

DISSERTATION

**CONJUNCTIVE MANAGEMENT METHODOLOGY
CONSIDERING STREAM-AQUIFER SYSTEMS AND
RESERVOIR OPERATIONS**

Submitted by

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In partial fulfillment of the requirements

for the Degree of Doctor of Philosophy

Colorado State University

Fort Collins, Colorado

Fall 2007

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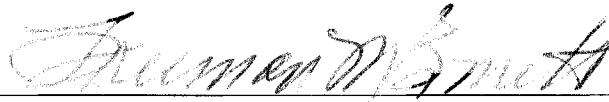
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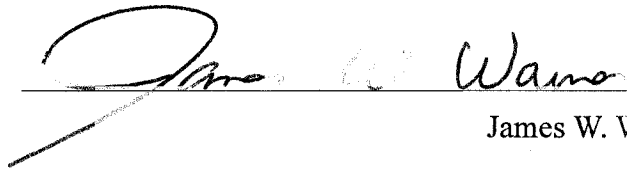
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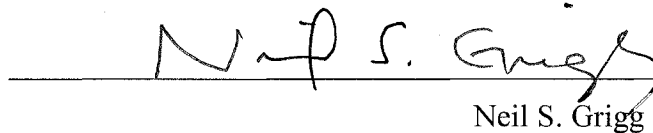
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ABSTRACT OF DISSERTATION

CONJUNCTIVE MANAGEMENT METHODOLOGY CONSIDERING STREAM-AQUIFER SYSTEMS AND RESERVOIR OPERATIONS

The purpose of this research was to develop a methodology to determine whether conjunctive surface water and groundwater management integrated with reservoir operation could significantly reduce deficits in a river basin with a relatively limited alluvial aquifer. The Geum River basin is one of major river basins in South Korea. The upper region of the Geum River basin is typical of many river basins in Korea where the shape of river basin is narrow with small alluvial aquifer depths from 10m to 20m and where most of the groundwater pumped comes quickly from the streamflow. The basin has two surface reservoirs, Daecheong and Yongdam. The most recent reservoir, Yongdam, provides water to a trans-basin diversion, and therefore reduces the water resources available in the Geum River basin. After the completion of Yongdam reservoir, the reduced water supply in the Geum basin resulted in increasing conflicts between downstream water needs and required instream flows, particularly during the low flow season.

Historically, the operation of groundwater pumping has had limited control and is administered separately from surface water diversions. Further, the operation of the two reservoirs has only considered the surface water demands and instream flow requirements in the development of operational policies. Given the limited size of the alluvial aquifer, it is apparent that groundwater pumping is essentially taking its water from the stream. Therefore, the operation of the surface water withdrawals and groundwater pumping must be considered together.

The major component of the proposed methodology is a goal-programming based, multi-objective optimization model that simultaneously considers reservoir release, surface water withdrawals, groundwater pumping and instream flow requirements. A 10-day time step is used in the model for a 24 year period of record. The interactions between groundwater pumping and the stream are handled through the use of response and lag coefficients. The impacts of pumping on streamflow are considered for multiple time periods. The model is formulated as a large linear goal-programming problem that is solved with the commercial LINGO optimization software package. The model was used to determine reservoir releases and groundwater pumping rates for a 10-day time step for a 24 year period of historical data from 1981 to 2004.

The results of the optimization model were analyzed and reservoir and groundwater

pumping operational rules were developed by linear regression. The developed operational rules were tested using the KMODSIM simulation model, which had been previously developed for the Geum river basin. The results of simulating various scenarios confirmed that the developed operation rules were able to yield an improved operation of the system, that is, meet the demands in the Geum river basin and reduce the deficits to the trans-basin diversion by as much as 15% as compared to a non-integrated operation.

In general this study was able to show that even though the aquifer system is small, there are possibilities to reduce deficits in the system through appropriate, integrated surface water withdrawal, groundwater pumping and reservoir release strategies.

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ACKNOWLEDGEMENTS

‘Happy those who do not follow the counsel of the wicked,
Nor go the way of sinners, nor sit in company with scoffers.
Rather, the law of the Lord is their joy; God’s law they study day and night.
They are like a tree planted near streams of water, that yields its fruit in season;
Its leaves never wither; whatever they do prospers. (PSALMS 1, 1-3)’

First of all I would like to thank the Lord to finish my dissertation successfully. I thank my father and mother, my wife’s father and mother, and all of my and wife’s families who devoted their time to us. I thank from the bottom of my heart to my wife Song Hellena, and young generation, Jun-Ho and Hyung-Min, to trust and follow me. I deeply thank my advisor, Professor Darrell G. Fontane to improve the quality of the dissertation and assist me in the process. I thank my committee members, Professor Neil S. Grigg, Professor Freeman Smith, and Professor James W. Warner, for the useful discussions and valuable suggestions. I thank Professor John W. Labadie, Professor Jose D. Salas, and Professor Jorge A. Ramirez, for their great teaching and considerations. I thank President Mr. Kwak Kyolho, ex-president Mr. Ko Seokgu to support. I thank Catholic Father Don and Kim Younggyo, and brothers and sisters, Cha Seunghwan, Oh Hyesuk, Ko Youngja, and Kang Hwanbee to peace and pray for us. I thank Mr. Won Inhee, Mr. Kim Seokhyon, and their family to encourage me. I thank Mr. Moon Taewan, Mr. Ko Ickhwan, Mr. Jeong Guyeol, Mr. Hwang Pilsun, Dr. Hwang Manha, Dr. Jeong Taesung, Mr. Yoon Bohun and Human Part staffs, and Water Resources Operations Center’s members to motivate the work. I thank Mr. Lee Hangu, Mr. Kim Bongjae, Mr. Jo Youngdae, Mr. Jeong Youngbae, Mr. Jo Youngduck, Mr. Shin Youngho, Mr. Lee Taesam, Mr. Lee Dongjin, Mr. Song Taejeong, Mr. Kim Jungyep, Mr. Lim Gwangsup, Mr. Jo Seongho, Ms. Ji Un, Mr. Park Daeryong to discuss the life. I thank Dr. Park Seongje, Dr. Lee Jaeyung, Dr. Park Sanggil, and Dr. Shin Hyonseok for their great advice. I thank my host families, Ms. Jean and Cindy, Fort Collins Korean Catholic families, and other Korean assiduous families to help in difficult situations. I often think the campus, big tree, house, people, sky, mountain, rain, snow, Canadian goose and other animals, and all of things that I love.

Cha Kee-uk
Fort Collins and Daejeon

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CHAPTER 1

INTRODUCTION

1.1 Background

Due to **population growth, industrial development, urbanization, and varied agricultural activities**, water resource engineers must manage water resources to identify the best solutions from both a regional and national standpoint. Concurrently, the concern over **environmental integrity of water resources and aquatic habitats is resulting in increasingly stringent environmental regulations that constrain water-resource development solutions**. The conflicts of **water resource development and environmental regulation** have created increasing challenges for hydrologists and environmental managers to design development strategies that simultaneously meet **water-resource demands and environmental goals**.

Along with **these problems**, water resource managers have searched for the best **management strategies and have used both optimization models and simulation models** to identify solutions. Conjunctive water management, including the stream, aquifer, and surface reservoirs is one potential management strategy. Conjunctive use must consider **the physical characteristics of the stream, aquifer, pumping, recharge, groundwater table**

variation, and surface reservoirs and related elements such as the technical, economic, environmental, and political factors that influence its application.

The conflicts that arise between resource development and conservation are particularly evident in the surface water resources used in the Geum River Basin located in the middle of South Korea. In the upstream portion of the Geum River, the management issues include managing the impact of a large trans-basin diversion, meeting instream flow requirements downstream of the reservoir, and existing groundwater use in close proximity to the river. Before the rise of downstream water quality problems and the trans-basin diversion after the completion of Yongdam reservoir in 2001, problems of water supply and environmental water use were virtually non-existent in this region. However, after the completion of Yongdam reservoir there are now many requirements for water use in the basin including downstream water rights, instream flow requirements for aquatic habitat and recreational use. These requirements have limited the amount of the streamflow that can be withdrawn from a stream and the amount of water that is stored in the reservoirs, particularly during the low flow season. These instream flow requirements, water rights, trans-basin diversions, and reasonable reservoir operations add great complexity to the evaluation of water resources planning alternatives. One method to solve these problems is to develop efficient water resources management strategies that incorporate conjunctive water management of surface and groundwater resources.

To develop conjunctive water management strategies it is necessary to formulate conjunctive water management models that consider the stream, the aquifer, and the reservoirs. These models could then be used to evaluate the potential water supply

deficits within the stream-aquifer system while considering consumptive water use and instream requirements. The models would be used to explore the performance of the stream-aquifer-reservoir system from a multi-objective management perspective, to develop surface reservoir operational policies that consider conjunctive use, and to evaluate the developed operational guide curves with appropriate simulation models.

The focus of conjunctive water management has mainly been on the stream-aquifer system and considers stream depletion response due to pumping. A particularly important variable in previous management studies that was often overlooked is the dynamic or transient behavior of the groundwater response (Barlow, 2003). However, optimal management strategies of conjunctive water use might be used to meet water resource needs while minimizing detrimental effects to stream and ground water, with an appropriate analysis of the dynamic and transient behavior of the ground water and surface water systems.

Substantial research in conjunctive use has been done in the United States and other countries (Bredehoeft and Young, 1970 and 1983; Danskin and Gorelick, 1985; Rastogi, 1989; Lefkoff and Gorelick, 1990a and 1990b; Matsukawa and others, 1992; Barlow and Hess, 1993; Fredericks and others, 1998; Belaine and others, 1999; Barlow and others, 1996, 2001, 2003; Fox, 2003; Barlow, 2003; Caldwell and Bowers, 2003; Dunning and others, 2004; Karamouz, 2004). However, comparatively little research in this field has taken place in South Korea.

Most streams in Korea have shallow alluvial aquifer zones from 10 m to 20 m in depth. The main source of water supply in most regions is surface water, including reservoirs. The Korean government has never progressively exploited groundwater. Due

to the difficulty in developing new reservoir projects, an increase in the NIMBY (Not In My Back Yard) attitude, and needs for environmental water use, water resources managers must manage the water resources much more carefully than in the past.

1.2 Objectives of Study

The objective of this study is to develop a multi-objective optimization framework to assist decision makers in improving the sustained yields of surface and ground water withdrawal. The framework will provide the ability to model the stream-aquifer system considering stream withdrawals and aquifer pumping (with single- and multiple-wells), to improve reservoir operational guide curves for conjunctive water management systems, and to determine allowable groundwater withdrawal during low flow seasons. The constraints of the system include instream flow needs, trans-basin diversion requirements at reservoirs, downstream reservoir water demands, limitations on groundwater withdrawal, withdrawal costs of groundwater and surface water, streamflow depletion factors due to well pumping, and reservoir characteristics. Of specific interest is the study of systems with small alluvial aquifers. The question is whether or not conjunctive management can provide any significant improvement in minimizing water shortages.

Optimization and simulation models were applied to evaluate the conjunctive water management systems that are representative of the river-valley and Yongdam reservoir in the upstream portion of the Geum River basin. Groundwater simulation models account for the physical characteristics of the stream-aquifer system, such as

streamflow depletion response and water table variations due to well pumping, whereas optimization models account for the management aspects of conjunctive water use, the optimal release from the reservoir, and the groundwater withdrawal. One of the primary goals of this research was to derive and evaluate optimization formulations that are appropriate for the conjunctive water management of the case study portion of the Geum River basin. This study evaluated integrated pumping strategies and reservoir operational guide curve strategies to attempt to minimize water shortages in the case study region. Conclusions about the effectiveness of conjunctive management in basins with small alluvial aquifers were drawn.

1.3 Organization of Dissertation

This study has been organized into seven chapters. Chapter One presents a background of the study and the objectives of this research. Chapter Two describes the previous work on surface water management, the analytical and numerical solutions of groundwater management equations, and the optimization formulations that deal with conjunctive water use. Chapter Three presents a statement of the methodology for stream-aquifer interaction, recharge, and optimization programming incorporated in this study. Optimization techniques for water resources system are discussed in this part. Chapter Four addresses the characteristics of the stream-aquifer system, recharge analysis, hydrologic features, and the system application in the case study area. The specific analysis for streamflow response due to well pumping, recharge in specific area, and groundwater table variation due to pumping is included. Chapter Five presents

conjunctive management models for the stream-aquifer system and the stream-aquifer-reservoir system. In the first section, the stream-aquifer-reservoir system model is formulated. The objective function and constraints of goal programming and response coefficient of conjunctive water management model are explained. In the second section, the stream-aquifer system model accounts for the potential yield of conjunctive water withdrawn from the stream and the groundwater zone. Chapter Six describes the application of the conjunctive water management system to the upstream portion of the Geum River in South Korea. The response of the stream-aquifer system including single withdrawal wells and multiple withdrawal wells, the use of the stream-aquifer-reservoir system to derive reservoir operational guide curves, and development and evaluation of operational guide curves are presented. The final chapter summarizes the results and conclusions obtained by this research. In addition to the summary and results, recommendations and improvements for future research are presented.

CHAPTER 2

LITERATURE REVIEW

Conjunctive use of surface water and ground water resources has a potential advantage during the drought season for efficiently managing the water resource. With proper management, conjunctive use of the water resources of an area can increase the efficiency, reliability, and cost-effectiveness of water use, particularly in regions with spatial or temporal imbalance of water demands and natural supplies (Louie and others, 1984; Willis and Yeh, 1987). Therefore conjunctive water use optimization modeling has been applied to a flow model of an alluvial aquifer and river system to develop withdrawal rates that could be sustained from both the alluvial aquifer and from the river.

This research study on conjunctive water management focuses primarily on system optimization of surface water and groundwater, stream-aquifer interaction by wells, groundwater recharge analysis, development of optimal reservoir operating rules considering the effects of instream flow and consumptive water use in downstream reaches, and evaluation of optimization model results.

There are a wide variety of theoretical and applied issues in stream-aquifer interaction and the optimal reservoir management. This chapter reviews the basic concepts of conjunctive water management.

2.1 Surface Water Management

Conjunctive water management combines surface and groundwater resources for consumptive water use, sustainable instream flow, and other purposes. Both surface and groundwater storage are used to distribute water in time to match supply to demands. However, surface and groundwater storage differ in terms of storage capacity, recharge and depletion rates, capital and operation costs and specific constraints. Jointly operating all manageable water resources in a region can potentially increase the yield, efficiency, supply reliability and cost effectiveness for a water system. River basin management has become increasingly more complex and dynamic due to growing conflicts among competing objectives such as municipal and industrial water demand, instream requirement for fish habitat and aquatic life, hydropower production, water quality, recreational water use, and flood control.

System analysis models used in studies to optimize river and reservoir system operations may be categorized as follows: descriptive simulation models, prescriptive optimization models, and integrated simulation and optimization models. Solving water management conflicts requires an efficient river basin water modeling system that provides both descriptive and prescriptive models that allow the decision maker to easily modify operating policy and physical and economic characteristics of a river basin (Eschenbach, 2001).

Yeh (1982, 1985) presents reviews of reservoir management and operation models, and includes an extensive list of references. Wurbs and others (1993) survey

systems techniques applied to reservoir operation. Labadie (1997, 2004) reviewed the optimization of reservoir system management and operation, and discussed future directions for additional research and applications.

2.2 Groundwater Management

Groundwater plays a principal role in water supply during the drought season. Groundwater storage is often a significant resource that can help in buffering surface water supply reductions. Compared with surface water storage, groundwater storage offers vast storage reserves in most watersheds.

Hall and Dracup(1970) describe groundwater management from the viewpoint of safe yield, which is the amount of water that can be withdrawn annually without causing an undesirable influence in the basin.

Bredehoeft and others (1982) describe the changes that occur in a groundwater system as a consequence of groundwater development. In most hydrogeologic settings, groundwater capture results in streamflow depletion, with little contribution from changes in mean recharge. Streamflow depletion consists of two components such as induced infiltration of streamflow and captured groundwater discharge that would have discharged to a stream in the absence of pumping. Variables that affect the timing of the sources of water to a well are the aquifer parameters, boundary conditions, and locations of development within the system.

The hydrologic-budget equation contains information on the relation between the two types of commonly practiced groundwater management strategies. Sustained

groundwater yield is equal to the capture and groundwater mining is equal to the permanent change in storage. There is a common misperception that the water budget of a groundwater basin determines the magnitude of sustained yield of a basin - that is, that the amount of sustained yield is equal to the natural recharge rate to the basin. The sustained yield of a basin is limited by the amount of water that a well is able to capture from recharge or discharge as noted by Barlow (1997, 2001, 2004). As noted by Bredehoeft and others (1982) and Bredehoeft (1997), the ultimate volume of sustained groundwater withdrawal depends on the level of hydrologic impacts that a community is willing to endure. The recharge rate affects the relative contributions of the two components of streamflow depletion such as induced infiltration and captured discharge. There are many regional studies on streamflow depletion rates, stream-aquifer interaction, and stream-aquifer optimization systems.

2.2.1 Analytical Solutions

The analysis of analytical solutions for well and stream interaction has been investigated in the previous studies. These studies looked at steady-state and transient conditions. Stream depletion rate (SDR) has primarily focused on hydraulic connection between a stream and an aquifer for pumping wells in alluvial valleys. Researchers include Theim (1906), Theis (1941), Glover and Balmer (1954), Glover (1960), Theis and Conover (1963), Hantush (1965, and 1967), Jenkins (1968), Wilson (1986, 1993), and Hunt (1999), Vitaly and Daniel (2005).

After a long period of pumping from a well or recharging to a well, the groundwater flow around the well approaches steady state. This means that the potentiometric head at any point in the groundwater flow field does not change with time. The time required to reach steady state depends on the hydraulic characteristics of the aquifer. Steady state solutions for confined and unconfined aquifers play an important role in analyzing the drawdown based on hydraulic characteristics of the aquifer. Theim (1906) was the first to derive the flow solution for a fully penetrating well in a confined aquifer under steady-state conditions. Dupuit (1863) derived the flow solution for a fully penetrating well for unconfined aquifers.

Theis (1941) and Glover and Balmer (1954) develop an analytical solution for the case of a stream perfectly connected and a fully penetrating homogeneous, isotropic aquifer. Jenkins (1968b) indicates that the volume of streamflow depletion approaches the volume of water pumped by the well as pumping time approaches infinity. This author described the use of a model represented in a set of curves and tables for transient conditions that relates the rate and volume of streamflow depletion to the rate of pumping at a well. Aquifer storage in the groundwater zone, captured discharge and induced infiltration by pumping in the study are analyzed. The study assumed a homogeneous, isotropic, and semi-infinite aquifer from a fully penetrating pumping well. All solutions treat transmissivity of the aquifer as constant with time; thus, for a water-table aquifer, drawdowns are assumed to be negligible compared to the saturated thickness of the aquifer. The stream that forms a boundary of the aquifer is assumed straight, hydraulically connected to the aquifer, and fully penetrates the aquifer.

Sophocleous and others (1988) applied three analytical solutions to the analysis of field data from a pumping test near the Arkansas River in central Kansas. Their results indicated that the amount of streamflow depletion caused by pumping during the eight day test was appreciably less than the computed depletion based on the analytical solutions. They attribute the difference between observed and calculated streamflow depletion to have been caused by the partial penetration of the stream and the fact that most of the water moving from the stream toward the well had to move vertically downward through a clay layer that was semiconfining the main aquifer.

Although the results of Sophocleous and others (1988) point to some of the deficiencies of analytical solutions, the solutions are useful for elucidating some of the factors that control streamflow depletion. Several of these factors have relevance to this research, including: (1) the ambient direction and rate of groundwater flow in the aquifer; (2) hydraulic characteristics of and distance to lateral boundaries; (3) transmissivity and storage properties of the aquifer; (4) distance of the well from the stream; (5) pumping rate of the well; and (6) duration of pumping.

The work of Newsom and Wilson (1988) and Wilson (1993) have shown that the relative contributions of captured discharge and induced infiltration to the total amount of streamflow depletion are related to the pumping rate of the well, distance of the well from the stream, and the ambient flow rate through the aquifer (which is a function of the recharge rate to the aquifer).

Wilson (1981, 1986) examines flow of groundwater to a well near a gaining stream. The effect of ambient flow direction on streamflow depletion by well pumping was analysed by Newsom and Wilson (1988). In their conceptual model, flow nets for

different angles of ambient flow and dimensionless pumping rates are used to illustrate how the model works. The direction of ambient groundwater flow with respect to the direction of the stream can vary from perpendicular to parallel. The approach includes the assumptions such as the aquifer extends to infinity away from the stream, flow is two dimensional in the aquifer, the stream is fully penetrating, straight, and hydraulically connected to a homogeneous and isotropic aquifer without a clogging layer in the streambed, and the well is fully penetrating.

Wilson (1993) extended the analysis of induced infiltration from a well pumping in a semi-infinite aquifer, a well between a stream and a barrier, and a well between two streams. This analysis employed almost all of the assumptions of Newsom and Wilson (1988), except flow was assumed to be perpendicular to the stream channel for all analyses.

Hantush (1965) developed a solution for stream-aquifer systems where a semipermeable layer exists. Since the stream is assumed to fully penetrate the aquifer, application of these solutions to a partially penetrating stream can overestimate the rate of depletion. Hunt (1999) developed transient solution for the stream depletion rate induced by a pumping well in a nonleaky aquifer near a shallow partially penetrating stream by assuming an impermeable aquitard, and the pumping rate supplied fully by the stream depletion.

Vitaly and Daniel (2005) derive a solution for drawdown and stream depletion for shallow aquifer penetration by a stream and hydraulic aquifer connection with the underlying bed. This solution extends the results obtained by Theis, Glover and Balmer, Hantush and Jacob, Hantush, and Hunt. The authors show that both hydraulic stream-

aquifer connection and hydro-stratigraphic conditions determine Maximum Stream Depletion Rate, which is defined as a maximum fraction of the pumping rate supplied by the stream depletion.

2.2.2 Numerical Solutions

Streamflow depletion induced by groundwater pumping is an increasing concern of water resources departments in many countries. This concern occurs especially in countries or states that undergo periodic droughts. A typical scenario is one in which groundwater pumping intercepts water that would have discharged to the stream under natural flow conditions. As streamflow of a river is reduced, water problems in relation to water use and natural habitat of fish and aquatic organisms can occur.

Analytical solutions provide simple models that are easy to apply but have the disadvantage that the boundary conditions simplify the complex nature of the real problem. Numerical approaches that simulate groundwater and surface water flow are widely used to evaluate and manage groundwater and surface water resources. Numerical models have become important tools for analyzing the effects of steady state and transient conditions such as areal recharge, discharge, drains, evapo-transpiration, and stream-aquifer interaction by pumping. The determination of stream stage as part of the numerical solution is an improvement over analytical solutions in which stream stage is a specified boundary condition. Numerical modeling approaches are often used in simulating stream-aquifer systems.

The three-dimensional, finite-difference model MODFLOW (McDonald and Harbaugh, 1988; Harbaugh and McDonald, 1996, 2000) is a widely used simulation model for the analysis of groundwater flow systems assuming a constant fluid density and alignment of the principle axes of hydraulic conductivity with the three coordinate directions x , y , and z . The model has four algorithms that represent the current breadth of approaches used for simulating stream-aquifer interaction and serve as a background for understanding numerical simulation of these systems. It has a modular structure that allows it to be easily modified to adapt the code for a particular application. MODFLOW-2000 simulates steady and unsteady flow in an irregularly shaped flow system in which aquifer layers can be confined, unconfined, or a combination of confined and unconfined. Flow from external stresses, such as flow to wells, areal recharge, evapotranspiration, flow to drains, and flow through river beds, can be simulated. The model can handle situations where hydraulic conductivities or transmissivities for any layer may differ spatially and be anisotropic, and the storage coefficient may be heterogeneous. Specified head and specified flux boundaries can be simulated as can a head dependent flux across the outer boundary of the model. This allows water to be supplied to a boundary block in the modeled area at a rate proportional to the current head difference between a source of water outside the modeled area and the boundary block.

A new Streamflow-Routing (SFR1) program was developed by Prudic (1989) for use with the MODFLOW groundwater model and the Groundwater Transport (GWT) Process. The package replaces an older Stream (STR1) Program that was developed for earlier versions of MODFLOW. The program is designed to simulate stream-aquifer

interactions and to route flow and a single solute through a network of surface-water channels. The new version offers improved efficiency in the input of data and offers more options in computing stream depth and width.

Swain and Wexler (1993) developed an approach for simulating stream-aquifer interaction that explicitly couples groundwater and surface water flow equations. Their model, called MODBRANCH, links MODFLOW with the surface-water flow model BRANCH (Schaffranek and others, 1981). The surface water flow model is based on the St. Venant equations of continuity and motion.

To facilitate the simulation of the surface water and groundwater interaction, the Diffusion Analogy Surface-Water Flow Model (DAFLOW) has been coupled to the modular, finite difference, groundwater flow model MODFLOW by Jobson and Harbaugh (1999). The program routes flows through a system of inter-connected one dimensional channels and subdivides the system into a series of branches. The models are coupled by adding an exchange between each subreach and a specified groundwater cell, with the amount of flow computed from the stream to groundwater. It is designed to simulate flow in upland stream systems where flow reversals do not occur and backwater conditions are not severe (Jobson and Harbaugh, 1999).

Barlow (1997, 2001) has shown that the amount extracted by pumping can be significantly increased without any significant increase in streamflow depletion by the location and timing of pumping (Ahlfeld, 2002).

Several researchers have used numerical models to evaluate the applicability and limitations of the steady-state and transient analytical solutions of stream-aquifer interaction. Conrad and Beljin (1996) found that areal recharge can not be neglected if

the groundwater basin receives more than 5 in/yr of areal recharge and that partial penetration can not be neglected if the ratio of horizontal to vertical hydraulic conductivity of the aquifer sediments is greater than 10.

Sophocleous and others (1995) evaluated some of the limitations of steady-state and transient analytical solutions developed by Glover (1974) for a well pumping in a semi-infinite stream-aquifer system. The authors compared streamflow depletions calculated by the analytical solutions to those calculated using finite-difference models and found that the finite-difference models simulated stream-aquifer systems more realistically than the analytical solutions. Streambed clogging and partial penetration of streams caused significant differences in the rates of streamflow depletion calculated by the analytical and numerical solutions. These results indicate that accurate representation of the stream-aquifer system at the stream-aquifer interface is particularly important to the source of well discharge. Varying specific yield, average hydraulic conductivity, penetration depth of the well, and stream-stage levels, however, resulted in relatively minor differences in rates of streamflow depletion calculated by the steady state and transient analytical solutions.

Numerical simulation is needed for stream-aquifer systems with varying specific yield, a clogging zone, and partial penetration of the aquifer where a simplified assumption of an analytical solution cannot be applied. Several example applications of numerical simulation of stream-aquifer systems provide further insight into the many variables that affect stream-aquifer interaction. Barlow and Hess (1993) provides an example of the use of MODFLOW with Prudic's (1989) algorithm to evaluate the response of the Quashnet River stream-aquifer in the glacial-outwash deposits of Cape

Cod, Massachusetts to proposed groundwater pumping. A sensitivity analysis of model parameters indicated that calculated streamflow was most sensitive to the specified value of hydraulic conductance of streambed deposits and also highly sensitive to recharge of the aquifer. Also calculated streamflow was not very sensitive to the specified values of horizontal hydraulic conductivity or the ratio of horizontal to vertical hydraulic conductivity of the aquifer deposits.

Pinder and Sauer (1971) used a two-dimensional numerical model based on the processes of flood-wave propagation and bank storage. Their numerical simulations indicated that flood waves may be reservoirped considerably by bank storage, particularly in the lower segments of a long reach, that the modification to the flood-wave was influenced strongly by the hydraulic conductivity of the aquifer and was nearly insensitive to the width of the aquifer, and that the response of the aquifer to the propagation of a flood wave decreases rapidly with distance from the stream.

The ability of numerical models to simulate partially penetrating wells and streams, semi-permeable stream bank and streambed materials, and aquifer heterogeneity provide advantages over analytical solutions for accurate representation of complex field conditions. These factors are particularly important in calculating accurate streamflow-depletion rates and relative contributions of captured discharge and induced infiltration to supply wells.

2.3 Optimization Water Management Systems

Conjunctive water management models for the physical behavior of stream-aquifer-reservoir systems are important for optimal water resources management. The conjunctive water use model provides an optimal allocation schedule for groundwater and surface water resources in a region to meet different water demands. Gorelick (1983) distinguishes two categories of combined management models as distributed aquifer simulation for hydraulic management models and policy evaluation and allocation models. Hydraulic management models are principally concerned with managing flow, heads and mass transport in the aquifer. In contrast, policy evaluation and allocation models are mainly concerned with the economically efficient allocation of surface and groundwater resources. Many conjunctive use optimization models are available in the literature. Such models typically use linear, non-linear or dynamic programming optimization techniques with a dynamic balance of relevant elements, appropriate constraints, and a single or a multiple objective (Lall 1995).

Linear programming has been the most widely used technique in conjunctive use optimization models. Linear formulation can express easily the relationship between water use and the stream, aquifer or reservoir. However, nonlinearities may arise due to the physical representation of the system, evaluation of energy from the reservoir operation, the relation between pumping and drawdown of unconfined aquifers, and the relationship of stream stage and discharge or reservoir release. Economic-driven conjunctive use optimization models have to address the nonlinearity of the pumping

costs, which is a function of the product of pumping head and pumping rate at the production well. The traditional approach is to express the drawdowns in term of pumping rates using response coefficients so as to express the objective function in a quadratic form. However, drawdowns are a function of not only well pumping rate but also the recharge to the aquifer, which depends on reservoir release, percolation rates, and infiltration from the recharge sub-basin (Velazquez, 1997).

For nonlinear systems, nonlinear programming and dynamic programming have been applied (Yeh, 1992). Alternatively separable programming techniques may lead to solutions using quadratic programming or by linear programming using piecewise approximations of the resulting quadratic functions. Application of classical dynamic programming to groundwater management problems is usually restricted to lumped parameter models, due to the constraints imposed by the “curse of the dimensionality” (Bellman, 1957). Labadie (1999) developed generalized dynamic programming software to solve the single and multi-dimensional problems. This software has been widely used in the dynamic programming field.

Complex and detailed groundwater management decisions require groundwater to be represented at a level of detail afforded only by simulation models. In coupled simulation-optimization models, a simulation model produces the response curve between the stream and aquifer and evaluates mass balance. The optimization model considering the lag time of streamflow depletion uses the information calculated in the simulation model. The models exchange data at each time step, daily or monthly. The response characteristic between the stream and aquifer incorporated into the surface water model is considered using the response matrix approach.

Despite the many different optimization models and techniques that have been applied, most conjunctive use optimization deals with hypothetical problems in stream-aquifer systems with steady state or transient state conditions. The models that can produce more detailed surface and groundwater interaction generally do not account for the economic aspects of water allocation. There is an absence of generalized large-scale optimization models for conjunctive use in which the surface and groundwater interaction is included with significant detail (Velaquez, 1997).

Management models that link the physical behavior of stream-aquifer and reservoir systems with the optimization formulation are important tools for optimal water resource management strategies. The use of operations-research methods for ground water planning and management has been reviewed by Gorelick (1983, 1990), Wills and Yeh (1987), Yeh (1992), Ahlfeld and Heidari (1994), Fredericks and others (1998), Belaineh and others (1999), and Ahlfeld and Mulligan (2000). A key component of a water resource optimization problem is the identification of design criteria and their formulation within the framework of an optimization problem (Ahlfeld and Heidari, 1994; Ahlfeld and Mulligan, 2000).

2.3.1 Stream-Aquifer System

In the management of stream-aquifer systems, the design objective is typically to maximize the water uses, under constraints of minimum cost, maximum allowable drawdowns, or minimum streamflow depletion. Alluvial valley stream-aquifer systems are important sources of water supply. These systems are typically characterized by an

aquifer that is narrow in width with alluvial zones near the stream. Many researchers have studied these kinds of systems (Maddock, 1972, 1974; Morel-Seytoux, 1975; Danskin and Gorelick, 1985; Rastogi, 1989; Lefkoff and Gorelick, 1990a and , 1990b; Matsukawa and others, 1992; Muller and Male 1993; Barlow and others, 1997, 2001, 2003; Belaineh and others, 1999; Karamouz and others, 2004).

With the shift in societal values toward a broader definition of water use that includes both economic and environmental criteria, design objectives for stream-aquifer systems also are shifting to the multi-objectives of maximization of net economic benefit with simultaneous minimization of streamflow depletions. In this context, Muller and Male (1993) have evaluated the allocation of groundwater withdrawal permits in the Charles River basin of Massachusetts with the objective of minimizing the amount of time that streamflow falls below a specified minimum value.

In many formulations in which pumping is the decision variable, costs of pumping are considered to be a quadratic function of the lift. Costs are quadratic because drawdown is a function of the pumping rate. Danskin and Gorelick (1985) used a method involving mixed-integer programming in order to constrain nonlinear pumping costs. Maddock (1974) used a probabilistic objective that minimized the expected value of the costs.

In conjunctive water management models, decision variables are typically the rates of water extraction from the stream-aquifer system, or diversion rates of streamflow or the rates of groundwater pumping. Constraints placed on the decision variables are typically upper and lower bounds on pumping and groundwater storage. Constraints are also included on state variables of the system. Probabilistic constraints also can be

defined, as was done by Hantush and Marino (1989) using chance-constraints on streamflow and on drawdowns.

Several researchers also have used nonlinear optimization approaches to relate decision and state variables (Rastogi, 1989; Hantush and Marino, 1989; and Matsukawa and others, 1992). While Rastogi (1989) and Matsukawa and others (1992) used numerical solutions of the stream and groundwater flow equations, Hantush and Marino (1989) used an analytical equation to relate pumping from an unconfined aquifer to streamflow depletion.

Danskin and Gorelick (1985) used an iterative formulation of the linear response functions described above for the analysis of nonlinear, unconfined aquifers, in which the response functions are updated iteratively as transmissivity from one iteration to the next is below a specified tolerance.

Gupta and Goodman (1985) used a three-dimensional hydrodynamic groundwater model with a multi-level management model to formulate a composite model of groundwater reservoir operation for drought management for regions of stratified drift of the Northeastern United States. The authors also evaluated artificial measures to reduce induced infiltration caused by groundwater withdrawals. Among the infiltration reduction measures studied, they found that a vertical semi-pervious core (constructed by a slurry trenching process) running parallel and in close proximity to the stream and extending through the entire thickness of the aquifer can reduce losses from over 60 percent to less than 40 percent.

Lettenmaier and Burges (1982) describe the concept of 'cyclic storage', which is the storage of excess surface-water runoff in an aquifer during high streamflow years and

withdrawal from groundwater storage during low-flow years. Cyclic storage has been viewed as the management of ground- and surface-water storage systems with emphasis on long-term performance, without necessarily considering the hydraulic connection between the groundwater and surface-water systems.

Barlow (1997) applied the simulation and optimization modeling approach to a hypothetical alluvial-valley stream aquifer system representative of the northeastern United States. He used an iterative response matrix technique to solve the nonlinearity of head dependent boundary conditions of the unconfined aquifer for the stream-aquifer system. He found that the iterative approach was actually not necessary to effectively model the stream-aquifer system.

Belaineh and others (1999) state that conjunctive water use simulation/optimization models will especially benefit irrigated agriculture in areas where there is significant interflow between surface and groundwater resources. Their model computes optimal groundwater pumping (extraction or injection), reservoir release, stream diversion, and the resulting system response, including aquifer hydraulic head, stream reach outflow, and reservoir storage. The authors tested improving stream aquifer interflow and this improvement permitted 13% more water use.

Barlow and others (2003) developed conjunctive management models to evaluate trade-offs between groundwater withdrawals and streamflow depletions for an alluvial-valley stream-aquifer system in Rhode Island. The developed model was used to assess the effect of interannual hydrologic variability on minimum monthly streamflow requirements. The model increased the amount of current withdrawal from the aquifer by

as much as 50% by modifying current withdrawal schedules and modifying the number and configuration of wells in the supply well network.

Mohammad and others (2004) studied monthly water resources and irrigation planning by conjunctive use in the Tehran metropolitan area. A systematic approach to surface and groundwater resources modeling in the study area is discussed. A dynamic programming optimization model was used for conjunctive water use planning in the study. The objective function of the model is developed to supply the agricultural water demands, to reduce pumping costs, and to control groundwater table fluctuations. The simulation of the optimal policies developed from the optimization model showed that groundwater table variations could be controlled.

2.3.2 Stream-Aquifer-Reservoir System

Many advances have been made in the development of mathematical models for conjunctive water management and reservoir management. However, conjunctive use models that have detailed stream/aquifer system interactions have not usually considered reservoir management rules. Conversely, reservoir management models have not usually simulated stream/aquifer interactions in detail. There is a need for enhanced linking of reservoir and stream/aquifer systems in conjunctive water management models (Lettenmaier and Burges, 1982; Belaineh and others, 1999; Fredericks and others, 1998).

Lettenmaier and Burges (1982) considered the cyclic performance of a water-resource system consisting of a single surface-water reservoir and adjacent aquifer storage that can be artificially recharged. The reservoir and aquifer are operated as a

coupled system to buffer the effects of annual high and low streamflows. Their study was concerned with two characteristics of the coupled system: reliability of the system in meeting a fixed demand and total system cost. There are a few optimization applications for the stream-reservoir problems. Belaineh and others (1999) used a simulation/optimization model for water resources management. This study managed the reservoir via a compromise linear reservoir decision rule (LDR), which can alternate between the S-Rule and SQ-Rule to avoid infeasibilities. The authors found that the enhancement of the simulation/optimization model to include aquifer recharge due to canal conveyance losses and percolating irrigation water allowed groundwater pumping to increase by 80%.

Fredericks and others (1998) presented a decision support system (DSS) for conjunctive management of surface water and groundwater under prior appropriation. The decision support system is constructed around the generalized river basin network flow model MODSIM. The decision support system includes the capability of utilizing groundwater response coefficients generated from a ground water flow model and a management capability for analysis of various conjunctive use scenarios. This model was applied to the South Platte River basin using two different sets of response coefficients: (1) numerical coefficients calculated using the MODRSP finite-difference ground water model; and (2) analytical coefficients calculated with the Glover equation using predefined stream-flow depletion factor (SDF) values. The authors found that the analytical method using SDF coefficients produces significantly lower net river return flow values when compared with results from the numerically based finite-difference

coefficients. This difference is primarily attributed to improved inclusion of tributary inflows in the simulation using the MODRSP finite-difference coefficients.

2.3.3 Operational Guide Curves

Real time operation of river and reservoir systems requires specific operating guide rules. These rules are guides for water conservation and release policies prepared for reservoir operators. There are several types of rules for guide curves ranging from simple and static to complex and dynamic. These rules typically consider the varying states of inflow and physical characteristics of a reservoir in each time period. Rule curves usually specify the target storage at the end of each month. Rule curves are forms of simple operating policies that often do not use forecasted information such as predicted inflows to the reservoirs in the coming months. Linear rules are as good as or sometimes better than nonlinear rules (Bhaskar and Whitlatch, 1980). Vasiliadis and Karamouz (1994) developed optimal operating policies derived from a stochastic dynamic programming model. Neural networks and fuzzy systems have been widely used for reservoir operation modeling. These techniques estimate a function without an underlying mathematical model of how an output depends on inputs (Hasebe and Nagayama, 1996). Application of these techniques for defining reservoir operation policies has been considered by Roman and Chandramouli (1996).

The preceding discussion has described the basic problem definition, analytical and numerical models, and solution methods for conjunctive management of stream-

aquifer and reservoir systems. Each conjunctive water management model consists of an objective function, a set of constraints, and decision variables.

For this study, the objective function attempts to meet water demand and instream requirement using both groundwater and surface water, and then to determine the storage rule based operational guide curves during drought conditions. The decision variables are the water demands, the instream requirement, rates of groundwater withdrawals, and reservoir release for specific periods. The constraints include pumping capacities, available drawdown, allowed streamflow depletions, instream requirement, allowed reservoir release, and reservoir storage levels.

CHAPTER 3

BASIC EQUATIONS FOR CONJUNCTIVE WATER MANAGEMENT

3.1 Summary of Conjunctive Water Management Systems

Many physical factors affect conjunctive water management systems including streams, aquifers, and reservoirs. Surface water management includes the operation of single- or multipurpose reservoirs, diversion canals, control structures, water and/or wastewater treatment plants, and flood control structures. System analysis considers the impacts resulting from those individual components on the other components in the surface water system. Integrated water resources management systems explore the performance of individual components and the performance of the entire system.

Groundwater models are physically based mathematical models derived from Darcy's law and the law of conservation of mass. Various established solutions utilizing either finite difference or finite element approximations are available for solving the governing equation of the model, provided that model parameters and initial and boundary conditions are properly specified. Groundwater models are used as tools for

decision making in the management of a conjunctive water system. The accuracy of model prediction depends on the reliability of the estimated parameters as well as on the accuracy of prescribed initial and boundary conditions. Hydrogeological information about an aquifer is used in the estimation of initial and boundary conditions. In practice model parameters are required to be estimated from historical observation data.

This chapter describes the basic equations for surface water and groundwater models, groundwater recharge, stream-aquifer interactions, groundwater flow, and optimization schemes for conjunctive water management system.

3.2 Stream-Aquifer Interaction

3.2.1 Analytical solution

3.2.1.1 Steady State Solution

Wilson (1993), assuming steady-state conditions due to well pumping, extended previous work for the analysis of induced infiltration from a well pumping in a semi-infinite aquifer, a well between a stream and a barrier, and a well between two streams. This author used almost all of the assumptions of the previous work, however flow was assumed to be perpendicular to the stream channel for all analyses. Equation 3.1 considers the case of a semi-infinite aquifer in which the ambient flow rate through the aquifer (q_a) supports the pre-pumping streamflow:

$$Q_s = -2q_a x' + \frac{2Q_w}{\pi} \tan^{-1} \left(\frac{x'}{d} \right) \quad (3.1)$$

where Q_s is streamflow depletion; Q_w is pumping rate of the well; q_a is uniform ambient flow rate in the aquifer; x' is half of the distance along the stream over which induced infiltration takes place; and d is perpendicular distance from the well to the stream.

The change in the rate of streamflow depletion (dQ_s) as a function of the change in the groundwater withdrawal rate beside the stream (dQ_w) is found by differentiation of Equation 3.1.

$$\frac{dQ_s}{dQ_w} = \frac{2}{\pi} \tan^{-1}\left(\frac{x'}{d}\right) \quad (3.2)$$

Also, Wilson (1993) derived Equation 3.3 for the case of well between a stream and a barrier and Equation 3.4 for the effect of well between two streams. Both equations can be similarly differentiated to give rates of streamflow depletion for an aquifer bounded by a stream and impermeable barrier in Equation 3.5 and for an aquifer bounded by two streams in Equation 3.6.

$$Q_s = -2q_a x' + \frac{2Q_w}{\pi} \tan^{-1}\left(\frac{x'}{d}\right) \quad (3.3)$$

$$Q_s = -2q_a x' + \frac{Q_w}{\pi} \cos^{-1}\left(\frac{1 - \cosh(\pi x' / L) \cos(\pi d / L)}{\cosh(\pi x' / L) - \cos(\pi d / L)}\right) \quad (3.4)$$

$$\frac{dQ_s}{dQ_w} = \frac{2}{\pi} \tan^{-1} \left(\frac{\sinh \pi x' / 2L}{\sin \pi d / 2L} \right) \quad (3.5)$$

$$\frac{dQ_s}{dQ_w} = \frac{1}{\pi} \cos^{-1} \left(\frac{1 - \cosh(\pi x' / L) \cos(\pi d / L)}{\cosh(\pi x' / L) - \cos(\pi d / L)} \right) \quad (3.6)$$

where L is the distance between the stream and barrier in Equation 3.5; and L is the distance between the two streams for Equation 3.6.

Equations 3.2, 3.5 and 3.6 are also used to calculate streamflow depletion between two points along a stream. For well pumping from a semi-infinite aquifer, streamflow depletion between points x_a and x_b is represented as:

$$\frac{dQ_s}{dQ_w} = \frac{1}{\pi} \left[\tan^{-1} \left(\frac{x_b}{d} \right) - \tan^{-1} \left(\frac{x_a}{d} \right) \right] \quad (3.7)$$

As the distance between the two points x_a and x_b reaches infinity, the total rate of streamflow depletion equals the total rate of withdrawal.

The author presents an equation that is related to induced infiltration from a stream as a function of a dimensionless pumping parameter β :

$$\frac{Q_s}{Q_w} = \frac{2}{\pi} \left[\frac{-\sqrt{\beta-1}}{\beta} + \tan^{-1} \sqrt{\beta-1} \right] \quad (3.8)$$

Where Q_s is the rate of induced infiltration and the dimensionless pumping parameter, β is:

$$\beta = \frac{Q_w}{\pi d q_a} \quad (3.9)$$

The only variables that affect the rate of streamflow depletion at steady state are the spatial dimensions of the stream-well-aquifer system and not the hydraulic properties of the aquifer such as transmissivity or hydraulic conductivity from these equations.

3.2.1.2 Transient Solution

For modeling the aquifer hydraulic and hydrologic situation, dimensionless constants that characterize certain aspects of streamflow and aquifer response systems are used. The reason for using such dimensionless constants is that these constants allow the evaluation of how different parameters or groups of parameters affect the response of the physical system (Barlow and others, 2003). In the case of stream-aquifer hydraulics, dimensionless constants provide insight into the relation between the timing of groundwater withdrawals and streamflow depletion.

