( K \), the geometrical factor (K) for an excitation voltage (V) is given in equation (30). This geometrical factor depends on shape and length of electrode probe.

\[
K = 2\pi \left( \frac{1}{a} - \frac{1}{b} - \frac{1}{d} + \frac{1}{c} \right) \tag{30}
\]

where \( a, b, c, d \) is distance between each points (Figure 36).

![Figure 36. Array of two current and two potential electrode](image-url)
Sample box for electrical resistivity is schematically depicted in figure 37. Freshwater–saltwater interface is formed in 12cm from the box bottom.

![Diagram of sample box with freshwater and saltwater phases and an electrode probe at 12cm depth.]

Figure 37. A small box filled with sands, two fluid phases and electrical probes.

(1) Schlumberger array

For the Schlumberger array the current electrodes (C1, C2) are spaced much further apart than the potential electrodes (P1, P2) as shown in Figure 38. The current electrodes are fixed with a large separation, and the potential pair is moved between them, also with fixed spacing for electrical profiling. Apparent resistivity is plotted against the midpoint of the potential electrodes.
Figure 38. Schlumberger array. (In this array two current electrodes are fixed, and two potential electrodes are moving between them)

Figure 39 and 40 is results of electrical resistivity test on sample box. They show the apparent electrical resistivity and EC values with distance from bottom of the box.

The location of interface is roughly coincided with physical condition.

Figure 39. Resistivity profile for sample box
(2) Lee Array

In this array, a third potential electrode is located at the center P0 of the array (Figure 41). Two measurements of potential are made, first between P1 and P0 ($\Delta V_1$), then between P0 and P2 ($\Delta V_2$). The resultant electrical resistivities are

$$\rho_{c1} = K^2 \frac{\Delta V_1}{I}, \quad \rho_{c2} = K^2 \frac{\Delta V_2}{I}. \quad (31)$$

In order to identifying an interface, the ratio of $\rho_{c1}$ to $\rho_{c2}$ or reverse of it is calculated and plotted against the location of P0.
Figure 41. Lee array (In this array five electrodes, two current electrodes and three potential electrodes are moving together)

Figure 42 and 43 represent the ratio of electrical resistivity in sample box. The convex of resistivity profile appears near the interface. Lee array method represents the position of interface better than Schlumberger array method.

Figure 42. Ratio of $\rho_{el}/\rho_{dl}$ for sample box
According to the preliminary test of electrical resistivity, the examination has been performed in the prototype sand tank. For multi-measurement of the electrical resistivity, connection part of measurement is modified (Figure 44~5).

Figure 43. Ratio of $\rho_{d1}/\rho_{d2}$ for sample box

Figure 44. Modified connection part
Figure 45. Modified connection part

Figure 46. Systemic diagram of EC measurement
The method is able to identify the sharp interface prepared in a sample tube or a box made of electrically nonconductive material. However, when the method is applied to the prototype sand tank, the result is not satisfactory (Figure 47). It is suspected that the stainless steel interfered flow of electricity.

3) Water Column Sample

The third method is the sampling of columns of water in the observation wells. A hollow tube with both ends open is lowered into an observation well where interface is to be measured. As the tube is lowered water in the observation well enters the tube. Once the tube is lowered to the desired level upper end is sealed before the tube is pulled out for inspection. The water sample then can be analyzed using computer software for image analysis. In this work glass tubes of 4mm inner diameter are used. Two
advantages are expected. First, the variation of color can be observed. Secondly, the water table elevation can also be measured. The sampling method is tested on a sharp-interface prepared in a 100ml flask. Although the sample has a sharp interface, the interface in the column of water sampled in the glass tube is dispersed (Figure 48).

![Figure 48. Detection of interface using a sample column of water and grayscale analysis](image)

Two causes are identified: dispersion occurs while the tube is lowered to sample the water, and while the tube is pulled out with the water inside. A number of tests are conducted to estimate the thickness of dispersed zone due to the sampling method. The thickness ranges from 5mm to 12mm, averaging 7mm.
3. Experiments

1) Cross Sectional Flow in Tank A

(1) Predevelopment Condition

Tank A is used to study cross sectional flow. The size of the small tank is 0.43m long, 0.08m wide and 0.30m high. Both constant water-level reservoirs are filled with saltwater of salinity measured at 46.3ppk. The corresponding specific gravity is 1.04, and the water levels in both chamber are 0.182 m. Freshwater is recharged at the center (x=0.22m) of the sand at the rate of 6ml/min. After several hours a steady-state freshwater 'lens' floating on the saltwater is formed. At the center the lens extends 8.5cm below the saltwater level in the reservoir (Figure 49). This data is used to calibrate the intrinsic permeability of the sand layer. It is estimated at $1.092 \times 10^{-10}$ $m^2$. 
Figure 49. The conceptual model of Tank A and photograph