One dimensionless parameter that has been used in many types of stream-aquifer problems is the characteristic response time factor described by Glover (1954) and Jenkins, (1968) as:

$$sdf = \frac{d^2 S}{T} \quad (3.10)$$

where sdf is streamflow depletion factor in time units, d is a perpendicular distance between the stream and pumping well, S is storativity or specific yield, and T represents transmissivity. This streamflow depletion factor has been referred to as “aquifer response time” by Sophocleous and others (1995). The streamflow depletion factor is a widely used model for estimating the rate and volume of stream depletions induced by groundwater pumping, as well as stream accretions induced by groundwater recharge.

Jenkins created the term “streamflow depletion factor (sdf)” to serve as a simple but useful input to the Glover analytical model. The Glover equation relates streamflow depletion rate (Q_s) to aquifer pumping rate (Q_w) as a function of aquifer properties and time such as perpendicular distance from the stream to the well, storativity, and aquifer transmissivity. The Glover equation has error function equations. Integrating the Glover equation gives a cumulative depletion volume that shows the relationship between depletion volume (v) and pumped volume ($Q_w \cdot t$). Jenkins defines the term “response curve” as “a curve showing the relation between pumping time and volume of streamflow depletion.

$$\frac{Q_s}{Q_w} = \operatorname{erfc} \left(\sqrt{\frac{d^2 S}{4 \cdot T \cdot t}} \right) \quad (3.11)$$

Equation 3.11 can be written as a function of the sdf :

$$\frac{Q_s}{Q_w} = \operatorname{erfc}\left(\sqrt{\frac{sd}{4t}}\right) \quad (3.12)$$

Integrating the Glover rate equation gives the cumulative depletion volume (v).

The cumulative depletion volume is a fraction of the total cumulative pumping volume.

$$\frac{v}{Q_w \cdot t} = \left(\frac{d^2}{2tT/S} + 1\right) \operatorname{erfc}\left(\frac{d}{\sqrt{4tT/S}}\right) - \left(\frac{d}{\sqrt{4tT/S}}\right) \left(\frac{2}{\sqrt{\pi}}\right) \exp\left(\frac{-d^2}{4tT/S}\right) \quad (3.13)$$

$$\frac{v}{Q_w \cdot t} = \left(\frac{sd}{2t} + 1\right) \operatorname{erfc}\left(\sqrt{\frac{sd}{4t}}\right) - \left(\sqrt{\frac{sd}{4t}}\right) \left(\frac{2}{\sqrt{\pi}}\right) \exp\left(\frac{-sd}{4t}\right) \quad (3.14)$$

Factors that affect transient streamflow depletion are the semi-infinite boundary conditions of the aquifer, the distance of the well from the stream, and hydraulic properties of the aquifer like specific yield and transmissivity.

Jenkins also showed that streamflow depletion could continue after pumping stops, which is important to the design of optimal, time-varying pumping schedules. Jenkins observed that the residual effects of pumping on streamflow might be greater than the effects during pumping. This means that streamflow depletion will continue after pumping stops. This observation points out the importance of transient effects in stream-aquifer interaction. In particular, in the management of stream-aquifer systems, time-varying pumping schedules may be needed to avoid streamflow depletions during critical periods that would cause degradation of aquatic habitats.

3.2.2 Numerical Models

Numerical models have become the most often used analysis tool for analysis of stream-aquifer interaction, including the determination of streamflow depletion rates. The additional benefits to stream-aquifer analysis by the development of two dimensional and three dimensional numerical models lie primarily in the ability of the models to simulate multi-dimensional, distributed-parameter systems. Also, the model can analyze the non-leaky, leaky, steady-state, and transient conditions.

3.2.2.1 Groundwater Flow Equation

Under the condition of a constant fluid density and alignment of the principal axes of hydraulic conductivity with the three coordinate directions x , y , and z , the three-dimensional equation of ground water flow for fully saturated, confining conditions is represented by McDonald and Harbaugh (1988) as:

$$\frac{\partial}{\partial x}\left(K_{xx} \frac{\partial h}{\partial x}\right) + \frac{\partial}{\partial y}\left(K_{yy} \frac{\partial h}{\partial y}\right) + \frac{\partial}{\partial z}\left(K_{zz} \frac{\partial h}{\partial z}\right) - W = S_s \frac{\partial h}{\partial t} \quad (3.15)$$

where K_{xx}, K_{yy}, K_{zz} are hydraulic conductivity along the x , y , and z coordinate axes, respectively, which are assumed to be parallel to the major axes of hydraulic conductivity, in units of length per time; h is potentiometric head, in units of length; W is volumetric

flux per unit volume, and represents sources and sinks, in units of inverse time; S_s is specific storage of the aquifer, in units of inverse length; and t is time, in units of time.

3.2.2.2 Numerical Approaches

The three-dimensional, finite-difference model MODFLOW (Harbaugh and McDonald, 1996; McDonald and Harbaugh, 1988) is a widely used simulation model for the analysis of groundwater-flow systems. The model represents the current breadth of approaches used for simulating stream-aquifer interaction and serves as a background for understanding numerical simulation of these systems.

As initially developed, MODFLOW contained a “River” package for simulation of stream-aquifer interaction, where the river stage is fixed during model stress periods. As long as the head in the aquifer remains greater than the elevation of the bottom of the riverbed, the rate of flow between the river and aquifer is proportional to the difference in head between the river stage and head in the aquifer. When head in the aquifer falls below the bottom of the riverbed, the aquifer is recharged with a constant flow rate that is proportional to the difference between river stage and elevation of the bottom of the riverbed.

The flow rate between the river and aquifer for each cell of the model through which a river passes, as a function of aquifer head, is:

$$Q_{riv} = C_{riv} (h_{riv} - h_{aq}) \quad h_{aq} > R_{bottom} \quad (3.16)$$

$$Q_{riv} = C_{riv} (h_{riv} - R_{bottom}) \quad h_{aq} \leq R_{bottom} \quad (3.17)$$

where Q_{riv} is flow rate between the river and aquifer, C_{riv} is the hydraulic conductance of the river-aquifer interconnection, h_{riv} is river stage, R_{bottom} is elevation of the riverbed, and h_{aq} is head in the aquifer. The hydraulic conductance term is often conceptualized as controlling the leakage of water through a low-permeability riverbed layer between the aquifer and river. The term is often set equal to the product of the hydraulic conductivity of the riverbed materials, length of the river within a model cell, and width of the river within a model cell, divided by the riverbed thickness.

If the simulated rivers are assumed to never lose water to the underlying aquifer, a second approach, the “Drain” package, can be used to simulate stream-aquifer interaction. In this approach, discharge to the drains (rivers) is proportional to the hydraulic conductance of the drain-aquifer interconnection, C_{drain} , when head in the aquifer is greater than the drain elevation, and zero when head in the aquifer is below the specified drain (or river bottom) elevation:

$$Q_{drain} = C_{drain} (h_{aq} - d_{el}) \quad h_{aq} > d_{el} \quad (3.18)$$

$$Q_{drain} = 0 \quad h_{aq} \leq d_{el} \quad (3.19)$$

where Q_{drain} is flow rate between the simulated drain and aquifer and d_{el} is the drain elevation, which may be assumed to equal river stage. Note in Equations 3.15 through 3.18 that simulated flow to river and drain cells are piecewise linear functions that add nonlinearities to the stream-aquifer problem.

Jobson and Harbaugh (1999) developed a diffusion analogy surface-water flow model (DAFLOW) for simulating surface water and groundwater interaction. To facilitate the simulation of the interaction, DAFLOW was coupled to the groundwater flow model (MODFLOW).

DAFLOW routes flows through a system of inter-connected one-dimensional channels and subdivides the system into a series of branches. The models are coupled by adding an exchange between each subreach and a specified groundwater cell. The water exchange for each subreach is computed on the basis of the stream-aquifer head difference, the streambed thickness, stream width, and streambed hydraulic conductivity.

The differential equations derived by Saint-Venant (1871) for one dimensional, unsteady flow are the basis for the diffusion analogy method used by DAFLOW. Assuming no lateral inflow, the Saint-Venant equations for channel flow are the continuity of mass equation:

$$\frac{\partial Q}{\partial X} + \frac{\partial A}{\partial t} = 0, \quad (3.20)$$

And the continuity of momentum equation:

$$\frac{1}{g} \frac{\partial U}{\partial t} + \frac{U \partial U}{g \partial X} + \frac{\partial Y}{\partial X} + S_f - S_0 = 0 \quad (3.21)$$

In which Q is discharge, X is distance along the channel, A is the cross sectional area of flow, t is time, g is the acceleration of gravity, U is velocity, Y is depth, S_f is the friction slope, and S_0 is the streambed slope.

The diffusive wave form of the flow equation is as follows:

$$\frac{\partial Q_s}{\partial t} + C \frac{\partial Q_s}{\partial X} - D_f \frac{\partial^2 Q_s}{\partial X^2} = 0 \quad (3.22)$$

The preceding discussions indicate that stream and groundwater flow systems are coupled by the flow of water at the interface between the two flow systems. In the simplest numerical modeling-approaches, the stream is treated as an imposed specified-head or head-dependent flux boundary to the groundwater flow system. Heads in the stream are unaffected by stresses occurring within the groundwater flow system. In more complicated numerical algorithms, stream stage can be calculated with groundwater heads and flow rates as part of the solution procedure.

3.3 Recharge Analysis by Streamflow

Several methods have been used to estimate groundwater discharge and groundwater recharge from streamflow records. The most widely used method is base-flow record estimation, which is the estimation of a continuous or daily record of base flow under the streamflow hydrograph. This procedure usually results in an estimate of the long term groundwater discharge.

An alternative to base flow record estimation is the recession-curve-displacement method. This procedure, often referred to as the Rorabaugh method, consists of a set of calculations that will estimate total recharge for each streamflow peak (Rorabaugh, 1964). The advantage of this method is that it is more theoretically based compared to base-flow-record estimation and actually includes groundwater variables.

The recession-curve-displacement method is intended for analysis of flow systems that are driven by a really diffuse recharge, where the stream can be considered as the discharge boundary of the groundwater flow system. Recharge is considered to be approximately concurrent with peaks in streamflow. The method is applicable only for streamflow records of catchments where regulation and diversion of flow can be considered negligible and may have only limited utility in flow systems dominated by snowmelt runoff. For expressing recharge in units of specific discharge, the area of contribution has to be known and it is usually considered equal to the drainage area of the surface-water system. If the streamflow record is continuous, there may be no lower limit to the size of the drainage area. However, if the record is incremental, then the drainage area should be of sufficient size that the time base of surface runoff will exceed the time increment of the data. The upper limit for drainage area size may depend on the degree of uniformity in weather systems or the accuracy of the time base of surface runoff.

The recession-curve-displacement method is based on the upward shift in the recession curve of groundwater discharge that occurs as a result of recharge. The only parts of the streamflow hydrograph that are used to measure this upward shift are parts that can be considered to be completely groundwater discharge. The decision that flow is completely groundwater discharge can be based on the antecedent recession. Linsley and others (1982) used the empirical relation:

$$N = A^{0.2} \quad (3.23)$$

for determining the time base of surface runoff (N) as a function of the drainage area(A) upstream from a streamflow gaging station. The time base of surface runoff is the number of days after a peak in the hydrograph of streamflow when the component of flow attributed to surface runoff can be considered negligible. A part of the streamflow hydrograph may be considered completely groundwater discharge if it is preceded by a period of recession equal to or greater than N.

Recharge will increase the total potential groundwater discharge, V, which is the total volume of water that will drain from the system if allowed to do so for infinite time without further recharge. Meyboom(1961) expressed V in the following equation, which is based on a linear relation between the logarithm of groundwater discharge and time.

$$V = \frac{Q \cdot K}{2.3026} \quad (3.24)$$

where V is total potential groundwater discharge; Q is groundwater discharge at initial time; and K is the recession index, the time required for groundwater discharge to decline through one log cycle.

Rorabaugh (1964) expressed groundwater discharge to a stream as a complex function of time after recharge; the function can be approximated after “critical time” by an equation that expresses the logarithm of groundwater discharge as a linear function of time.

$$T_c = \frac{0.2 \cdot a^2 \cdot S}{Tr} \quad (3.25)$$

where T_c is critical time; a is the average distance from the stream to the hydrologic divide; S is the storage coefficient; and Tr is transmissivity.

A formulation that gives critical time as a function of the recession index can be obtained by combining Equation 3.25 with the following equation from Rorabaugh and Simons (1966):

$$K = \frac{0.933 \cdot a^2 \cdot S}{Tr} \quad (3.26)$$

By solving for $a^2 S / Tr$ and substituting into Equation 3.25, T_c can be expressed as:

$$T_c = 0.2144 \cdot K \quad (3.27)$$

The formulations of Glover (1964) and Rorabaugh (1964) show that the total potential groundwater discharge at critical time after the peak in streamflow is equal to approximately one-half of the total volume of water that recharged the system. This finding, combined with the principle of superposition, is the basis for the recession-curve-displacement method; thus, total recharge is calculated by use of the following equation:

$$R = \frac{2 \cdot (Q_2 - Q_1) \cdot K}{2.3026} \quad (3.28)$$

where R is total volume of recharge; Q_1 is groundwater discharge at critical time as extrapolated from the stream-flow recession preceding the peak; and Q_2 is groundwater discharge at critical time as extrapolated from the streamflow recession following the peak.

3.4 Groundwater Table Variation by Pumping

The groundwater withdrawal by well pumping in municipal and agricultural zones caused the groundwater table variation described for the study area discussed in Chapter 6. The aquifer in the study area is unconfined. The facilities for a municipal water use are located close to the stream but those for an agricultural water use are located far from the stream as the paddy lands lie approximately 1,000 meter distance away from the stream.

In order to evaluate the groundwater table variation, the Dupuit-Forchheimer equation is described in this section. A solution for the groundwater head uses the following assumptions:

- a) The aquifer is homogeneous and isotropic and its boundaries go to infinity.
- b) The water table is horizontal before pumping.
- c) Darcy's law is valid for the flow in the aquifer.
- d) Water is instantaneously removed from storage as the head declines.
- e) The pumping rate of well is constant.
- f) The flow is symmetric with respect to the axis of the well.

g) The seepage face of the well is negligible and the aquifer receives a constant recharge rate.

With the assumptions given above, the governing differential equation is derived. The continuity principle for the portion of aquifer between two zones of radius r and $r + \Delta r$ gives

$$Q(r + \Delta r) - Q(r) + (2 \cdot \pi \cdot r \cdot \Delta r)I = 0 \quad (3.29)$$

where $I(L/T)$ represents the volume of water entering in a unit horizontal area of the aquifer per unit time due to the precipitation recharge. The head distribution, $h(r)$, can be determined from the governing differential equation and the boundary conditions, where H_w is the head at the well face, r_w is the well radius, H is the head before pumping, and R is the influence radius at which the drawdown is zero. The solution is

$$h^2 = H^2 + \frac{I}{2K}(R^2 - r^2) + \frac{Q}{\pi \cdot K} \ln\left(\frac{r}{R}\right) \quad (3.30)$$

with $I=0$, Equation 3.30 becomes:

$$h^2 = H^2 + \frac{Q}{\pi \cdot K} \ln\left(\frac{r}{R}\right) \quad (3.31)$$

Equations 3.29 and 3.30 represent the head distributions with recharge and without recharge cases, respectively. For the boundary condition, the well discharge rate, Q , can be expressed as:

$$Q = \frac{\pi \cdot K (H^2 - H_w^2)}{\ln\left(\frac{R}{r_w}\right)} \quad (3.32)$$

Between any two distances (r_1 and $r_2, r_2 > r_1$), the head takes the form:

$$h_1 = \sqrt{h_2^2 + \frac{Q}{\pi \cdot K} \ln\left(\frac{r_1}{r_2}\right)} \quad (3.33)$$

where h_1 and h_2 are piezometric heads in distance r_1 and r_2 respectively, Q is the well discharge rate, and K is hydraulic conductivity.

Equations 3.31 and 3.32 are known as forms of the Dupuit-Forchheimer well discharge formula. For multiple wells with drawdowns that overlap, the principle of superposition can be used. Drawdown at any point in the area of influence of several pumping wells is equal to the sum of drawdowns from each well in unconfined aquifers. The summation of drawdowns from each pumping well provides the correct drawdown.

$$h = \sqrt{H^2 + \frac{Q_p}{\pi \cdot K} \ln\left(\frac{\sqrt{(x-x_p)^2 + (y-y_p)^2}}{R}\right)} \quad (3.34)$$

where h is piezometric head at any point (x, y) , x_p and y_p are the coordinates for each well, R is influence radius at which drawdown is zero, Q_p is the each well discharge rate, H is the piezometric head before pumping, and K is hydraulic conductivity.

3.5 Synthetic Streamflow

A multivariate auto-regressive lag-1 model with periodic parameters, MPAR(1)(Multi-variate Periodic Auto-Regressive lag-1 model) was selected for the long term 10day streamflow generation of the case study area, and the goodness of fit was tested. The MPAR(1) model can be expressed following

$$Y_{v,t} = G_t(X_{v,t}) \quad (3.35)$$

$$Z_{v,t} = \frac{Y_{v,t} - \hat{u}_t}{\hat{s}_t^2} \quad (3.36)$$

$$Z_{v,t} = A_{1,t}Z_{v,t-1} + B_t e_{v,t} \quad (3.37)$$

Where, $X_{v,t}$ are observations of year v , at time t , $n \cdot 1$ column vector; G_t is transformation function; $Y_{v,t}$ is transformed series of $X_{v,t}$ assumed normally distributed with periodic mean \hat{u}_t and variance \hat{s}_t^2 , $n \cdot 1$ column vector; $Z_{v,t}$ is a standardized version of $Y_{v,t}$, $n \cdot 1$ column vector; $A_{1,t}$ and B_t are $n \cdot n$ periodic coefficient matrices; $\underline{e}_{v,t}$ is $n \cdot 1$ vector of time independent, normally distributed deviation; and n is the number of sites. The parameters of the MPAR model of equation are obtained from the following equations:

$$\hat{A}_{1,t} = \hat{M}_{1,t} \hat{M}_{0,t-1} \quad (3.38)$$

$$\hat{B}_t \hat{B}_t^T = \hat{M}_{0,t} - \hat{M}_{1,t} \hat{M}_{0,t-1}^{-1} \hat{M}_{1,t}^T \quad (3.39)$$

Where, $\hat{M}_{0,t}$, $\hat{M}_{0,t-1}$, $\hat{M}_{1,t}$ are the estimated periodic correlation matrices defined by Salas et al(1980). The elements of the matrix \hat{B}_t of Equation 3.39 can be obtained by the procedures given by Graybill (1983). The parameter and estimated value are calculated by the Stochastic Analysis, Modeling, and Simulation software (SAMS, version 2000; Salas and others, 2000).

3.6 Optimization Programming

3.6.1 Linear Programming

Linear programming models have been extensively applied to optimal water resources problems. The general form of a linear programming model can be expressed as

$$\text{Max(or Min)} \quad z = \sum_{j=1}^n c_j x_j \quad (3.40)$$

Subject to

$$\sum_{j=1}^n a_{ij} x_j = b_i, \quad \text{for } i = 1, 2, \dots, m, \quad j = 1, 2, \dots, n$$

$$x_j \geq 0, \quad \text{for } j = 1, 2, \dots, n$$

Where c_j represent objective function coefficients, a_{ij} are constraint coefficients, b_i are right-hand side values, and x_j are decision variables.

Four basic assumptions are implicitly built into Linear Programming models: proportionality, additivity, divisibility, and a deterministic assumption (Mays and others, 1996).

3.6.2 Goal Programming

The goal programming approach for a multi-objective programming problem allows the flexible expression of policy constraints as objectives. Specified goals may be met, underachieved, or overachieved. For each objective, two types of goals can be used in a goal programming formulation, depending on the nature of the goal: a two-sided goal or one-sided goal.

A two-sided goal applies a penalty when the goal is not met exactly by the solution. A one-sided goal incurs a penalty only when the specified goal is either underachieved or overachieved, depending upon the definition of the goal. For example, for the objective of profit maximization, the goal type is one-sided because the penalty would not occur if the solution results in a higher benefit than the specified goal. For the objective of maintaining the groundwater table at a fixed level, the goal type is a two-sided one because undesirable consequences would occur if such a level is either too high or too low.

In the linear goal programming formulation, the multi-objective general form is:

$$\min \sum_{k=1}^K (w_k^+ d_k^+ + w_k^- d_k^-) \quad (3.41)$$

Subject to

$$\begin{aligned} f_k(x) - d_k^+ + d_k^- &= G_k, \quad \text{for } k=1,2,\dots,K \\ g(x) &= 0 \end{aligned}$$

Where d_k^+ and d_k^- are the additional nonnegative decision variables representing respectively, the deviations due to overachieving and underachieving the specified goal G_k for the k-th objective, $f_k(x)$; w_k^+ and w_k^- are weighting factors associated with the two types of deviation, respectively; x and $g(x)$ is the vector of the original decision variables and constraints, respectively. The two nonnegative decision variables d_k^+ and d_k^- satisfy the condition that both will not be simultaneously greater than zero. The weighting factors w_k^+ and w_k^- can be used to indicate the relative importance of different types of deviations for the various objective functions considered in the problem. The solution technique for solving a goal programming problem depends on the nature of the original objective functions $f_k(x)$, and the constraints $g(x)$.

3.6.3 Conjunctive Water Optimization

Even if a conjunctive water management system is applied in a basin, shortages to water supply and instream requirements may still occur during the low flow or drought season. For example in the case study area streamflows are very low except during July to September and the aquifer is shallow. Multiple objective optimization of the reservoir, surface withdrawals and groundwater pumping is needed to help develop options to manage these anticipated shortages. Essential to a basin-wide water resources management practice is a unified plan that simultaneously considers all planning aspects. Such basin wide planning has long been recognized as a management problem with multiple objectives (Louie and others, 1984; Yeh and others, 1992).

The goal of this study is to analyze the operation of stream-aquifer-reservoir systems considering multiple uses. Conjunctive water management is used to satisfy multiple purposes such as water supply, irrigation, low flow augmentation, and reservoir operation. Also guide or rule curves were derived for the reservoir operational policy. Rule curves are considered strategic and long-term policy (Loganathan and Bhattacharya, 1990).

Because of interactions between streamflow and groundwater withdrawal, deriving optimal policies from multi-objective optimization is complicated. The formulation can include a variety of objectives such as minimization of cost, minimization of water shortage, and minimization of deviations from rule curve targets. The streamflows involved in the constraints may be treated as deterministic or random. For this study, a deterministic model is used in the conjunctive management optimization model.

This research uses a goal programming approach. Different weights can be applied to the achievement of the various goals. In this way, a variety of nondominated solutions using different options for balancing the deviations from the goals can be found. Because there are several nondominated points, the decision maker has alternatives to choose from and additionally gains more insight about how the system performs. It has also been shown that goal programs can yield dominated points as solutions if the goals are not properly established (Loganathan and Bhattacharya, 1990).

CHAPTER 4

CHARACTERISTICS OF STREAM-AQUIFER, HYDROLOGY, AND THE RESERVOIR IN THE CASE STUDY AREA

The case study for the conjunctive water management modeling was analyzed using both optimization and simulation models. The study area is located in the upper region of the Geum River basin in the middle of South Korea. Issues have arisen in the case study area regarding the ability of the system to meet water demands for the municipal, industrial, and the agricultural purposes while still maintaining instream flows important for environmental and wildlife habitat concerns. In spite of constructing the Yongdam multi-purpose reservoir in 2001, the management problems such as the competition for water resources are still important. Efficient water management policies on the conjunctive use of surface and groundwater may help to solve these problems. This chapter addresses the stream-aquifer interaction, groundwater recharge, streamflow in time series analysis, and simulation of water table variations for the case study area.

4.1 Study Area

The case study was performed in the upper region of the Geum River basin where there are two multi-objective reservoirs, Daecheong and Yongdam. Figure 4.1 illustrates the Geum River basin located in the middle of South Korea, with a drainage area is 9,810km² and a mainstream length of 396km. The Geum River basin is one of four major river basins in South Korea. These basins are the Han River basin, Nakdong River basin, Geum River basin, and the Seomjin River basin. The characteristics of Daecheong reservoir and Yongdam reservoir are listed in Table 4.1. Mountains and forests cover approximately 75 percent of the study area, and the rest is composed of agricultural lands, reservoir water surface, and streams.

Table 4.1 Characteristics of multipurpose reservoirs in Geum River basin

Reservoir	Height (m)	Basin Area (km ²)	Inflow (m ³ /s)	Water supply(10 ⁶ m ³)	Capacity (10 ⁶ m ³)	Eff.capacity (10 ⁶ m ³)	Power (kW)
Daecheong	72	4,134	88.6	1,649	1,490	790	90,000
Yongdam	70	930	26.5	650	815	672	24,400

As for the typical climate features of the study area, there is a flood season during summer and a drought season during between fall and late spring. The monthly average precipitation and inflow data for Daecheong and the Yongdam reservoir are listed in Table 4.2. The average annual precipitation is estimated to be 1,203.0mm (45.7in). The average annual inflow of the Daecheong reservoir is 88.6 m³/sec over the 24-year periods from 1981 to 2004. The Yongdam reservoir inflow is 26.5m³/sec over the 3-year period

from 2002 to 2004. This amount is influenced by Typhoons “Rusa” and “Maemi” that occurred in 2002 and 2003 respectively. To compare the monthly average precipitation to the time, monthly average precipitation of Daecheong and Yongdam reservoirs are plotted in Figure 4.2. Figure 4.2 shows that monthly precipitation is unevenly distributed over the year with a minimum of 26.5mm and a maximum of 275.3mm. Over 65 percent of the average annual precipitation occurs between late June and September. The weather of this period is influenced by the monsoon characteristics of the South Pacific Ocean. The monthly average inflows of Daecheong and Yongdam reservoirs obtained after the completion of both reservoirs are plotted in Figure 4.3. Figure 4.3 shows the monthly average inflow to Daecheong is larger than it is for Yongdam reservoir. The monthly average inflow is also concentrated in the flood season like the average monthly precipitation.

Table 4.2 Monthly average precipitation and inflow of Daecheong and Yongdam Reservoirs

(Unit: mm, m³/sec, 10⁶m³)

Items		Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Sum
Precipitation	Daech.	29.0	33.4	46.6	70.5	81.9	159.6	275.3	253.5	132.6	49.5	44.7	26.5	1203.0
	Yongdam	39.1	33.6	45.4	165.4	131.1	144.3	368.5	480.7	173.4	25.0	48.9	37.9	1693.3
inflow	Daech.	19.8	28.6	44.1	54.6	52.2	94.9	277.8	223.9	169.0	41.9	26.5	23.7	2792.8
	Yongdam	4.6	7.7	8.9	23.3	18.2	23.2	99.6	66.3	50.0	4.8	4.2	4.5	834.0

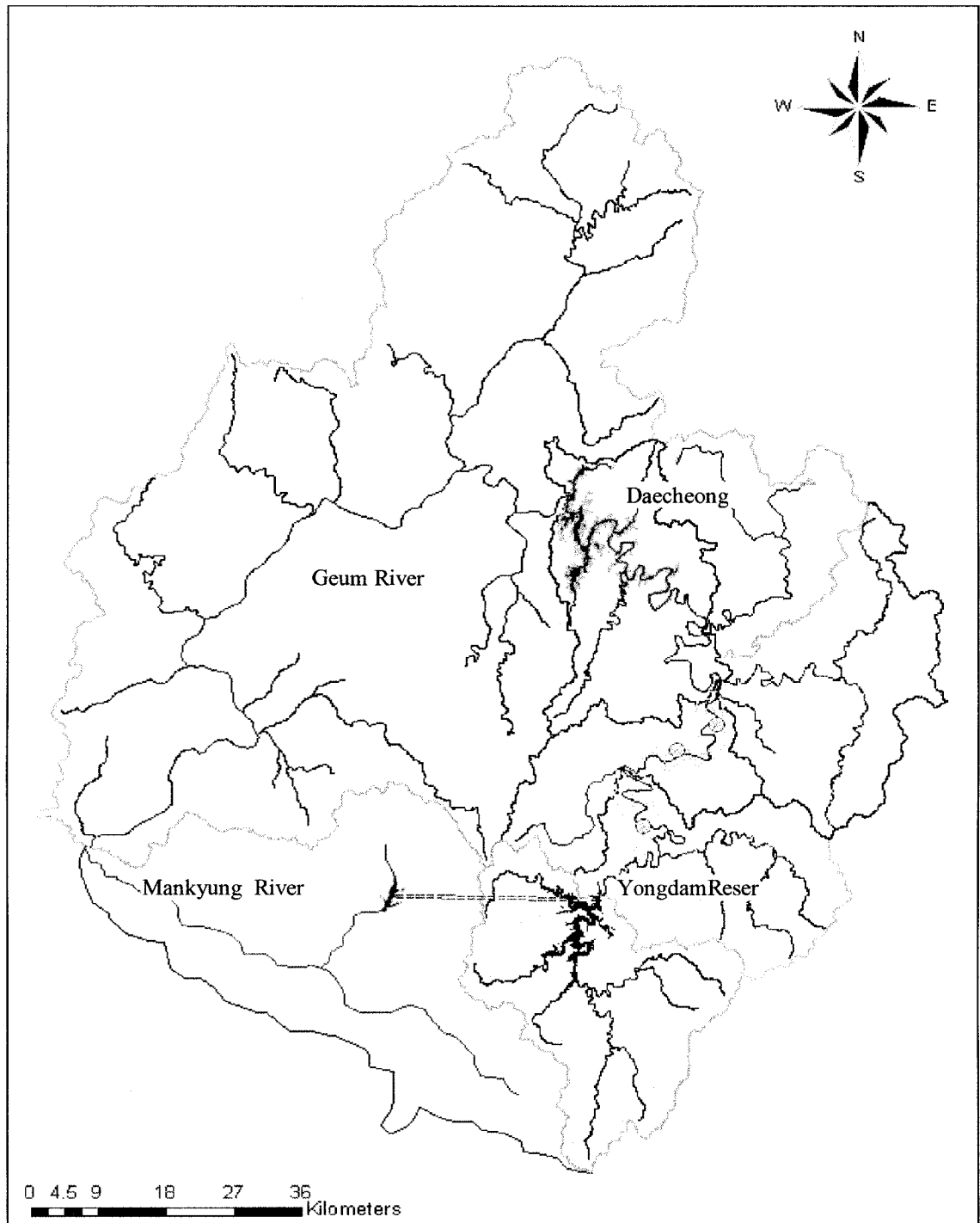


Figure 4.1 The drainage area of Geum River basin located in middle of South Korea

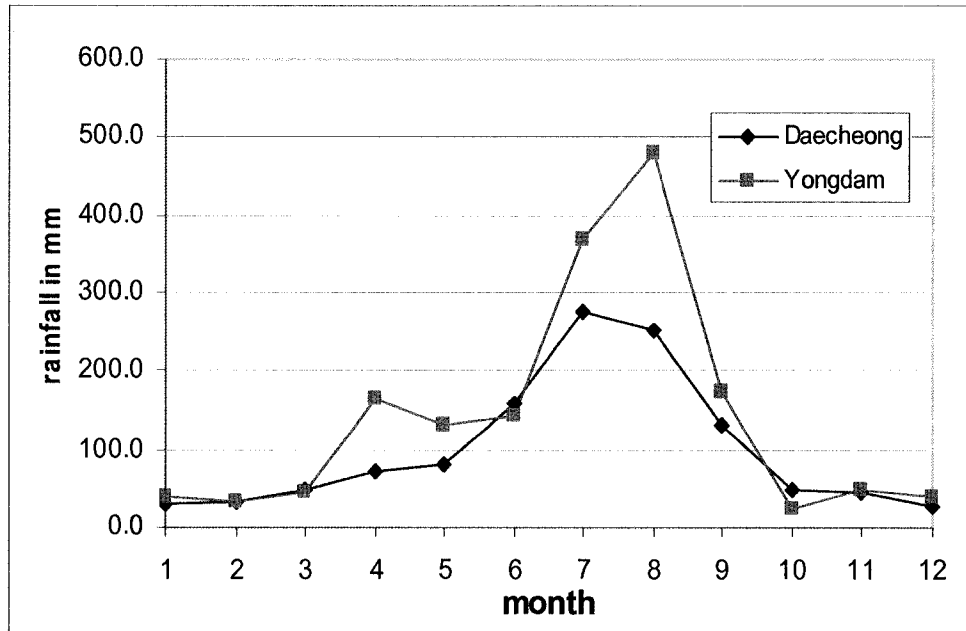


Figure 4.2 Monthly average precipitation of Daecheong and Yongdam reservoirs

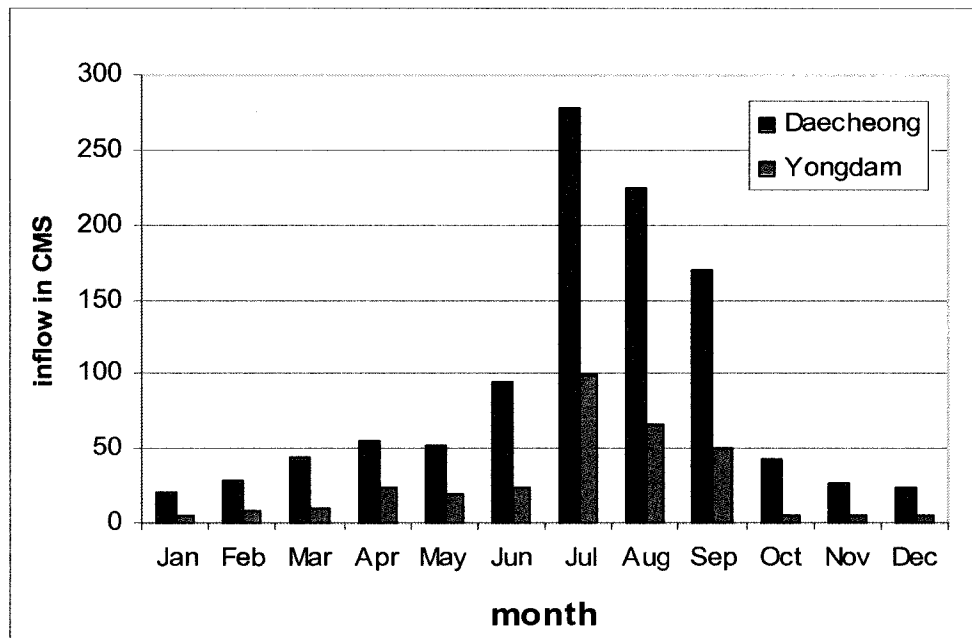


Figure 4.3 Monthly average inflow of Daecheong and Yongdam reservoirs

Since Yongdam reservoir was completed in 2001, conflicts have arisen over meeting water demands downstream of Yongdam reservoir and to supply water requests from Jeonju. In addition to water supply, there is considerable concern about the water quality on the Geum River which depends on the Yongdam reservoir release. Downstream users and state agencies have suggested minimum flows of $12 \text{ m}^3/\text{sec}$ for the Geum River based on consideration of the downstream sub-basin area, population density, and the instream flow needs for fish and aquatic habitat. However, the Korean government and upstream users have suggested a value of $5.0 \text{ m}^3/\text{sec}$ as an allowable instream flow for the Geum River. The Geum River basin survey(MOCT/KOWACO, 1998) reported the historical minimum drought flow of $2.25 \text{ m}^3/\text{sec}$ measured at Yongdam reservoir for senior water-right holders. For this situation, state agencies and non-governmental organizations require more water for both the demands and the instream flow requirements.

Two serious droughts in 1995 and 2001 have impacted the water supply in the Geum River basin. In the fall of 2001, the Daecheong reservoir reached its lowest storage level since beginning operations. The intensity and magnitude of drought in this region has created conflicts between the interested parties. Additionally, instream flow requirements, regional water needs, and the trans-basin diversion have resulted in a high level of conflict among stake holders in the basin. This research aims to explore methodologies for effective water resources management, and to evaluate potential conflicts of water resources management alternatives in the upstream area of the Geum River basin. Models are helpful to support decision making for water resources management as well as to evaluate the conjunctive management strategies for stream-

aquifer-reservoir systems. In addition, the results of this study can help promote long-term water planning with conjunctive water use between groundwater and surface water including reservoir operations.

A schematic diagram of the case study area is illustrated in Figure 4.4. It consists of one multi-purpose reservoir, one trans-basin diversion to Jeonju, six non-storage nodes, four groundwater pumping wells, one instream flow requirement, the return flow of each demand, two municipal and agricultural withdrawals, upstream inflow to the Yongdam Reservoir, two inflows on the downstream of Yongdam reservoir, and four recharge sites. Figure 4.5 illustrates the diagram of management process. This management study is for the streamflow and water demand analysis, the stochastic analysis for generation of the 10days streamflows, the analysis of stream-aquifer characteristics, the analysis of groundwater recharge, the calculation of groundwater table variation and the cost factor, the evaluation of stream-aquifer interactions for single well and multiple wells, the development of performance in measuring and evaluation of stream-aquifer-reservoir systems, the sensitivity analysis, the derivation of a reservoir operational guide curve, and the evaluation of the guide curves by the KModSim model.

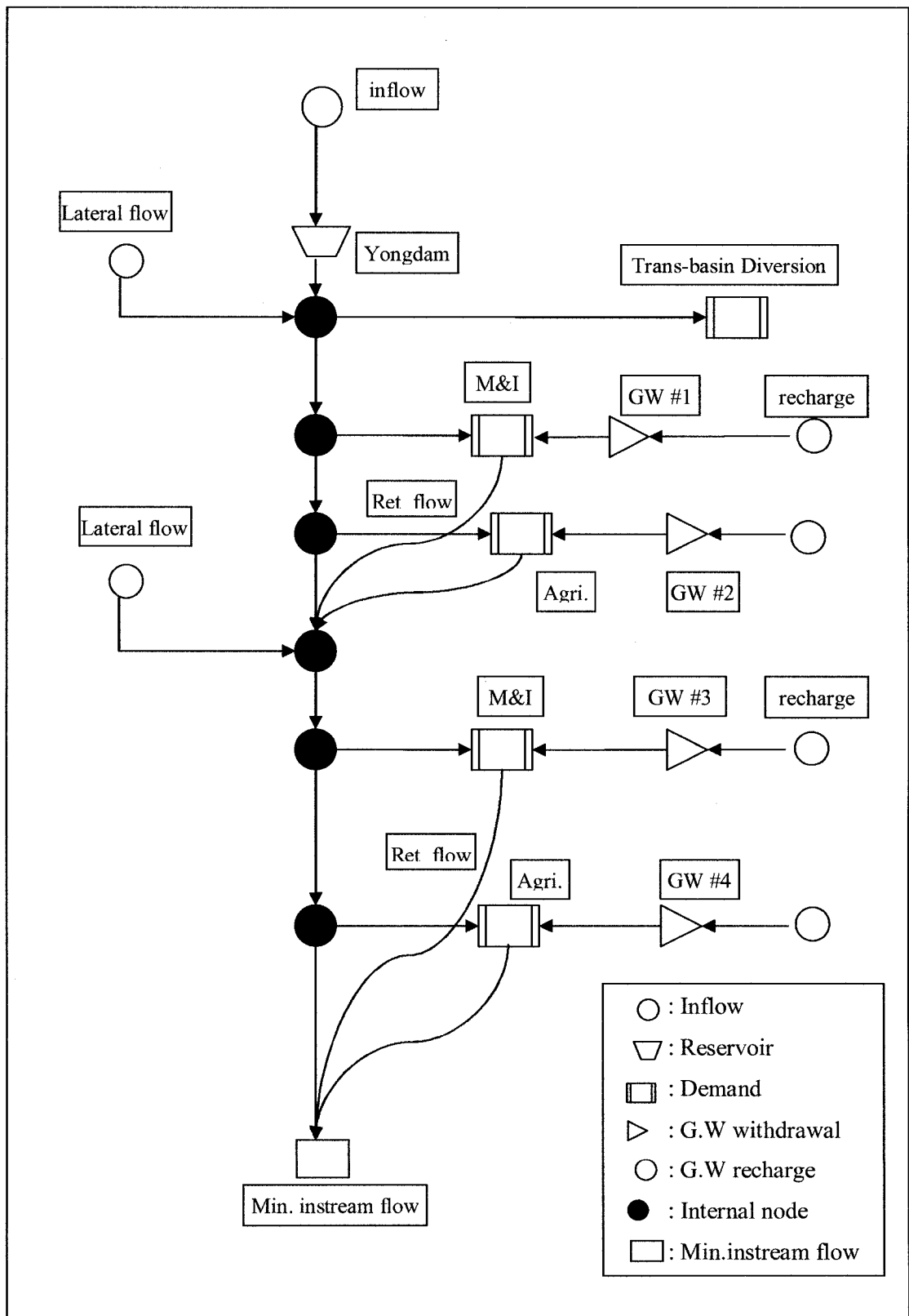


Figure 4.4 Schematic diagram of case study site

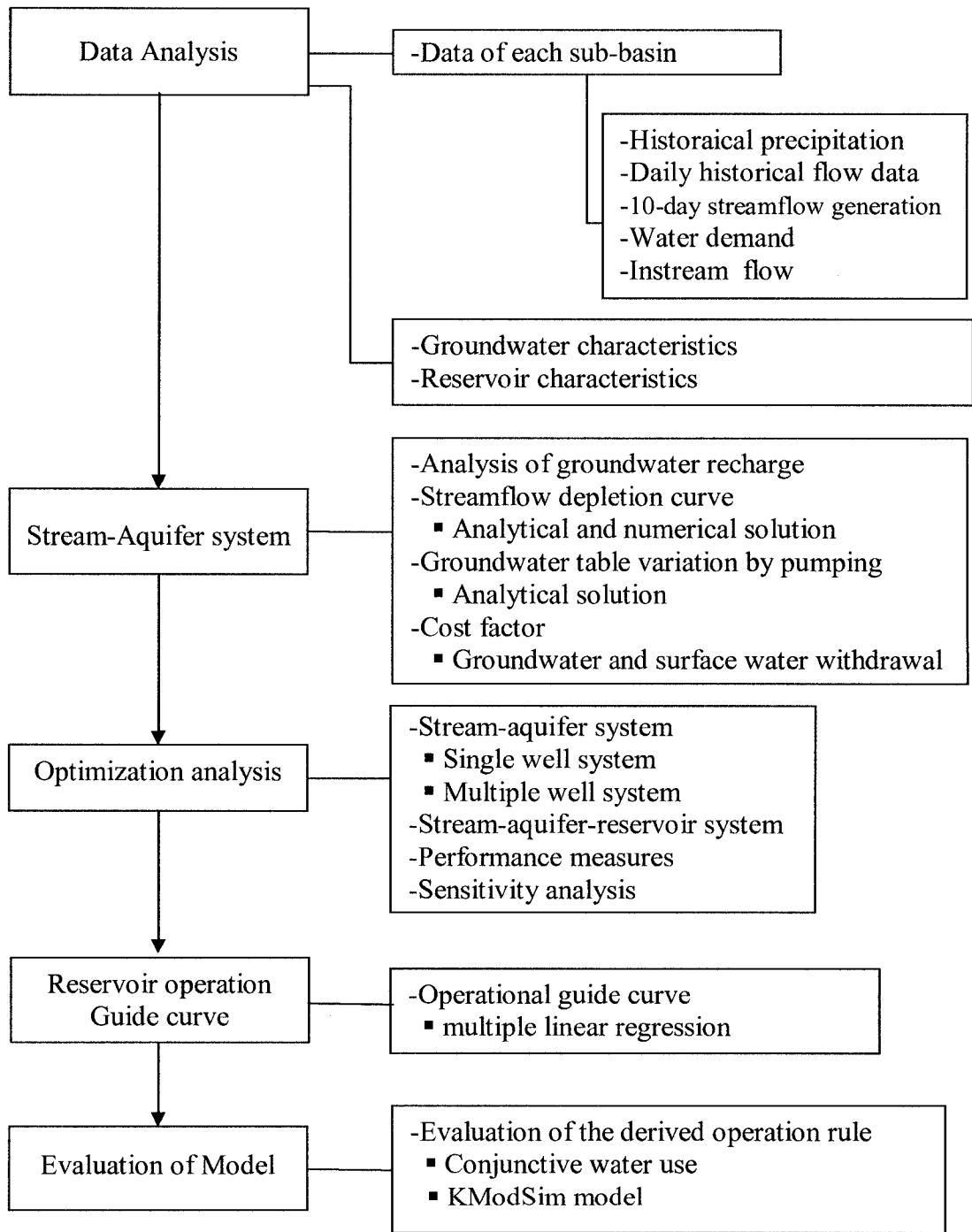


Figure 4.5 Research process

4.2 Streamflow and Water Demand Analysis

Stochastic simulation of hydrologic time series has been used to evaluate the reliability of a reservoir operations and the adequacy of a water resources management strategies such as the effectiveness of an operational guide curve under various potential hydrologic scenarios (Salas, 1993). Three locations of Site 1 (Yongdam reservoir), Site 2 (Muju), and Site 3 (Okcheon) are incorporated in the conjunctive water management case study. A synthetic trace of 100 years of 10-day inflows for the Yongdam reservoir, the Muju, and the Okcheon were generated to develop the operating guide curve.

It is generally known that 10 day hydrological series show periodicity rather than stationarity. The periodic characteristics are usually expressed by periodic means, standard deviations, and skewness. The Multivariate Periodic Auto-Regressive (MPAR) process is used to generate the inflow data by using the Stochastic Analysis, Modeling, and Simulation software (SAMS, version 2000, Salas and others) developed by Colorado State University Computing Hydrology Laboratory.

The major demand categories in this area include the trans-basin diversion for the purpose of municipal and industrial water use, the municipal and agricultural demand downstream of the Yongdam Reservoir, and the instream flow for fishery and aquatic habitat. Water use conflicts arise in the basin because water required for the trans-basin diversion represents three fourths of the reservoir storage (see table 6.5).

4.2.1 Historical Streamflow and Time Series Analysis

Streamflow and precipitation upstream of Daecheong reservoir have been observed systematically since the reservoir was constructed in 1981. Therefore the inflow data before the completion of the Yongdam reservoir is homogeneous and not intermittent. After the completion of the Yongdam reservoir, the inflow data above Daecheong reservoir is controlled by the release from Yongdam reservoir.

In order to test a reservoir operational guide curve for Yongdam reservoir, long-term inflow data is needed. The streamflow data observed upstream of Daecheong Reservoir was used in this analysis. For the homogeneous time series analysis, the unregulated monthly streamflow, 24 years from 1981 to 2004, to the Daecheong reservoir was used. The Yongdam reservoir has a small number of reliable streamflow series from 2002 to 2004. The unregulated streamflow for two subbasins, Muju and Okcheon, are not measured. Therefore the 10-day streamflow data measured in Daecheong reservoir during the 3 year period from 2002 to 2004 were used to estimate the unregulated flow of each subbasin based upon the ratio of the area and the precipitation of each subbasin. The autocorrelation analysis results of each site are plotted in Figure 4.6, 4.7, and 4.8 respectively.

These figures show that a Lag-1 autoregressive model is suitable to analyze the historical streamflow data. The statistical characteristics of the historical streamflow data for each site are provided in Table 4.3. The skewness coefficients show that the historical streamflow records are not normally distributed. Each streamflow series was transformed to normal by various transformation methods. The comparison of several transformations

of the original data showed that the Logarithmic and Power equations are better to represent normality. The transformed skewness of the historical data is also shown in Table 4.3. The skewness calculated from the historical data for each site are plotted in Figures 4.9-4.11. These figures of the normality test for 95 percent confidence level show that the skewness coefficients of all sites are within the limit of ± 0.711 .

Table 4.3 Statistics of the historical streamflow measured in three sites during 1981-2004.

Item	site	1	2	3	4	5	6	7	8	9	10	11	12
mean	1	8.40	4.17	15.02	9.84	12.62	7.73	60.56	42.34	38.81	18.31	5.74	10.56
	2	3.70	1.89	5.48	5.24	9.02	5.08	34.85	25.21	21.57	9.39	3.27	6.58
	3	4.38	2.19	18.77	4.89	9.50	3.85	26.66	17.75	22.42	5.64	2.61	2.95
skewness	1	4.38	1.05	3.93	1.94	1.27	1.29	1.63	1.16	2.05	2.81	0.23	4.23
	2	2.80	0.72	2.54	1.21	1.45	1.21	1.33	1.15	2.81	4.00	0.35	4.32
	3	4.22	3.91	4.53	3.46	3.30	2.33	1.61	1.17	2.13	2.38	0.60	1.89
transformed skewness	1	0.18	-0.07	-0.17	-0.11	-0.04	0.01	-0.29	-0.23	-0.02	0.25	-0.03	-0.02
	2	0.04	-0.01	0.16	0.36	-0.02	-0.22	-0.08	-0.26	-0.08	-0.26	0.16	-0.15
	3	0.02	-0.04	-0.45	-0.21	-0.04	-0.05	-0.17	0.14	-0.08	0.05	0.02	-0.23
Item	site	13	14	15	16	17	18	19	20	21	22	23	24
mean	1	6.32	7.35	8.31	15.13	11.34	11.88	70.13	57.48	42.85	11.99	6.51	5.32
	2	3.68	4.98	8.46	9.22	5.72	7.28	46.23	37.02	27.40	5.98	3.40	2.84
	3	2.94	3.17	4.33	6.67	5.52	6.33	37.44	22.93	19.38	4.69	2.91	2.14
skewness	1	1.10	1.36	1.22	1.26	1.39	3.51	0.72	2.54	1.85	2.15	0.95	0.49
	2	0.84	1.96	3.72	1.72	1.44	4.15	0.93	2.64	1.92	3.30	1.24	0.61
	3	1.16	2.48	0.34	2.37	0.95	4.04	0.68	2.76	1.85	3.25	1.66	0.82
transformed skewness	1	-0.02	0.06	-0.04	0.00	-0.07	-0.32	0.10	0.07	-0.27	0.26	0.00	-0.17
	2	-0.01	-0.19	0.32	-0.05	-0.11	-0.16	0.18	-0.03	-0.07	0.20	0.26	0.09
	3	0.12	-0.04	-0.01	-0.22	-0.31	-0.11	0.02	-0.04	-0.02	-0.25	0.24	-0.14
Item	site	25	26	27	28	29	30	31	32	33	34	35	36
mean	1	15.60	8.64	12.48	13.23	7.02	36.68	45.79	53.31	15.29	6.41	6.78	5.07
	2	2.87	4.81	8.63	8.05	4.76	21.18	34.13	34.32	10.29	4.05	3.45	2.73
	3	7.51	5.30	5.73	7.13	2.66	18.68	26.20	29.48	9.39	3.33	3.18	2.22
skewness	1	4.50	1.83	0.28	2.45	1.83	1.11	1.24	1.51	3.58	1.32	1.28	1.07
	2	1.89	2.91	2.08	2.67	2.20	1.41	1.31	1.77	3.71	0.47	2.25	0.11
	3	4.53	3.12	0.15	2.37	2.15	1.53	1.47	1.48	3.35	1.33	1.95	2.23
transformed skewness	1	-0.06	-0.19	0.02	-0.04	0.10	0.03	-0.02	-0.10	-0.004	0.09	-0.17	-0.17
	2	-0.04	-0.17	0.16	-0.20	-0.02	0.20	0.00	-0.06	0.07	0.05	-0.45	0.11
	3	-0.03	0.23	-0.01	0.01	-0.30	0.05	0.22	0.16	-0.29	0.04	0.25	-0.09

Site 1: The Yongdam reservoir; Site 2 : Muju; and Site 3 : Okcheon

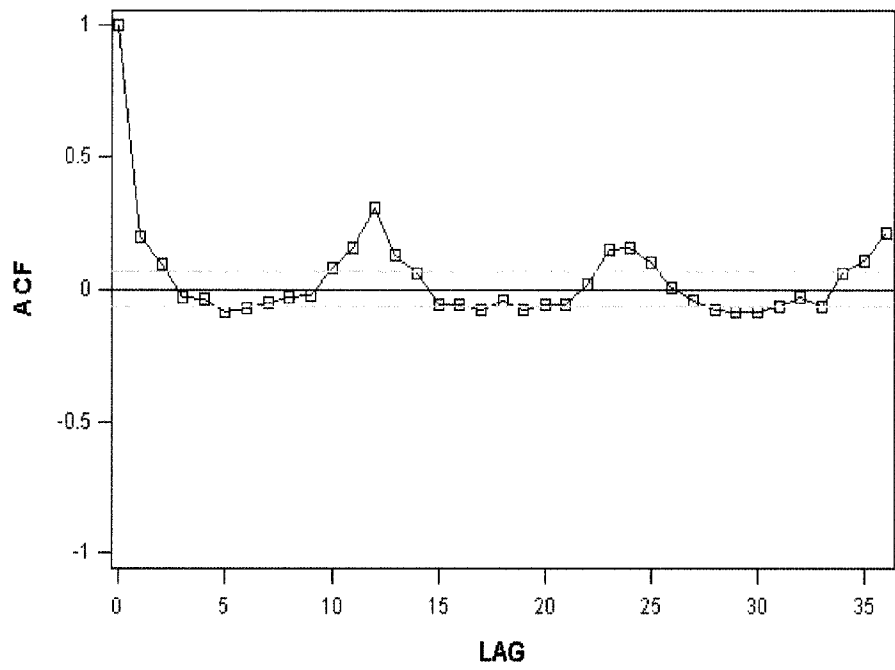


Figure 4.6 Autocorrelation function of site 1, Yongdam reservoir

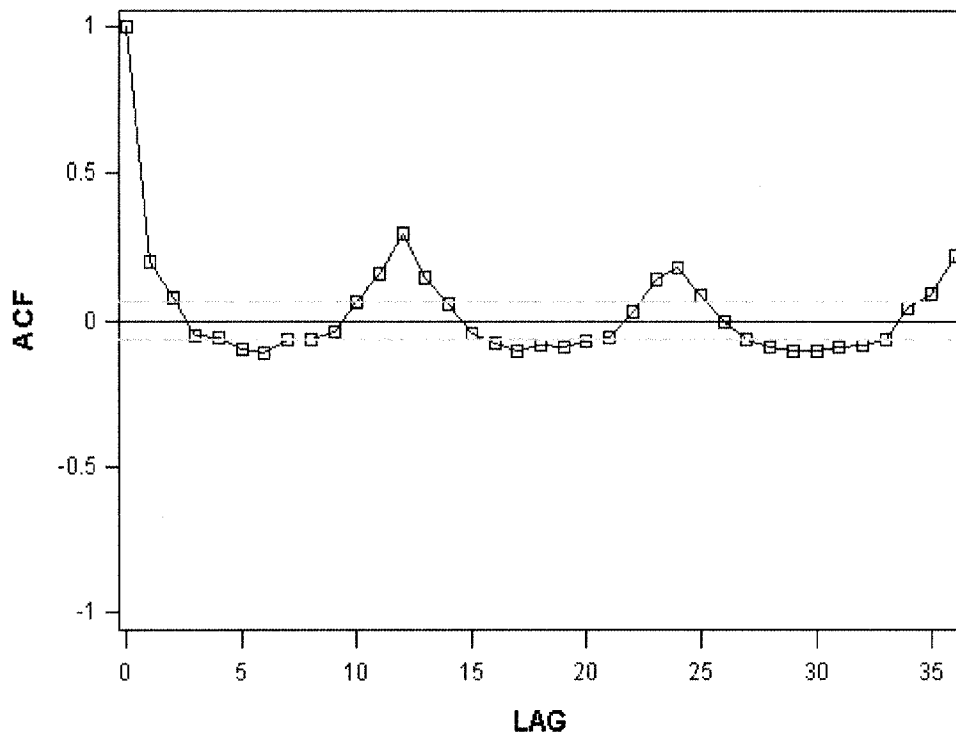


Figure 4.7 Autocorrelation function of site 2, Muju

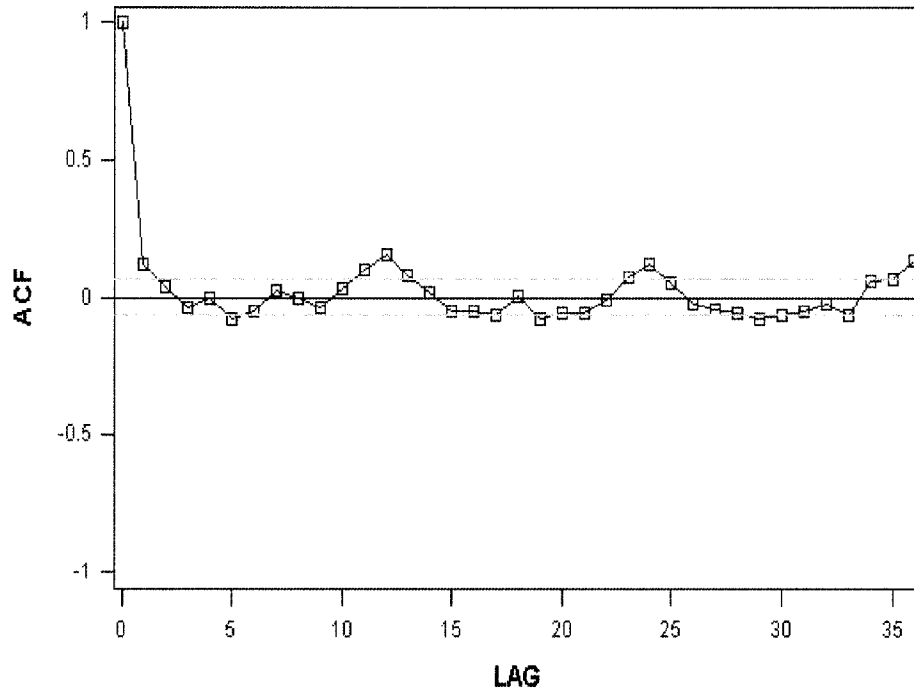


Figure 4.8 Autocorrelation function of site 3, Okcheon

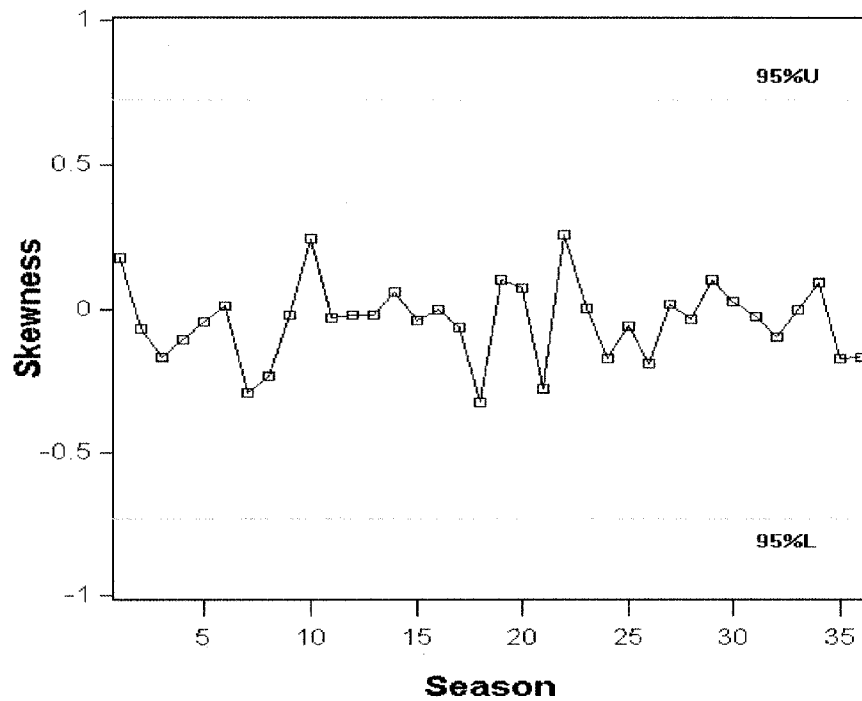


Figure 4.9 Skewness test of normality on site 1, Yongdam reservoir

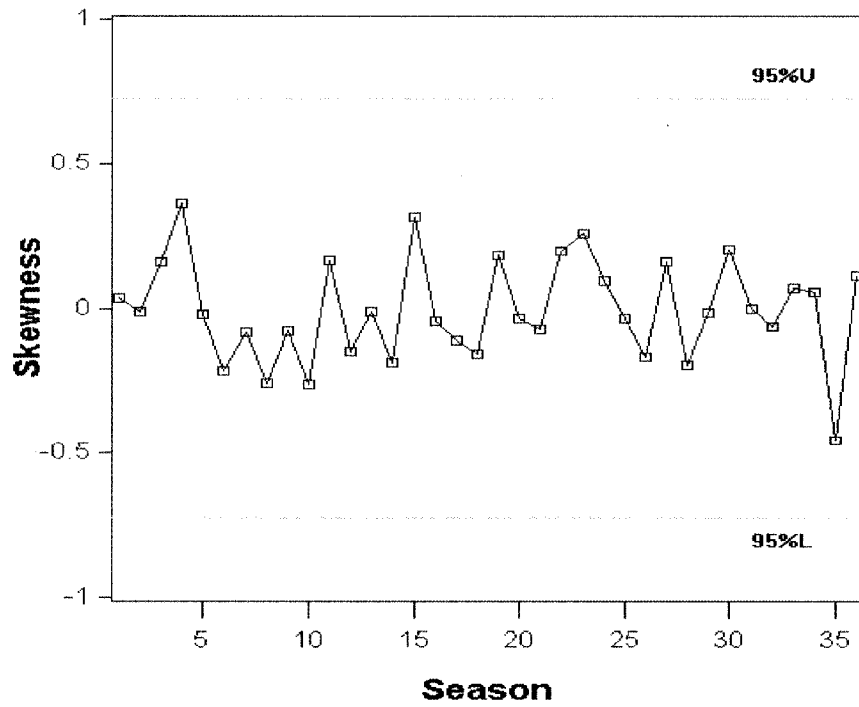


Figure 4.10 Skewness test of normality on site 2, Muju

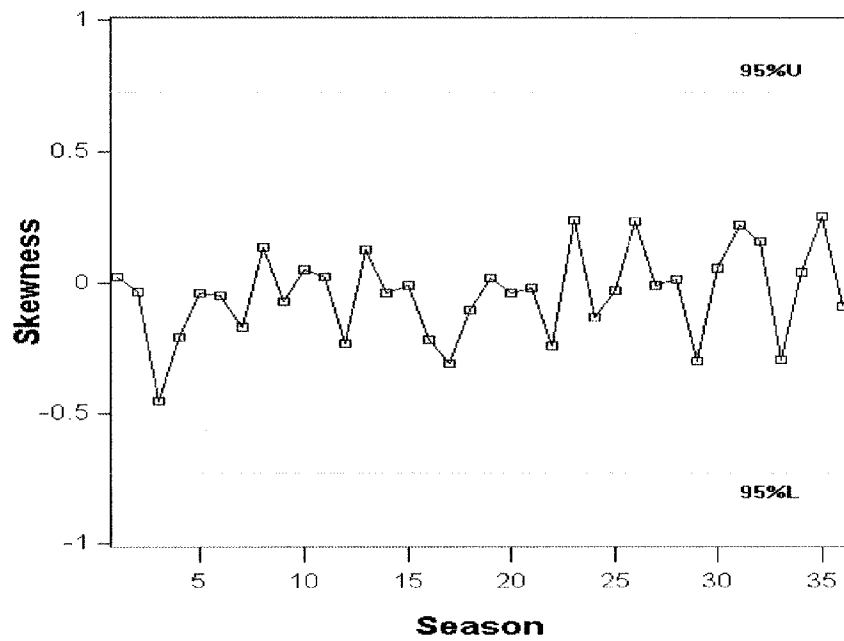


Figure 4.11 Skewness test of normality on site 3, Okcheon

The 10-day historical inflow data into the Daecheong reservoir for 24 years from 1981 to 2004 are given in Table 4.4. These inflows data are also plotted in Figure 4.12. This figure shows that the mean of 10-day inflow for a year is about 77.9MCM and over 72 percent of the average of 10-day inflow occurs between late June and September.

Table 4.4 Historical inflows to Daecheong reservoir

(Unit: 10^6m^3)

year	day	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Sum
1981	1-10	15.0	12.3	29.9	29.9	8.0	3.3	369.9	103.3	339.4	40.5	32.4	14.0	2,609
	11-20	14.1	42.5	51.8	38.3	5.9	5.2	267.6	102.9	33.9	25.1	20.7	13.3	
	21-30	13.7	34.1	52.7	16.8	7.0	115.5	106.0	457.4	125.0	29.8	16.0	16.2	
1982	1-10	12.6	9.3	25.8	25.4	33.3	7.4	7.7	108.3	21.5	6.6	11.0	37.2	1,144
	11-20	7.5	10.4	28.4	8.9	18.8	3.2	4.3	288.6	15.5	10.1	10.7	23.9	
	21-30	9.6	13.1	52.6	11.3	17.7	5.3	53.3	153.6	11.8	12.3	55.1	12.1	
1983	1-10	14.1	23.6	64.3	35.5	91.9	4.2	82.7	34.3	175.3	40.4	13.9	17.4	2,445
	11-20	12.4	17.3	34.3	46.9	29.7	48.2	156.6	27.7	168.1	36.9	13.7	15.0	
	21-30	9.8	12.0	110.6	167.7	15.0	165.9	543.8	118.6	49.1	19.1	17.3	11.7	
1984	1-10	13.5	5.5	8.1	18.0	86.7	63.9	459.3	40.7	751.4	72.0	20.3	23.0	2,752
	11-20	8.9	5.4	7.9	90.6	57.4	42.8	299.8	31.8	80.1	23.8	62.4	19.9	
	21-30	7.4	5.4	13.2	100.3	17.0	74.8	54.3	80.8	35.1	14.8	40.1	15.4	
1985	1-10	10.7	12.0	34.4	53.4	163.9	14.8	555.1	40.7	222.2	88.6	30.0	74.9	4,710
	11-20	10.9	22.6	47.2	54.4	142.4	20.2	499.5	688.0	863.0	244.1	68.0	30.6	
	21-30	10.8	14.2	79.9	29.7	33.5	99.9	58.2	67.9	161.4	46.7	87.3	28.9	
1986	1-10	36.2	12.2	16.5	25.3	24.6	26.4	62.2	43.1	83.7	42.6	34.1	22.2	2,691
	11-20	24.0	15.9	71.2	20.2	85.4	29.5	239.9	347.2	38.2	95.9	28.0	44.4	
	21-30	22.7	12.8	55.7	17.2	55.8	416.7	176.9	194.1	127.6	70.2	20.9	51.8	
1987	1-10	109.6	27.9	32.0	35.6	39.1	179.6	15.4	511.7	306.1	19.1	53.3	32.4	4,768
	11-20	65.6	136.0	31.9	18.2	19.6	47.1	444.6	202.3	44.9	12.0	19.7	15.4	
	21-30	47.1	37.6	80.1	40.4	35.1	22.1	972.7	1008.8	25.9	27.1	36.5	15.5	
1988	1-10	10.4	11.2	7.5	11.8	18.0	13.0	5.8	27.8	13.7	9.0	2.1	1.4	1,253
	11-20	11.8	10.0	24.0	16.5	24.4	8.1	648.3	23.4	21.3	5.4	1.4	1.8	
	21-30	13.8	7.4	20.4	25.3	8.9	5.4	181.7	44.6	10.3	3.8	1.6	2.1	
1989	1-10	4.2	11.8	162.5	27.1	8.9	69.2	19.2	66.2	102.7	30.4	37.0	17.0	3,012
	11-20	22.4	11.8	34.2	24.3	13.7	74.5	491.6	20.4	418.3	22.5	30.7	11.8	
	21-30	48.9	37.2	45.9	17.3	4.6	26.0	875.4	106.1	66.3	18.2	18.9	14.9	
1990	1-10	13.4	19.6	53.0	56.8	122.3	20.1	109.1	16.7	103.0	23.2	12.9	13.3	3,099
	11-20	21.3	113.4	44.2	59.0	47.7	70.3	790.6	32.0	153.2	16.1	14.5	13.6	
	21-30	15.1	197.6	51.2	68.8	33.4	415.9	134.0	156.8	44.6	12.1	18.9	11.5	

1991	1-10	10.7	11.0	39.6	35.5	15.4	49.5	95.8	377.2	115.7	28.6	10.5	8.7	2,394
	11-20	8.2	34.0	101.3	112.0	9.7	95.5	426.7	89.5	33.2	15.6	8.4	9.7	
	21-30	15.1	17.9	76.0	54.4	12.0	12.2	231.7	138.5	55.8	11.2	8.5	18.7	
1992	1-10	14.3	14.9	19.5	22.1	22.4	9.7	2.2	6.5	117.1	66.9	17.6	23.2	1,663
	11-20	12.6	12.2	14.3	138.1	24.3	6.1	298.3	112.9	90.9	35.4	28.2	33.8	
	21-30	10.7	8.0	40.4	39.4	13.5	1.6	51.6	130.7	140.5	31.2	22.7	29.4	
1993	1-10	25.2	12.7	40.3	24.7	48.0	73.0	111.3	519.3	39.8	21.9	27.5	45.3	3,687
	11-20	21.2	29.6	37.3	14.1	179.8	47.9	846.4	341.5	133.8	16.7	62.3	26.0	
	21-30	17.9	88.9	43.5	23.3	82.3	253.2	93.4	216.9	47.6	20.7	31.2	22.6	
1994	1-10	16.8	15.2	30.0	21.9	7.9	8.8	158.0	28.3	14.3	5.6	11.2	9.7	827
	11-20	15.9	16.3	32.0	25.1	77.1	6.3	14.1	39.6	6.0	33.5	10.0	9.8	
	21-30	17.6	17.7	25.8	12.6	29.6	16.8	7.3	25.5	7.4	33.7	11.5	8.1	
1995	1-10	6.5	6.2	7.0	13.6	13.0	14.8	3.8	10.6	238.3	238.3	6.4	6.7	1,884
	11-20	5.3	6.6	23.8	7.1	7.6	3.4	16.9	37.8	51.7	38.4	11.9	5.5	
	21-30	8.0	7.1	23.4	83.1	12.3	2.7	75.9	849.7	0.0	23.4	11.2	6.1	
1996	1-10	5.2	4.9	7.4	34.1	53.9	10.0	254.2	26.5	24.0	18.3	25.3	24.8	2,245
	11-20	7.7	4.1	39.1	15.3	21.7	456.3	84.7	18.6	14.6	13.4	29.3	22.6	
	21-30	7.3	4.1	77.6	21.7	12.6	597.8	201.6	55.5	12.1	9.3	12.5	17.3	
1997	1-10	13.3	7.1	52.4	74.7	80.2	54.7	897.1	559.5	18.2	9.8	4.9	77.7	3,043
	11-20	11.7	7.3	41.4	28.8	157.5	16.0	279.6	135.3	14.1	6.5	19.8	42.2	
	21-30	9.1	18.8	26.3	11.8	39.4	150.4	52.3	30.5	13.4	5.5	46.1	29.0	
1998	1-10	17.5	12.7	27.1	203.2	32.2	27.4	394.7	323.3	31.7	392.8	23.6	13.0	4,525
	11-20	20.9	24.9	21.2	119.9	118.5	30.4	189.9	1508.4	17.2	90.0	15.4	16.8	
	21-30	19.3	58.2	33.3	73.9	27.4	260.7	61.9	114.8	136.0	38.8	17.1	10.4	
1999	1-10	8.1	7.3	10.6	56.6	97.4	24.1	65.1	196.0	207.6	75.4	65.7	17.2	2,748
	11-20	6.5	7.3	34.5	165.2	22.3	48.9	24.7	33.6	180.0	146.0	31.3	14.9	
	21-30	8.8	7.5	64.0	27.7	39.7	182.8	84.2	117.8	588.1	47.2	19.5	14.3	
2000	1-10	16.9	13.6	9.1	13.0	10.2	5.8	84.0	179.6	217.7	36.5	16.8	15.4	3,583
	11-20	17.4	9.9	12.3	14.8	4.8	39.1	100.1	73.2	888.1	16.1	14.2	16.7	
	21-30	16.0	7.7	14.1	42.1	7.9	170.3	568.5	788.3	84.7	23.2	22.2	12.9	
2001	1-10	13.7	11.2	101.0	20.4	11.1	2.5	72.6	66.3	10.8	25.1	19.8	15.7	1,092
	11-20	12.5	12.7	39.6	15.8	5.4	20.2	105.2	30.8	25.7	22.2	12.8	14.4	
	21-30	12.0	64.9	38.7	9.9	6.1	132.8	53.7	13.6	15.3	31.2	13.4	13.0	
2002	1-10	10.7	17.4	14.7	19.1	272.8	23.1	72.7	279.2	812.6	33.8	24.5	26.5	2,844
	11-20	45.2	15.1	17.5	106.4	56.9	20.1	32.0	400.1	78.0	27.0	18.9	26.8	
	21-30	30.3	11.9	27.7	33.3	25.1	16.9	34.5	96.6	38.4	31.9	17.3	29.0	
2003	1-10	20.8	18.9	59.8	34.0	213.0	59.2	699.4	118.5	346.0	25.3	23.0	21.9	5,624
	11-20	20.0	29.7	50.1	41.4	59.0	63.7	787.7	302.2	670.0	26.3	22.8	20.7	
	21-30	20.4	58.6	47.1	285.8	98.7	159.2	684.6	418.5	53.2	23.3	19.9	21.6	
2004	1-10	19.4	12.5	19.3	18.8	21.6	10.5	205.4	29.5	63.6	27.7	15.1	22.7	2,630
	11-20	18.4	12.9	29.4	11.3	48.1	25.6	577.9	165.1	108.6	16.3	13.5	17.1	
	21-30	16.7	18.0	20.5	19.3	19.8	537.4	60.9	261.4	120.5	13.3	15.6	16.3	

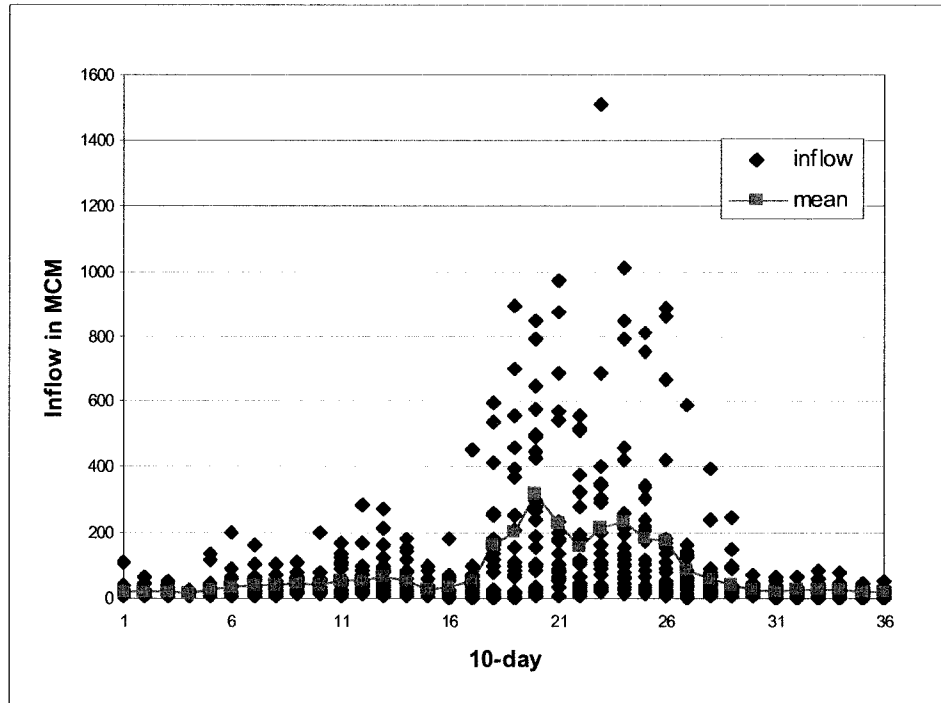


Figure 4.12 Monthly inflow distribution to Daecheong reservoir

4.2.2 Water Demand

The water demand data used in the analysis is given in Table 4.5. There are municipal and agricultural demands for two sites downstream of Yongdam Reservoir, the trans-basin diversion for municipal water in the Mankyung basin, and the instream flow for the Geum River (Figure 4.13). Population growth, urbanization, industrialization, and public interests in environmental conservation create a lot of pressure on water supply for all users in this area. It is essential to share available water in order to meet the target of water demands and instream flows.

Table 4.5 Water demand and diversion to the Jeonju in the upper region of Geum River basin.

(Unit: $10^6 m^3$)

Mon	10days	M&I water		agricultural		Subbasin sum	Trans-basin diversion	Instream flow		total sum
		muju	okcheon	muju	okcheon			reservoir	down	
Jan	1	0.056	0.249			0.305	13.48	1.94	4.787	6.731
	2	0.056	0.249			0.305	13.48	1.94	4.787	6.731
	3	0.061	0.274			0.336	14.83	2.14	5.264	7.404
Feb	4	0.062	0.276			0.338	13.48	1.94	4.787	6.731
	5	0.062	0.276			0.338	13.48	1.94	4.787	6.731
	6	0.049	0.221			0.270	10.78	1.56	3.830	5.385
Mar	7	0.058	0.260	-	-	0.318	13.48	1.94	4.787	6.731
	8	0.058	0.260	-	-	0.318	13.48	1.94	4.787	6.731
	9	0.064	0.285	0.046	0.271	0.667	14.83	2.14	5.264	7.404
Apr	10	0.060	0.268	0.018	0.108	0.454	13.48	1.94	4.787	6.731
	11	0.060	0.268	0.019	0.113	0.460	13.48	1.94	4.787	6.731
	12	0.060	0.268	0.011	0.067	0.407	13.48	1.94	4.787	6.731
May	13	0.064	0.283	0.034	0.202	0.583	13.48	1.94	4.787	6.731
	14	0.064	0.283	0.096	0.569	1.012	13.48	1.94	4.787	6.731
	15	0.070	0.312	0.107	0.632	1.120	14.83	2.14	5.264	7.404
Jun	16	0.069	0.307	0.220	1.299	1.895	13.48	1.94	4.787	6.731
	17	0.069	0.307	0.304	1.794	2.474	13.48	1.94	4.787	6.731
	18	0.069	0.307	0.222	1.311	1.908	13.48	1.94	4.787	6.731
Jul	19	0.071	0.318	0.258	1.526	2.174	13.48	1.94	4.787	6.731
	20	0.071	0.318	0.262	1.548	2.199	13.48	1.94	4.787	6.731
	21	0.078	0.349	0.240	1.415	2.082	14.83	2.14	5.264	7.404
Aug	22	0.075	0.335	0.164	0.967	1.540	13.48	1.94	4.787	6.731
	23	0.075	0.335	0.255	1.509	2.174	13.48	1.94	4.787	6.731
	24	0.083	0.368	0.160	0.944	1.554	14.83	2.14	5.264	7.404
Sep	25	0.078	0.349	0.108	0.637	1.172	13.48	1.94	4.787	6.731
	26	0.078	0.349	0.072	0.424	0.923	13.48	1.94	4.787	6.731
	27	0.078	0.349	0.060	0.353	0.840	13.48	1.94	4.787	6.731
Oct	28	0.071	0.318	0.049	0.289	0.726	13.48	1.94	4.787	6.731
	29	0.071	0.318			0.389	13.48	1.94	4.787	6.731
	30	0.078	0.349			0.428	14.83	2.14	5.264	7.404
Nov	31	0.060	0.268			0.328	13.48	1.94	4.787	6.731
	32	0.060	0.268			0.328	13.48	1.94	4.787	6.731
	33	0.060	0.268			0.328	13.48	1.94	4.787	6.731
Dec	34	0.056	0.249			0.305	13.48	1.94	4.787	6.731
	35	0.056	0.249			0.305	13.48	1.94	4.787	6.731
	36	0.061	0.274			0.336	14.83	2.14	5.264	7.404
sum		2.37	10.59	2.71	15.98	31.64	492.00	70.97	174.72	245.69

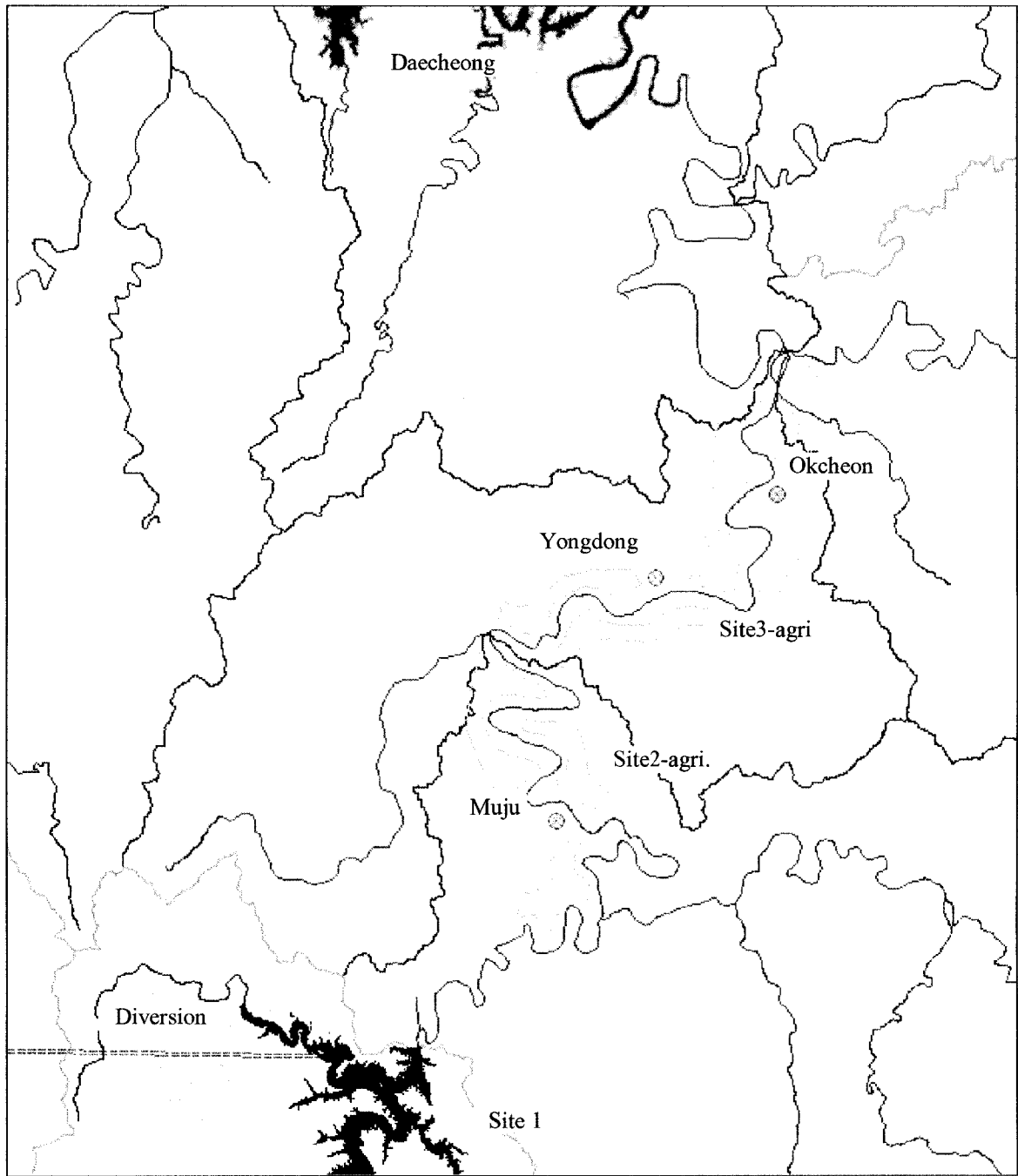


Figure 4.13 Water demands and diversion sites

In this study area, the largest demand is for the Jeonju diversion, second is for instream flow, third is for agricultural usage, and the smallest demand is for the municipal usage. Each water demand is plotted on a 10-day period as shown in Figure 4.14. Most demands are almost constant during the 10-day period except the agricultural demands which are concentrated in 10-day time periods from period 16 to 26. The instream flow in the Geum River and the trans-basin diversion for municipal water in the Mankyung basin are plotted in Figure 4.15. This figure shows that water uses for both demands are similar.

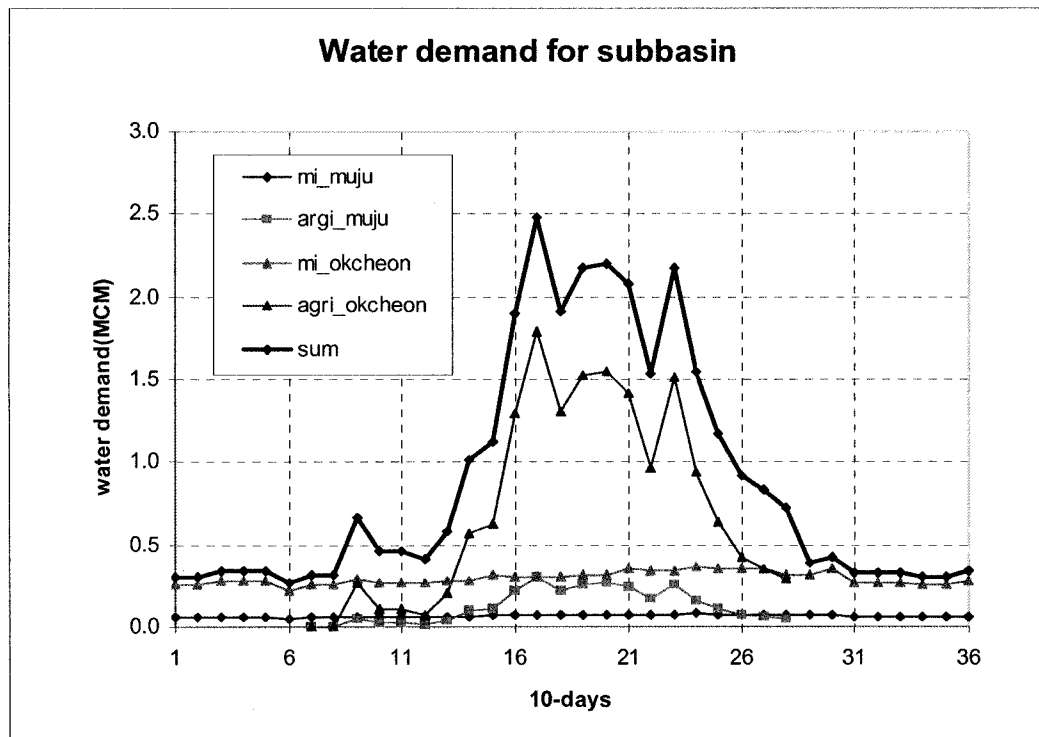


Figure 4.14 Seasonal water demand in Geum River basin

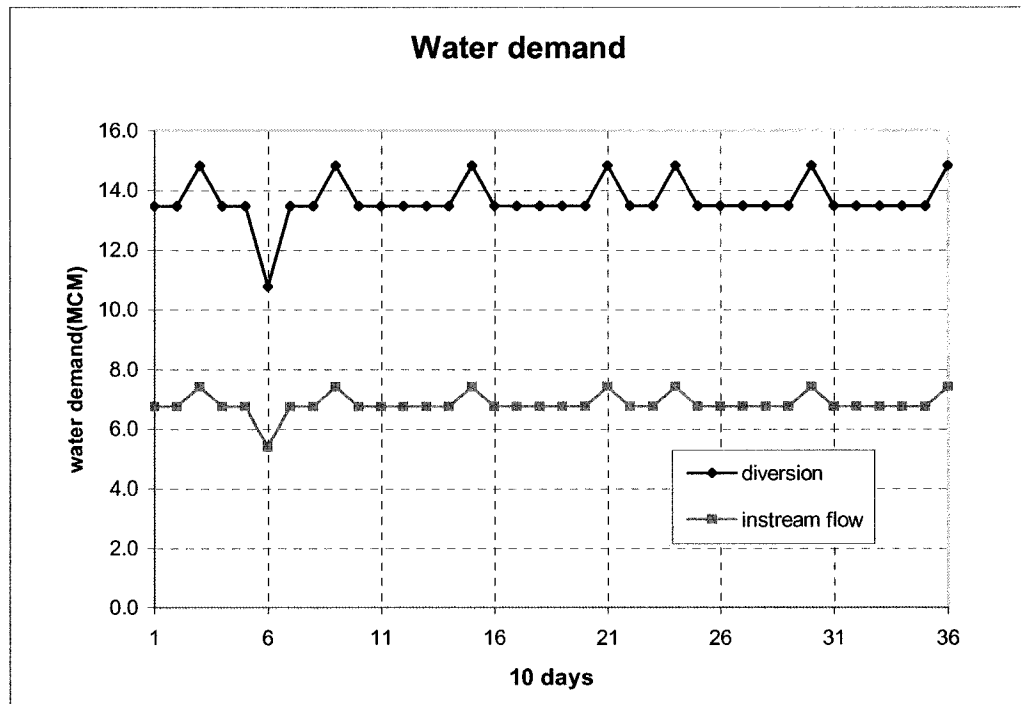


Figure 4.15 Trans-basin diversion to the Mankyung River and instream flow

4.3 Response Coefficient for Pumping

Specific yields, hydraulic conductivity, and stream depletion factors for stream-aquifer systems in the upper region of the Geum River as a function of distance from the well to the stream are obtained in the Geum river basin by a survey conducted by KOWACO. These data are listed in the Table 4.6. Streamflow depletion factors ranged from about 5days at a 50m distance between the well and the stream to above 100days at a 500m distance. The comparisons of stream depletion factors to the transmissivity are shown in Figure 4.16. Table 4.6 and Figure 4.16 show that the streamflow depletion factor decreases as the aquifer thickness increases and the distance between the well and the stream decreases. The stream-aquifer systems equilibrate quickly to new pumping

stresses in the case of large values of transmissivity and short distances between the well and the stream. Consequently, long periods of time may be needed for equilibrium to be re-established in the case of pumping from wells which are far from a stream. These wells can withdraw substantial groundwater storage amounts before capturing significant amounts of streamflow. For the case of a well very close to a stream, a quick return to equilibrium is expected. The representative aquifer thickness of alluvial sites in the Geum River is 15m and a representative value of hydraulic conductivity is 16m/day. The representative transmissivity of the alluvial aquifer is 240m²/day. The water demand is composed of mostly municipal water and agricultural uses. The municipal water demand is uniformly distributed through the year, but agricultural water demand is concentrated between March and October.