(2) Optimal Freshwater Pumping

To investigate the effect of groundwater pumping on the shape of the lens a pumping well is installed at 12cm from the left saltwater reservoir (Figure 50a). The length of the screen of the pumping well is 3.5cm and the well is installed such that the bottom part of the screen is 3.5cm below the saltwater level in the reservoir. The experimental work is performed using the results of optimization model and optimization model is validated by comparing observed and calculated results. The optimal pumping calculated by the optimization model for the well is 2.4m³/min at 12cm (given location). After the flow reached to a new equilibrium state, the interface rises by 1.5cm below the well, and the salinity of the pumped
water is 115ppm.

Figure 50. (a) Schematic diagram of pumping scenario  
(b) Shift in interface position due to pumping

The comparison of the computed and observed interface is presented in Figure 51. Except for the center position the agreement is very good.
As it is seen from the figure agreement between computed and observed interfaces quite reasonable. The largest deviation occurs near the center part of tank. It is suspected that vertical flow component neglected in the numerical simulation affects the solution.

The pumping rate is raised to 3.5ml/min to investigate the impact of excessive pumping. The interface rises by 1.69cm from the pre-pumping position (Figure 51b). In addition to the shift in the interface position the well is intruded with saltwater. The salinity of the pumped water is measured at 1.69ppk.
The volumetric percentage of the saltwater was evaluated using the following equations.

\[ C_s Q_s + C_f Q_f = C_p Q_p \]  \hspace{1cm} (32)

\[ Q_s + Q_f = Q_p \]  \hspace{1cm} (33)

where \( C_s \), \( C_f \) and \( C_p \) are the salinity of saltwater, freshwater and pumped water. \( Q_s \) is the pumping rate in a well, and \( Q_s \) and \( Q_f \) are the pumping rate of saltwater and freshwater, respectively.

Computed and observed percentages of saltwater are 4.25% and 3.4%, respectively. Even though the two-phase model is used, water quality can be handled with reasonable accuracy.

(3) Control of Saltwater Wedge (Optimal Saltwater Pumping Rate)

As a second validation problem, the amount of saltwater pumping to protect the freshwater pumping well is computed and tested with experiment. It is shown in the previous section that the pumping well is contaminated when the water is withdrawn at the rate of 3.5 m³/min. In this section the possibility of lowering the interface so as to make the well fresh is tested while maintaining the pumping rate. At the same location of the freshwater pumping well a deeper well was added to extract saltwater (Figure 52).
Figure 52. (a) Schematic diagram of pumping freshwater and saltwater scenario (b) Shift in interface position due to pumping both waters

The optimization model predicts that the optimal saltwater pumping rate is 3 ml/min until a steady state is reached. The interface is lowered by 1.09cm while the salinity of the pumped water is lowered to 118ppm (0.05%), practically freshwater.

Thus saltwater pumping is effective in lowering the interface for the sake of keeping the freshwater well fresh while pumping freshwater at a higher rate than will be otherwise advisable.
1)

2) Three-Dimensional Saltwater Intrusion in Tank B

A three-dimensional saltwater intrusion experiment is conducted with IN Tank B. The size of the three-dimensional sand tank is 2.0 m long, 1.6 m wide and 0.6 m high. The thickness of the sand layer is approximately 0.5 m. There are two water supply chambers on both sides of the container. One is filled with freshwater and the other is filled with saltwater. The saltwater is produced and supplied from another tanks (mixing tank and saltwater supply tank, Figure 53). The water level in the freshwater reservoir is 1.8 cm higher than the saltwater level in the corresponding saltwater reservoir. For the given head difference the toe of the interface is located at 75 cm from the freshwater chamber, and the freshwater flow rate is 270 L min.

Figure 53. Picture of mixing tank and saltwater supply tank

For this experiment it is assumed that there is an existing well (1) pumping at 15 m³ min, predetermined non-optimal rate, near the center part
of the tank (Figure 54).

![Figure 54. Well locations for three dimensional experiment](image)

A new well (2), where optimal pumping is desired, is installed at (x=1.2m, y=0.5m). The optimization model estimates the maximum pumping, that would protect both wells from saltwater intrusion, to be 27.6 m³ min⁻¹.