Table 4.6 Streamflow depletion factor of aquifers in Geum River basin

Distance of well to stream, m	Specific yield	Aquifer thickness, m	Hydraulic conductivity, m/day	Transmissivity, m ² /day	Streamflow depletion factor, days
50	0.22	10	16	160	3.438
50	0.22	15	16	240	2.292
50	0.22	20	16	320	1.719
50	0.22	25	16	400	1.375
100	0.22	10	16	160	13.750
100	0.22	15	16	240	9.167
100	0.22	20	16	320	6.875
100	0.22	25	16	400	5.500
200	0.22	10	16	160	55.000
200	0.22	15	16	240	36.667
200	0.22	20	16	320	27.500
200	0.22	25	16	400	22.000
500	0.22	10	16	160	343.750
500	0.22	15	16	240	229.167
500	0.22	20	16	320	171.875
500	0.22	25	16	400	137.500

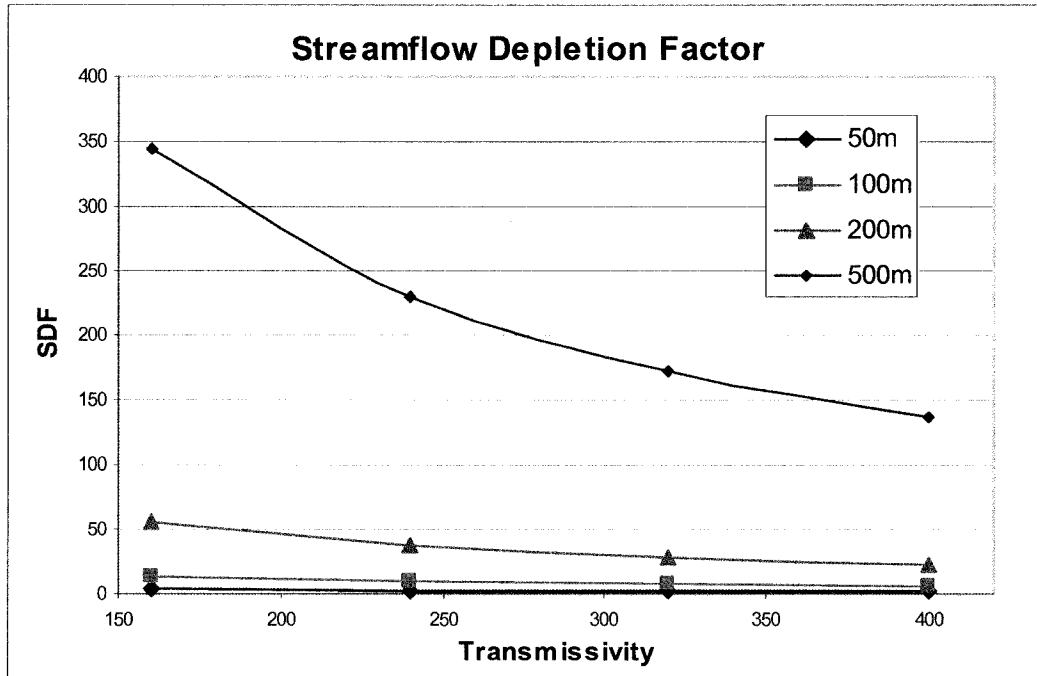


Figure 4.16 Streamflow depletion factor curves with distance

For the Geum River basin, the season from late June to September is the flood season and there are often severe droughts during from April to June. Even though a new multi-purpose reservoir was built at upper region in the Geum River basin in 2001, most of the water stored in Yongdam reservoir will be transferred to the ManKyung River basin through the diversion tunnel. Therefore, water supply for the downstream area of the Yongdam reservoir is limited. A stream depletion analysis was done for the case study of pumping in the downstream of Yongdam reservoir. For three pumping periods, 10 days, 20 days, and 30 days, streamflow and storage depletions as a function of distance between the well and the stream were calculated from the solution of the superposition of the complementary error function derived by Jenkins. To consider the withdrawal periods of the groundwater for agricultural or municipal uses, the pumping time is divided by the length of each period. The distances between the well and stream

are selected based on the streamflow depletion response coefficients (Q_s/Q_w) in which Q_w is the unit withdrawal of well and Q_s is streamflow depletion responses to the unit withdrawal. Stream depletion curves for all periods are plotted in Figure 4.17 and the stream depletion factor for a 10-day pumping period is plotted in Figure 4.18.

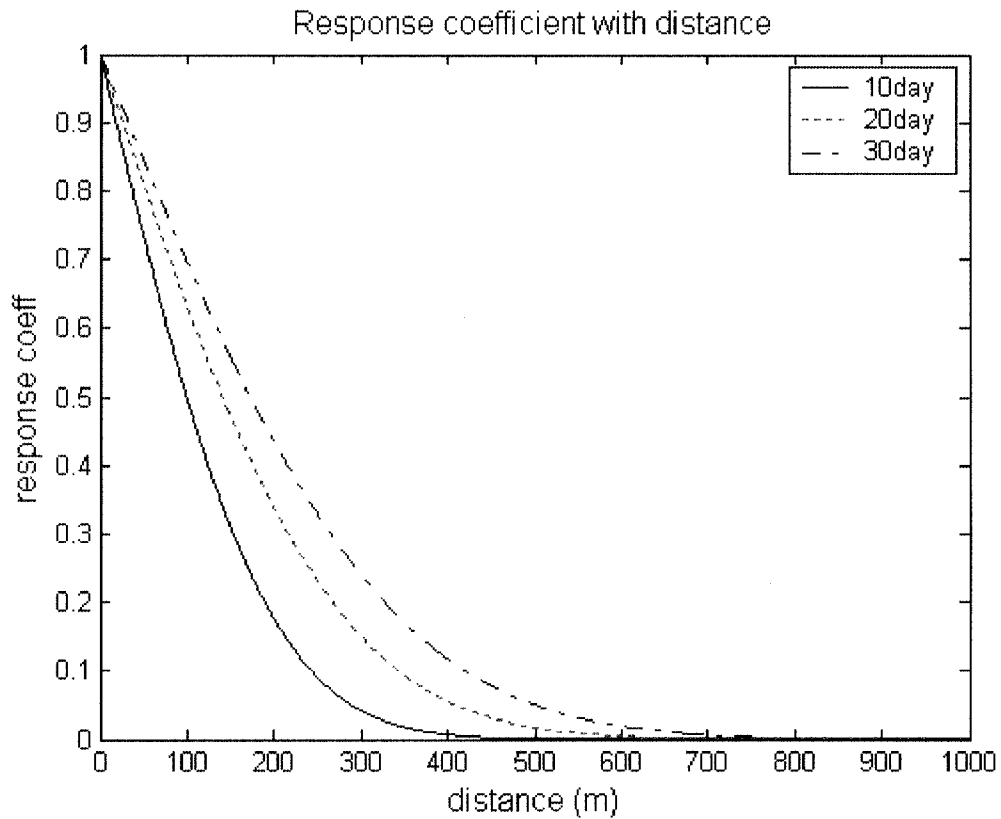


Figure 4.17 Streamflow depletion curve for periods

In Figure 4.18, when the distance between the well and the stream is small, the stream depletion occurs quickly and also stabilizes quickly to pumping stresses. Also stream depletion quits very soon after pumping stops, which is advantageous during periods of low recharge or drought. Wells far from a stream will withdraw groundwater

for a long time by capturing significant amounts of streamflow; consequently, long time periods may be needed for equilibrium to be restored.

The results for a 10-day pumping period considered for the agricultural demand in which horizontal distance between the well and stream is 50m show that stream depletion rates asymptotically approach the pumping rates. As the well pumping is turned off, the streamflow depletion rates decreases rapidly. The stream depletion rate in the case of the pumping with further distances between the well is nearly constant throughout the year. This stream depletion rate is approximately equal to the annual pumping rate pumped from a well. The stream depletion caused by pumping at a distance of 500m responds much more slowly than it does from pumping located close to the stream and the storage depletion comprises a significant part of the well discharge for the entire pumping horizon.

This simple example illustrates how conjunctive management strategies might be found to minimize streamflow depletions. If the groundwater is used for water supply purpose during periods of high recharge and high streamflow, pumping should occur from a well located close to a stream, and nearly all water captured by the well would be from streamflow depletion. During periods of low recharge and low streamflow, pumping should occur from wells that are located far from a stream and the stress imparted to the aquifer at the well would not cause substantial streamflow depletion. Consequently, the effect of the streamflow depletion factor continues for a long time horizon, and then the pumping impact occurs over the following time periods. Groundwater managers devising pumping schedules for groundwater that minimize instream flow deficits and maximize water supply during the drought season have to be concerned with the relation between

the timing of withdrawals and streamflow depletions, and this relation is a function of streamflow depletion factor of the system and withdrawal schedules (Barlow, 2001, 2003).

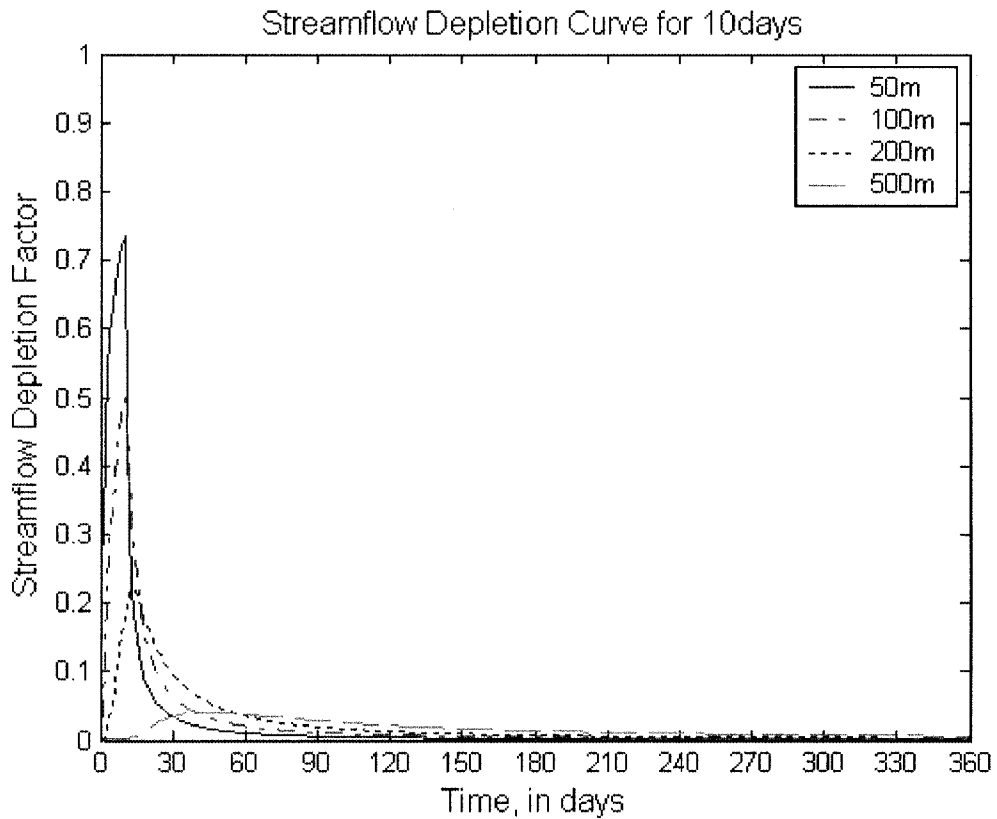


Figure 4.18 Streamflow depletion factor for 10-day pumping

In summary, the streamflow depletion factor, sdf , described in this section is a parameter that characterizes the response time of a particular stream-aquifer system. The parameter shows the streamflow effect due to well pumping in both hydraulic properties of the aquifer and the distance between the pumping well and the stream. For small values of sdf and short distances between the well and the stream, high rates of

streamflow depletion will occur for a short time after pumping begins and the stream-aquifer system will quickly attain a new dynamic equilibrium.

Conversely, for large values of sdf which occur for wells far from a stream, substantial amounts of water are withdrawn from the aquifer storage before large rates of streamflow depletion occur, and the stream-aquifer system will slowly attain a new dynamic equilibrium. These differences in streamflow depletion factor will affect optimal management strategies in the stream-aquifer systems and stream-aquifer-reservoir systems.

4.4 Stream-Aquifer Interaction

The purpose of this section is to use numerical groundwater flow models to evaluate streamflow depletion and induced infiltration in baseflow in alluvial stream aquifers for stream-aquifer systems of the upper region in Geum River basin.

4.4.1 Simulation of Stream-Aquifer System

For the baseflow stream aquifer system analysis, the MODFLOW finite-difference model was used. The representative, hypothetical case study system is assumed to be 2,400m long and 2,000m wide. The aquifer is unconfined and bounded by till uplands, which are not simulated. Sand and gravel deposits fill the valleys with varying values of saturated thickness. The thickness is assumed to be 15m equally along

the upland boundary and in the center of the valleys. The bottom of the aquifer is assumed to be impermeable bedrock.

The characteristics of the baseflow system include a gradient of 0.002m/m from upland to the stream and a stream gradient of 0.001m/m from the northern part to the southern part of the basin, which is consistent with the definition of baseflow-dominated system. The gradient comes from the average stream-channel gradient from the Geum River groundwater basic survey Report (MOCT/KOWACO, 2004). In the baseflow system, there is no down-valley component of groundwater flow. It is assumed that groundwater flow is in perpendicular to the stream and all groundwater discharges run to the stream. The model grid shown in Figure 4.19 was developed to consist of 60 rows, 60 columns, and 3 layers. The top layer of the model is assumed to be 20m thick. The second layer is 10m thick and the third layer is 2m thick from the bottom. The elevations of the layer in the base flow system slope downward from north to south. Grid cells are a uniformly distributed with length of 50m in the north to south direction and various with varied length in the east to west direction. The single stream that flows through the center of valley is 60m wide, which is the average width of streams studied by KOWACO. The grid size for modeling has a relatively minor effect on calculated seepage rates as noted by Sophocleous and others (1995). Recharge to the ground water surface is assumed to be 231mm/yr, which is based on typical recharge rates to aquifers in the upstream portions

of the Geum River Basin. These rates are estimated by the recession-curve-displacement method. Recharge from upland till was applied to each model using head-dependent flux boundaries at all cells along the upland contacts.

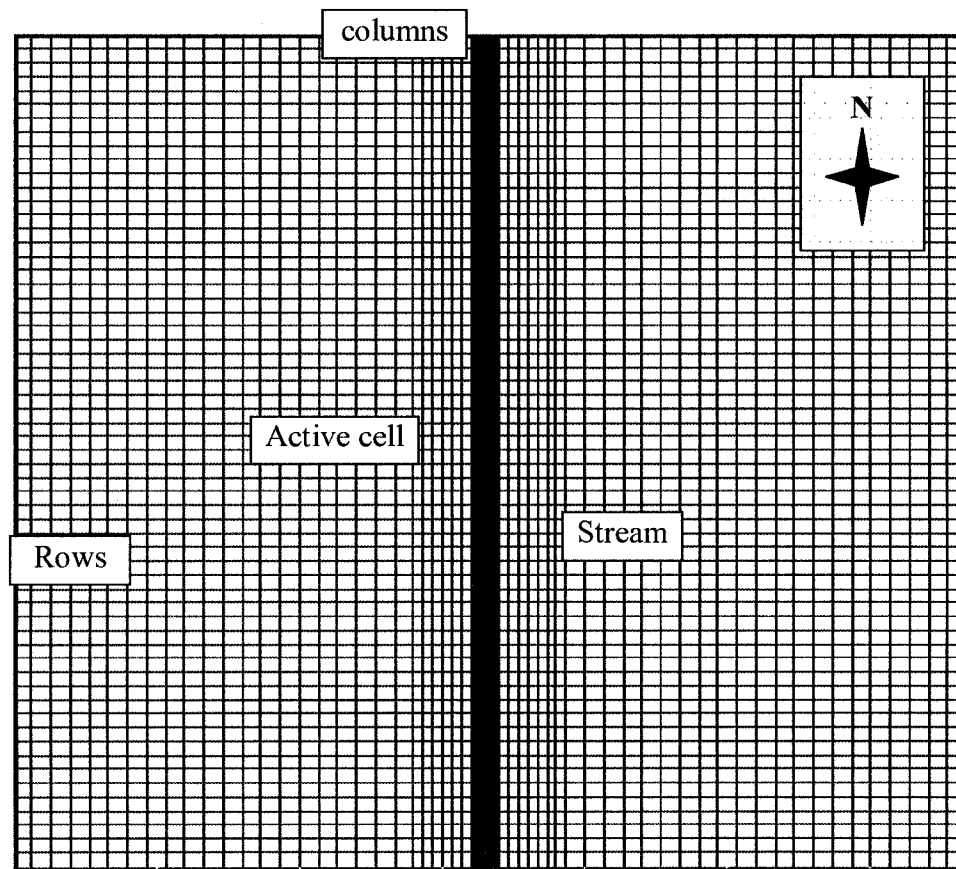


Figure 4.19 Baseflow model in river-valley stream-aquifer system

Head-dependent flux boundaries were also used on the northern and southern boundaries of the baseflow system to isolate the system from the upgradient and downgradient areas of the valley that are not simulated. Heads along the northern boundary range from 31m at the stream to 33m at the upland contacts. Heads along the southern boundary range from 30m at the stream to 32m at the upland contacts. Heads

were distributed along the northern and southern boundaries in such a way as to maintain a nearly uniform rate of baseflow throughout the system.

Streams in the upstream region of the Geum River are typically shallow and fully penetrate the aquifers. Based on the discharge obtained by KOWACO a uniform stream depth value is below 1-2m. Streambanks and streambeds are simulated using hydraulic leakages along the lateral and bottom faces of the stream cells. Horizontal leakage is simulated by use of the horizontal-flow barrier algorithm developed for MODFLOW by Hsieh and Freckleton (1992); vertical leakage is simulated by use of the VCONT parameter in MODFLOW (McDonald and Harbaugh, 1988).

The alluvial sediments have a uniform value of horizontal hydraulic conductivity of 16m/d and a uniform anisotropic ratio of 10:1 between horizontal and vertical values, which is typical of alluvial river valley aquifers of the Geum River. Specific yields of 0.22 and storage coefficient (or storativity) of 0.015 are used for transient simulations and these are typical values reported by KOWACO.

A horizontal hydraulic conductivity of 50-100m/d is specified for the stream cells so that there is very little resistance to flow within the simulated stream channels. Horizontal and vertical leakages of 50-100m/d and 5-10m/d are assumed for the streambanks and streambeds, respectively. The horizontal leakages correspond to a 50cm thick streambank with hydraulic conductivity equal to the horizontal hydraulic conductivity of the aquifer. The vertical leakages correspond to a 50cm thick streambed with hydraulic conductivity equal to the vertical conductivity of the aquifer.

A steady-state hydrologic budget for the system is shown in Table 4.7. Total inflow rate through the baseflow system is 3,417m³ per day. The only source of inflow to

the baseflow system is recharge directly to the deposits and from adjacent uplands. Groundwater flow directions in the baseflow system are perpendicular to the stream.

Table 4.7 Hydrologic budget for the baseflow aquifer

(unit : m³/day)

Inflow Component		Outflow Component	
Recharge to alluvial zone	2,849.26	Stream baseflow	3,416.7
Recharge from uplands	475.97		
Recharge of stream	91.47		
Total	3,416.7	Total	3,416.7

4.4.2 Steady State Stream-Aquifer Interaction

For analyzing the response between streamflow and aquifer well pumping, stream-aquifer interaction in the baseflow system was evaluated for steady-state conditions. In the first stage, a pumping rate of 1,000m³/d was simulated in which a single well pumped water from the system. The well was placed at varying distances of 50m, 100m, 200m, and 500m from the stream.

The contributing area to well pumping is simulated by the water particle pathlines shown in Figure 4.20. The well in the baseflow system captures all of the streamflow induced into the aquifer. In these particle tracking simulations, the rate of captured streamflow at the well was calculated by summing the individual contributions of particles along the stream. A total of fifty particles, located inside the stream, were

tracked in the baseflow system. Twelve particles were released at each stream cell. The particle tracking algorithm MODPATH was used in the simulation.

Induced infiltration of streamflow caused by groundwater pumping during flood events was evaluated based upon the results reported by Barlow (2001). Barlow found that short duration flood waves may not substantially affect the amount of streamflow captured by wells, and conversely, that pumping does not substantially affect the amount of streamflow seepage, induced infiltration, and bank storage that occurs during a flood event. Along with this result, flood waves are not considered in the conjunctive management formulations for the stream-aquifer-reservoir case study system.

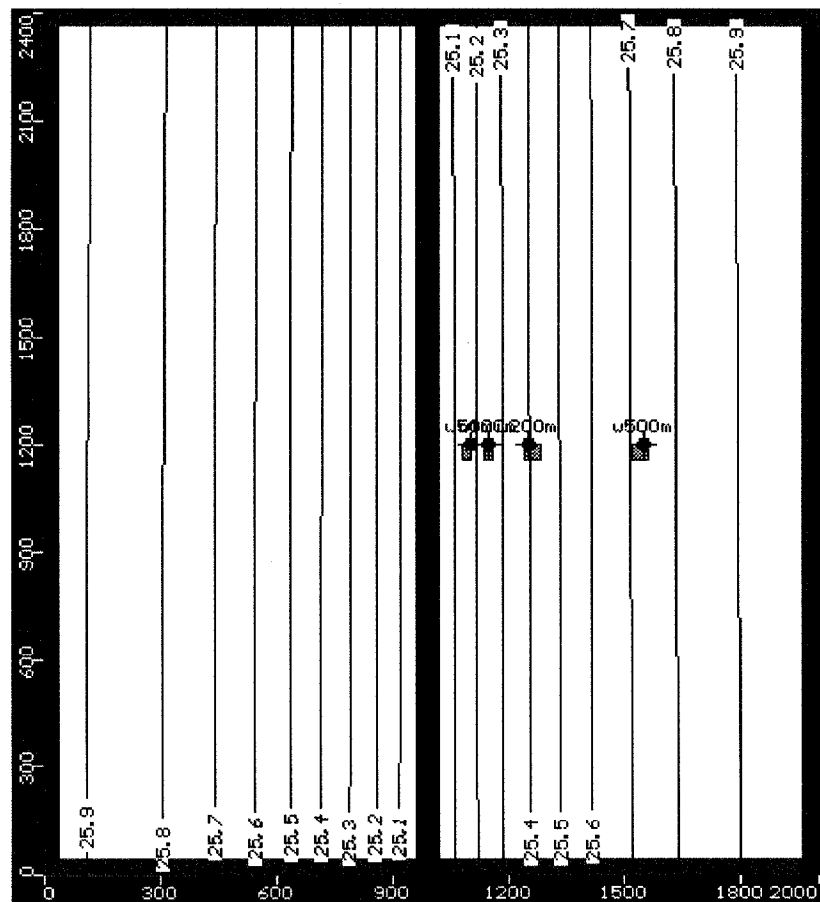


Figure 4.20 Water table contours in baseflow aquifer

4.5 Drought Analysis for Streamflow Records

This section describes low flow statistics used for long-term analysis of streamflow. The low flow statistics are evaluated for selecting the characteristics of drought analysis of the streamflow for use in the case study.

4.5.1 Low Flow Analysis

Good estimates of low flow statistics depend on having a long period of streamflow data. The longer the period selected for analysis, the more representative the hydrology will be of long-term climate conditions at the station (Bent, 1995). Flow statistics are typically estimated for the period of record at a streamflow-gaging station. However, concurrent records of equal length are best for a regional analysis (Fennessey and Vogel, 1990). A period of record of at least 20 years is commonly used to ensure stable estimates of streamflow predictability. For this study, monthly flow durations will be calculated for approximately a 20 year period using water year data for the specific streamflow-gaging station evaluated in this case study.

A flow-duration curve is a cumulative frequency plot that shows the percentage of time that a daily or monthly mean flow is equaled or exceeded during a given period (Gordon and others, 1992). For example, the 98-percent-duration flow (Q_{98}), considered a low flow, is equaled or exceeded 98 percent of the time, and the 2-percent-duration flow (Q_2), considered a high flow, is equaled or exceeded only 2 percent of the time. Both

annual and monthly drought frequency analysis were conducted. Annual analysis is to determine drought information about the overall hydrological condition in the specified region. Monthly analysis is used to determine the variability in the magnitude and frequency of streamflow within different months of the year.

4.5.1.1 Annual Statistics

Annual flow-duration curves show the relation between streamflow and the percentage of time that streamflow is exceeded on an annual basis. Annual flow-duration curves were developed for this case study by ranking annual total mean discharge for the period of analysis from largest to smallest and assigning the flows a plotting position by using the Weibull $(m/n+1)$ formula. The estimated probability that each discharge is equaled or exceeded was determined and a curve drawn through the plotted points.

The shape of flow-duration curves is determined by the hydrologic and geologic characteristics of a specified basin. The curves may be used to compare the characteristics of different sub-basins. A curve with a steep slope at the high-flow end indicates a flashy river where flow is largely from direct runoff. A curve with a gentle slope at the low-flow end indicates a river that is dominated by base flow from groundwater discharge or surface storage (Gordon and others, 1992). Annual flow-duration curves were used for analysis of the overall drought frequency based on an annual analysis. This result was compared with annual historical rainfall and streamflow in the region.

4.5.1.2 Monthly Statistics

Monthly flow-duration plots were constructed to perform a monthly streamflow analysis. The variability in the magnitude and frequency of monthly streamflows at a gaging station can be characterized by evaluation of monthly flow-duration curves for a period of analysis (Vogel and Fennessey, 1994). The distribution of selected flow durations was calculated to characterize streamflow variability over a 20 year period. Monthly flow durations were calculated for each month of a 20 year period. The monthly mean streamflows for each month of a given year were ranked in ascending order of discharge and the exceedance probability of each streamflow was calculated by use of the Weibull formula (Helsel and Hirsch, 1992). An exceedance percentile was calculated from the cumulative distribution frequency value of the sorted data. The 90-percent-monthly-duration-flow (10 year drought) means that the flow duration is equaled or exceeded 90 percent of the time for the period analyzed. Similarly 20 year and 50 year drought conditions were calculated using the 95-percent and 98-percent monthly flow duration. These results were used for evaluating the stream-aquifer response analysis and determining the water supply considering the stream, aquifer, and the reservoir.

4.6 Recharge Analysis due to Streamflow

The recharge of groundwater is used for the stream-aquifer interaction and groundwater supply evaluation in conjunctive water management. The numerical model

is representative of a baseflow dominated stream-aquifer system developed in Chapter 4. This forms the basis of the evaluation of conjunctive management formulations for stream-aquifer systems in the case study area.

To make the stream-aquifer system formulation accurate, the baseflow stream-aquifer system and recharge of the area have to be evaluated correctly. Program RORA, developed by Rorabaugh (1964), was used and the results of the program were compared with results of the manual application of the Rorabaugh method (Rutledge and Daniel, 1994). Rutledge (1998, 2000, 2004) present more information about the application of the method.

The RORA computer program allows automated analysis of streamflow hydrographs to estimate groundwater recharge. Output from the program, which is based on the recession-curve-displacement method, was compared to estimates of recharge obtained from a manual analysis of 156 years of streamflow record and 15 streamflow gaging stations in Eastern United States. Statistical tests showed that there was no significant difference between paired estimates of annual recharge by these two methods. (Rutledge and Daniel, 1993)

The RORA program estimates ground water recharge using the recession-curve-displacement method. It requires that a recession index is specified for a site, which is the time required for baseflow to decline from one logarithmic cycle on a semi logarithmic plot between streamflow and time. The computer program RECESS was used to estimate the recession index and to define the master recession curve for the basin. The computer program PART was used to estimate monthly and annual baseflow rates that were the actual net groundwater recharge rates specified to the models. PART used streamflow

partitioning to estimate a daily record of baseflow under the streamflow record. PART also calculates daily streamflow rates for the streamflow record.

The monthly recharge pattern of the aquifer was analyzed for the alluvial zone along the stream. The daily inflow data of Daecheong reservoir was used for the recharge analysis where Daecheong reservoir has a daily inflow data series recorded since 1981. Therefore the inflow data from 1981 to 2004 are analyzed to represent a homogeneous time series. The evaporation loss is already considered in the inflow data of Daecheong reservoir. So, evaporation and evapo-transpiration are not considered in the recharge analysis. The results of the analysis are shown in Figures 4.21 and 4.22. These figures show that the recession curves decrease with time.

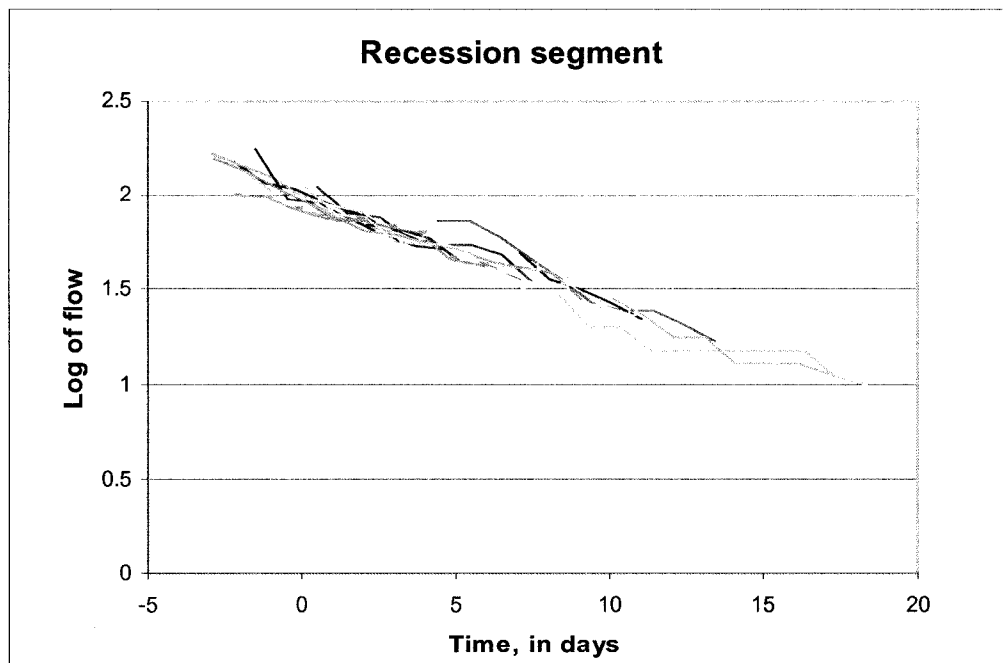


Figure 4.21 Recession segment for the case study area

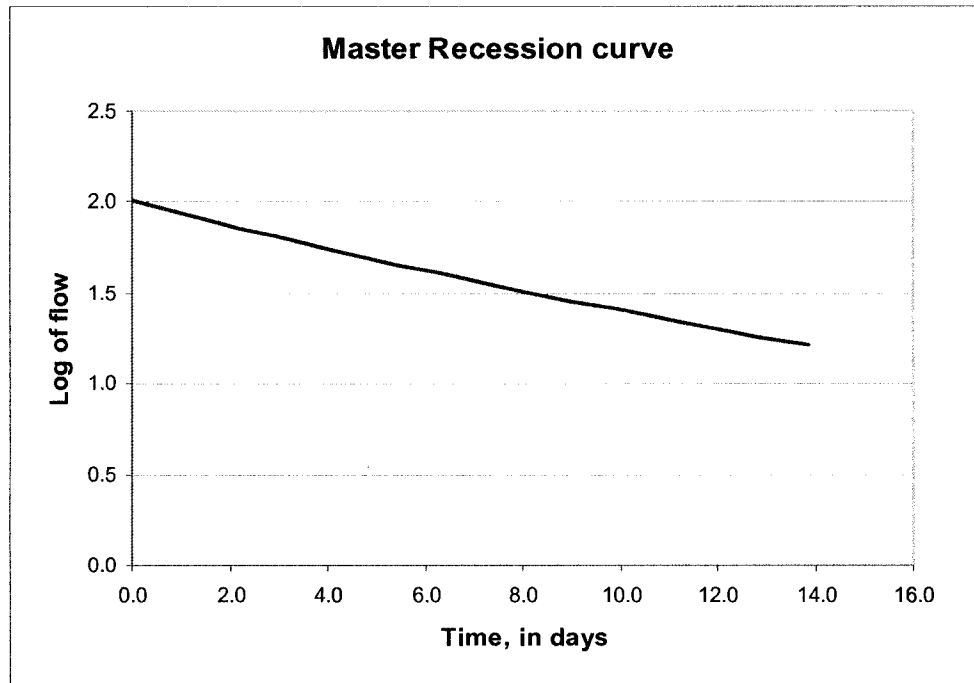


Figure 4.22 Master recession curve for the case study area

4.7 Groundwater Table Variations and Pumping Power

The groundwater table variation due to pumping is calculated by the Dupuit-Forchheimer well discharge formula. The input parameters used in this analysis are 2,400m of width, 1000m of length, 0.002 of regional gradient, 15m of aquifer thickness, and 500m of influence radius. There are two withdrawal conditions of municipal and agricultural pumping. The municipal water pumping facilities are close to the stream and the hydraulic conductivity is faster. On the other hand the agricultural water pumping facilities are far from the stream and the hydraulic conductivity is slower.

The equations of the Section 3.5 were programmed into MATHCAD software. MATHCAD was used to solve the equations and to draw groundwater table variations for

two- and three-dimensional representation. The maximum groundwater drawdown by municipal water supply pumping is 7.8m below the ground surface and that by pumping for agricultural water supply is 9.5m below the ground surface. The detailed calculation of drawdown is shown in appendix 1. The drawdown results due to municipal and agricultural water pumping are illustrated in Figures 4.23 and 4.24. The pumping power due to the groundwater withdrawal is calculated as a function of pumping discharge, table depth, and pumping efficiency as follows:

$$P_{pump} = \frac{G \cdot H}{0.102 \cdot \eta} \quad (4.1)$$

where P_{pump} is pumping power (kW); G is pumping discharge (m^3 / sec); H is groundwater table depth (m); and η is pumping efficiency (0.5).

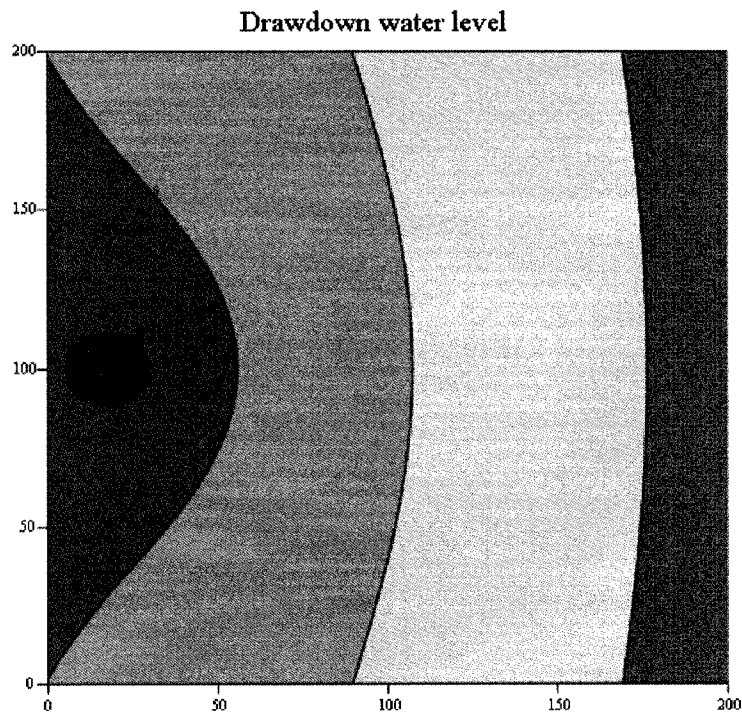


Figure 4.23 Drawdown at municipal water pumping site

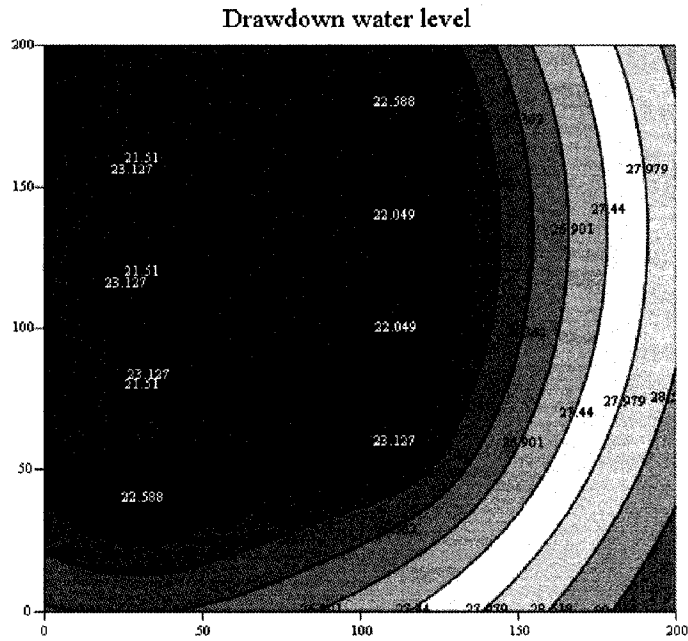


Figure 4.24 Drawdown at an agricultural water pumping site

4.8 System Application

The LINGO, optimization software, version 10, was developed by Lind Systems Inc. in USA. It is a comprehensive tool designed to make building and solving linear, nonlinear and integer optimization models faster and easier. LINGO provides a completely integrated package that includes a powerful language for expressing optimization models, a full featured environment for building and editing problems, and a set of fast built-in solvers. LINGO has been applying widely in many fields since the 1980s.

The LINGO software is a good choice to solve the conjunctive water management optimization problem considering the types of objective functions, decision variables, and constraints. Compared to the earlier LINDO version of the software, LINGO is easier to

use and provides enhanced results. Finally the results of the LINGO model were used to develop reservoir operational policies and groundwater pumping strategies.

4.9 Evaluation using KModSim Network Programming

The water allocation simulation model, KModSim is used to evaluate the developed operational rules and conjunctive water management between the streamflows and the groundwater. KModSim is a modified version of the MODSIM model developed at Colorado State University. Various versions of MODSIM are described by Shafer (1979), Labadie and Shafer (1979), Labadie (1983), Labadie and others (1984), and Labadie (1992, 2000). MODSIM has been used extensively for modeling water rights allocations in the United States such as the Rio Grande River Basin, and the Upper Snake River Basin (Fredericks, 1998). KModSim model also has been developed over the past 5 years to serve as the water allocation simulation model for the entire Geum River Basin in partnership with Korea Institute of Water and Environment of KOWACO. Because of its application to the Geum basin it is an ideal choice for use in this research.

A major consideration in selecting a river basin simulation model is the ease and flexibility to specify various operational conditions. This includes the priorities of meeting the various demands and the desires to maintain target storage levels in the reservoir. The generalized river basin network model KModSim employs a network optimization algorithm for simultaneously assuring that water is allocated according to physical, hydrological, and institutional aspects of river basin management included stream-aquifer interactions.

The basic principle underlying KModSim is that most physical water resources system can be simulated as capacitated flow networks. The term capacitated refers to imposition of strict upper and lower bounds on all flows in the network. Components of the system are represented as a network of nodes, both storage (i.e., reservoirs, groundwater basins, and storage right accounts) and non-storage (i.e., river confluences, diversion points, and demand locations) and links or arcs (i.e., canals, pipelines, natural river reaches, and decreed water rights) connecting the nodes. Links and nodes in KModSim are not confined to representing physical components of a river basin system, but may also be utilized to symbolize artificial and conceptual elements for modeling complex administrative and legal mechanisms governing water allocation. In addition to the links and nodes defined by users, several accounting nodes and links are automatically created. These nodes and links are essential to insuring mass balance is satisfied throughout the entire network. KModSim simulates water allocation mechanisms in a river basin through sequential solution of the following generalized network flow optimization problem for each time period $t = 1, \dots, T$:

$$\min \sum_{\ell \in A} c_{\ell} q_{\ell} \quad (4.2)$$

subject to:

$$\sum_{k \in O_i} q_k - \sum_{\ell \in I_i} q_{\ell} = b_{it} \quad \text{for all nodes } i \in N \quad (4.3)$$

$$l_{\ell t} \leq q_{\ell} \leq u_{\ell t} \quad \text{for all links } \ell \in A \quad (4.4)$$

where A is the set of all arcs or links in the network; N is the set of all nodes; O_i is the set of all links originating at node i (i.e., outflow links); I_i is the set of all links terminating at node i (i.e., inflow links); b_{it} is the (positive) supply or (negative) demand at node i ; q_ℓ is the flow rate in link ℓ ; c_ℓ are costs, weighing factors, or water right priorities per unit flow rate in link ℓ ; and $l_{\ell t}$ and $u_{\ell t}$ are specified lower and upper bounds, respectively, on flow in link ℓ . Equations 4.2-4.4 are solved with a highly efficient lagrangian relaxation algorithm based on a dual coordinate ascent called RELAX-IV (Bertsekas and Tseng, 1994). This algorithm is up to two orders of magnitude faster than the revised simplex method of linear programming. Since solution of Equations 4.2-4.4 is executed period by period, rather than as a fully dynamic optimization, flows in the carryover storage arcs shown in Figure 4.25 become initial storage levels for the next period optimization.

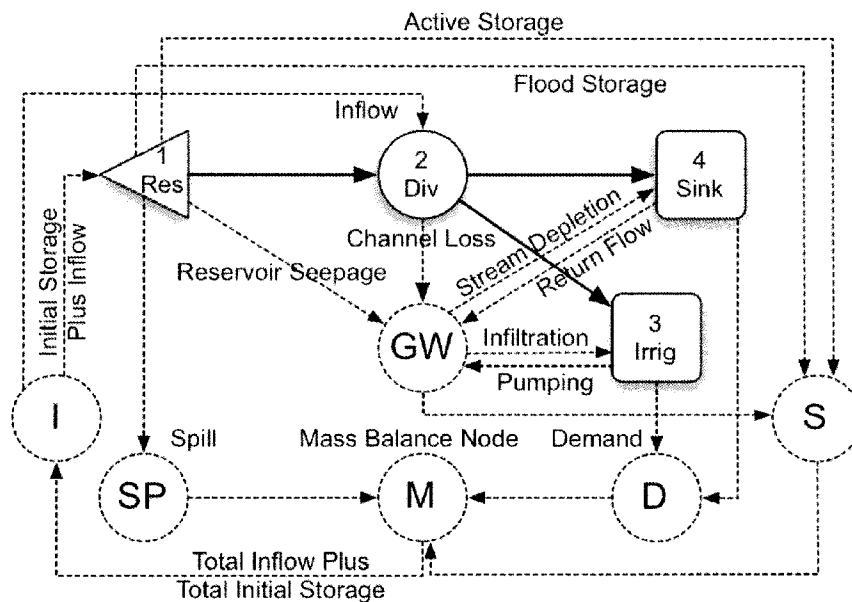


Figure 4.25 Illustration of KModSim network structure with accounting nodes and links

The stream-aquifer module within KModSim performs dynamic calculation of reservoir seepage, irrigation infiltration, pumping, channel losses, return flows, and river depletion due to pumping. Stream-aquifer return/depletion flows are simulated using response coefficients calculated using the one dimensional equations developed by Glover (1977). These are similar to groundwater response coefficients estimated from the stream depletion factor method of Jenkins (1968). Fredericks, et al. (1998), showed that response coefficients can be generated from three-dimensional finite difference groundwater models such as MODFLOW (Harbaugh and McDonald, 1996) and read into KModSim from external data files. This allows response coefficients to be calculated based on spatially distributed aquifer characteristics and complex boundary conditions.

For demand node i and any current time period considered, the total return flow IRF_{ik} from previous and current time periods due to groundwater recharge is calculated as:

$$IRF_{ik} = \sum_{\tau=1}^k I_{i\tau} \delta_{ik-\tau+1}; \quad \delta_{ik-\tau+1} = 0 \text{ for } k - \tau + 1 > N \quad (4.5)$$

in which $I_{i\tau}$ is the recharge rate at node i , time period τ , and $\delta_{ik-\tau+1}$ is the discrete kernel coefficient defined for node i , period $k - \tau + 1$, specified period k . The discrete kernel coefficients are calculated internally in KModSim using either the Glover equations or the SDF method, or they can be precalculated externally by a model such as MODRSP or MODFLOW and then read into KMODSIM as an external data file. In

effect, the use of precalculated discrete kernel response coefficients essentially represents the selected groundwater model, but without the need to explicitly run the groundwater model during KModSim calculations, resulting in considerable savings in computer time. The same approach used for calculating return flows is also applied to calculation of stream depletion due to pumping PSD_{ik} , where

$$PSD_{ik} = \sum_{\tau=1}^k P_{i\tau} \alpha_{ik-\tau+1}; \alpha_{ik-\tau+1} = 0 \text{ for } k - \tau + 1 > N \quad (4.6)$$

In the case of groundwater withdrawal $P_{i\tau}$, the same principles described above are applicable to determining response coefficient kernels $\alpha_{ik-\tau+1}$, but for river depletion rather than return flows

Jenkins (1968) solved the Glover equation graphically by developing dimensionless curves and tables to compute the rate and volume of stream depletion by wells. Under ideal conditions, the stream depletion factor ($SDF = a^2 S / T$) represents the time in days where the volume of stream depletion is 28% of the volume pumped during time t , a is perpendicular distance from the pumped well to the stream, L , S is specific yield of the aquifer, and T is transmissivity.

In a complex system, the value of SDF can represent integrated effects of irregular impermeable boundaries, stream meanders, aquifer properties, areal variation, distance from the stream, and hydraulic connection between the stream and aquifer. The SDF is intended to be a calibrated parameter determined from numerical modeling of stream-aquifer systems. The basic assumption is that SDF generated response functions

can have shapes similar to those calculated from 3-dimensional numerical finite difference groundwater models, such as developed by Hurr and Burns (1980) and Warner et al., (1986) for regional stream-aquifer system modeling. The same Glover equation can be used to represent stream accretion rate to aquifer recharge rate over time. The SDF concept can also be applied to the Maasland parallel drain analogy for calculating irrigation return flows, although it can no longer be interpreted as exactly representing the time in days.

4.10 Performance Measures

Objectives for managing the water resources in this study relate to meeting the diversion and downstream water demands, complying with instream flow regulation, and maintaining satisfactory groundwater use. To facilitate quantitative comparison among alternatives, several performance measures are adopted. Performance measures for meeting water resources management are total reliability, total vulnerability, and total resiliency of water supply. All performance measures are consistently constructed to have values from zero (0) to one (1), with zero (0) indicating undesirable performance and one (1) indicating desirable performance.

(1) Total reliability of water supply

Gabriel and others (2000) defined reliability of water supply as the probability that a reservoir system resides in a satisfactory. A satisfactory state occurs when downstream instream flow and diversion water demands are fully satisfied.

$$P_{rel} = \frac{1}{N_T} \sum_{t=1}^N Z_t \quad (4.7)$$

Where N_T is total number of time steps within the operational period, the indicator function $Z_t = 1$ if the performance is in a satisfactory state for time step t , or a value of $Z_t = 0$ if the performance is not satisfactory.

(2) Total vulnerability of water supply

Vulnerability in water supply usually is measured by the ratio of the amount of water delivered relative to the amount water required (Djordjevic, 1993).

$$P_{vul} = \frac{1}{N_T} \sum_{t=1}^N \frac{R_t}{D_t} \quad (4.8)$$

where R_t is release in each time steps, D_t is required water demand over a time, and N_T is total number of time steps within the operating period.

Suppose multiple reservoirs or multiple demand points, represented as M , are also considered. Reliability and vulnerability can also be represented as composite metrics by summing the metrics at each location over all possible locations:

$$P_{rel} = \frac{1}{M} \sum_{i=1}^M \left[\frac{1}{N_T} \sum_{t=1}^{N_T} Z_t \right] \quad (4.9)$$

$$P_{vul} = \frac{1}{M} \sum_{i=1}^M \left[\frac{1}{N_T} \sum_{t=1}^{N_T} \frac{R_t}{D_t} \right] \quad (4.10)$$

(3) Total resiliency of water supply

Resiliency in water supply addresses how quickly a system returns to a satisfactory state after any reservoir in the system fails to meet demand. During the simulation, a reservoir is checked to ensure that it meets the minimum instream flow and diversion water demand. If at least one reservoir fails to meet the demands, then a system failure is considered to have occurred (Gabriel and others, 2000). The performance measure developed Burn et al. (1991) is as follows.

$$P_{res} = \frac{1}{\left[N_f \left(\frac{N_{cf}}{N_T} \right) \right]} \quad \text{for } N_f \geq 1 \quad (4.11)$$

where N_f is total number of occurrences of system failure over total time T, N_{cf} is the maximum number of consecutive periods of failure over total time T, and N_T is total number of time steps within the operating period. Gabriel and others (2000) change Equation 4.11 into Equation 4.12:

$$P_{res} = \begin{cases} 1 / [N_f \cdot N_{cf}] & \text{for } N_f \geq 1 \\ 1 & \text{for } N_f = 0 \end{cases} \quad (4.12)$$

Equation 4.12 describes a performance measure that approaches 0 as the number of consecutive periods increases and equals 1 for either a single failure or no failures.

CHAPTER 5

CONJUNCTIVE MANAGEMENT SYSTEMS

This study focuses on evaluating conjunctive water management using a multi-objective optimization model that considers instream flow requirements for fish and aquatic life, municipal and industrial water supply and irrigation, a trans-basin diversion for water supply, and the cost of ground water and surface water use. There is no best solution of this conjunctive water management problem unless all the objectives can be fully met. Rather the decision maker must decide on a satisfactory closeness for meeting the various goals.

In the upper region of the Geum River basin, the focus of any conjunctive management plan must be on managing the impact of existing and proposed wells that pump in close proximity to the river. Before the trans-basin diversion of the Yongdam multi-purpose reservoir, groundwater pumping was not necessary to meet the demands. Today, however, increasing instream flow requirements for aquatic habitats and recreational use and increasing water demands need more available water resources, the amount of streamflow is limited, particularly during low streamflow conditions.

The approach in this research was to formulate a conjunctive water management optimization model including reservoir to meet the various demands on the system. It is

considered the streamflow depletion due to pumping to reflect the influence between surface and groundwater withdrawal. The model can set different priorities for allocating shortages in the river system. Additionally the model had to be able to evaluate the impact of pumping by either single- or multiple-wells located at varying distances from the river. The optimization results of the stream-aquifer-reservoir system were analyzed to develop the reservoir operational guide curves. The performance of these guide curves was then evaluated by the KModSim model.

Conjunctive water management studies have mainly considered the stream-aquifer system only without directly considering reservoir operation. A particularly important variable which is often overlooked in conjunctive management of stream-aquifer system is the transient behavior of these systems. However, since the withdrawals from pumping may vary by time period and well location, transient behavior is especially important.

Formulation of an optimization model requires definition of the decision variables, an objective function, and a set of constraints. Decision variables represent the quantifiable decisions that are to be determined by solution of the optimization problem. For each time period the decision variables are the pumping rate of groundwater at each well location, the withdrawal from surface water, the instream flow at control points, the amount of water provided to the trans-basin diversion, and the reservoir release. The goal programming objective function is a mathematical expression of management goals and priorities. Constraints represent physical, hydraulic, and other specific restrictions that define the mass balance equation, the characteristics of the reservoir and pumping wells, consumptive water use, diversions, instream flows, return flows, stream and aquifer

depletions and recharge, and the cost weighting factors for stream and ground water withdrawal.

5.1 Stream-Aquifer-Reservoir System

5.1.1 Model Formulation

In this section, the specific formulation for evaluating optimal operating policies of the stream-aquifer-reservoir system is described. The objective function was also formulated to satisfy the instream flow requirements, downstream water demands, the trans-basin diversion, and allowable surface and groundwater withdrawal. The constraints consider the minimum and maximum reservoir storage and release, the interaction of the stream and aquifer, and upper limits on water demands.

Linear goal programming was selected as a tool to solve the problem for the upper region of the Geum River basin. The problems are formulated as a linear function in which hydropower and evaporation are not considered. The commercial LINGO software was used to solve this problem.

The purposes of the modeling are to minimize water shortage of instream flow and water demands for trans-basin diversion and downstream in the Geum River, to determine optimal reservoir storage and release, to determine optimal sequences of pumping considering streamflow depletion factors, and to minimize pumping costs. The model accounts for pumping costs, reservoir storage capacity, diversion and spill capabilities of the Yongdam reservoir, streamflow depletion, minimum and maximum

withdrawal rates by pumping, return flows from both municipal and agricultural water use, and the lagged time of return flow for agricultural water.

The stream-aquifer-reservoir system model uses linear goal programming to identify trade-offs of the water shortage among diversion, downstream water demand, instream flow, and find the optimum operating policy of reservoir release and withdrawal between the streamflow and the aquifer.

The stream-aquifer-reservoir system is defined for 10-day time step reservoir operation considering surface and groundwater withdrawals, diversion, and instream flow. The objective function, Equation 5.1, is the minimization of the weighted sum of the goal deviations associated with the instream flow, diversion, downstream water demands, reservoir spill, and cost of surface and groundwater withdrawals. Equation 5.1 does not appear to be in the general programming format given by Equation 3.41. However, as will be described below each of the terms in the objective function represents a goal violation. In the case that a particular deviation is not considered (for example, there is no problem if the goal is over-achieved) it is effectively multiplied by a weight equal to zero. In these cases it is simply omitted from the objective function.

The first constraint, Equation 5.2, represents the state equation that describes the water balance for the reservoir. The second constraint, Equation 5.3, represents the water balance at downstream of the reservoir including reservoir release and spill, lateral flow, stream flow depletion due to pumping, and return flows from municipal and agricultural water use. Equation 5.4 describes streamflow depletion due to groundwater pumping and it is described in detail in Section 5.1.3.

Equation 5.5, 5.6, and 5.7 represent the goal deviations or the under-achievement or over-achievement of instream flow, the trans-basin diversion and the downstream water demand respectively. Note that in the objective function, Equation 5.1, there are also terms for spill, cost of groundwater and the cost of surface water. These terms do not require constraints to compute goal deviations because their implied goal value equals zero and they can not be negative. Therefore, these three terms actually represent the over-achievement of the goal of zero.

Equation 5.8 and 5.9 show the return flow for municipal and agricultural water use respectively. The rate of the return flow for municipal and agricultural water use is 0.6 and 0.3 respectively expressed in Geum River Basin Survey Report by KOWACO. Particularly agricultural return flow has a lagged time interval with 0.7, 0.2, and 0.1 based upon three 10-day time steps. These parameters come from hydraulic conductivity and distances between stream and wells in accordance with shape of alluvial zone in the upper region of Geum River Basin. The next three sets of Equations, 5.10, 5.11, 5.12, ensure that the operation will satisfy physical constraints of the system.

$$\min \sum_{t=1}^{np} \left\{ \begin{array}{l} C_r(Q_{sm,t}) + C_d(D_{def,t} + D_{sw,t}) + \sum_{i=1}^{nw} C_i(Q_{def,i,t} + Q_{sw,i,t}) \\ + C_{sp}(SP_t) + \sum_{l=1}^{nw} C_l Q_{gw,l,t} + \sum_{m=1}^{nw} C_m Q_{sw,m,t} \end{array} \right\}; \quad (5.1)$$

subject to

$$S_{t+1} = S_t - R_t + I_t - D_t - SP_t, t=1, \dots, np \quad (5.2)$$

$$Q_{s_{j,t}} = R_t + SP_t + Q_{lf_{j,t}} - Q_{sd_{j,t}} - \sum_{i=1}^{nw} Q_{sw_{i,t}} + Q_{ret-m_{j,t}} + Q_{ret-ag_{j,t}} ; \quad (5.3)$$

$$j=1,\dots,ns, i=1,\dots,nw, t=1,\dots,np$$

$$Q_{sd_{j,t}} = \sum_{i=1}^{nw} \sum_{k=1}^{36} r_{j,i,k} Q_{gw_{i,t'}} , \quad i=1,\dots,nw, \quad t=1,\dots,np ; \quad (5.4)$$

$$t' = t - k + 1, \quad \text{for } t-k+1 > 0;$$

$$t' = 36 + (t - k + 1), \quad \text{for } t-k+1 \leq 0;$$

$$Q_{s_{j,t}} - (Q_{s_{j,t}})_{\min} + Q_{sm_{j,t}} - Q_{s.sw_{j,t}} = 0 ; \quad j=1,2,\dots,ns, t=1,2,\dots,np \quad (5.5)$$

$$Q_{sm_{j,t}} \geq 0, \quad Q_{s.sw_{j,t}} \geq 0$$

$$Q_{div_t} - D_t + D_{def_t} - D_{sw_t} = 0 ; \quad t=1,2,\dots,np \quad (5.6)$$

$$D_{def_t} \geq 0 , \quad D_{sw_t} \geq 0$$

$$Q_{d_{i,t}} - Q_{gw_{i,t}} - Q_{sw_{i,t}} + Q_{def_{i,t}} - Q_{sw_{i,t}} = 0 ; \quad i=1,2,\dots,nw, t=1,2,\dots,np \quad (5.7)$$

$$Q_{def_{i,t}} \geq 0 , \quad Q_{sw_{i,t}} \geq 0$$

$$Q_{ret-m_{j,t}} = 0.6 \cdot rm_t \cdot \left(\sum_{i=1}^{nw} Q_{gw_{i,t}} + \sum_{i=1}^{nw} Q_{sw_{i,t}} \right) , \quad j=1,\dots,ns, i=1,\dots,nw, t=1,\dots,np, ; \quad (5.8)$$

$$[rm_t = 1]$$

$$Q_{ret-ag_{j,t}} = 0.3 \cdot ra_t \cdot \left(\sum_{i=1}^{nw} Q_{gw_{i,t}} + \sum_{i=1}^{nw} Q_{sw_{i,t}} \right), \quad j = 1, \dots, ns, i = 1, \dots, nw, t = 1, \dots, np \quad ; \quad (5.9)$$

$$\left[ra_t : ra_t = 0.7, \quad ra_{t+1} = 0.2, \quad ra_{t+2} = 0.1 \right]$$

$$(R_t)_{\min} \leq (R_t) \leq (R_t)_{\max}, \quad t = 1, \dots, np \quad ; \quad (5.10)$$

$$(S_t)_{\min} \leq (S_t) \leq (S_t)_{\max}, \quad t = 1, \dots, np \quad ; \quad (5.11)$$

$$\left(Q_{gw_{i,t}} \right)_{\min} \leq \left(Q_{gw_{i,t}} \right) \leq \left(Q_{gw_{i,t}} \right)_{\max} \quad \forall i = 1, 2, \dots, nw, \quad \forall t = 1, 2, \dots, np \quad (5.12)$$

where nw : number of wells;

np : number of time periods;

ns : number of streamflow constraint sites such as control points;

C_r : weighted value for instream requirement deficit;

C_d : weighted value for diversion water supply;

C_i : weighted value for water supply deficit;

C_{sp} : weighted value for reservoir spill;

C_l : weighted value for groundwater supply;

C_m : weighted value for surface water supply;

$\left(Q_{s_{j,t}} \right)_{\min}^l$: instream requirement at site j in time period t ;

$Q_{s_{j,t}}$: streamflow at constraint site j in time period t ;

$Q_{sm,j,t}$: shortage instream flow target at constraint site j in time period t ;

$Q_{s, sur,j,t}$: excess instream flow at constraint site j in time period t ;

$Q_{div,t}$: target diversion rate of reservoir in time period t ;

D_t : diversion rate of reservoir in time period t ;

$D_{def,t}$: shortage diversion water supply in time period t ;

$D_{sur,t}$: excess diversion water supply in time period t ;

$Q_{d,i,t}$: target volume for water supply;

$Q_{def,i,t}$: under-achievement of downstream water supply at i in time period t ;

$Q_{sur,i,t}$: over-achievement of downstream water supply at i in time period t ;

$Q_{gw,i,t}$: pumping water rate at well i in time period t ;

$Q_{sw,i,t}$: surface-water withdrawal rate at i in time period t ;

$Q_{gw,l,t}$: pumping water rate at well l in time period t ($l = i$);

$Q_{sw,m,t}$: surface-water withdrawal rate at m in time period t ($m = i$);

D_t : diversion water of the reservoir in time period t ;

SP_t : spill of the reservoir in time period t ;

$Q_{sd,j,t}$: stream depletion at constraint site j in time period t ;

$Q_{lf,j,t}$: lateral flow at constraint site j , in time period t ;

$Q_{d,i,t}$: required water demand rates at well site i in time period t ;

R_t : the reservoir release in time period t ;

I_t : inflow in time period t ;
 $(R_t)_{\min}, (R_t)_{\max}$: minimum and maximum reservoir release in time period t ;
 $r_{j,i,k}$: streamflow-depletion response coefficients at constraint site j ,
well i , and each periods k ;
 $Q_{ret-m_{j,t}}$: return flow of municipal water use at constraint site j , time t ;
 $Q_{ret-ag_{j,t}}$: return flow of agricultural water use at constraint site j , time t ;
 rm_t : lagged time of return flow of municipal water use in time t ;
 ra_t : lagged time of return flow of agricultural water use in time t ;
 S_t : reservoir storage in time period t ;
 S_{t+1} : reservoir storage in time period $(t+1)$;
 $(S_t)_{\min}, (S_t)_{\max}$: minimum and maximum reservoir storage in time period t ;
 $(SP_t)_{\min}, (SP_t)_{\max}$: minimum and maximum reservoir spill in time period t ;
 $(Q_{gw_{i,t}})_{\min}, (Q_{gw_{i,t}})_{\max}$: minimum and maximum groundwater pumping
rates at each well i in time period t .

5.1.2 Time Period of the System

The determination of potential yields under different system-specific restrictions regarding pumping, drawdown, and streamflow depletion is an important application of ground-water optimization models (Gorelick, 1983). As discussed by Atwood and

Gorelick (1985), the global optimization strategy solves a management problem over the entire management horizon and accounts for constraints at each time period. Sequential formulations can consider constraints that are in effect during each of the independent planning periods.

Global, transient formulations are used in this stream-aquifer-reservoir system because they allow for greater design flexibility over the management horizon by accounting for interactions over many planning periods. They also provide insights into how dynamic equilibrium is established in the groundwater system. When potential pumping patterns in each specific period have been established, the stream-aquifer – reservoir system is in a condition of dynamic equilibrium during the specific period.

One of the important steps in evaluating the potential yields for the water supply, and the interactions between the streamflow and the aquifer in a particular basin is to select a period of analysis which represents the hydrologic conditions of the basin. The minimum time period used for the stream-aquifer system is 10 days in this study. This minimum period is selected to ensure unit periods of adequate agricultural water supply and to obtain relatively straightforward cyclic pumping schedules for water users. The water supply plan of Korea is changed every 10 days and this plan is set based on the agricultural water use. On the other hand, municipal and industrial water supply plans are changed on a monthly basis.

5.1.3 Response Coefficients

The optimization method used to solve the conjunctive water management is based on a technique which uses response coefficients to represent the impact of

groundwater pumping. The basic assumption is that the streamflow depletion rate at a specified site can be described as a linear function of the rates of groundwater withdrawal at each well impacting that site. By assuming linearity, it is possible to determine the total streamflow depletion at a constraint site by summation of the individual stream depletions represented for each well. Linearity of the system makes it possible to isolate the response of each state variable such as head or streamflow to an individual stress. The individual responses have been referred to the unit solutions or the response coefficients (Gorelick and others, 1993).

A unit solution can be written in a differential form in which the state variable changes constantly with the stress. In the case of a streamflow depletion at constraint location j due to a unit pumping rate at a well i , a unit response coefficient can be written as $(\partial Q_{sd})_{j,tp} / (\partial Q_{gw})_i$. The notation is used as $r_{j,i,t}$ is the streamflow-depletion response coefficients at constraint site j , well i , and time period t . The response coefficient is dimensionless and ranges from zero to one. For the assumption of linearity to be valid, the values of the response coefficients for each well must remain constant for all simulated withdrawal and hydrologic conditions. The single-well conjunctive water model uses an independent response coefficient based on distance between the well and the stream. The multiple-well conjunctive water model uses multiple response coefficients for wells by varying distance between the well and the stream. Because of the assumed linearity of the system, the total streamflow depletion $(Q_{sd,tp})$ at a constraint site j for each specified period, tp , can be calculated with the response coefficients by the summation of all changes in streamflow caused by pumping at each individual well, which depend on each unit response coefficient and the pumping rate at each well. The

length of the specified period varies according to the time step. For a one year period, the 10-day total streamflow depletion for 36 time steps is calculated as shown in Equation 5.13.

The two-part definition of t' is required as a consequence of the annual cycle of withdrawals. Streamflow depletion in the current period can be affected by pumping in previous periods. The response coefficients represent the stream depletion changes with the state variable at a particular point for some specified time step such as 10-day, one month, or one year.

Response coefficients for stream depletion can be calculated from analytical solutions described by Jenkins (1968a, b), or from numerical ground-water flow models such as MODFLOW. In this research, response coefficients are evaluated from both an analytical method and the finite-difference numerical model MODFLOW. The response coefficients of single- and multiple-wells are calculated based upon the distance between the well and the stream.

The response coefficient is incorporated into the management optimization formulation through the constraint set. Each streamflow depletion constraint is rewritten to replace $Q_{sd_{j,sp}}$ by $r_{j,i,t}Q_{gw_{i,t'}}$, as shown in the Equation 5.13. The resulting modified formulation for systems with withdrawal wells follows:

$$Q_{sd_{j,sp}} = \sum_{i=1}^{nw} \sum_{t=1}^{np=36} r_{j,i,t} Q_{gw_{i,t'}} \quad (5.13)$$

$$t' = tp - t + 1, \text{ for } tp - t + 1 > 0$$

$$t' = 36 + (tp - t + 1), \text{ for } tp - t + 1 \leq 0$$

where

tp : specified period;

$r_{j,i,t}$: streamflow-depletion response coefficients at constraint site j ,
well i , and each periods t ;

$Q_{sd_{j,tp}}$: streamflow at site j in specified period tp ;

$Q_{gw_{i,t'}}$: rate of withdrawal at well i , in time t' .

5.2 Stream-Aquifer System

The stream-aquifer system accounts for the potential yield of conjunctive water withdrawn only from the stream and the ground water zone. The analysis is necessary to calculate pure stream and groundwater shortage volume without considering reservoir operation and to evaluate water supply and deficits for single- and multiple-wells for transient conditions considering distance from the stream.

5.2.1 Model Formulation

The objective function minimizes both the shortage of instream requirements and all demands and the excess or oversupply to these demands. The intent is to provide only that amount of water to actually meet the demand. In fact, deviations from the goals of this objective function, Equation 5.14, are not the same if the goal is overachieved or

underachieved. If the downstream flow is greater than the instream flow requirement, there is no problem with overachieving the instream flow goal. Therefore the problem is formulated as a goal programming problem where each deviation from the goal could be weighted separately. The solution to this problem is a weighted compromise solution according to the decision maker's preference (Mays and Tung, 1996).

The constraint of Equation 5.15 represents streamflow considering inflow, withdrawal of surface water and groundwater, and return flow of water demands at a control site for a specific time period. The goal formulation for instream flow target and downstream water demand is represented into Equations 5.16 and 5.17, respectively. Equation 5.19 constrains minimum and maximum groundwater pumping rates at each well.

The groundwater supply described in Equation 5.15 is replaced by a streamflow-depletion constraint, Equation 5.18, which states that streamflow depletion at site j in stress period t , $(Q_{sd_{j,tp}})$, is related with groundwater withdrawal rates $Q_{gw_{i,tp}}$. The constraints to be changed between the pumping rate and streamflow depletion are described in the Section 5.1.3. The link between these management formulations and the response of the water resources system to specific withdrawal schedules is handled through the use of response coefficients for each well. These response coefficients were developed from the groundwater flow simulation models.

$$\min \sum_{t=1}^{np} \left\{ C_r(Q_{sm_{j,t}}) + \sum_{i=1}^{mw} C_i(Q_{def_{i,t}} + Q_{sw_{i,t}}) + \sum_{l=1}^{mw} C_l Q_{gw_{l,t}} + \sum_{m=1}^{mw} C_m Q_{sw_{m,t}} \right\} \quad (5.14)$$

subject to

$$Q_{s_{j,t}} = I_t - \sum_{i=1}^{nw} Q_{gw_{i,t}} - \sum_{i=1}^{nw} Q_{sw_{i,t}} + Q_{ret-m_{j,t}} + Q_{ret-ag_{j,t}}, \quad (5.15)$$

$$j = 1, 2, \dots, ns, i = 1, 2, \dots, nw, t = 1, 2, \dots, np$$

$$Q_{s_{j,t}} - (Q_{s_{j,t}})_{\min} + Q_{sm_{j,t}} - Q_{s,sw_{j,t}} = 0 ; j = 1, 2, \dots, ns, t = 1, 2, \dots, np \quad (5.16)$$

$$Q_{sm_{j,t}} \geq 0, \quad Q_{s,sw_{j,t}} \geq 0$$

$$Q_{d_{i,t}} - Q_{gw_{i,t}} - Q_{sw_{i,t}} + Q_{def_{i,t}} - Q_{sw_{i,t}} = 0 ; i = 1, 2, \dots, nw, t = 1, 2, \dots, np \quad (5.17)$$

$$Q_{def_{i,t}} \geq 0, \quad Q_{sw_{i,t}} \geq 0$$

$$Q_{s_{j,t}} = I_t - \sum_{i=1}^{nw} \sum_{t=1}^{36} r_{j,i,t} Q_{gw_{i,t}} - \sum_{i=1}^{nw} Q_{sw_{i,t}} + Q_{ret-m_{j,t}} + Q_{ret-ag_{j,t}} \quad (5.18)$$

$$j = 1, 2, \dots, ns, i = 1, 2, \dots, nw, t = 1, 2, \dots, np$$

$$(Q_{gw_{i,t}})_{\min} \leq (Q_{gw_{i,t}}) \leq (Q_{gw_{i,t}})_{\max} \quad \forall i = 1, 2, \dots, nw, \quad \forall t = 1, 2, \dots, np \quad (5.19)$$

where nw : total number of wells;

np : total planning periods;

ns : total number of streamflow constraint sites for control points;

C_r : weighted value for instream requirement deficit;

C_i : weighted value for water supply deficit;

C_l : cost weighted value for groundwater supply;

C_m : cost weighted value for surface water supply;

I_t : inflow in time period t ;

$Q_{s,j,t}$: streamflow at site j in time period t ;

$(Q_{s,j,t})_{\min}$: instream flow requirement at j in time period t ;

$Q_{d,i,t}$: required water demand rate at well site i in time period t ;

$Q_{ret-m,j,t}$: return flow of municipal water use at control site j , time t ;

$Q_{ret-ag,j,t}$: return flow of agricultural water use at control site j , time t ;

$(Q_{gw,i,t})_{\min}$, $(Q_{gw,i,t})_{\max}$: minimum and maximum groundwater pumping rates at each well i in time period t ;

$Q_{gw,i,t}$: groundwater withdrawal rate at well i in time period t ;

$Q_{sw,i,t}$: surface water withdrawal rate at i in time period t ;

$Q_{sm,j,t}$: shortage instream flow at constraint site j in time period t ;

$Q_{s,sw,j,t}$: excess instream flow at constraint site j in time period t ;

$Q_{def,i,t}$: under-achievement of water supply at i in time period t ;

$Q_{sur,i,t}$: over-achievement of water supply at i in time period t ;

CHAPTER 6

OPTIMAL CONJUNCTIVE MODEL APPLICATION

6.1 Purpose of the Case Study

The Geum River provides municipal and industrial water supply, irrigation, hydropower energy, and recreation water in central regions of South Korea. There are two major multi-purpose reservoirs: Daecheong and Yongdam reservoir constructed in 1981 and 2001 in the Geum River basin.

After the Yongdam reservoir was completed, downstream state agencies, senior water right users, and non-government organizations were concerned with how the operation of Yongdam reservoir would affect the operation of Daecheong reservoir and impact on water use and instream flow downstream of Yongdam Dam. Unfortunately, the operation of Yongdam reservoir has produced some negative impacts for water supply and instream flow in the area between the Yongdam Dam and the Daecheong Dam. The limited release of the Yongdam reservoir has an impact on the downstream water supply during the low flow season. Therefore conflicts related to enough water supplies and releases occur when the available water is not sufficient to meet all demands. It is very

important to evaluate the available water resource of the stream-aquifer-reservoir system using groundwater and surface water including reservoir storage.

The conjunctive water management system evaluates reservoir operation and groundwater pumping operations using single-wells and multiple-wells. The reservoir operation based upon conjunctive water use is assessed under the condition of the various water demands and multiple-well withdrawals. An appropriate operational guide curve for Yongdam reservoir operation is proposed from the historical record and evaluated by a simulation approach. For the case studies, the descriptions of the case study area described in Chapter 4 are used.

6.2 Stream-Aquifer Characteristics

The evaluation of groundwater capacity and stream-aquifer interaction in the case study, the numerical groundwater flow model and recharge analysis of a stream-aquifer system are developed in the following sections.

6.2.1 Recharge Analysis

Groundwater recharge from precipitation was estimated for the case study area. Annual and monthly precipitation recharge and baseflow rates are estimated by the RORA, RECESS and PART programs with daily streamflow data measured at the

Daecheong reservoir. The hydrologic characteristics of the daily streamflow measured in the case study area are shown in Table 6.1. The precipitation rates at each rain gaging station were collected by the Korea Water Resources Corporation and the Korea Meteorological Administration for areas upstream of the Daecheong reservoir. Streamflow data are collected using the inflows to Daecheong reservoir. Surface evaporation and the direct runoff components between the precipitation and the streamflow were estimated. Evapo-transpiration was estimated from the groundwater recharge and baseflow.

Table 6.1 Hydrologic characteristics of the case study area

(unit:mm/yr)

Site	Precipitation	Evaporation	Stream flow	Ground water recharge	Baseflow	Direct Runoff	Evapo-transpiration
Daecheong Reservoir Basin	1,203.0	559.0	644.0	231.9	205.6	438.4	26.3

Figure 6.1 indicates a period of low and high recharge by resulting from the 24-year period of monthly recharge rates throughout this area. The maximum estimated annual recharge rate occurred in 1998 and the minimum estimated annual recharge rate in 1994. The average annual recharge rate is 231.9mm per year and it was used to compute groundwater storage in the stream-aquifer simulations.

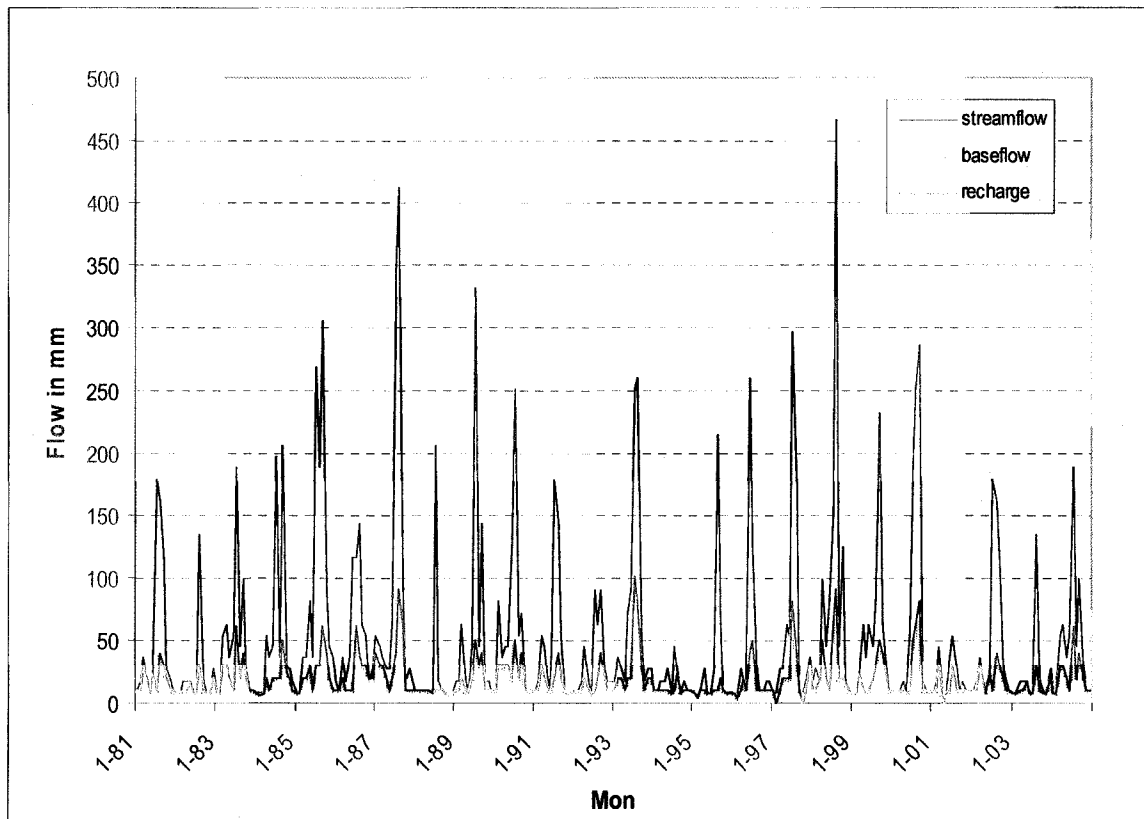


Figure 6.1 Monthly groundwater recharge

Average monthly recharge rates from 1981 to 2004 range between a minimum of 10mm for January and a maximum of 37mm for July from Figure 6.2. Figure 6.2 shows the high recharge rates in summer during the flood season and low recharge rates in winter during the drought season.

The 10-day period recharge rate used for available groundwater was calculated by the monthly recharge rates and the ratio of 10-day and monthly cumulative rainfall obtained in the case study area.

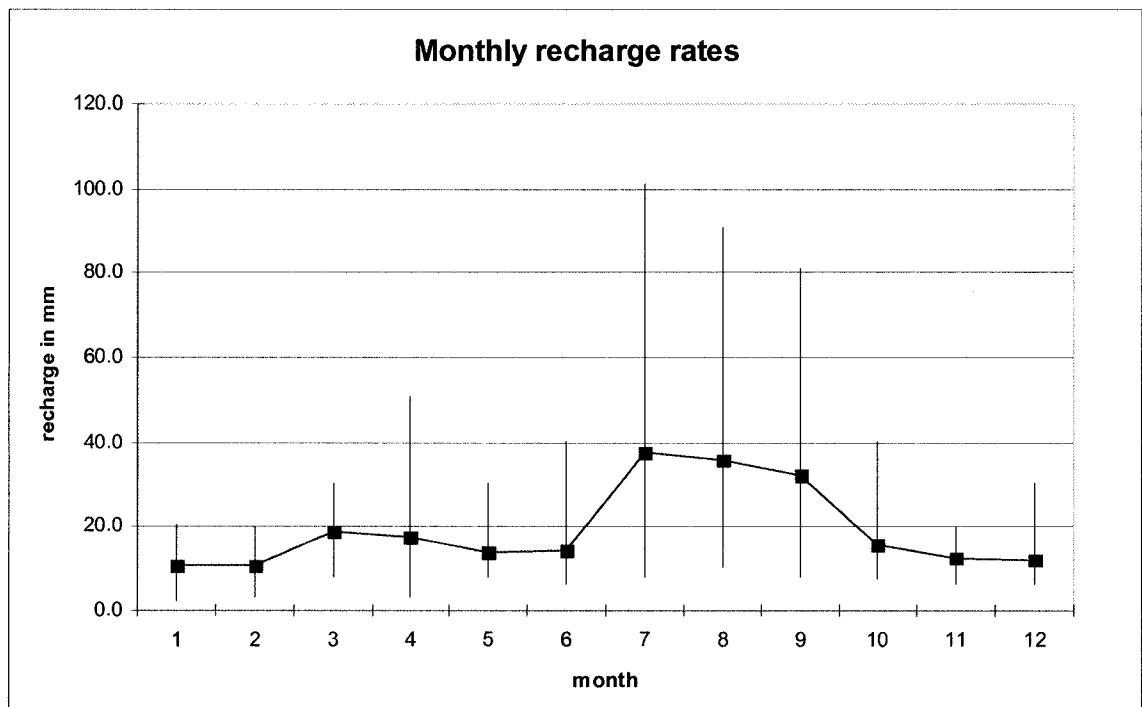


Figure 6.2 Average monthly recharge rate

6.2.2 Stream-Aquifer Interaction

The interaction between the stream and the aquifer under steady-state conditions for single wells at distances of 50m, 100m, 200m, and 500m from the stream are discussed in Section 4.4.2. This section evaluates the induced infiltration of streamflow depletion due to all of the wells pumping at the hypothetical well sites. It is necessary to compare streamflow induced rates calculated for each simulated stream cell from the MODFLOW model with conditions of no withdrawals at the simulated wells. Therefore no withdrawals from the wells are used as a first step and then withdrawals from all of the wells are used as a second step for calculating streamflow depletion due to pumping. Only two lines of wells at a distance of 50m and 200m from the stream are considered in

the case study area because the municipal water pumping wells are located close to the stream and the agricultural water pumping wells are located away from the stream.

Flow boundaries on the northern and southern ends of the model (described in Section 4.4) were treated as specified-flux boundaries for the management evaluations. This avoids the need to modify heads along these boundaries in each stress period for the transient simulations. There are total of 18 sites of agricultural lands with a total area of 13.6km² in the study area. The average area of agricultural lands is approximately 0.8 km² in Table 6.2. A sample site size for the stream-aquifer response study is assumed to 2.4km² including uplands. The total number of wells in a sample site is estimated to be 20 wells by considering wells of agricultural lands in the vicinity of the Yeosu and Icheon. These were investigated by the Groundwater survey report of KOWACO in 2004. The response coefficients of induced infiltration for this site obtained from the MODFLOW model are given in Table 6.3 and shown in Figure 6.3. Figure 6.3 shows that the response coefficients decrease with time and that their largest value is 0.112. These response coefficients were calculated based on spatially distributed aquifer characteristics and complex boundary conditions.

Table 6.2 Average well-pumping site area and well numbers

Items	Area or well number
Stream-aquifer analysis area	
- Total agricultural lands	13,605,000 m ² / 18 sites
- Average land	755,800 m ² /site
Wells	
Wells numbers applied in study area	20 wells
- Yeosu region	18 wells/km ²
- Icheon region	22 wells/km ²

Table 6.3 Response coefficients of induced infiltration by pumping

10days	1	2	3	4	5	6	7	8	9	10	11	12
coeff.	0.112	0.092	0.063	0.063	0.063	0.060	0.055	0.049	0.044	0.040	0.036	0.033
10days	13	14	15	16	17	18	19	20	21	22	23	24
coeff.	0.030	0.028	0.026	0.024	0.022	0.020	0.018	0.017	0.015	0.014	0.012	0.011
10days	25	26	27	28	29	30	31	32	33	34	35	36
coeff.	0.010	0.009	0.008	0.007	0.006	0.004	0.003	0.003	0.002	0.000	0.000	0.000

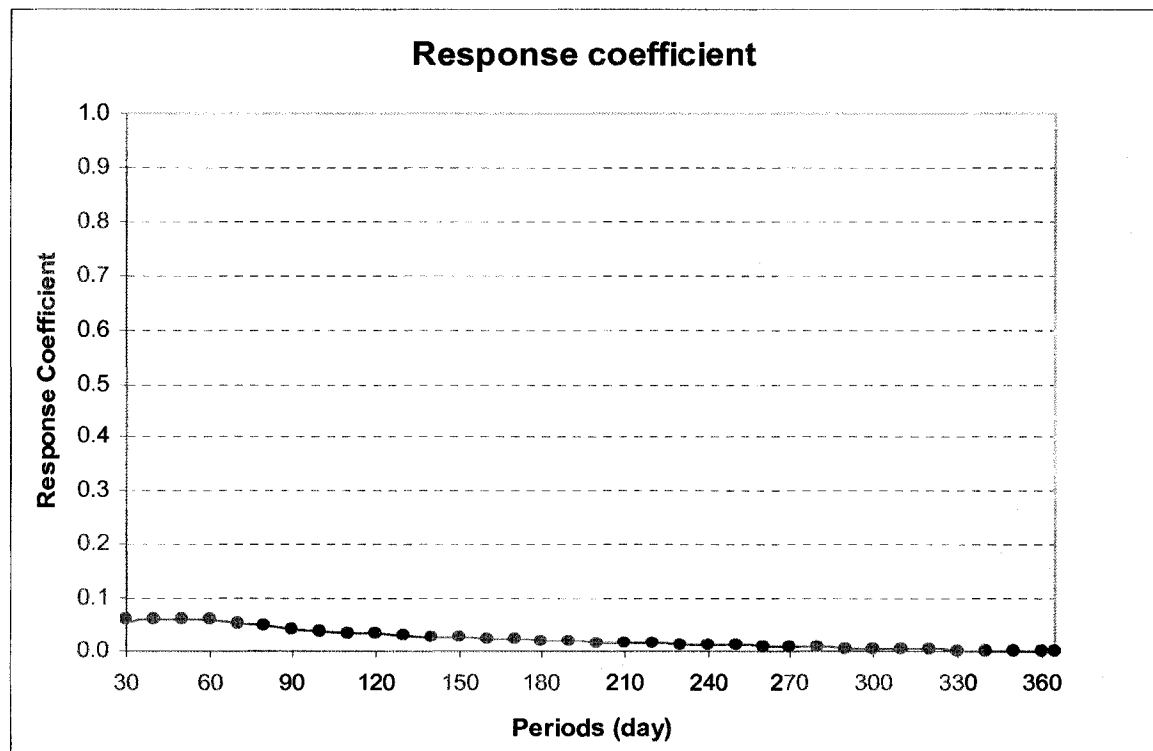


Figure 6.3 Response coefficients for the site

6.2.3 Cost Factors of Groundwater and Surface Water

The groundwater pumping cost and surface water withdrawal cost should be applied in a conjunctive management model as cost factors. The cost for municipal and agricultural pumping is a function of pumping quantity, pumping power, groundwater head, and unit price of power rate. The equations for estimating 10-day water table variations were obtained for each agricultural zone based upon a linear regression analysis. In this study, the frequency analysis of the groundwater simulation model and the principle of superposition are considered to develop the regression equation. For calculating the pumping power, groundwater table depth due to pumping rates is needed. The regression relationship between pumping rates and water table variations can be easily developed by the groundwater simulation model using the MATHCAD software described in Section 4.7. An iterative process of changing groundwater pumping rates and then evaluating the drawdown of the groundwater table produce the data to develop the regression relationship. Base power rates and additional power rates due to pumping withdrawal and the associated drawdown were calculated by the model. Also the power rate and municipal and agricultural water rates are assessed using the price list from two Government Corporations, KOWACO and KEPCO (Korea Electric Power Corporation). The unit price and withdrawal cost of groundwater and surface water are shown in Table 6.4 and Table 6.5, respectively. The cost ratio of surface water and groundwater withdrawal for municipal purposes is shown in Figure 6.4. Figure 6.4 shows the cost of groundwater withdrawal for municipal purposes is linearly increased with the cost of

surface withdrawal for municipal purposes. The cost ratio of surface water and groundwater withdrawal for agricultural purposes is shown in Figure 6.5. Figure 6.5 shows the cost of groundwater withdrawal for agricultural purposes linearly increases with the cost of surface withdrawal for agricultural purposes.

Table 6.4 Unit price of groundwater and surface water

Item	Groundwater pumping power rate		Surface water rate
	Base (\$)	Base+Usage(\$)	(\$)
Municipal water	\$5.7/KW	\$0.07/KWh	\$0.149/m ³
Agricultural water	\$1.1/KW	\$0.038/KWh	\$0.048/m ³

Table 6.5 Withdrawal cost of groundwater and surface water

No	Municipal water			Agricultural water		
	Withdrawal	Groundwater	Surface	Withdrawal	Groundwater	Surface
	10 ⁶ m ³	\$1000	\$1000	10 ⁶ m ³	\$1000	\$1000
1	0.050	16.58	7.45	0.050	2.45	2.40
2	0.100	33.60	14.90	0.100	4.90	4.79
3	0.150	51.11	22.35	0.200	9.81	9.59
4	0.200	69.91	29.80	0.300	14.73	14.38
5	0.300	107.61	44.70	0.500	24.57	23.97
6	0.400	147.14	59.60	0.700	34.43	33.55
7	0.500	188.50	74.50	1.000	49.25	47.93
8	0.700	270.08	104.30	2.000	98.97	95.86
9	0.800	315.98	119.20	3.000	149.17	143.79
10	0.900	363.71	134.10	4.000	199.83	191.72
11	1.100	464.66	163.90	5.631	283.95	269.89

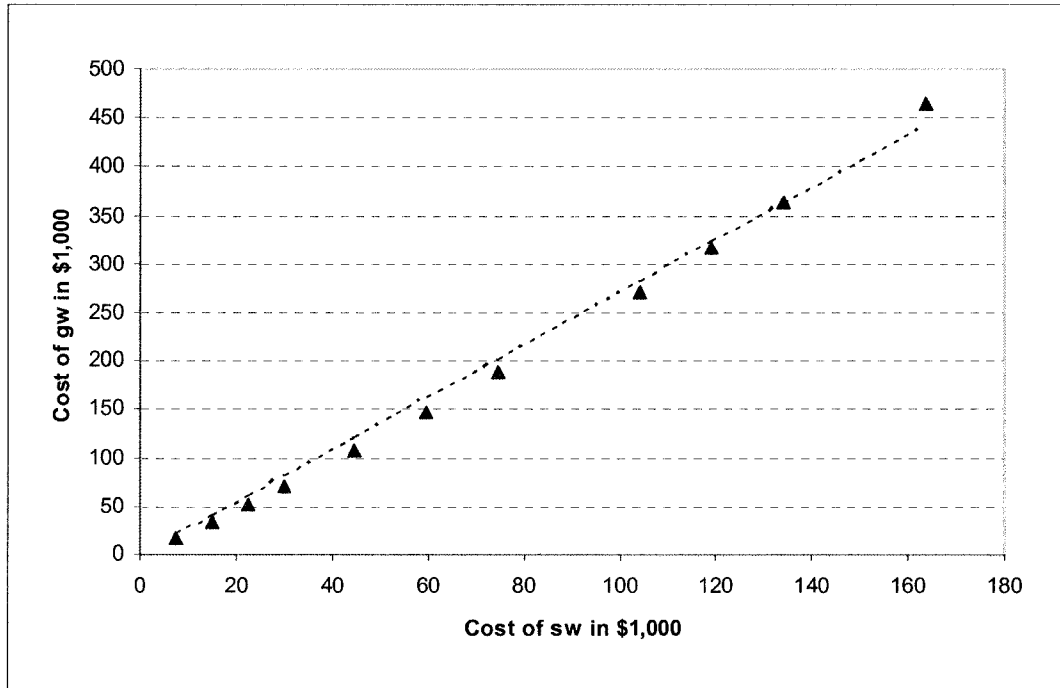


Figure 6.4 Cost ratio of surface- and groundwater withdrawal for municipal purposes

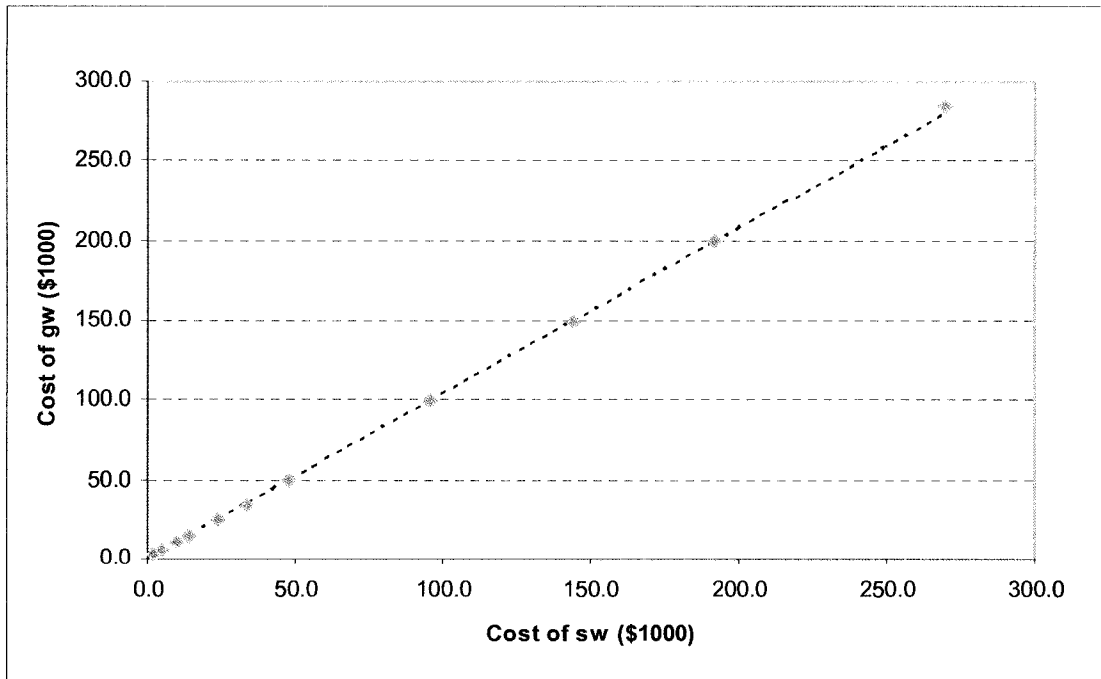


Figure 6.5 Cost ratio of surface- and groundwater withdrawal for agricultural purposes

6.3 Stream-Aquifer Systems

6.3.1 Single Withdrawal Well

This section is for evaluation of water supply and deficits for a 10-day time period using a single withdrawal well for the transient model of the stream-aquifer system without considering the reservoir system. Wells are placed in a fully penetrated condition in the aquifer zone at distances of 50m, 100m, 200m, and 500m from the stream. The model accounts for water supply and deficits for each distance. In the upper region of the Geum River basin, most of the wells are located in the alluvial zones within 500m distance from the stream. Wells for the municipal water withdrawal are located close to the stream and wells for agricultural water withdrawal are located away from the stream.