The calculated pumping rate is applied to the well 2. The saltwater wedge responds to the pumping by intruding into the freshwater region of the aquifer. After several days, the interface reaches a new equilibrium position. The maximum intrusion length is 63.0 cm indication 6.0 cm of additional intrusion due to the pumping from well 2 (Figure 55).
Figure 55. Maximum intrusion length (toe) before and after pumping in well 2

Increase height of interface in well 2 is approximately ten times higher than that of interface in well 1 due to the pumping from well 2 (Table 13, Figure 13)

Table 13. Comparison of interfaces

<table>
<thead>
<tr>
<th>Well</th>
<th>Interface (cm)</th>
<th>Increase height (cm)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Well 1 pumping</td>
<td>Well 1 and 2 pumping</td>
</tr>
<tr>
<td>1</td>
<td>-26.62</td>
<td>-25.827</td>
</tr>
<tr>
<td>2</td>
<td>-23.732</td>
<td>-16.515</td>
</tr>
</tbody>
</table>
The interface is measured using the glass tubes. A total of 28 samples are obtained (Figure 57). The photographs of the samples are analyzed. Fifty percent are taken arbitrarily as the relative transparency value for the fresh-side limit of the transition zone, and twenty percent are used for the other limit of the transition zone. Thirty five percent are used for the middle of the transition zone.
Figure 57. (a) Water column samples obtained from observation wells
Figure 57. (b) Water column samples obtained from observation wells

The resulting interface positions along rows A, B, C, and D of the observation wells are given in Figure 58. The calculated interface agrees with the freshwater side limit of the transition zone. The effect of pumping well B can be seen clearly in the profile of the transition zone measured along row C. And the salinity of well A and well B is measured at 112ppm and 156ppm (0.13%). The effect of pumping well A is not clear from the interface measurement. The pumping rate is probably not high enough to cause changes in interface positions in the observation well.
Figure 58. Comparison of observed interfaces with calculated interfaces
I. Conclusions

An optimization model for development and management of groundwater in coastal area is developed. The proposed model is unique in the sense that it can be used to evaluate not only optimal freshwater pumping rates and well locations but also freshwater injection or saltwater pumping rates, which can be used as a decision support tool for most groundwater problems encountered in coastal areas. Main conclusions from this study are:

- A four-stage strategy to develop and manage groundwater in coastal areas is proposed.
- An optimization model is developed as a decision support system.
- The model is verified numerically and validated experimentally.

The strategy proposes a step-by-step approach to develop groundwater in a cost-effective manner. Each step identifies the most groundwater resource within the budget allowed. The first step centers on optimal pumping from wells of optimal or non-optimal locations. The second step expands pumping rates by actively controlling the saltwater wedge. The third step augments groundwater resources through artificial recharge, and the last step exploits virtually entire groundwater by building a subsurface dam. Each step ensures more water to be harvested. The strategy is cost effective since each step can be implemented in addition to measures based on earlier steps without sacrificing previous investments. The proposed model is designed to handle problems for the first two steps. Although not
included in this work the model can be easily extended to deal with third-step problems.

The optimization model is composed of a simulation model and an optimization model. The model maximizes total freshwater pumping while minimizing adverse impacts to groundwater environment and development and management cost. Optimal pumping/injection rates and locations of freshwater and saltwater wells can be determined. The use of a two-phase model as the simulation model provides a capacity to deal more broad problems. Unlike models proposed in the previous studies the current model can handle situations involving saltwater flows. Therefore, the model can provide solutions to problems of the first two steps. With minor modifications it can be extended for the third step. A parallel genetic algorithm is used for the optimization for fast turn-around results.

Solutions of the optimization model depend on a number of parameters, both physical and numerical. Sensitivities of optimal solutions to variations of major parameters are investigated. In the analysis an optimal solution is expressed as the ratio of optimal pumping, defined as the net pumping rate divided by the recharge rate. Hydraulic conductivities, recharge rate (defined as recharge rate divided by precipitation), aquifer thickness, and well locations are selected parameters for sensitivity analyses.

The sensitivity of the ratio of optimal pumping rate to recharge rate varies depending on the hydraulic conductivity: in the highly conductive aquifer the ratio of pumping varies nearly linearly with the recharge rate. The maximum ratio of optimal pumping rate reached at about 50%. In the low conductive aquifer the relationship is nonlinear: in the high recharge
rate region the optimal pumping ratio is less sensitive to the variation of the recharge rate than in the low recharge rate region. In the low conductive aquifer the maximum ratio reached over 70%.