The system uses the historical 10-day streamflow data for 24 years of record at Okcheon (Site 3) located downstream of Yongdam Reservoir. Three rivers, Geum River, Namdaecheon, and Bonghwangcheon are flowing into this site area. Groundwater recharge is assumed to be the 10-day recharge rates based on the agricultural area at the Okcheon site. Total water demand at the Okcheon site considers the sum of municipal and agricultural water for Muju and Okcheon in Section 4.2.2 and instream water requirement for the Okcheon site given in Table 6.6. Return flows are assumed to return to the river at the next downstream location in the modeled system. The groundwater infiltration coefficient at the demand site is 0.3 with lag coefficients of 0.7, 0.2, and 0.1 by 10-day period. The 10-day instream flow of $2.25\text{m}^3/\text{sec}$ was used for the Yongdam reservoir site and $7.79\text{m}^3/\text{sec}$ was used for the Okcheon site.

Table 6.6 10-day instream flow for Okcheon in site 3

(Unit: 10^6m^3)

10-day	1	2	3	4	5	6	7	8	9	10	11	12
min. flow	6.73	6.73	7.40	6.73	6.73	5.38	6.73	6.73	7.40	6.73	6.73	6.73
10-day	13	14	15	16	17	18	19	20	21	22	23	24
min. flow	6.73	6.73	7.40	6.73	6.73	6.73	6.73	6.73	7.40	6.73	6.73	7.40
10-day	25	26	27	28	29	30	31	32	33	34	35	36
min. flow	6.73	6.73	6.73	6.73	6.73	7.40	6.73	6.73	6.73	6.73	6.73	7.40

Barlow and others (2003) evaluated a set of four formulations which consisted of one 12-month period, two 6-month periods, four 3-month periods, and twelve 1-month time periods for their conjunctive management model considering only well pumping. They suggested that substantial increases in annual withdrawal were possible by increasing the number of time periods each year because flexible pumping patterns produced more water than a fixed pumping pattern. The total withdrawal rate for a single well was first determined using the stream-aquifer system. Changing the weighting factors among instream flow deficit, downstream water supply deficit, surface water supply, and groundwater supply resulted in different total water deficits.

The total water deficits from the system increase for the well closest to the stream and decrease for the well furthest from the stream. When the weighting factor for the

downstream water supply is bigger than that for the instream flow, the total water deficit is reduced by up to 46 MCM at distance of 500m from the stream. Compared to water deficits considering only surface water supply, water deficits from conjunctive use were reduced to 64 MCM when the furthest well from the stream was used. If the weighting value or cost for the groundwater withdrawal is small, the groundwater pumping withdraws up to the maximum capacity of the groundwater. These results are illustrated in terms of the water deficits from wells located 50m, 100m, 200m, and 500m from the stream in Table 6.7 and Figure 6.6 and Figure 6.7 as a function of different weighting factors for downstream water supply and instream flow.

Water deficits for the condition that the downstream water supply weighting factor is larger than the weighting factor for instream flow are reduced. On the contrary, water deficits for the condition that the downstream water supply weighting factor is smaller than the weighting factor for instream flow are increased. Therefore groundwater supply using the well furthest from the stream combined with the surface water supply alleviates water deficits better as compared to using the well closest to the stream.

Table 6.7 Total withdrawals from the stream-aquifer system for a single well

(Unit: 10^6m^3)

distance (m)	Weighting Factor				water deficit			water supply	min. flow
	w.supply	min.flow	SW	GW	w.supply	min.flow	sum		
50	100	1	1	2	-4.7	-556.3	-561.0	754.6	5344.8
				1	-4.7	-543.6	-548.3	754.6	5352.9
	1	100	1	2	-159.6	-467.9	-627.5	599.8	5428.6
				1	-168.8	-467.9	-636.7	590.6	5428.6
100	100	1	1	2	-3.0	-568.3	-571.3	756.3	5328.2
				1	-3.0	-522.5	-525.5	756.3	5373.9
	1	100	1	2	-158.3	-467.9	-626.2	601.1	5428.5
				1	-158.3	-467.9	-626.2	601.1	5428.5
200	100	1	1	2	-0.3	-568.3	-568.6	758.1	5378.2
				1	-0.3	-518.3	-518.6	759.1	5378.2
	1	100	1	2	-130.3	-466.0	-596.3	629.0	5430.4
				1	-131.0	-466.0	-597.0	628.3	5430.4
500	100	1	1	2	0.0	-569.0	-569.0	759.4	5327.4
				1	0.0	-513.4	-513.4	759.4	5383.0
	1	100	1	2	-97.2	-462.3	-559.5	662.2	5434.1
				1	-97.2	-462.3	-559.5	662.2	5434.1
surface	100	1	1		-9.1	-568.3	-577.4	750.3	5328.2
water	1	100	1		-145.5	-469.9	-615.4	613.9	5426.5

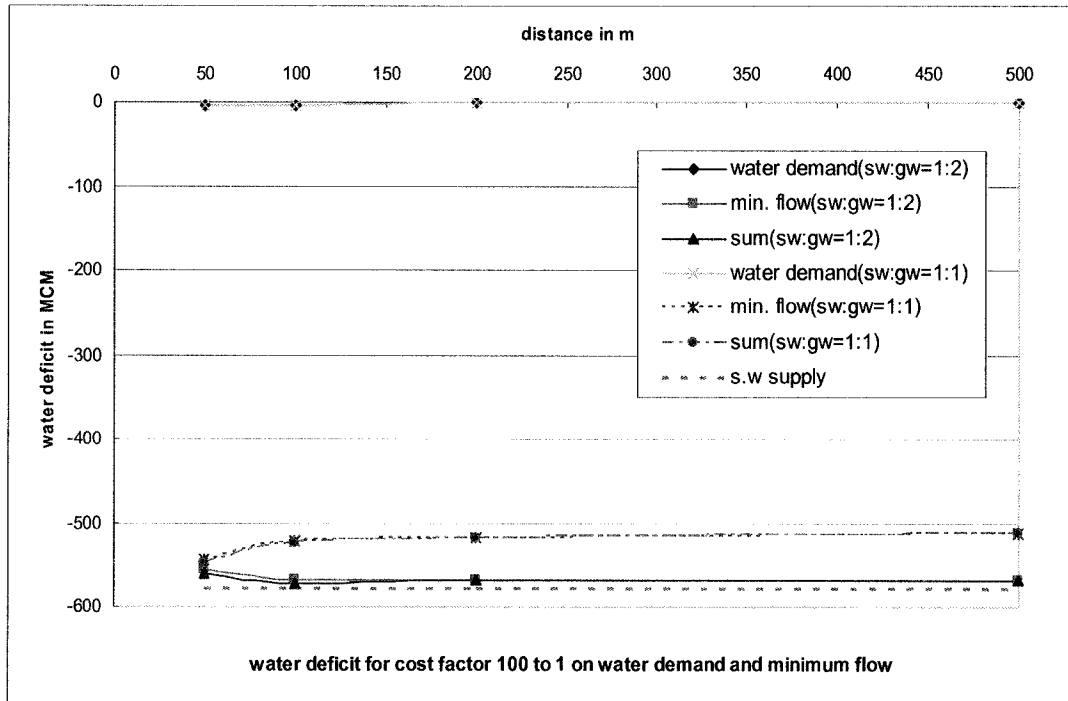


Figure 6.6 Water deficits for cost factor 100 to 1 on water demand and minimum flow

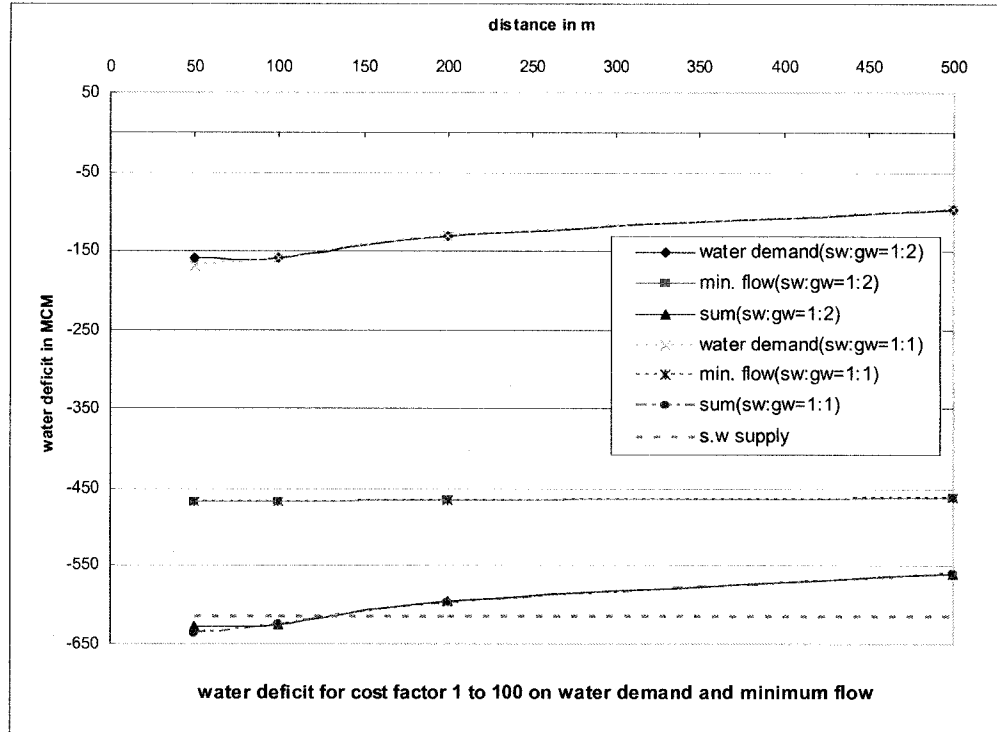


Figure 6.7 Water deficits for cost factor 1 to 100 on water demand and minimum flow

6.3.2 Multiple Withdrawal Wells

The multiple-well formulation was used to determine the response of multiple-well withdrawal patterns at varying distances from the stream. This formulation was also used to assess the impact on withdrawal as weighting values were changed. Water supply systems using multiple wells have the flexibility to alter pumping patterns to meet downstream water supply and instream flow targets. In this formulation, all four wells and surface water withdrawals were included in the analysis, which increased the number of decision variables and streamflow depletion factors in each constraint. The objective function of the model was to minimize total water shortage for the entire operational time periods. The solution was limited by a set of constraints on water demands, instream flows, recharge, and response coefficients for 10-day time periods.

The groundwater supply amounts from the wells located at 50m and 100m from the stream are 47.9 MCM for both sets of weighting factors shown in Table 6.8. Comparisons of the groundwater supply amounts from the each well due to weighting values and distance from the stream are plotted in Figure 6.8. The Figure 6.8 and Table 6.8 show that the groundwater withdrawals of the well located at 500m from the stream are used to provide more water than the wells closer to the stream. This is likely due to the fact that groundwater withdrawals at the well located 500m from the stream have constant response coefficients throughout the year and the weighting factors of surface and groundwater withdrawal were the same. This is an indication that the farther wells should be used when the impact on the stream must be kept small. These simulations

indicate that when a constant annual pumping rate is needed during a drought period, a combination between the closest well and the furthest well from the stream should be used.

Table 6.8 Groundwater supply for various weighting factors

Weighting Factor				Groundwater supply (MCM)			
w.supply	min. flow	SW	GW	50	100	200	500
100	1	1	2	47.9	47.9	0	3.3
			1	47.9	47.9	55.1	77.9
1	100	1	2	47.9	47.9	6.1	57.8
			1	47.9	47.9	7.4	56.7

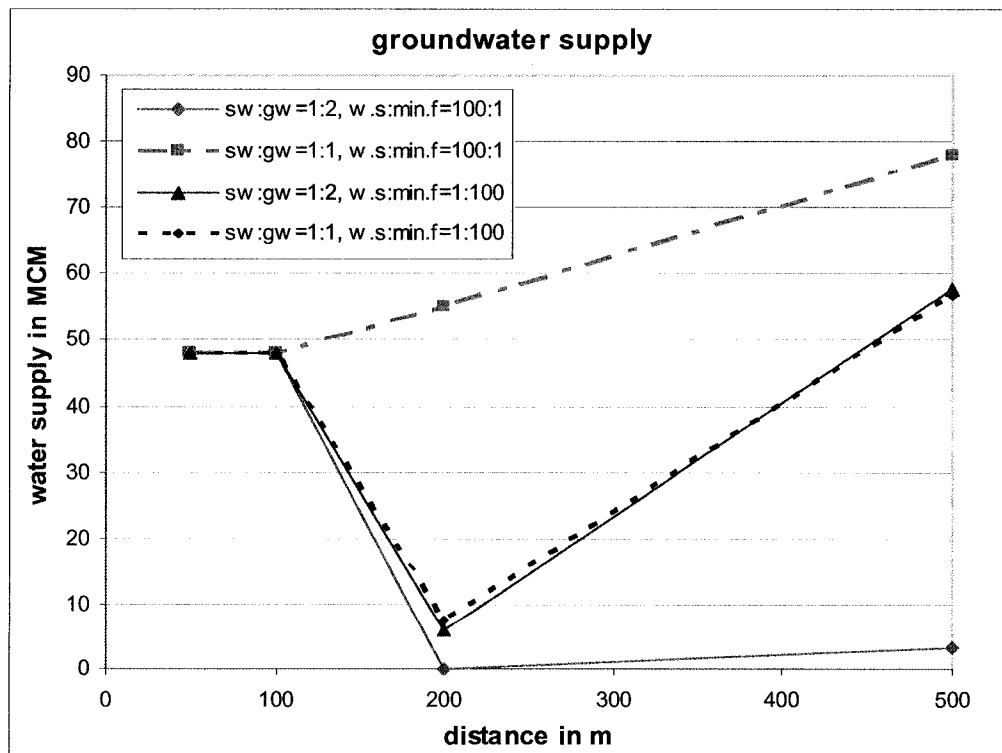


Figure 6.8 Groundwater supply due to weighting values

Weighting factors for ground water and surface water supplies are considered for evaluating water deficits in each case and are shown in Table 6.9. The water deficit of minimum flow increases with large weighting factors for water supply and the water deficit of water supply decreases with large weighting factors for water supply. However, when the groundwater weighting factor is with the same as that for surface water, the water deficit in Geum River basin is decreased up to 54MCM.

Table 6.9 Water deficits and supplies for various weighting factors

Weighting Factor				Water deficit		Water supply	
w.supply	min. flow	SW	GW	w.supply	min. flow	w.supply	min. flow
100	1	1	2	0	-529.3	759.4	5367.2
			1	0	-475.3	759.4	5421.1
1	100	1	2	-91.1	-461.8	668.2	5434.7
			1	-92.1	-461.8	667.3	5434.7

6.4 Stream-Aquifer-Reservoir Management System

The multi-objective optimization in this case study is based on the methodology and techniques represented in the previous Chapter 5. The conjunctive management model considering reservoir operation, surface water and groundwater withdrawals, and response coefficients of streamflow depletion due to groundwater pumping is developed using the generalized linear and nonlinear optimization software, LINGO, to obtain optimal reservoir release and groundwater withdrawal.

The model is formulated on a 10-day time step to determine reservoir releases and surface water and groundwater withdrawals subject to meeting a set of constraints, which include the reservoir capacity, the streamflow depletion by pumping in time, water

demands such as municipal and agricultural water, instream flows of the reservoir and the downstream site, groundwater pumping rates, interactions between the streamflow and the groundwater pumping, the return flow, the detention time of return flow. The model uses the data for the groundwater recharge, the inflow to and storage of the reservoir, downstream lateral flows, the estimated groundwater recharge, estimated response coefficients between the streamflow and the groundwater pumping, estimated return flow coefficients and the detention time of return flow, instream flows of the Yongdam Reservoir and the Okcheon site, and the demand and diversion for municipal and agricultural water uses. The multi-objective formulation is applied in the Geum River Basin case study considering:

- A) Minimization of shortages from downstream instream flow requirements
- B) Minimization of deficits for municipal and agricultural water supply
- C) Minimization of water deficits for the trans-basin diversion
- D) Minimization of the total withdrawal costs of groundwater and surface water
- E) Minimization of reservoir spill

Energy production is relatively minor and water supply has the highest priority in the case study. Also not considering energy makes the problem linear instead of non-linear. The cost of surface water and groundwater withdrawal was minimized to determine the cheapest way to meet the demands. This objective function helps to define groundwater pumping strategies possibilities during the drought season.

The LINGO software is used to solve the optimization problem. This problem has 56,884 decision variables and 33,573 constraints. It takes 5 minutes to solve the problem. On the contrary, solving the problem considering only the surface water supply

system without including groundwater supply has 54,432 decision variables and 22,465 constraints. Therefore the run time was only 1 minute and 30 seconds.

Weighting factors for conjunctive water and surface water supply considered for evaluating water deficits are shown in Table 6.10. Water deficit decreases up to 104 MCM due to changes in streamflow depletion due to groundwater pumping as shown in Table 6.11. By comparing the total amount of conjunctive water with the surface water alone, the amount of conjunctive water is 104 MCM greater than the surface water amount. Note that this allows the deficit to the trans-basin diversion to be reduced by that amount. The reduction in the deficit is about 15% if only the deficit amount (719.8 MCM) is considered. The total demand for the trans-basin diversion is 11,808 MCM for 24 years and the additional 104 MCM represents less than 1% of that value. In downstream area of reservoir, however, groundwater supply amounts to 18.5% of water supply, 759.4 MCM. Particularly when severe droughts with 5 to 30 year recurrence intervals occurred (1982, 1988, 1992, 1994, 1995, 2002) during the 24 year period as shown in Table 6.12, the deficit for the trans-basin diversion diminishes to 151.4 MCM for the case of conjunctive water during these drought periods. The additional supply amounts to 6.1 % of trans-basin diversion. This indicates that proper groundwater withdrawal can contribute to a more effective water allocation in the Geum River basin.

The important question to be answered is where does this reduction in deficits come from? Since aquifer storage is relatively small then essentially the same amount of water is available and in fact Table 6.11 shows that the water balance for the system is preserved. The time for the impact of well-pumping on the stream is the same order of magnitude as the time required for return flows.

The answer can be found by examining the distribution of water between instream flows and ending reservoir storage. Both the surface water only case and the conjunctive use case meet the minimum instream flow constraint. However, the conjunctive use operation provides less total flow for instream purposes, or in other words, less over-supply of the instream flow requirement. This results in higher ending values for reservoir storage. The conjunctive management strategy allows a better spatial and temporal distribution of water in the system.

Table 6.10 Weighting factors for conjunctive water and surface water supply

Supply	Min. instream flow	diversion	Water demand	spill	Surface water	Ground Water
Conjunctive water	100	1	100	0.001	1	1.2
Surface water	100	1	100	0.001	1	-

Table 6.11 Water supplies and deficits

(Unit:MCM)

Items	detailed	Water supply deficit	Conjunctive water	Surface water	Drought years	
					Conj.w	Surface w.
Ground water	sum	supply	140.82	-	92.2	-
	M&I muju	supply	12.53	-	7.1	-
	Agri. Muju	supply	11.63	-	8.2	-
	M&I Oc	supply	53.01	-	30.2	-
	Agri. Oc	supply	63.65	-	46.7	-
Surface water	sum	supply	618.5	759.4	97.6	189.8
		deficit	0.00	0.00	0.00	0.00
	M&I Muju	supply	44.40	56.90	7.1	14.2
		deficit	0.00	0.00	0.00	0.00
	Agri. Muju	supply	53.30	64.9	8.1	16.2
		deficit	0.00	0.00	0.00	0.00
	M&I Oc	supply	201.00	254.0	33.3	63.5
		deficit	0.00	0.00	0.00	0.00
	Agri. Oc	supply	319.80	383.5	49.1	95.9
		deficit	0.00	0.00	0.00	0.00

Trans-basin diversion	supply	11,192.2	11,088.2	2,653.3	2,501.9
	deficit	-615.8	-719.8	-298.7	-450.1
Total instream flow		24,547.4	24,968.3	3,910.3	3,863.5
Reservoir storage	beginning	510.0	510.0		
	ending	533.6	215.3		
Total mass balance		36,522.5	36,521.2		

Table 6.12 Annual drought analysis using Log Pearson type III in Yongdam reservoir
(Unit:MCM)

Historical annual inflow			Drought frequency analysis	
Order	Inflow(Oct.-Sep.)	Date	Return period (years)	Drought
1	204.3	1994	5	421.8
2	247.9	2002	10	316.6
3	344.9	1982	20	245.8
4	369.4	1995	30	214.1
5	421.8	1988	50	181.6
6	440.8	1992	80	157.6
Mean	747.9			
Max	1,515.3			
Min	204.3			

The comparisons of water supplies in each demand node are plotted in Figure 6.9. This figure shows that the agricultural demands of each site are larger than the municipal demands and that the amount of surface water supply is larger than the amount of ground water. Total water supplies between conjunctive water and surface water alone at the downstream site are similar, 759.32MCM and 759.4MCM respectively, and that of instream flow is the same. This shows that the water balance between conjunctive water and surface water is the same. According to transient groundwater withdrawal harmonized with surface water withdrawal due to conjunctive water use, it is possible to reduce the water deficits using conjunctive water management.

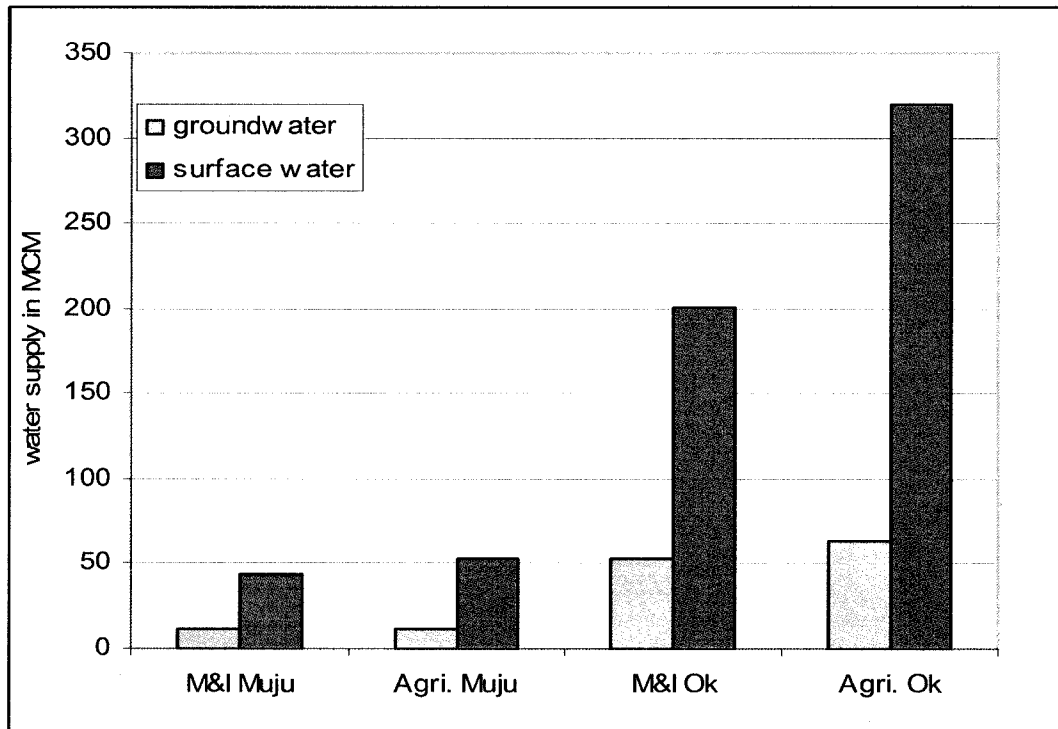


Figure 6.9 Water supplies from groundwater and surface water

The next step in the analysis was to find an effective storage rule for the reservoir that considered both groundwater and surface water withdrawal. Reservoir operators of KOWACO have used storage rule based guide curves. The optimal 10-day reservoir storage, reservoir release and groundwater withdrawal due to release and lateral flow policies were created based upon the results of the optimization model. The creation of these policies is presented in Section 6.6.

6.5 Sensitivity Analysis

In groundwater and surface water management, there are various uncertainties associated with weighting parameters such as the cost of groundwater and the surface water withdrawal, and priorities among the instream flow, the diversion water, and

downstream water demands. A sensitivity analysis was used to estimate the effects of parameter and input data uncertainties on optimal conjunctive management strategies. These uncertainties were estimated by performing sensitivity analysis according to the cost variation of surface water and groundwater withdrawal and the priorities of water demands. A sensitivity analysis was also performed for the weighting factors.

6.5.1 Cost Weighting Factor

Table 6.13 indicates that water shortages can be reduced if the groundwater withdrawal costs were lower than or equal to the cost of surface water withdrawals. The water supplies due to weighting factor variation are plotted in Figure 6.10. This figure also shows that surface water supplies are increased with weighting value and ground water supplies are decreased with weighting value. The conjunctive water use is affected by lagged streamflow depletion due to groundwater withdrawal. Therefore efficient surface water and groundwater withdrawal patterns are needed. Groundwater supply decreases from 188.79MCM to 10.08MCM as the relative cost weighting factor varies from 0.8 to 2.0. A maximum water deficit of 711.7MCM occurred when the groundwater withdrawal cost was two times higher than the cost of surface water withdrawal. The water deficit when only surface water is used was 719.8MCM. Conjunctive water use was able to reduce the water deficit. The performance measures of several alternative scenarios are shown in Table 6.14. Water deficit due to weighting factor variation is plotted in Figure 6.11. This figure shows that water deficit is decreased with weighting value.

Table 6.13 Sensitivity analysis by cost weighting factors

(unit:MCM)

Items	detailed	supply or cost weighting factor of groundwater (gw/sw)						Surface water
		content	0.8	1	1.2	1.5	2	
Ground water	sum	supply	188.79	188.49	140.82	127.82	10.08	-
	M&I muju	supply	16.42	16.36	12.53	11.31	1.14	-
	Agri. Muju	supply	16.36	16.36	11.63	10.83	2.77	-
	M&I OK	supply	78.13	77.88	53.01	48.48	3.96	-
	Agri. OK	supply	77.88	77.88	63.65	57.20	2.21	-
Surface water	sum	supply	570.60	570.90	618.5	631.50	749.30	759.4
		deficit	0.00	0.00	0.00	0.00	0.00	0.00
	M&I Muju	supply	40.50	40.60	44.40	45.60	55.80	56.90
		deficit	0.00	0.00	0.00	0.00	0.00	0.00
	Agri. Muju	supply	48.60	48.60	53.30	54.10	62.20	64.9
		deficit	0.00	0.00	0.00	0.00	0.00	0.00
	M&I OK	supply	175.90	176.20	201.00	205.60	250.10	254.0
		deficit	0.00	0.00	0.00	0.00	0.00	0.00
	Agri. OK	supply	305.60	305.60	319.80	326.30	381.20	383.5
		deficit	0.00	0.00	0.00	0.00	0.00	0.00
Trans-basin diversion	supply	11192.2	11192.2	11192.2	11186.6	11096.3	11088.2	
	deficit	-615.8	-615.8	-615.8	-621.4	-711.7	-719.8	
Min. instream flow	supply	5896.4	5896.4	5896.4	5896.4	5896.4	5896.4	
	deficit	0.0	0.0	0.0	0.0	0.0	0.00	
Sum of supply			17,847.69	17,847.99	17,847.9	17,842.3	17,752.0	17,747.0

Table 6.14 Performance measures by cost weighting factor

Items	performance measures	cost weighting factor of groundwater and surface water (gw/sw)					Surface water
		0.8	1	1.2	1.5	2	
diversion	Prel	0.94	0.93	0.94	0.94	0.94	0.93
	Pvul	0.91	0.90	0.91	0.94	0.90	0.90
	Pres	0.0055	0.0039	0.005	0.0044	0.0039	0.0039
min. instream flow	Prel	1.0	1.0	1.0	1.0	1.0	1.0
	Pvul	1.0	1.0	1.0	1.0	1.0	1.0
	Pres	1.0	1.0	1.0	1.0	1.0	1.0
sum	Prel	0.97	0.97	0.97	0.97	0.97	0.97
	Pvul	0.95	0.95	0.95	0.95	0.95	0.95
	Pres	0.503	0.502	0.503	0.502	0.502	0.502

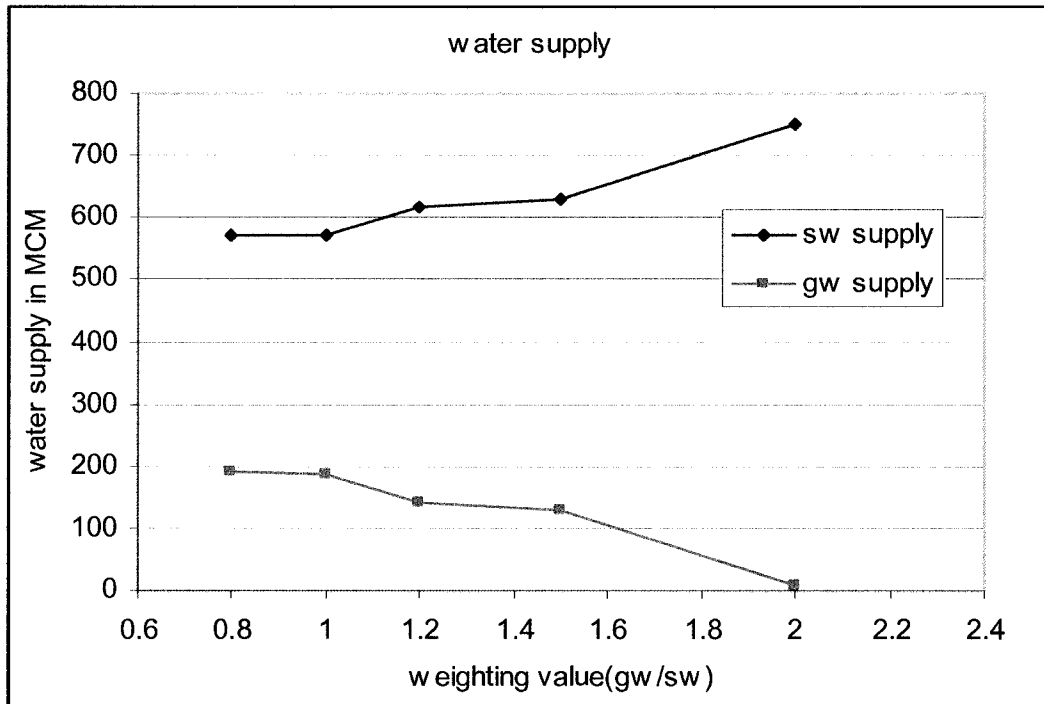


Figure 6.10 Water supplies due to weighting factor variation

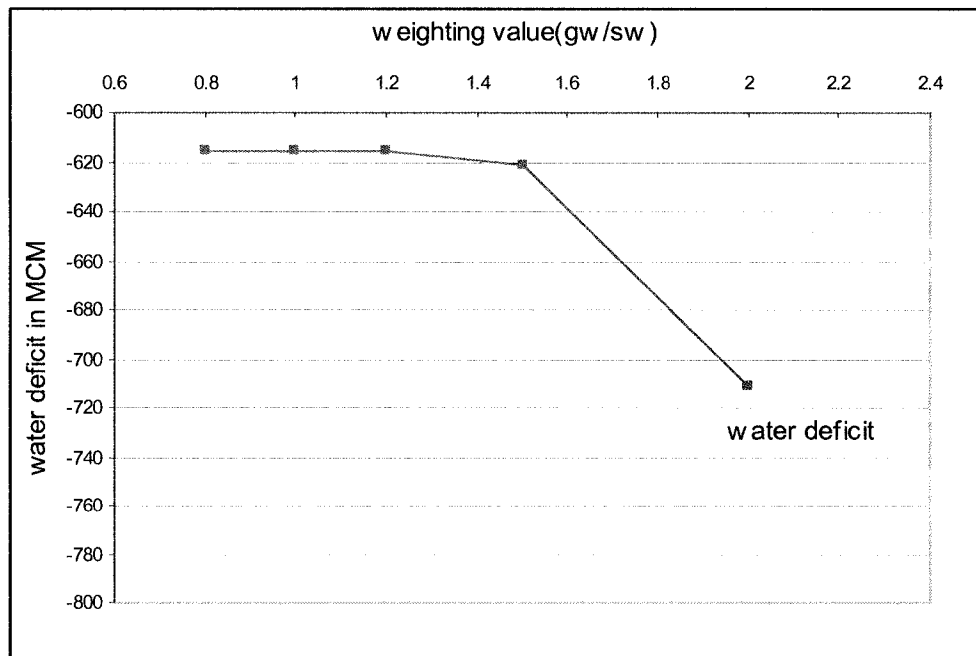


Figure 6.11 Water deficits due to weighting factor variation

6.5.2 Water Demand Factor

The purpose of the water demand sensitivity analysis was to illustrate the effects due to variability of the demand weighting factor. The ultimate objective of this analysis is to encourage efficient water resource planning. Variable cost weighting factors between downstream water demand and instream flow were evaluated.

Table 6.15 show the weighting factors for three cases which demonstrate the impact of weighting values for the instream flow, the diversion, and the downstream water demands. In Case 1 is water demand and instream flow have the highest priority and the spill has the lowest priority. In Case 2 and Case 3 are that the trans-basin diversion has the highest priority and the spill has the lowest priority. The priority of downstream water demand in Case 2 is higher than it is in Case 3.

Table 6.15 Weighting factors for each case

Item	Instream flow	Diversion	Downstream water demand	spill	Surface water	Ground water
Case 1	100	1	100	0.001	1	1.2
Case 2	10	100	10	0.001	1	1.2
Case 3	10	100	1	0.001	1	1.2

The water demand sensitivity analysis was performed and the results of each case are listed in the Table 6.16. When the weighting factor for instream flow and downstream water demands is higher than that for the diversion demand, the water deficit for the trans-basin diversion is 615.8MCM. Water deficits to instream flow and downstream

water demands do not occur in Case 1. On the other hand, Case 2 has water deficits to both the trans-basin diversion and instream flow. Case 3 has deficits to both the trans-basin diversion and instream flow plus deficits for some of the downstream demands. The maximum water deficit for downstream water demands was 67.4 MCM.

Table 6.16 Sensitivity analysis for various weightings

(unit:MCM)

Items			Weighting factor		
			Case 1	Case 2	Case 3
Ground water	sum	supply	140.82	140.82	101.84
	M&I muju	supply	12.53	12.53	11.61
	Agri. Muju	supply	11.63	11.63	5.62
	M&I Oc	supply	53.01	53.01	51.40
	Agri. Oc	supply	63.65	63.65	33.21
Surface water	sum	supply	618.5	618.5	550.1
		deficit	0.00	0.00	-67.4
	M&I Muju	supply	44.40	44.40	42.6
		deficit	0.00	0.00	-2.7
	Agri. Muju	supply	53.30	53.30	50.1
		deficit	0.00	0.00	-9.2
	M&I Okcheon	supply	201.00	201.00	186.3
		deficit	0.00	0.00	-16.4
	Agri. Okcheon	supply	319.80	319.80	271.1
		deficit	0.00	0.00	-39.1
Trans-basin diversion		supply	11192.2	11486.4	11486.4
		deficit	-615.8	-321.6	-321.6
Minimum Instream flow		supply	5896.4	5602.2	5607.3
		deficit	0.0	-294.2	-289.1
Total water deficit			-615.8	-615.8	-678.1

The performance measures for various weightings are listed in Table 6.17. For diversion to Jeonju, the trans-basin diversion, the performance measures of Case 2 and 3 are the same and are larger than those of Case 1. For instream flow, performance measures of Case 2 and 3 are the same and these are smaller than those of Case 1. The average values of performance measures of Case 1 are larger than for Case 2 and Case 3.

Table 6.17 Performance measures for various weightings

Items	Sub-items	Performance measures		
		Case 1	Case 2	Case 3
Diversion to Jeonju	Prel	0.94	0.97	0.97
	Pvul	0.91	0.93	0.93
	Pres	0.01	0.01	0.01
Instream flow	Prel	1.0	0.83	0.83
	Pvul	1.0	0.95	0.95
	Pres	1.0	0.0017	0.0019
Average	Prel	0.97	0.90	0.90
	Pvul	0.95	0.94	0.94
	Pres	0.503	0.006	0.007

6.6 Development of an Operational Guide Curve

A reservoir operational policy is needed to incorporate the results of the optimization model. Using the results of the stream-aquifer-reservoir system, 10-day reservoir operational guide curves are derived by linear regression analysis of the optimal set of releases, surface water and groundwater withdrawals, downstream lateral flows, and amounts of the trans-basin diversion.

6.6.1 Procedure of the Regression Analysis

Optimal releases obtained from the conjunctive management model are computed based on the independent variables of the inflow and the storage to derive 10-day reservoir operating rules. The optimization model determined the optimal 10-day values for reservoir storage, reservoir release, surface water withdrawals, and groundwater withdrawals. The procedure for estimating operational guide curves consisted of a linear regression analysis among the storage, the forecasted inflow, the total release, the separation of downstream release, the trans-basin diversion, the groundwater withdrawal and the downstream release and lateral flows.

The general form of a storage rule is:

$$S_t = a_0 + a_1(R_t + D_t) + a_2S_{t-1} + a_3\hat{I}_t \quad (6.1)$$

Where reservoir storage at the end of the period is S_t , storage at the beginning of each period is S_{t-1} , current period forecasted inflow is \hat{I}_t , and total outflow is composed of downstream release (R_t) and the trans-basin diversion (D_t). Note that the current period forecasted inflow is an unknown. For the purpose of this research it was assumed that the historical inflow would be used to represent this value. In fact this assumes that the inflow could be forecasted without error and the results of the regression are better than they should be. KOWACO does have an inflow forecasting system for the Geum basin

and could produce forecasted 10-day values. It is assumed that the use of the forecasted inflow values will not significantly impact the results of the operational policies.

Once the end of period storage has been computed by the storage rule, total release is computed from Equation 6.2, where \hat{I}_t is the forecasted inflow, R_t is the downstream release and D_t is the diversion in the current time period.

$$S_t = S_{t-1} + \hat{I}_t - (R_t + D_t) \quad (6.2)$$

The separation of downstream release and diversion in total outflow is estimated by:

$$R_t = f(R_t + D_t) + b \quad (6.3)$$

The groundwater withdrawal, GW_t is also estimated from a linear regression analysis between the groundwater withdrawal and the downstream release and lateral flows, Lf_t , as shown in Equation 6.4:

$$GW_t = f(R_t + Lf_t) + c \quad (6.4)$$

6.6.2 Reservoir Operational Guide Curve

The developed storage rules are illustrated in Table 6.18 and 6.19. Table 6.18 shows the 10-day regression equations between ending storage and beginning storage.

Most correlations have a large R^2 value except for July, August, and September. Table 6.19 illustrates correlation between downstream release and total outflow of the reservoir including the trans-basin diversion. The regression equation fits very well to all months because the trans-basin diversions are constant. Minimum downstream release of the reservoir is 2MCM per 10-day period because the instream flow from the Yongdam reservoir is legally established at $2.25\text{m}^3/\text{sec}$.

Groundwater withdrawal is constant from November to March but it is different for the other periods which are estimated from the regression equations shown in Table 6.19. It is assumed that municipal water is used at a constant rate all year round but agricultural water is used differently according to the time of the year. Figures in relation to this analysis are shown in the appendix.

Table 6.18 Storage rule based operational guide curves

Month	$S_t = \alpha f(S_{t-1}) + \beta$	R^2	Month	$S_t = \alpha f(S_{t-1}) + \beta$	R^2
Oct	$\alpha = 1.0048, \beta = -8.0634$	0.98	Apr	$\alpha = 0.9746, \beta = 3.8076$	0.99
Nov	$\alpha = 1.0093, \beta = -15.702$	0.99	May	$\alpha = 0.9926, \beta = -4.8071$	0.99
Dec	$\alpha = 1.02278, \beta = -19.843$	0.99	Jun	$\alpha = 0.9724, \beta = 9.0637$	0.97
Jan	$\alpha = 0.9657, \beta = 5.6878$	0.97	Jul	$\alpha = 0.9262, \beta = 61.774$	0.85
Feb	$\alpha = 1.0054, \beta = -11.781$	0.99	Aug	$\alpha = 0.959, \beta = 44.094$	0.83
Mar	$\alpha = 1.0131, \beta = -9.763$	0.98	Sep	$\alpha = 0.8968, \beta = 62.638$	0.91

Table 6.19 Regression for release and groundwater withdrawal

Mon	10-day	Release		Groundwater withdrawal		
		$R_t = \alpha f(R_t + D_t) + \beta$	$R_t + D_t$	$GW_t = \alpha f(R_t + Lf_t) + \beta$	$R_t + Lf_t$	R^2
Oct	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.53, \beta = 4.05$	≤ 7.86	0.47
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.53, \beta = 4.05$	≤ 7.86	
	21-31	$\alpha = 1, \beta = -14.827$	≥ 16.83	$\alpha = 0.53, \beta = 4.05$	≤ 7.86	
Nov	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.31$	≤ 6.70	
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.31$	≤ 6.70	
	21-30	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.31$	≤ 6.70	
Dec	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.31$	≤ 6.89	
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.31$	≤ 6.89	
	21-31	$\alpha = 1, \beta = -14.827$	≥ 16.83	$\alpha = 0.0, \beta = 0.35$	≤ 7.54	
Jan	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.31$	≤ 6.87	
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.31$	≤ 6.87	
	21-31	$\alpha = 1, \beta = -14.827$	≥ 16.83	$\alpha = 0.0, \beta = 0.35$	≤ 7.55	
Feb	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.34$	≤ 6.86	
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.34$	≤ 6.86	
	21-28	$\alpha = 1, \beta = -10.784$	≥ 16.83	$\alpha = 0.0, \beta = 0.27$	≤ 5.39	
Mar	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.32$	≤ 6.71	
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = 0.0, \beta = 0.32$	≤ 6.71	
	21-31	$\alpha = 1, \beta = -14.827$	≥ 16.83	$\alpha = 0.0, \beta = 0.67$	≤ 7.27	
Apr	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -1.17, \beta = 8.2$	≤ 7.05	0.91
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -1.17, \beta = 8.2$	≤ 7.05	
	21-30	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -1.17, \beta = 8.2$	≤ 7.05	
May	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.64, \beta = 4.88$	≤ 6.77	0.70
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.64, \beta = 4.88$	≤ 6.77	
	21-31	$\alpha = 1, \beta = -14.827$	≥ 16.83	$\alpha = -1.07, \beta = 8.7$	≤ 8.09	
Jun	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -1.62, \beta = 12.1$	≤ 7.43	0.89
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -1.62, \beta = 12.1$	≤ 7.43	
	21-30	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -1.62, \beta = 12.1$	≤ 7.43	
Jul	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.68, \beta = 6.5$	≤ 9.42	0.90
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.68, \beta = 6.5$	≤ 9.42	
	21-31	$\alpha = 1, \beta = -14.827$	≥ 16.83	$\alpha = -0.68, \beta = 6.5$	≤ 9.42	
Aug	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.64, \beta = 5.77$	≤ 9.61	0.74
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.64, \beta = 5.77$	≤ 9.61	
	21-31	$\alpha = 1, \beta = -14.827$	≥ 16.83	$\alpha = -0.64, \beta = 5.77$	≤ 9.61	
Sep	1-10	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.54, \beta = 4.5$	≤ 8.61	0.68
	11-20	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.54, \beta = 4.5$	≤ 8.61	
	21-30	$\alpha = 1, \beta = -13.479$	≥ 15.48	$\alpha = -0.54, \beta = 4.5$	≤ 8.61	

6.7 Evaluation of the Operational Guide Curves

The guide curves developed by regression analysis do not fit the optimization results exactly. Therefore the developed equations will produce some error of prediction. To evaluate the overall performance of the guide curves, the KModSim river basin simulation model was used. The KModSim model allows the user to include the priorities of meeting the various demands and the desire to maintain target storage levels in the reservoir.

6.7.1 Network Formulation of KModSim

A critical problem facing the upper region of the Geum River basin is how to manage the intensifying competition for water by expanding municipal and agricultural water uses and yet meet instream flows. Confronted with the prospect of heightened competition for available water and the increased difficulty in constructing large-scale reservoirs, water users should depend on better management of existing reservoirs through integrated, basin-wide strategies that include conjunctive water use between surface water and groundwater. In this study, KModSim was used to simulate the allocation of available water resources through integrated, basin-wide strategies that include conjunctive use of surface water and ground water resources. The KModSim network for the upper region of the Geum River is designed to evaluate 10-day management scenarios and to forecast effects from extreme conditions such as drought. The network designed for the upper region of Geum River basin is illustrated in Figure

6.12. The network contains Yongdam reservoir, 17 major demand nodes, 13 confluence points, 3 inflows, 5 flowthrough demand nodes including one instreamflow demand, and other important locations in the basin. All the preexisting water rights in the basins are protected after the instream flows are satisfied. The priorities of water allocation in the basin are instream flows, municipal, industrial, agricultural, and the Jeonju supply in descending order. The highest priority at the Yongdam Reservoir is on the instream flow requirement in the Geum River and the second highest priority is on the Jeonju flow. The hydropower generation at Yongdam Reservoir is not considered in this network. The municipal and industrial water are estimated from records of previous water usage. The amount of return flow for both the municipal and industrial water is assumed to be 60% of the diversion amount. The agricultural water is estimated from an agricultural consumptive use model and the amount of return flow of agricultural water is assumed to be 30% of the diversion. Lagging of municipal and industrial return flows is not considered so the return flows are returned to the river in the same time period as the diversion. The consecutive three lagging time periods for the agricultural return flow is considered so the return flows are returned to the river by lag coefficients of [0.7 0.2 0.1].

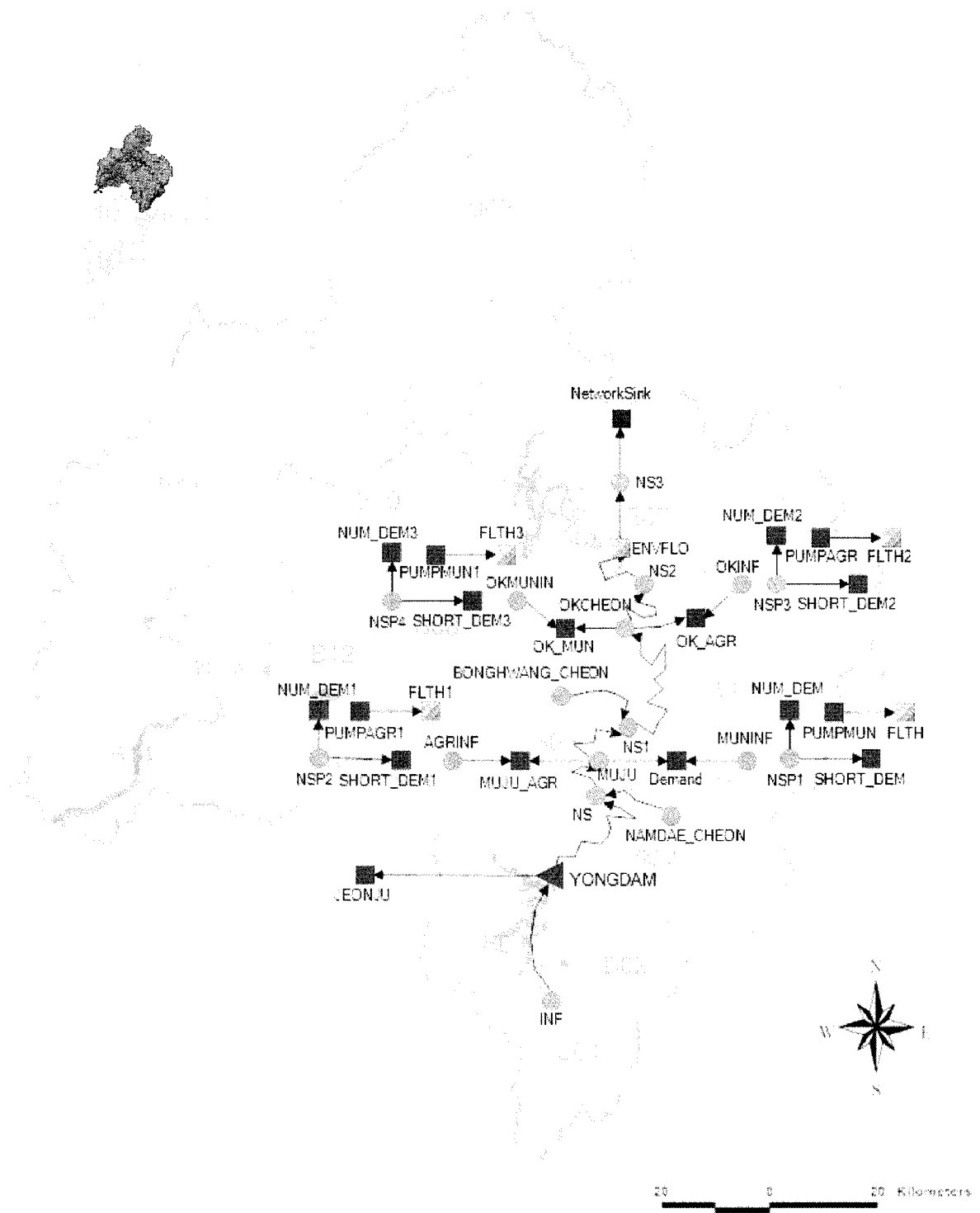


Figure 6.12 The 10-day network is developed for the upper region of Geum River basin. There are three inflow points including one reservoir and some demand or diversion locations for three subbasins in the upper region of Geum River basin.

6.7.2 Evaluation of Operational Guide Curves Using KModSim

To analyze the operational equations based on the results of the stream-aquifer-reservoir model, inflow data and the developed 10-day operational rules were applied by using the system states table in KModSim. For the KModSim simulation, stream flow and ground water interactions were considered. In this study, KModSim simulates groundwater pumping with response coefficients estimated from the MODFLOW model. The historical inflows and demands used in the KModSim simulation of the upper region of Geum River basin are listed in Table 6.20.

Table 6.20 The historical inflows and demands in the upper region of Geum River basin.

Items	Node Points	Minimum (MCM)	Maximum (MCM)	Average (MCM)	Total (MCM)
Surface Water	Upstream of Yongdam Reservoir	0.20	383.20	19.86	17159.1
	Namdae_Chun	0.10	248.6	12.01	10377.4
	Bonghwang_Chun	0.06	347.7	10.00	8644.23
Demands	Jeonju	10.8	14.82	13.66	11807.6
	M&I Muju	0.05	0.083	0.065	56.9040
	Agriculture Muju	0.00	0.304	0.075	64.9200
	M&I OKcheon	0.22	0.368	0.294	254.020
	Agriculture Okcheon	0.00	1.794	0.443	383.470
	Instream Flow	5.39	7.404	6.824	5896.34

The ground water pumping was simulated in KModSim to compare simulation results with stream-aquifer-reservoir system optimization results and historical results in which the groundwater was not used but only surface water was used. For the simulation, infiltration and depletion are considered for groundwater pumping in which all parameters used in KModSim simulation are estimated from MODFLOW. The pumping rates are used in KModSim to calculate the amount of groundwater necessary to satisfy all demands. The results of KModSim simulation with groundwater pumping showed that optimal integrated operation of Yongdam reservoir with the groundwater pumping satisfied most demands. The water deficit at the trans-basin Jeonju diversion is similar to that of the optimization model as shown in Table 6.21. Compared to water deficits considering only surface water supply, water deficits from conjunctive use simulated from the optimization method and the KModSim were reduced somewhat at all demands locations in the upper region of the Geum River basin.

Table 6.21 Various statistics calculated from the 10-day KModSim results using the developed linear operating rules and ground water pumping. In this simulation, the total number of time period is 864.

Demands	Shortage (MCM)	Number of Failures	Reliability	Vulnerability	Resiliency
Jeonju	615.0	78	0.91	0.96	0.0045
M&I Muju	0.000	0	1.0	1.0	1.0
AGR Muju	0.000	0	1.0	1.0	1.0
M&I Okcheon	0.000	0	1.0	1.0	1.0
AGR Okcheon	0.000	0	1.0	1.0	1.0
Instream Flow	0.000	0	1.0	1.0	1.0

The total ground water pumping rates generated from KModSim for each demand are listed in Table 6.22. The groundwater pumping for the municipal demand and the agricultural demand at Muju was smaller than the total ground water pumping for municipal demand and agricultural demand at Okcheon. These pumping rates are similar to the pumping rates obtained from LINGO when the pumping costs are assumed to be 1.7.

Table 6.22 Total ground water pumping rates for each demands generated from a KModSim with response coefficients estimated from MODFLOW.

Node Points	Minimum (MCM)	Maximum (MCM)	Average (MCM)	Total (MCM)
M&I Muju	0.00	0.078	0.0015	4.625
AGR Muju	0.00	0.304	0.0148	5.200
M&I Okcheon	0.00	0.349	0.0057	19.33
AGR Okcheon	0.00	1.794	0.0117	27.06

For validation of the developed 10-day optimal operational rules for operation of the Yongdam reservoir, stochastically generated inflow sequences were used. At first, 100 years of 10-day period data were generated from the SAMS statistical software package and then three sets of 24 years of 10-day data were selected. The minimum, maximum, average, and total amounts of three sets of 24 years of 10 days flows for the simulation of KModSim are listed in Table 6.23. The generated inflows are little larger than historical inflows but the inflow patterns of each case is similar to the historical inflows in Figure 6.13.

Table 6.23 The three sets of 24 years of 10-day inflow data in the upper region of Geum River basin (unit:MCM)

Demands	Minimum	Maximum	Average	Total (24Yrs)
Case I				
Upstream of Yongdam Reservoir	0.022	881.03	21.275	18,382.01
Namdae_Chun	0.016	648.47	12.932	11,173.45
Bonghwang_Chun	0.061	349.05	10.616	9,173.01
Case II				
Upstream of Yongdam Reservoir	0.200	2586.0	23.226	20,067.18
Namdae_Chun	0.020	717.76	14.281	12,338.99
Bonghwang_Chun	0.008	1127.7	12.642	10,922.96
Case III				
Upstream of Yongdam Reservoir	0.082	912.31	20.413	17,637.52
Namdae_Chun	0.018	592.56	13.133	11,347.25
Bonghwang_Chun	0.031	514.20	9.9291	85,79.153

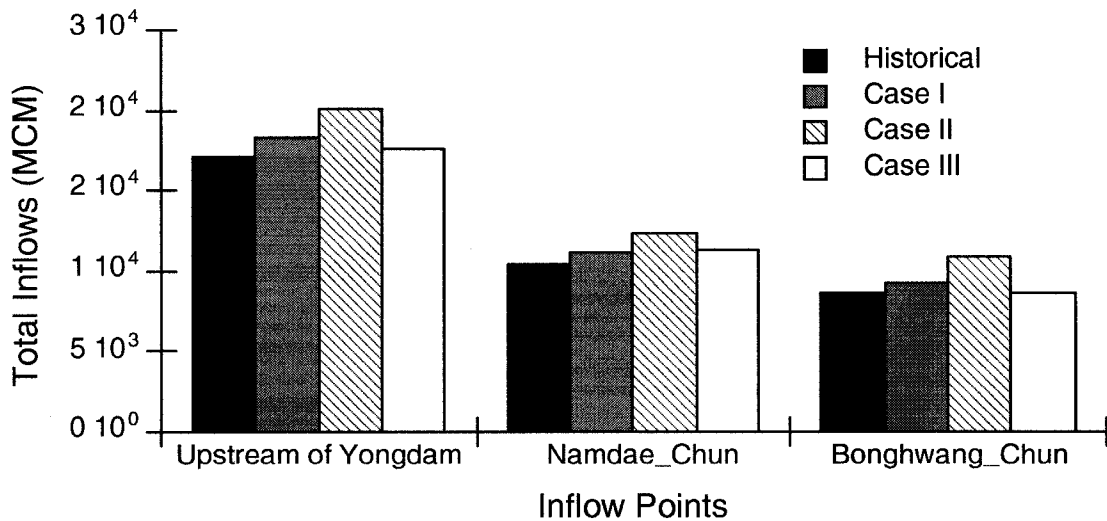


Figure 6.13 Comparisons of inflows between the total inflows and the total generated inflows of three cases.

The system states feature in KModSim was used to estimate the optimal target storage from the derived 10-day operational rules. The result shows that optimal integrated operation of Yongdam reservoir with the groundwater satisfies most demands with some failures at the Jeonju diversion in Table 6.24.

Table 6.24 Various statistics calculated from the KModSim results using the developed linear operating rules, generated inflows, and groundwater pumping

Demands	Shortage	Number of Failures	Temporal Reliability	Vulnerability	Resiliency
Case I					
Jeonju	517.4	45	0.95	0.97	0.01
M&I Muju	0.00	0	1.0	1.0	1.0
AGR Muju	0.00	0	1.0	1.0	1.0
M&I Okcheon	0.00	0	1.0	1.0	1.0
AGR Okcheon	0.00	0	1.0	1.0	1.0
Instream Flow	0.00	0	1.0	1.0	1.0
Case II					
Jeonju	457.2	41	0.95	0.97	0.016
M&I Muju	0.00	0	1.0	1.0	1.0
AGR Muju	0.00	0	1.0	1.0	1.0
M&I OKcheon	0.00	0	1.0	1.0	1.0
AGR Okcheon	0.00	0	1.0	1.0	1.0
Instream Flow	0.00	0	1.0	1.0	1.0
Case III					
Jeonju	535.1	47	0.93	0.96	0.15
M&I Muju	0.00	0	1.0	1.0	1.0
AGR Muju	0.50	2	0.99	1.0	0.5
M&I Okcheon	0.00	0	1.0	1.0	1.0
AGR Okcheon	0.00	0	1.0	1.0	1.0
Instream Flow	0.00	0	1.0	1.0	1.0

Ground water pumping was simulated in KModSim to estimate the total pumping rates used for each demand and to compare the simulation results with the conjunctive water management system results. For the simulation, three sets of 24 years of 10-day

sequential flows were used and infiltration and depletion are considered for ground water pumping in which all parameters used in the KModSim simulation are estimated from MODFLOW. The pumping rates are used in KModSim to calculate the amount of ground water necessary to satisfy all demands. The results of the KModSim simulation with groundwater pumping show that optimal integrated operation of Yongdam reservoir combined with groundwater pumping satisfied most of the demands in Table 6.25.

Table 6.25 various statistics calculated from the KModSim results using the developed linear operating rules, the total groundwater, and stream-aquifer interaction

Demands	Shortage	Number of Failures	Temporal Reliability	Vulnerability	Resiliency
Case I					
Jeonju	436.1	97	0.89	0.99	0.008
M&I Muju	0.000	0	1.0	1.0	1.0
AGR Muju	0.000	0	1.0	1.0	1.0
M&I OKcheon	0.000	0	1.0	1.0	1.0
AGR Okcheon	0.000	0	1.0	1.0	1.0
Instream Flow	0.000	0	1.0	1.0	1.0
Case II					
Jeonju	371.8	63	0.92	0.97	0.0097
M&I Muju	0.000	0	1.0	1.0	1.0
AGR Muju	0.000	0	1.0	1.0	1.0
M&I OKcheon	0.000	0	1.0	1.0	1.0
AGR Okcheon	0.000	0	1.0	1.0	1.0
Instream Flow	0.000	0	1.0	1.0	1.0
Case III					
Jeonju	326.2	53	0.96	0.99	0.0096
M&I Muju	0.000	0	1.0	1.0	1.0
AGR Muju	0.000	0	1.0	1.0	1.0
M&I OKcheon	0.000	0	1.0	1.0	1.0
AGR Okcheon	0.000	0	1.0	1.0	1.0
Instream Flow	0.000	0	1.0	1.0	1.0

The total groundwater supplies estimated from a KModSim for each inflow set are listed in Table 6.26. The results show that the all cases of groundwater pumping rates generated from KModSim were similar to the total ground water pumping rates estimated from LINGO. The comparisons of pumping rates from KModSim with inflows of historical, Case I, Case II, and Case III are shown in Figure 6.14. In Figure 6.14, Case II uses the largest amount of groundwater for each demand and the Case III uses the smallest amount of groundwater for each demand.

Table 6.26 Total ground water amounts for each demands estimated from a KModSim with response coefficients

Demands	Minimum	Maximum	Average	Total (24Yrs)
Case I				
Muju_Mun	0	0.07	0.0038	6.692
Muju_Agr	0	0.30	0.0145	7.260
OK_Mun	0	0.31	0.0058	21.17
Ok_Agr	0	1.31	0.0164	28.10
Case II				
Muju_Mun	0	0.71	0.0032	6.643
Muju_Agr	0	0.30	0.0147	8.106
OK_Mun	0	0.31	0.0032	22.61
Ok_Agr	0	1.79	0.0131	36.55
Case III				
Muju_Mun	0	0.08	0.0030	5.971
Muju_Agr	0	0.30	0.0146	6.652
OK_Mun	0	0.31	0.0043	21.76
Ok_Agr	0	1.32	0.0121	24.99

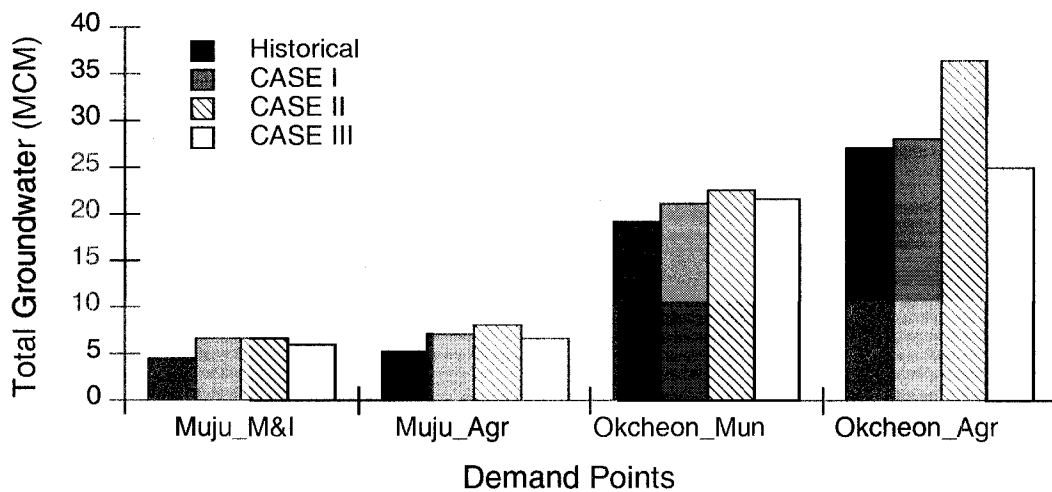


Figure 6.14 Comparisons of groundwater pumping in KModSim in each case.

In summary, the operational scenarios tested in the 10-day KModSim network were to evaluate various operational strategies. The results show that optimal conjunctive operation of Yongdam reservoir satisfied all demands including the instream flows in the Geum River and slightly increased the Jeonju diversion flows in the system.

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary and Conclusions

In this research, a modeling framework was developed to evaluate the potential for conjunctive management of groundwater and surface water in a river system with reservoirs, a trans-basin diversion and a relatively small, shallow alluvial aquifer. A set of models was used in this research to consider the groundwater flow, the streamflow depletion response, the stream-aquifer interaction, impacts on the stream-aquifer system from single wells and multiple wells, and the operation of a reservoir.

A goal-programming based optimization model was developed to determine integrated reservoir releases and groundwater pumping rates. The objectives of the optimization model are addressed to solutions that minimize water shortages of demands for a trans-basin diversion, downstream water needs, and instream flows. The multiple objectives used in the optimization model were considered in terms of weighted deviations from specified goals. The model used lag and response coefficients to consider streamflow depletion, infiltration to the aquifer, and return flows.

The upper region of the Geum River basin, used as a case study in this research, is typical of many river basins in Korea where the shape of river basin is narrow with small alluvial aquifer saturated thickness and where most groundwater pumping comes quickly from the streamflow. The narrow basin has a limited ability have wells located a long distance away from the streams. In this study, groundwater withdrawal from wells located at 50m, 100m, 200m, and 500m from the river were considered based upon the conditions in a typical river valley in Korea.

The recharge and groundwater table variations of the aquifer are calculated from daily streamflow records to estimate the stream-aquifer response. The water table variations due to well pumping are important factors to calculate withdrawal cost of groundwater in the optimization model.

According to this analysis, conjunctive water yield can be increased as the distances between the well and the stream increase. The reason is that pumping at the wells closest to the stream has a relatively large impact on the stream in each time period, while pumping wells at further distance allows some of the transient groundwater storage to be exploited. Therefore, additional benefits to a water supply system using multiple wells are possible. These simulations indicate that the best combination of various well distances is possible when a constant annual pumping rate is needed.

The results of the analysis showed that conjunctive management allowed for a better distribution of water in the system in both space and time. The conjunctive use strategy was able to reduce the amount of excess water (above the minimum instream flow requirements) that went downstream. This allowed more water to be kept in the reservoir.

A key purpose of this conjunctive water management model is to determine an operational guide curve for the reservoir that considers both the groundwater withdrawal and the surface water withdrawal. The results of this study showed that groundwater pumping should primarily be used to supply the agricultural water demand located in the second site, Okcheon, in the case study area. Municipal water withdrawal from groundwater pumping should not be used much because the withdrawal cost of municipal water is higher than that for agricultural water.

In groundwater and surface water management having long term planning horizons, there are various uncertainties associated with the cost weighting factor of groundwater and surface water withdrawal, and with the water demand weighting factor between instream flow and downstream water demand. The effects of these uncertainties were evaluated by performing a sensitivity analysis with respect to the variation of the cost and water demand weighting factor for groundwater and surface water withdrawal. The results of the sensitivity analysis suggest that water deficits increase as the cost ratio between groundwater and surface water increases from 0.8 to 2. However, even if the cost ratio lies in a value of 0.8 or 1.0, the groundwater discharge rates are similar because the total allowable groundwater supply is being withdrawn from the aquifer.

Using the results of the conjunctive management optimization model, 10-day reservoir operational policies were derived by a linear regression. Operational guide curves were developed to consider downstream water demand, trans-basin diversions, and instream flow requirements. To evaluate the reservoir operational policies, a 10-day time step KModSim network model was used in the upper region of the Geum River basin. Reservoir operational guide curves are applied in KModSim by specified

hydrologic states. Reservoir target storage levels are specified from the values estimated from the regression equations developed in this study, along with same priorities, all of which are conditioned on user defined system state information. In the KModSim simulation, the municipal and the agricultural demands, trans-basin water supply, the interactions between the groundwater and streamflow, instream flow in the Geum River, and lagged time intervals for agricultural return flow, depletion, and infiltration were considered.

The total ground water pumping rates generated from KModSim for the municipal demand and the agricultural demand of the Muju site were smaller than the total ground water pumping for municipal demand and agricultural demand of the Okcheon site. These pumping rates are similar to the pumping rates obtained from LINGO. The results of the KModSim simulation with groundwater pumping show that optimal integrated operation of Yongdam reservoir with the groundwater pumping satisfied most demands. The total ground water supplies estimated from a KModSim for each inflow show that the groundwater pumping rates generated from KModSim were similar to the total ground water pumping rates estimated from LINGO.

The KModSim results show that optimal integrated operation of Yongdam reservoir satisfies all demands including the instream flows in the Geum River, however, shortages still exist at the trans-basin diversion. The deficits at the trans-basin Jeonju diversion, can be reduced by up to 100 MCM. While this is a small amount relative to the trans-basin diversion demand, 100 MCM is still a lot of water. In the KmodSim simulation, the developed reservoir operational guide curves were able to improve the

water shortage situation during a dry year. This shows that the use of the developed operational rules improve the operation of the system.

In general this study was able to show that even though the aquifer system is small, there are possibilities to reduce deficits in the system through appropriate groundwater pumping and reservoir release strategies. While the amount of deficit reduction is small compared to the total demand and even the amount of deficit, it should be recognized that the results show that the system is unable to meet all of its demands. To meet all the demands in the Geum River basin requires that the trans-basin diversion be shorted. In such a situation any amount of additional water that can be used to reduce the deficits to the diversion might be considered to be significant.

Perhaps the more relevant question is whether the developed reservoir operation and groundwater pumping strategies could be effectively applied in practice. To apply the developed operational rules would require the coordinated operation of both the reservoir and the groundwater pumping. Currently the operational agency for the reservoir, K-Water, does not have operational control of the groundwater pumping. Further the agency with the operational control of groundwater pumping would need to be able to specify the location, rates and timing of the desired pumping. This is not currently the case. Without a high level of administrative and technical control of the groundwater pumps, it is unlikely that such a sophisticated operational policy could be implemented. Perhaps this research can serve as an incentive to decision makers to pursue a coordinated operation strategy for the surface water and groundwater resources in the Geum River Basin.

7.2 Recommendations for Further Study

This study considered only groundwater quantity. In future studies groundwater quality should also be considered. The assumption of this study was that the quality of both surface water and groundwater were comparable. If this is not the case, then conjunctive management could be adversely impacted.

This study used a 10-day time step. Future studies should consider a daily time step. The use of a shorter time step would potentially improve the understanding of the transient stream-aquifer interactions in the case study area.

This study demonstrated that an integrated conjunctive use of surface and groundwater has potential to reduce deficits in the Geum River Basin. However, an administrative structure that allows this conjunctive use needs to be implemented in South Korea. Future studies should be conducted on the type of administrative structure that would allow this level of integrated conjunctive use.

REFERENCES

- Ahlfeld, D.P., and Mulligan, A.E., "Optimal management of flow in groundwater systems", Academic Press, San Diego, California, 2000, 185 p.
- Ahlfeld, D.P., and Heidari, Manoutchehr, "Application of optimal hydraulic control to ground-water systems", *Journal of Water resources Management and Planning*, Vol. 120, No. 3, pp 350-365, 1994
- Ahmad Abrishamchi, Ali Ebrahimian, Massoud Tajrishi, and Miguel A. Marino, "Case study: application of multicriteria decision making to urban water supply", *Journal of Water resources Management and Planning*, ASCE, Vol. 131. No.4, pp 326-335, 2005
- Armstrong, D.S., Parker, G.W., and Richards, T.A., "Evaluation of streamflow requirements for habitat protection by comparison to streamflow characteristics at index streamflow-gaging stations in Southern New England", U.S. Geological Survey Water-Resources Investigations Report 4332, 2003, 99 p.
- Barlow, P.M., and Hess, K.M., "Simulated hydrologic response of the Quashnet river stream-aquifer system to ground-water withdrawals, Cape Cod, Massachusetts", U.S. Geological Survey Water-Resources Investigations Report 4064, 1993, 52 p.
- Barlow, P.M., Ahlfeld, D.P., and Dickerman, D.C., "Conjunctive-management models for sustained yield of stream-aquifer systems", *Journal of Water resources Management and Planning*, Vol. 129, No. 1, pp 35-48, 2003
- Barlow, P.M., and Dickerman, D.C., "Numerical-simulation and conjunctive-management models the Hunt-Annaquatucket- Pettaquamscutt stream-aquifer system, Rhode Island", U.S. Geological Survey Professional Paper 1636, 2001, 88 p.
- Basagaoglu, H., and Marino, M.A., "Joint management of surface and ground water supplies", *Ground Water*, Vol. 37, No.2, pp 214-222, 1999
- Batu, V., "Aquifer hydraulics : a comprehensive guide to hydrogeologic data analysis", John Wiley & Sons, Inc., New York, 1998, 727 p.
- Belaine, G., Peralta, R.C. and Hughes, T.C., "Simulation/optimization modeling for water resources management", *Journal of Water resources Management and Planning*, Vol. 125, No. 3, pp 154-161, 1999

- Bredehoeft, J.D. and Young, R.A., "The temporal allocation of ground water - a simulation approach", *Water Resources Research*, Vol. 6, No. 1, pp 3-21, 1970
- Bredehoeft, J.D. and Young, R.A., "Conjunctive use of ground water and surface water for irrigated agriculture : risk aversion", *Water Resources Research*, Vol. 19, No. 5, pp 1111-1121, 1983
- Caldwell, R.R., and Bowers, C.L., "Surface-water/ground-water interaction of the Spokane river and the Spokane valley/ Rathdrum Prairie aquifer, Idaho and Washington", U.S. Geological Survey Water-Resources Investigations Report 4239, 2003, 60 p.
- Cary A. Talbot, Charles W. Downer, Hsin-Chi Lin, Stacy E. Howington and David Richards, "Computational methods for simulating interaction between surface and subsurface hydrologic system", *Proceedings of the 24th International Conference on Computational Methods in Water Resources*, June 23-28, 2002, The Netherlands, pp 1511-1518
- Charles Revelle, Erhard Joeres, and William Kirby, "The linear decision rule in reservoir management and design. 1, development of the stochastic model", *Water Resources Research*, Vol. 5, No. 4, pp 767-777, 1969
- Conrad, L.P., and Bejin, M.S., "Evaluation of an induced infiltration model as applied to glacial aquifer systems", *Water Resources Bulletin*, Vol. 32, No. 6, pp 1209-1220, 1996
- C.T. Jenkins, "Computation of rate and volume of stream depletion by wells", U.S. Geological Survey Techniques of Water-Resources Investigations, Ch. D1, Bk 4, 1968, 17 p.
- Danskin, W.R., and Gorelick, S.M., "A policy evaluation tool : management of a multiaquifer system using controlled stream recharge", *Water Resources Research*, Vol. 21, No. 11, pp 1731-1747, 1985
- Das, A., and Datta, B., "Application of optimization techniques in groundwater quantity and quality management", Vol. , No. , pp 293-316, 2001
- Daubert, J.T. and Young, R.A., "Ground-water development in western river basins: large economic gains with unseen costs", *Ground Water*, Vol. 20, No.1, pp 80-85, 1982
- David P. Ahlfeld, Michele R. Minihane, "Numerical issues in management of stream/aquifer interaction", *Proceedings of the 24th International Conference on*

Computational Methods in Water Resources, June 23-28, 2002, The Netherlands, pp 1471-1478

- DeMeo, G.A., Lacznia, R.J., Boyd, R.A., Smith, J.L., and Nylund, W.E., "Estimated ground-water discharge by evapotranspiration from Death valley, California, 1997-2001", U.S. Geological Survey Water-Resources Investigations Report 4254, 2003, 27 p.
- D.K. Singh, "Conjunctive use of surface and groundwater"
- Dunning, C.P., Feinstein, D.T., Hunt, R.J., and Krohelski, J.T., "Simulation of ground-water flow, surface-water flow, and a Deep Sewer Tunnel System in the Menomonee valley, Milwaukee, Wisconsin", U.S. Geological Survey Scientific Investigations Report 5031, 2004, 40 p.
- Eggleston, J.R., "Evaluation of strategies for balancing water use and streamflow reductions in the Upper Charles river basin, Eastern Massachusetts", U.S. Geological Survey Water-Resources Investigations Report 4330, 2003, 85 p.
- Elizabeth A. Eschenbach, Timothy Magee, Edith Zagana, Morgan Goranflo, and Richard Shane, "Goal Programming Decision Support System for Multiobjective Operation of Reservoir Systems", Journal of Water resources Management and Planning, Vol. 127, No. 2, pp 108-120, 2001
- Emre K. Can and Mark H. Houck, "Real-time Reservoir Operations by Goal Programming", Journal of Water resources Management and Planning, Vol. 110, No. 3, pp 297-309, 1984
- Fontane, D.G., Labadie, J.W. and Loftis, B., "Optimal control reservoir discharge quality through selective withdrawal", Water Resources Research, Vol. 17, No. 6, pp 1594-1604, 1981
- Fontane, D.G., and Donald K. Frevert, "Water management under drought conditions : Overview of practices", Journal of Irrigation and Drainage Engineering, Vol. 121, No. 2, pp 199-206, 1995
- Fredericks, J.W., Labadie, J.L., and Altenhofen, J.M., "Decision support system for conjunctive stream-aquifer management", Journal of Water resources Management and Planning, Vol. 124, No. 2, pp 69-78, 1998
- Flynn, R.H., and Tasker, G.D., "Generalized estimates from streamflow data of annual and seasonal ground-water-recharge rates for drainage basins in New Hampshire", U.S. Geological Survey Scientific Investigations Report 5019, 2004, 61 p.

- G. V. Loganathan, and D. Bhattacharya, "Goal-Programming Techniques for Optimal Reservoir Operations", *Journal of Water resources Management and Planning*, Vol. 116, No. 6, pp 820-838, 1990
- Gorelick, S.M., "A review of distributed parameter groundwater management modeling methods", *Water Resources Research*, Vol. 19, No. 2, pp 305-319, 1983
- Gorelick, S.M., "Large scale nonlinear deterministic and stochastic optimization: formulations subsurface contamination", *Mathematical Programming*, Vol. 48, pp 19-39, 1990
- Gorelick, S.M., Freeze, R.A., Donohue, D., and Keely, J.F., "Groundwater contamination : optimal capture and containment", Lewis Publishers, Boca Raton, Florida, 1993, 385 p.
- Granato, G.E., Barlow, P.M., and Dickerman, D.C., "Hydrogeology and simulated effects of ground-water withdrawals in the Big river area, Rhode Island", U.S. Geological Survey Water-Resources Investigations Report 4222, 2003, 76 p.
- Grigg, N.S., "Water resources planning", McGraw-Hill, New York, NY, 1985
- Grigg, N.S., "New paradigm for coordination in water industry", *Journal of Water resources Management and Planning*, Vol. 119, No. 5, pp 572-587, 1993
- Gupta, Ram S., and Goodman, Alvin S., "Ground-water reservoir operation for drought management", *Journal of Water resources Management and Planning*, Vol. 111, No. 3, pp 303-320, 1985
- Hantush, M.S.M., and Marino, M.A., "Chance-constrained model for management of stream-aquifer system", *Journal of Water resources Management and Planning*, Vol. 115, No. 3, pp 259-277, 1989
- Hansen, C.V., and Aucott, W.R., "Status of ground-water levels and storage volume in the Equus Beds aquifer near Wichita, Kansas, January 2000 - January 2003", U.S. Geological Survey Water-Resources Investigations Report 4298, 2003, 36 p.
- Haugh, C.J., Carmichael, J.K., and Ladd, D.E., "Hydrogeology and ground-water-flow simulation in the Former Airfield Area of Naval support activity Mid-South, Millington, Tennessee", U.S. Geological Survey Scientific Investigations Report 5040, 2004, 31 p.
- Illangasekare, Tissa and Morel-Seytoux, H.J., "Stream-aquifer influence coefficients as tools for simulation and management", *Water Resources Research*, Vol. 18, No. 1, pp 168-176, 1982

- James A. Momoh, M.E. El-Hawary, and Ramababu Adapa, "A review of selected optimal power flow literature to 1993 part I: nonlinear and quadratic programming approaches", IEEE Transactions on Power Systems, Vol. 14, No.1, pp 96-104, 1999
- J.D. Salas, J.W. Delleur, V.Yevjevich and W.L.Lane, "Applied modeling of hydrologic time series", Water resources publications, LLC., 1980, 484p
- J.D. Salas, N.Saada, C.H.Chung, W.L.Lane, and D.K.Frevert, "Stochastic analysis, modeling, and simulation (SAMS) Version 2000", User's manual, Colorado State University, Computing Hydrology Laboratory, Water Resources, Hydrologic and Environmental Sciences Engineering Research Center, 2000,92p
- J.D. Salas, R.A. Smith, G.Q. Tabios III, J.H. Heo, "Statistical computing techniques in water resources and environmental engineering ", Course packet, Civil Engineering Department, Colorado State University, 2002
- Jay R. Lund, Discussion of "Optimal Operation of Multireservoir Systems:State-of-the-Art Review" by John W. Labadie, Journal of Water Resources Planning and Management,ASCE, pp 406-407, Sep/Oct. 2005
- Jih-Shyang Shih, and Charles ReVelle, "Water supply operations during drought: A discrete hedging rule", European Journal of Operational Research 82, pp 163-175, 1985
- John B. Czarnecki, Brian R. Clark, and Thomas B. Reed, "Conjunctive-use optimization model of the Mississippi river valley alluvial aquifer of Northeastern Arkansas",U.S. Geological Survey Water-Resources Investigations Report 4230, 2003, 29 p.
- John B. Czarnecki, Brian R. Clark, and Gregory P. Stanton, "Conjunctive-use optimization model of the Mississippi river valley alluvial aquifer of Southeastern Arkansas",U.S. Geological Survey Water-Resources Investigations Report 4230, 2003, 26 p.
- John W. Labadie, "Optimal Operation of Multireservoir Systems:State-of-the-Art Review", Journal of Water Resources Planning and Management,ASCE, Vol. 130, No.2, pp 93-111, 2004
- Karamaouz, M., Tabari, M.R., and Kerachian, R., "Conjunctive use of surface and groundwater resources : application of genetic algorithm and neural network",ASCE Environmental and Water Resources Institute ConferenceProceedings, pp 1-10, 2004

- Ko Ick Hwan, "Integrated river basin operational planning considering water quantity and water quality", Ph.D, Dissertation-Colorado State University, Fort Collins, Colorado, 1997
- Ko S-K., "Optimizing reservoir systems operation with multiobjective decision analysis", Ph.D, Dissertation-Colorado State University, Fort Collins, Colorado, 1989
- Kwanyuen, Banacha, "Improved multiobjective groundwater planning model", Ph.D, Dissertation - Colorado State University, Fort Collins, Colorado, 1996
- K. Mohammadi and H.R. Eslami, "Conjunctive stream-aquifer management using an object oriented model : Case study", Proceedings of the 24th International Conference on Computational Methods in Water Resources, June 23-28, 2002, The Netherlands, pp 1519-1526
- Kyu-Cheoul Shim, Darrell G. Fontane, and John W. Labadie, "Spatial Decision Support System for Integrated River Basin Flood Control", Journal of Water Resources Management and Planning, Volume 128, No. 3, pp. 190-201, May/June 2002
- Labadie, J.W., "Optimal operation of multi-reservoir systems : a state-of-the-art review", Journal of Water Resources Management and Planning, Vol. ,No. , pp , 2003
- Larry W. Mays and Yeou-Koung Tung, "Systems Analysis: Water Resources Handbook", pp. 6.1-6.50, McGraw-Hill, U.S.A, 1996
- Lettenmaier, D.P., and Burges, D.R., "Cyclic storage : a preliminary assessment", Ground Water, Vol. 20, No.3, pp 278-288, 1982
- Lefkoff, L.J., and Gorelick, S.M., "Simulating physical processes and economic behavior in saline, irrigated agriculture: model development", Water Resources Research, Vol. 26, No. 7, pp 1359-1369, 1990
- Lefkoff, L.J., and Gorelick, S.M., "Benefits of an irrigation water rental market in a saline stream-aquifer system", Water Resources Research, Vol. 26, No. 7, pp 1371-1381, 1990
- Lefkoff, L.J., and Gorelick, S.M., "AQMAN : linear and quadratic programming matrix generator using two-dimensional ground-water flow simulation for aquifer management modeling", U.S. Geological Survey Water-Resources Investigations Report 4061, 1987, 164 p.
- Maddock, T., III., "Algebraic technological function from a simulation model", Water Resources Research, Vol. 8, No. 1, pp 129-134, 1972

- Maddock, T., III., "The operation of a stream-aquifer system under stochastic demands", *Water Resources Research*, Vol. 10, No. 1, pp 1-10, 1974
- Male, J.W., and Mueller, F.A., "Model for prescribing ground-water use permits", *Journal of Water Resources Management and Planning*, Vol. 118, No. 5, pp 543-561, 1992
- Marion W. Jenkins, Jay R. Lund, Richard E. Howitt, Andrew J. Draper, Siwa M. Msangi, Stacy K. Tanaka, Randall S. Ritzema, and Guilherme F. Marques, "Optimization of California's Water Supply System: Results and Insights", *Journal of Water Resources Planning and Management*, ASCE, Vol. 130, No. 4, pp 271-280, 2004
- Mark H. Houck, "Real-time daily reservoir operation by mathematical programming", *Water Resources Research*, Vol. 18, No.5, pp 1345-1351, 1982
- Matsukawa, Joy, Finney, B.A, and Willis, Robert, "Conjunctive-use Planning in MAD River Basin, California", *Journal of Water Resources Planning and Management*, ASCE, Vol. 118, No. 2, pp 115-132, March/April 1992
- McKee, P.W., Clark, B.R., and Czarnecki, J.B., "Conjunctive-use optimization model and sustainable-yield estimation for the Sparata aquifer of Southeastern Arkansas and Northcentral Louisiana", *U.S. Geological Survey Water-Resources Investigations Report 4231*, 2003, 30 p.
- Ming-Yen Tu, Nien-Sheng Hsu, and William W.-G. Yeh, "Optimization of reservoir management and operation with hedging rules", *ASCE*, Vol. 129, No.2, pp 86-97, 2003
- Ministry of Construction and Transportation (MOCT), and Korea Water Resources Corporation (KOWACO), "Report of Geum River basin investigation in Korea", 2004
- Ministry of Construction and Transportation (MOCT), and Korea Water Resources Corporation (KOWACO), "Geum River groundwater basic survey report", 2004
- Ministry of Construction and Transportation (MOCT), and Korea Water Resources Corporation (KOWACO), "Longterm comprehensive water resources plan (2006-2020)", 2006
- Mohammad Karamouz, and Mark H. Houck, "Annual and monthly reservoir operating rules generated by deterministic optimization", *Water Resources Research*, Vol. 18, No.5, pp 1337-1344, 1982
- Mohammad Karamouz, Reza Kerachian, and Banafsheh Zahraie, "Monthly water resources and irrigation planning: case study of conjunctive use of surface and groundwater resources", *Journal of Water Resources Management and Planning*, ASCE, Vol. 130, No.5, pp 391-402, 2004

- Morel-Seytoux, H.J., "A combined model of water table and river stage evolution",
Water Resources Research, Vol. 11, No. 6, pp 968-972, 1975
- Morel-Seytoux, H.J., and Daly, C.J., "A discrete kernel generator for stream-aquifer studies", Water Resources Research, Vol. 11, No. 2, pp 253-260, 1975
- Mueller, F.A., and Male, J.W., "A management model for specification of groundwater withdrawal permits", Water Resources Research, Vol. 29, No. 5, pp 1259-1368, 1993
- Nageshwar R. Bhaskar, and Elbert E. Whitlatch, Jr., "Derivation of monthly reservoir release policies", Water Resources Research, Vol. 16, No. 6, pp 987-993, 1980
- Newson, J.M., and Wilson, J.L., "Flow of groundwater to a well near a stream-effect of ambient groundwater flow direction", Ground Water, Vol 26, No 6, pp 703-711, 1988
- Nisai Wanakule and Larry W. Mays, and Leon S. Lasdon, "Optimal Management of Large-Scale Aquifers : Methodology and Applications", Water Resources Research, Vol. 22, No. 4, pp 447-465, 1986
- Paul M. Barlow, Brian J. Wagner, and Kenneth Belitz, "Pumping Strategies for Management of a Shallow Water Table: The Value of the Simulation-Optimization Approach", Ground water, Vol. 34, No. 2, pp 305-317, 1996
- Philip D. Crawley and Graeme C. Dandy, "Optimal Operation of Multiple-Reservoir System", Journal of Water resources Management and Planning, ASCE, Vol. 119, No. 1, pp 1-17, 1993
- Pinder, G.F., and Sauer, S.P., "Numerical simulation of flood wave modification due to bank storage effects", Water Resources Research, Vol. 7, No. 1, pp 63-70, 1971
- Prince, K.R., Reilly, T.E., and Franke, Lehn, "Analysis of the shallow groundwater flow system near Connetquot brook, Long island, New York", Journal of Hydrology, Vol. 107, pp 223-250, 1989
- R.A. Longenbaugh, "Determining optimum operational policies for conjunctive use of ground and surface water using linear programming", ASCE, 18th Annual Speciality Conference, University of Minnesota, 1970
- Ralph A. Wurbs, "Reservoir-system simulation and optimization model", Journal of Water resources Management and Planning, ASCE, Vol. 119, No. 4, pp 455-472, 1993

- Rastogi, A.K., "Optimal Pumping Policy and Groundwater Balance for The Blue Aquifer, California, Involving Nonlinear Groundwater Hydraulics", *Journal of Hydrology*, Vol. 111, pp 177-194, 1989
- Remson, I., Hornberger, G.M., and Molz, F.J., "Numerical methods in subsurface hydrology with an introduction to the finite element method", John Wiley & Sons, Inc., New York, 1971, 389 p.
- Richard N. Palmer, and K. John Holmes, "Operational guidance during droughts: expert system approach", *Journal of Water resources Management and Planning*, ASCE, Vol. 114. No.6, pp 647-666, 1988
- Robert H. Prucha, Tissa Illangasekare, "An integrated conceptual and modeling framework for studying groundwater recharge in arid and semi-arid regions", *Proceedings of the 24th International Conference on Computational Methods in Water Resources*, June 23-28, 2002, The Netherlands, pp 1503-1510
- Ross, S., "Introduction to stochastic dynamic programming", Academic Press, Inc., New York, 1983, 164 p.
- S. Pezeshk, O. J. Helweg, and K. E. Oliver, "Optimal Operation of Ground-water Supply Distribution Systems", *Journal of Water resources Management and Planning*, Vol. 120, No. 5, pp 573-586, 1994
- Schrage, L., "Linear, integer, and quadratic programming with LINDO : user's manual", The Scientific Press, Palo Alto, California, 1984, 86 p.
- Schwartz, F.W., and Zhang, H., "Funreservoir rentals of ground water", John Wiley & Sons, Inc., New York, 2003, 583 p.
- Smith, B.J., "Ground-water flow and ground- and surface- water interaction at McBaine Bottoms, Columbia, Missouri - 2000-02", U.S. Geological Survey Water-Resources Investigations Report 4234, 2003, 83 p.
- Sophocleous, Marios, Koussis, Antonis, Martin, J.L., and Perkins, S.P., "Evaluation of simplified stream-aquifer depletion models for water rights administration", *Ground Water*, Vol. 33, No.4, pp 579-588, 1995
- Spalding, C.P., and Khaleel, Raziuddin, "An evaluation of analytical to estimate drawdowns and stream depletions by wells", *Water Resources Research*, Vol. 27, No. 4, pp 597-609, 1991
- Stephens, D.B., "Vadose zone hydrology", Lewis Publishers, Boca Raton, Florida, 1996, 347 p.