The ratio of optimal pumping rate is more sensitive to well location in the low conductive aquifer than in the highly conductive aquifer if the well is placed away from the coastline. Variation of the ratio is nearly 20% in the low conductive aquifer whereas it is only about 5% in the highly conductive aquifer. Thus well location is more important in less transmissive aquifers.

Aquifer thickness is the one of the parameters that are not easy to evaluate. The sensitivity analyses indicate that the ratio of optimal pumping is independent of the thickness.

The optimization model is numerically verified in a hypothetical cross-sectional unconfined aquifer. The behavior of interface in response to groundwater pumping is examined for the first two steps of the fore-mentioned strategy. Additional groundwater resources are secured by the optimization model while protecting the well and while minimizing adverse impacts. The optimal solutions are verified by examining the behavior of the objective function value and adverse impacts in accordance with locations and comparing the behavior of pumping rate and groundwater protection index. The optimal solutions may vary depending on the importance of groundwater environment or groundwater development.

Experiments are conducted using two sand tanks to validate the
simulation model and the optimization method. Tank A is used to examine various cross sectional flow problems, and Tank B is used to investigate three-dimensional flow problems. To date three-dimensional experiments on saltwater intrusion have not been reported in the literature. Measurement of three-dimensional distribution of interface itself is a challenging problem. After a few attempts, a simple but, effective method is devised. This method is composed of water column sampling and image analysis. Extreme care must be exercised in taking the sample since sampling disturbs the interface. The disturbance mixes water in the observation well, and measured transition zone comes out thicker than the thickness of the actual transition zone. The magnitude of disturbance is proportional to the speed of the tube. The same is true to field measurement of interface as the sensor disturbs water as it is lowered. Thus operation speed of a sensor must be lowered to a value practically possible to minimize disturbance.

Despite its limitation the two phase model can predict with reasonable accuracy the proportion of saltwater in the pumping well. The capability of predicting water quality of pumping water expands the utility of the two-phase model. The interface position calculated by the model is in good agreement with experimental result. More specifically calculated interface from the model agrees with the freshwater side limit of the transition zone.

Prior to the field application of the optimization model, the accuracy and the goodness of this model are verified numerically and validated experimentally for groundwater development and management in coastal areas.

For guaranteeing the application of this model, a few works such as a
periodic and/or intermittent pumping, a time-dependent groundwater recharge and a tidal fluctuation should be included in the optimization model in the future.

The model can be applied to a wide variety of problems. However, it is not without limitations. The limitations of the model are as follows:

- The model assumes that the groundwater flow field is in a steady-state.
- The model’s application is limited to aquifers where the saltwater-freshwater transition zone is relatively thin.
- The model can only be applied to aquifers that can be treated as a single layer.
- Computing time may increase dramatically as the number of computational nodes and the size of parameter space grow.
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ABSTRACT

Optimization Model for Development and Management of Groundwater in Coastal Areas

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An optimization model consists of flow simulation model and optimization technique. There are many groundwater flow models that consider the characteristics of coastal aquifers, such as density-variable flow and saltwater intrusion. But the optimization model for effective groundwater development and management is a development stage and early stage for application in advanced nations.

In this work a four-stage strategy is suggested for development and management of groundwater in coastal areas and an optimization model which can be applied as a making decision support tool is developed for the first two steps. The four-stage strategy is consists of an optimal arrangement of pumping wells, a control of saltwater wedge, an artificial recharge and an aquifer storage and recovery and a subsurface dam. Later stages guarantee more groundwater production and require more cost compared to earlier stages. Appropriate methods are suggested in this strategy according to groundwater demand.

Optimization model preserves groundwater environment and existing wells and assesses maximum pumping rate and optimal well location. It is used for estimating an optimal solution of freshwater injection or saltwater pumping to control saltwater...
wedge. Sensitivity analysis of the optimal solutions is analyzed according to major parameters such like a recharge rate, a hydraulic conductivity and an aquifer thickness. Ratio of optimal pumping rates are sensitive to the recharge rate and optimal well locations are sensitive to the hydraulic conductivity. And the optimization model is numerically verified and validated using a laboratory sand tank model. As a result of studies, optimization model guarantees the accuracy and it is shown that the optimization model is well applied to a real three-dimensional flow in the laboratory sand tank model. This optimization should be a useful tool in the development and management of groundwater in coastal areas.
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간산공과대학 은성용 교수님,
상주대학교 최윤영 교수님,
김윤권 교수님,
서울대학교 박창근 교수님,
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김창완 박사님,
최철순 부정남과 이성 면 과정님, 김 진

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