- Theis, C.V., and Conover, C.S., "Chart for determination of the percentage of pumped water being diverted from a stream or drain : in Bentall, Ray, compiler, Shortcuts and special problems in aquifer tests", U.S. Geological Survey Water Supply Paper 1545-C, pp C106-C109
- Theis, C.V., "The effect of a well on the flow of a nearby stream", American Geophysical Union Transactions, Vol. 22, part 3, pp 734-738, 1941
- T.R. Neelakantan, and N.V. Pundarikanthan, "Neural network-based simulation-optimization model for reservoir operation", Journal of Water resources Management and Planning, ASCE, Vol. 126. No.2, pp 57-64, 2000
- Tsuyoshi Hashimoto, Reliability, "resiliency, and vulnerability criteria for water resources system performance evaluation", Water Resources Research, Vol. 18, No.1, pp 14-20, 1982
- U.S. Army Corps of Engineers, "Optimization of Multiple-Purpose Reservoir System Operations: A Review of Modeling and Analysis Approaches", Research Document No. 34, 1991
- Wallace, R.B., Darama, Yakup, and Annable, M.D., "Stream depletion by cyclic pumping of wells", Water Resources Research, Vol. 26, No. 6, pp 1263-1270, 1990
- Weaver, J.C., and Fine, J.M., "Low-flow characteristics and profiles for the Rocky river in the Yadkin-Pee Dee river basin, North Carolina, through 2002", U.S. Geological Survey Water-Resources Investigations Report 4147, 2003, 50 p.
- White, D.J., "Dynamic programming", Holden-day, San Francisco, California, 1969, 181 p.
- Willis, R., and Yeh, W. W-G, "Groundwater systems planning and management", Prentice-Hall, Eaglewood Cliffs, New Jersey, 1987, 416 p.
- William W-G. Yeh, "Groundwater Systems: Water Resources Handbook ", pp. 16.1-16.4, McGraw-Hill, U.S.A, 1996
- Wilson, J.L., "Induced infiltration in aquifers with ambient flow", Water Resources Research, Vol. 29, No.10, pp 3503-3512, 1993
- Winter, T.C., "Recent advances in understanding the interaction of groundwater and surface water", Reviews of Geophysics, Supplement, U.S. National Report to International Union of Geodesy and Geophysics, pp 985-994, 1995
- Yeh, William W-G., "Systems analysis in ground-water planning and management", Journal of Water resources Management and Planning, Vol. 118, No. 3, pp 224-237, 1992

Yeh, William W-G., "Reservoir management and operations models : a state-of-the-art review", *Water Resources Research*, Vol. 21, No. 12, pp 1797-1818, 1985

Young, R.A. and Bredehoeft, J.D., "Digital computer simulation for solving management problems of conjunctive groundwater and surface water systems", *Water Resources Research*, Vol. 8, No. 3, pp 533-556, 1972

Yung-Hsin Sun, William W-G. Yeh, Nien-Sheng Hsu, and Peter W.F. Louie, "Generalized network algorithm for water-supply-system optimization ", *Journal of Water resources Management and Planning, ASCE*, Vol. 121. No.5, pp 392-398, 1995

APPENDIX-A : Groundwater Table Variations using MATHCAD

Agricultural water pumping in Okcheon

1. Input parameters

$$W := 2400r \quad K := 16 \frac{m}{day} \quad B := 15r \quad slope := 0.00\% \quad L := 1000r$$

$$r_o := 500r \quad N_{num} := 20$$

$$Q_{min} := K \cdot slope \cdot W \cdot B \quad Q_{min} = 1.152 \times 10^3 \frac{m^3}{day}$$

- One site agricultural water use :

$$Q_{agri} := 12800 \frac{m^3}{day}$$

Use the analytical solutions to drawdown due to pumping with a width of 2,400m and length of 1000m moving with the regional gradient of 0.002 in an aquifer with a hydrologic conductivity of 16m/day and a thickness of 15m under 5m of the ground surface.

The purpose of the study is to get a drawdown from ground surface due to pumping for stream-aquifer-reservoir optimal operation.

2. Assumptions/Approximations

Assume that the principle of superposition and the the following steady-state equation will be reasonably accurate for this case:

$$s(Q, r) := \sqrt{B^2 + \left(\frac{Q}{\pi \cdot K} \cdot \ln \left(\frac{r}{r_o} \right) \right)} - 15r$$

- Drawdown with Q_{min} : $s(Q_{min}, 0.6m) = -6.582m$

Then each drawdown is calculated with each pumping well about 300m apart and 200m width.

- a minimum pumping (Q_{p1}) : $Q_{p1} := \frac{Q_{agri}}{N_{num}} \quad Q_{p1} = 640 \frac{m^3}{day} \quad s(Q_{p1}, 1m) = -2.922m$

3. Total drawdown evaluation

- set coordinates from p1 to p20
- each drawdown evaluation with coordinates
- total drawdown evaluation to sum each drawdown values

3.1 set coordinates

- p1 coordinates : $\begin{pmatrix} x_{p1} \\ y_{p1} \end{pmatrix} := \begin{pmatrix} 150.5m \\ 200m \end{pmatrix}$ - p11 coordinates : $\begin{pmatrix} x_{p11} \\ y_{p11} \end{pmatrix} := \begin{pmatrix} 550.5m \\ 300m \end{pmatrix}$

$$\text{- p2 coordinates : } \begin{pmatrix} x_{p2} \\ y_{p2} \end{pmatrix} := \begin{pmatrix} 150.5\text{m} \\ 400\text{m} \end{pmatrix} \quad \text{- p12 coordinates : } \begin{pmatrix} x_{p12} \\ y_{p12} \end{pmatrix} := \begin{pmatrix} 550.5\text{m} \\ 500\text{m} \end{pmatrix}$$

$$\text{- p3 coordinates : } \begin{pmatrix} x_{p3} \\ y_{p3} \end{pmatrix} := \begin{pmatrix} 150.5\text{m} \\ 600\text{m} \end{pmatrix} \quad \text{- p13 coordinates : } \begin{pmatrix} x_{p13} \\ y_{p13} \end{pmatrix} := \begin{pmatrix} 550.5\text{m} \\ 700\text{m} \end{pmatrix}$$

$$\text{- p4 coordinates : } \begin{pmatrix} x_{p4} \\ y_{p4} \end{pmatrix} := \begin{pmatrix} 150.5\text{m} \\ 800\text{m} \end{pmatrix} \quad \text{- p14 coordinates : } \begin{pmatrix} x_{p14} \\ y_{p14} \end{pmatrix} := \begin{pmatrix} 550.5\text{m} \\ 900\text{m} \end{pmatrix}$$

$$\text{- p5 coordinates : } \begin{pmatrix} x_{p5} \\ y_{p5} \end{pmatrix} := \begin{pmatrix} 150.5\text{m} \\ 1000\text{m} \end{pmatrix} \quad \text{- p15 coordinates : } \begin{pmatrix} x_{p15} \\ y_{p15} \end{pmatrix} := \begin{pmatrix} 550.5\text{m} \\ 1100\text{m} \end{pmatrix}$$

3.2 Matrice setup

$$i := 0, 1.. 200 \quad j := 0, 1.. 200$$

$$\Delta x := 5\text{m} \quad \Delta y := 5\text{m}$$

$$x_i := i \cdot \Delta x \quad y_j := j \cdot \Delta y$$

3.3 Drawdown from pumping wells

$$s_{p1}(x, y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x - x_{p1})^2 + (y - y_{p1})^2}}{r_o} \right] \right]^2} - 15\text{m}$$

$$s_{p2}(x, y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x - x_{p2})^2 + (y - y_{p2})^2}}{r_o} \right] \right]^2} - 15\text{m}$$

$$s_{p3}(x, y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x - x_{p3})^2 + (y - y_{p3})^2}}{r_o} \right] \right]^2} - 15\text{m}$$

$$s_{p4}(x, y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x - x_{p4})^2 + (y - y_{p4})^2}}{r_o} \right] \right]^2} - 15\text{m}$$

$$s_{p5}(x,y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x-x_{p5})^2 + (y-y_{p5})^2}}{r_o} \right] \right]} - 15r$$

$$s_{p11}(x,y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x-x_{p11})^2 + (y-y_{p11})^2}}{r_o} \right] \right]} - 15r$$

$$s_{p12}(x,y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x-x_{p12})^2 + (y-y_{p12})^2}}{r_o} \right] \right]} - 15r$$

$$s_{p13}(x,y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x-x_{p13})^2 + (y-y_{p13})^2}}{r_o} \right] \right]} - 15r$$

$$s_{p14}(x,y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x-x_{p14})^2 + (y-y_{p14})^2}}{r_o} \right] \right]} - 15r$$

$$s_{p15}(x,y) := \sqrt{B^2 + \left[\frac{Q_{p1}}{\pi \cdot K} \cdot \ln \left[\frac{\sqrt{(x-x_{p15})^2 + (y-y_{p15})^2}}{r_o} \right] \right]} - 15r$$

- Calculating drawdown for each wells

$$s_{p1i,j} := s_{p1}(x_i, y_j)$$

$$s_{p2i,j} := s_{p2}(x_i, y_j)$$

$$s_{p3i,j} := s_{p3}(x_i, y_j)$$

$$s_{p4i,j} := s_{p4}(x_i, y_j)$$

$$s_{p5i,j} := s_{p5}(x_i, y_j)$$

$$s_{p11i,j} := s_{p11}(x_i, y_j)$$

$$s_{p12i,j} := s_{p12}(x_i, y_j)$$

$$s_{p13i,j} := s_{p13}(x_i, y_j)$$

$$s_{p14i,j} := s_{p14}(x_i, y_j)$$

$$s_{p15i,j} := s_{p15}(x_i, y_j)$$

Total Drawdown :

$$\text{downgw1} := s_{p1} + s_{p2} + s_{p3} + s_{p4} + s_{p5}$$

$$\text{downgw2} := s_{p11} + s_{p12} + s_{p13} + s_{p14} + s_{p15}$$

$$\text{downgw} := \text{downgw1} + \text{downgw2} - 5r$$

Initial surface

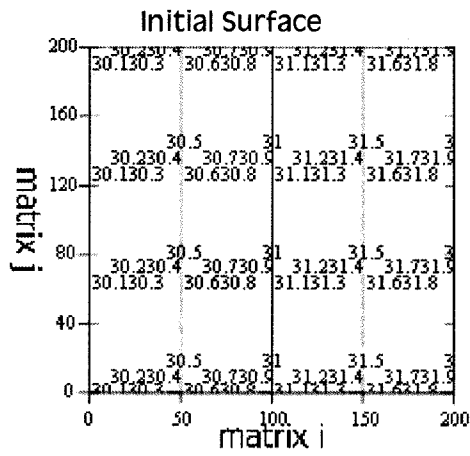
$$\text{boundary} := 32r$$

$$h_i := \text{boundary} - \text{slope} \cdot \Delta x \cdot (200 - i)$$

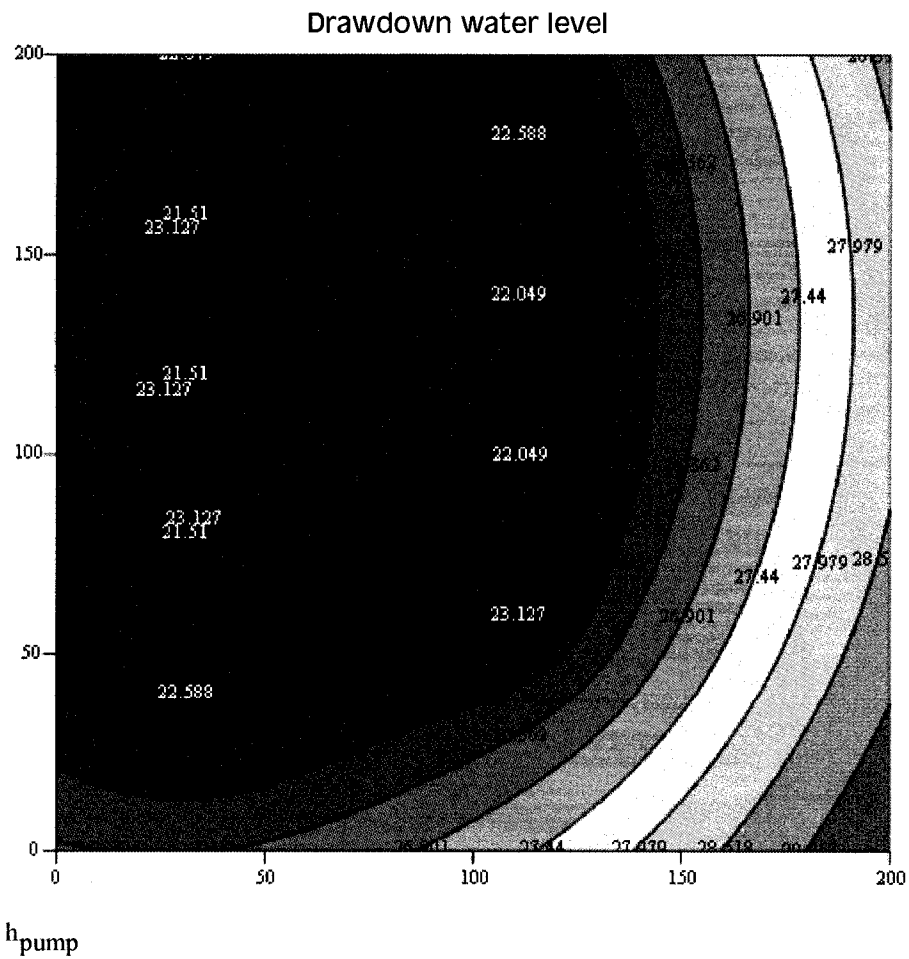
$$h_{\text{init},i,j} := h_i$$

$$h_{\text{pump}} := h_{\text{init}} + \text{downgw}$$

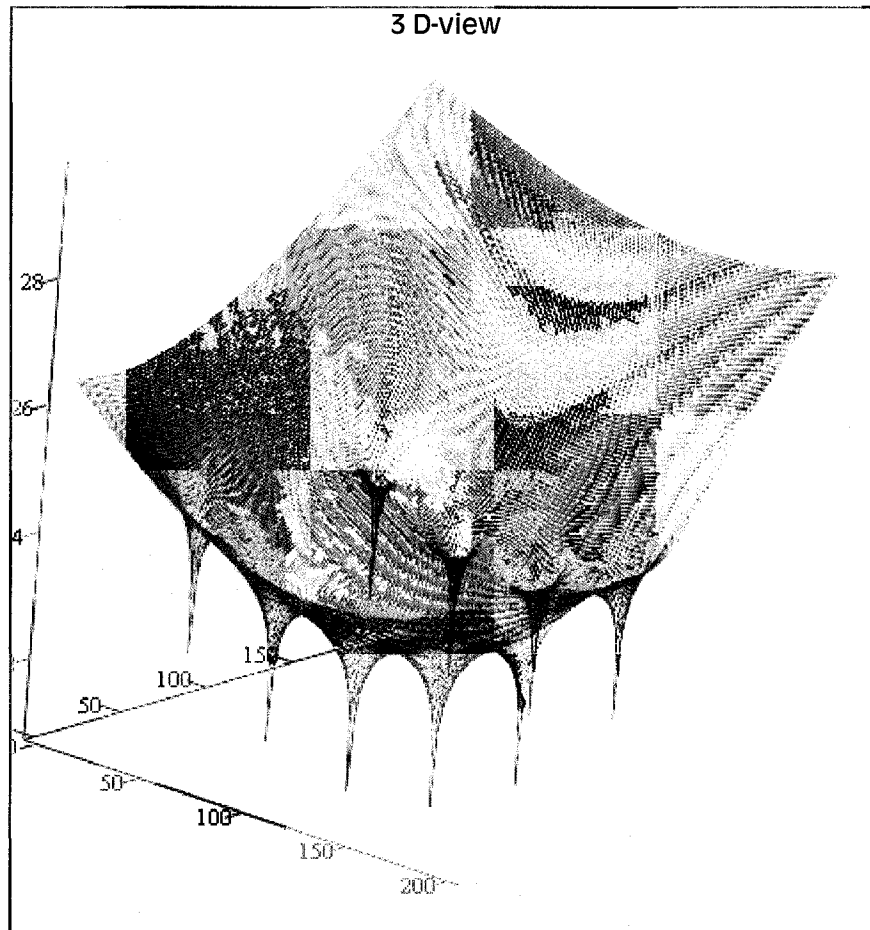
	0	1	2	3	4	5	6	7
0	-3.575	-3.603	-3.631	-3.659	-3.688	-3.716	-3.745	-3.774
1	-3.591	-3.62	-3.648	-3.677	-3.706	-3.735	-3.764	-3.793
2	-3.608	-3.637	-3.666	-3.694	-3.724	-3.753	-3.782	-3.812
3	-3.625	-3.653	-3.683	-3.712	-3.741	-3.771	-3.801	-3.831
4	-3.641	-3.67	-3.7	-3.729	-3.759	-3.789	-3.819	-3.85
5	-3.657	-3.687	-3.716	-3.746	-3.776	-3.807	-3.837	-3.868
6	-3.673	-3.703	-3.733	-3.763	-3.794	-3.824	-3.855	-3.886
downgw = 7	-3.689	-3.719	-3.749	-3.78	-3.811	-3.842	-3.873	-3.905
8	-3.704	-3.735	-3.766	-3.797	-3.828	-3.859	-3.891	-3.923
9	-3.72	-3.751	-3.782	-3.813	-3.844	-3.876	-3.908	-3.941
10	-3.735	-3.766	-3.797	-3.829	-3.861	-3.893	-3.925	-3.958
11	-3.75	-3.781	-3.813	-3.845	-3.877	-3.91	-3.942	-3.975
12	-3.764	-3.796	-3.828	-3.86	-3.893	-3.926	-3.959	-3.992
13	-3.779	-3.811	-3.843	-3.876	-3.909	-3.942	-3.975	-4.009
14	-3.793	-3.825	-3.858	-3.891	-3.924	-3.958	-3.992	-4.026
15	-3.807	-3.839	-3.872	-3.906	-3.939	-3.973	-4.007	-4.042



h_{init}



h_{pump}



h_{pump}

$j := 118..122$ distance : $5m \cdot \text{initial}(i)$ to $5m \cdot \text{end}(i) = 590m$ to $610m$

$$\text{diff150}_j := h_{\text{pump}30,j} - h_{\text{init}30,j} \quad \sum_{j=118}^{122} \frac{\text{diff150}_j}{4} = -10.3 \text{ m}$$

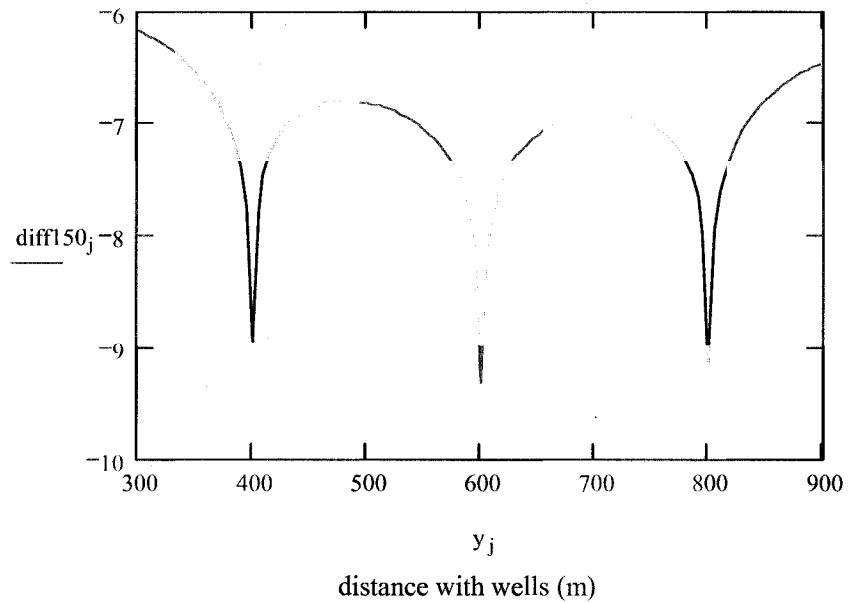
$$\text{diff550}_j := h_{\text{pump}110,j} - h_{\text{init}110,j} \quad \sum_{j=118}^{122} \frac{\text{diff550}_j}{4} = -8.656 \text{ m}$$

$j := 60..180$

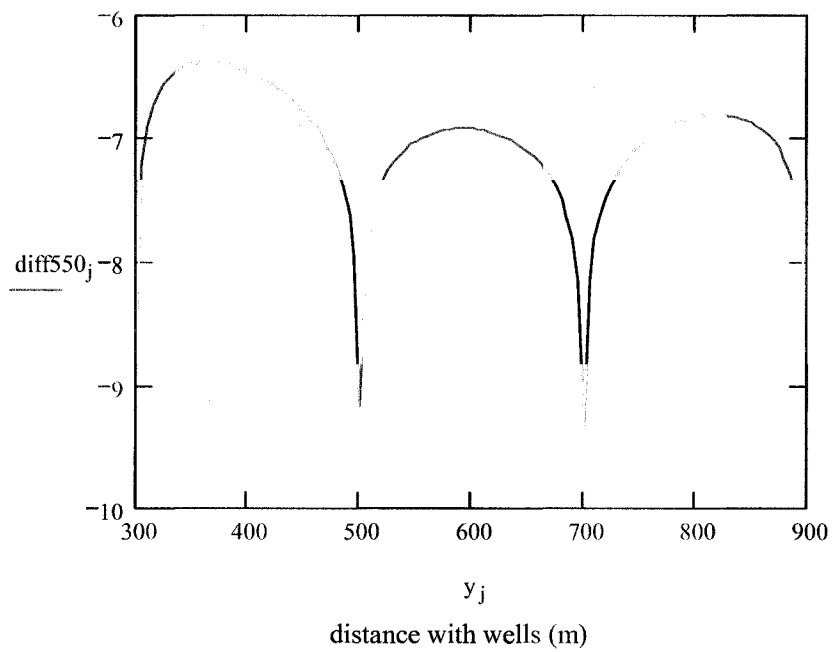
$\text{diff150}_j := h_{\text{pump}_{30,j}} - h_{\text{init}_{30,j}}$

$\text{diff550}_j := h_{\text{pump}_{110,j}} - h_{\text{init}_{110,j}}$

Drawdown of 1st well line



Drawdown of 2nd wells line



APPENDIX-B : Streamflow Depletion Factor using MATLAB

```
% response coefficients for four wells in the baseflow system for  
% 10days(1/3month) planning period
```

```
% distance(d) = 50m from stream, td=transmissivity(m2/day)  
% s=specific yield, t1=time(days), tp=pumping stop time  
% t= pumping time  
% 1-365 days analysis  
% 10days analysis for water demand variation
```

```
d=50;  
td=240;  
s=0.22;  
t1=1:360;  
tp=10;  
sum1=0.;  
for t=1:1:10  
    x=sqrt((d^2*s)/(4*td*t));  
    tau50(t) = erfc(x);  
    sum1=sum1+tau50(t);  
end  
sum2=0.;  
for t=11:1:20  
    x=sqrt((d^2*s)/(4*td*t));  
    x1=sqrt((d^2*s)/(4*td*(t-tp)));  
    tau50(t) = erfc(x) - erfc(x1);  
    sum2=sum2+tau50(t);  
end  
sum3=0.;  
for t=20:1:30  
    x=sqrt((d^2*s)/(4*td*t));  
    x1=sqrt((d^2*s)/(4*td*(t-tp)));  
    tau50(t) = erfc(x) - erfc(x1);  
    sum3=sum3+tau50(t);  
end  
sum4=0.;  
for t=30:1:40  
    x=sqrt((d^2*s)/(4*td*t));  
    x1=sqrt((d^2*s)/(4*td*(t-tp)));  
    tau50(t) = erfc(x) - erfc(x1);  
    sum4=sum4+tau50(t);  
end  
sum5=0.;  
for t=40:1:50  
    x=sqrt((d^2*s)/(4*td*t));
```

```

    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum5=sum5+tau50(t);
end
    sum6=0.;
for t=50:1:60
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum6=sum6+tau50(t);
end
    sum7=0.;
for t=60:1:70
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum7=sum7+tau50(t);
end
    sum8=0.;
for t=70:1:80
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum8=sum8+tau50(t);
end
    sum9=0.;
for t=80:1:90
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum9=sum9+tau50(t);
end
    sum10=0.;
for t=90:1:100
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum10=sum10+tau50(t);
end
    sum11=0.;
for t=100:1:110
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum11=sum11+tau50(t);
end
end

```

```

sum12=0.;
for t=110:1:120
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum12=sum12+tau50(t);
end
sum13=0.;
for t=120:1:130
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum13=sum13+tau50(t);
end
sum14=0.;
for t=130:1:140
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum14=sum14+tau50(t);
end
sum15=0.;
for t=140:1:150
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum15=sum15+tau50(t);
end
sum16=0.;
for t=150:1:160
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum16=sum16+tau50(t);
end
sum17=0.;
for t=160:1:170
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum17=sum17+tau50(t);
end
sum18=0.;
for t=170:1:180
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));

```

```

    tau50(t) = erfc(x) - erfc(x1);
    sum18=sum18+tau50(t);
end
    sum19=0.;
for t=180:1:190
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum19=sum19+tau50(t);
end
    sum20=0.;
for t=190:1:200
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum20=sum20+tau50(t);
end
    sum21=0.;
for t=200:1:210
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum21=sum21+tau50(t);
end
    sum22=0.;
for t=210:1:220
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum22=sum22+tau50(t);
end
    sum23=0.;
for t=220:1:230
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum23=sum23+tau50(t);
end
    sum24=0.;
for t=230:1:240
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum24=sum24+tau50(t);
end
    sum25=0.;

```

```

for t=240:1:250
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum25=sum25+tau50(t);
end
sum26=0.;
for t=250:1:260
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum26=sum26+tau50(t);
end
sum27=0.;
for t=260:1:270
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum27=sum27+tau50(t);
end
sum28=0.;
for t=270:1:280
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum28=sum28+tau50(t);
end
sum29=0.;
for t=280:1:290
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum29=sum29+tau50(t);
end
sum30=0.;
for t=290:1:300
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);
    sum30=sum30+tau50(t);
end
sum31=0.;
for t=300:1:310
    x=sqrt((d^2*s)/(4*td*t));
    x1=sqrt((d^2*s)/(4*td*(t-tp)));
    tau50(t) = erfc(x) - erfc(x1);

```

```

sum31=sum31+tau50(t);
end
sum32=0.;
for t=310:1:320
x=sqrt((d^2*s)/(4*td*t));
x1=sqrt((d^2*s)/(4*td*(t-tp)));
tau50(t) = erfc(x) - erfc(x1);
sum32=sum32+tau50(t);
end
sum33=0.;
for t=320:1:330
x=sqrt((d^2*s)/(4*td*t));
x1=sqrt((d^2*s)/(4*td*(t-tp)));
tau50(t) = erfc(x) - erfc(x1);
sum33=sum33+tau50(t);
end
sum34=0.;
for t=330:1:340
x=sqrt((d^2*s)/(4*td*t));
x1=sqrt((d^2*s)/(4*td*(t-tp)));
tau50(t) = erfc(x) - erfc(x1);
sum34=sum34+tau50(t);
end
sum35=0.;
for t=340:1:350
x=sqrt((d^2*s)/(4*td*t));
x1=sqrt((d^2*s)/(4*td*(t-tp)));
tau50(t) = erfc(x) - erfc(x1);
sum35=sum35+tau50(t);
end
sum36=0.;
for t=350:1:360
x=sqrt((d^2*s)/(4*td*t));
x1=sqrt((d^2*s)/(4*td*(t-tp)));
tau50(t) = erfc(x) - erfc(x1);
sum36=sum36+tau50(t);
end

```

```

sum1001=sum1+sum2+sum3+sum4+sum5+sum6+sum7+sum8+sum9+sum10+sum11+s
um12;
sum1002=sum13+sum14+sum15+sum16+sum17+sum18+sum19+sum20+sum21+sum22
+sum23+sum24;
sum1003=sum25+sum26+sum27+sum28+sum29+sum30+sum31+sum32+sum33+sum34
+sum35+sum36;

```

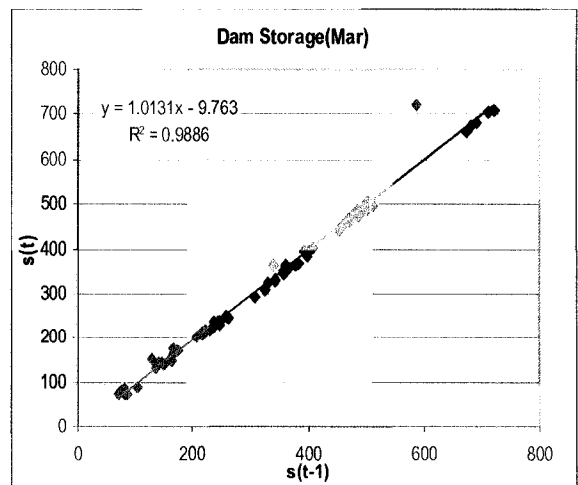
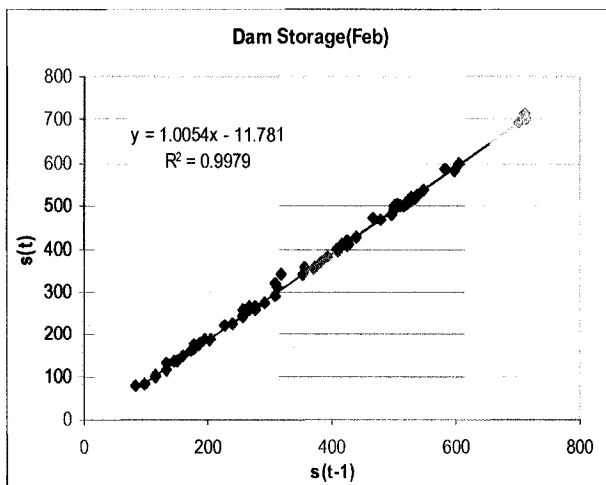
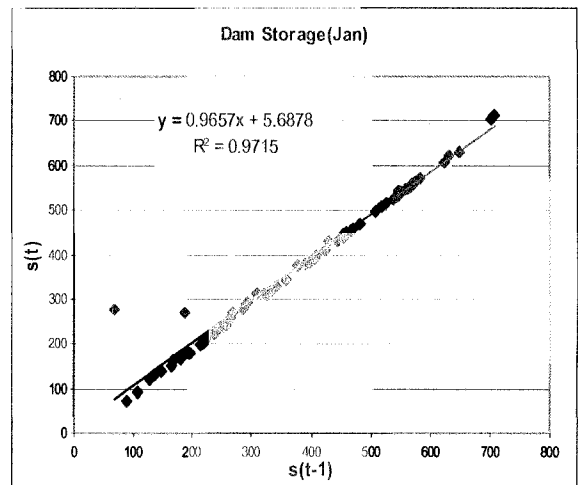
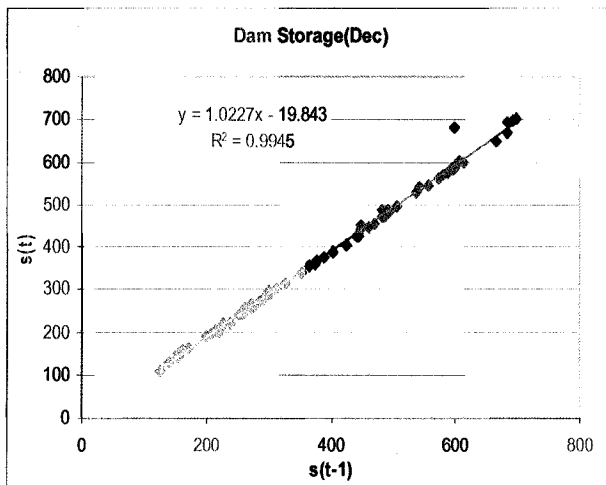
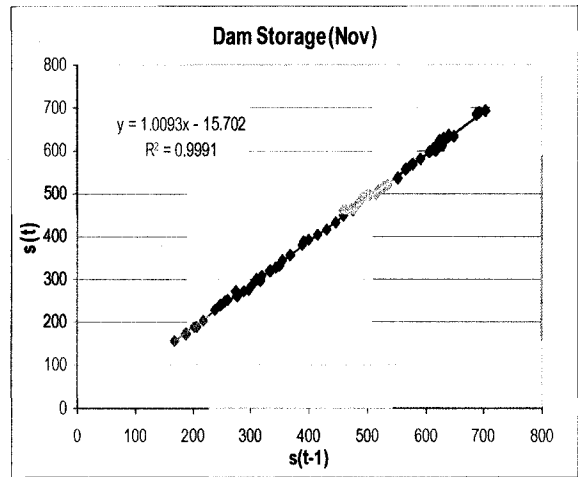
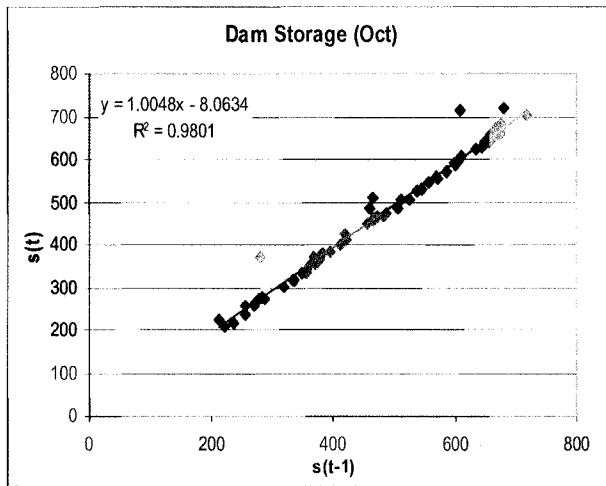
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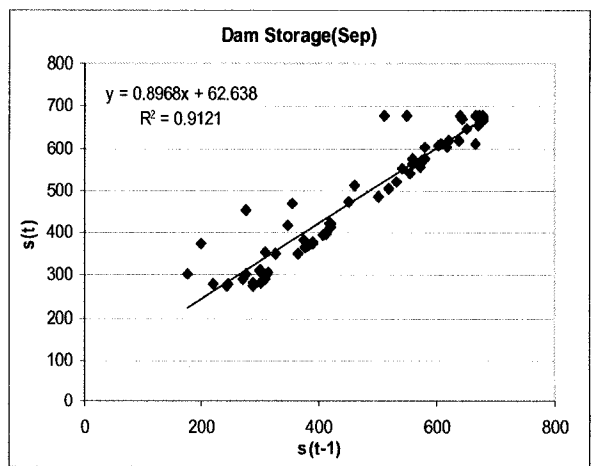
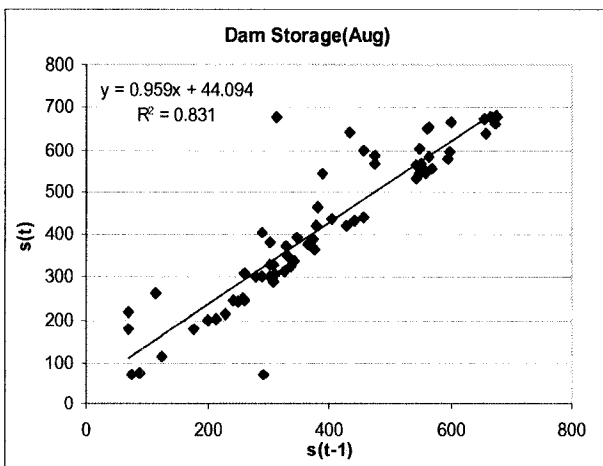
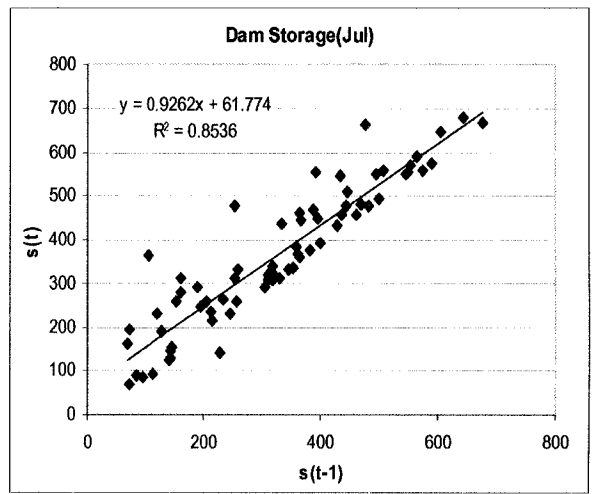
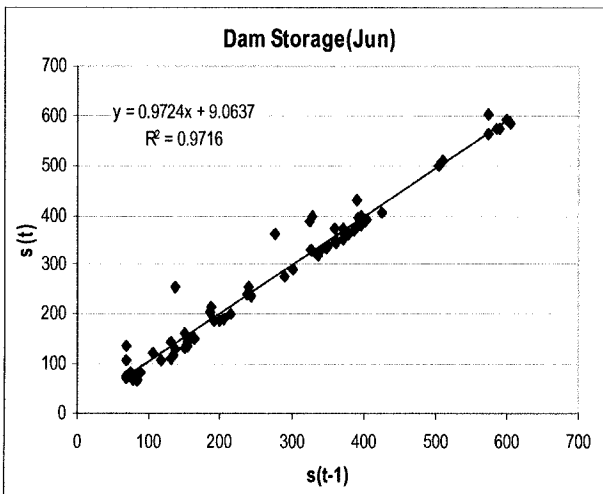
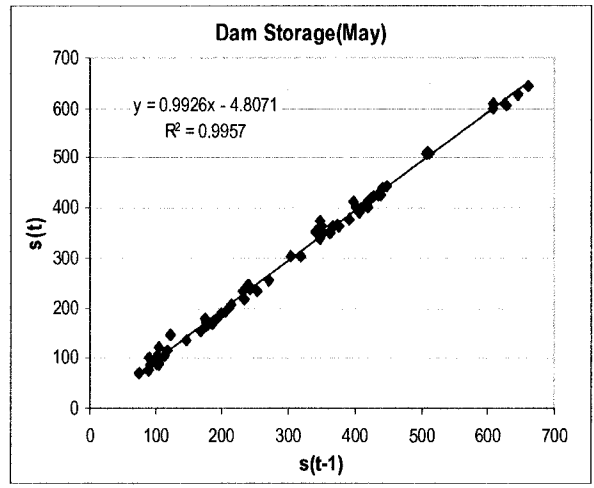
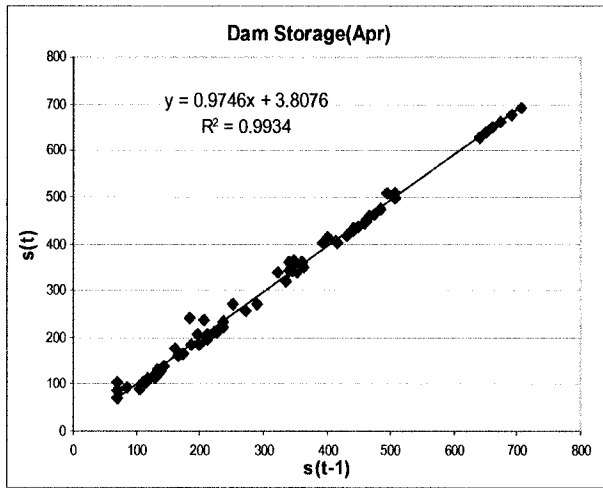
st50=sum1001+sum1002+sum1003;

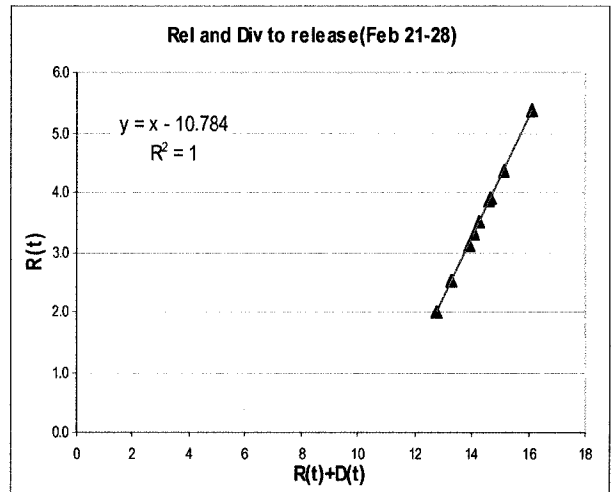
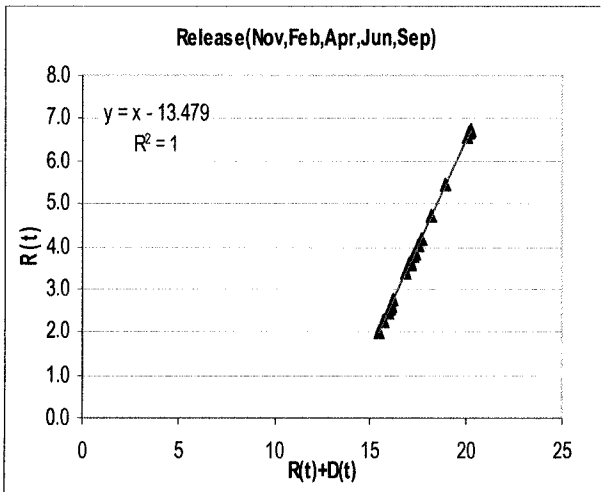
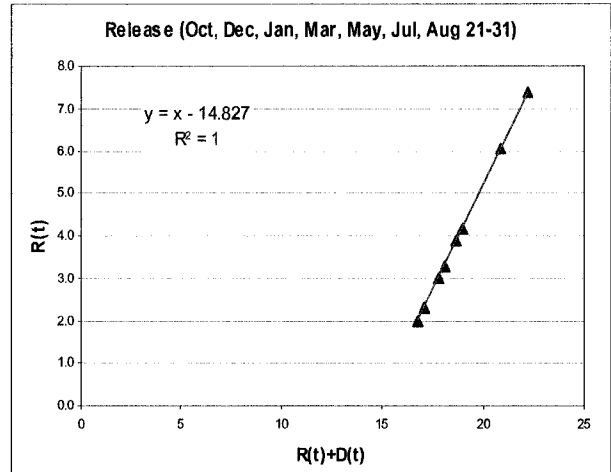
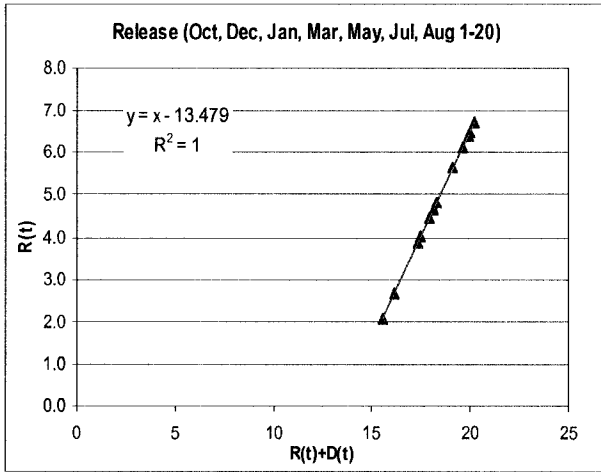
% sum = sum of daily response coefficient
% r = response coefficient each month ex)1-30days, 60-90days,
r1=sum1/st50;
r2=sum2/st50;
r3=sum3/st50;
r4=sum4/st50;
r5=sum5/st50;
r6=sum6/st50;
r7=sum7/st50;
r8=sum8/st50;
r9=sum9/st50;
r10=sum10/st50;
r11=sum11/st50;
r12=sum12/st50;
r13=sum13/st50;
r14=sum14/st50;
r15=sum15/st50;
r16=sum16/st50;
r17=sum17/st50;
r18=sum18/st50;
r19=sum19/st50;
r20=sum20/st50;
r21=sum21/st50;
r22=sum22/st50;
r23=sum23/st50;
r24=sum24/st50;
r25=sum25/st50;
r26=sum26/st50;
r27=sum27/st50;
r28=sum28/st50;
r29=sum29/st50;
r30=sum30/st50;
r31=sum31/st50;
r32=sum32/st50;
r33=sum33/st50;
r34=sum34/st50;
r35=sum35/st50;
r36=sum36/st50;
r1001=r1+r2+r3+r4+r5+r6+r7+r8+r9+r10+r11+r12;
r1002=r13+r14+r15+r16+r17+r18+r19+r20+r21+r22+r23+r24;
r1003=r25+r26+r27+r28+r29+r30+r31+r32+r33+r34+r35+r36;
rt50=r1001+r1002+r1003;

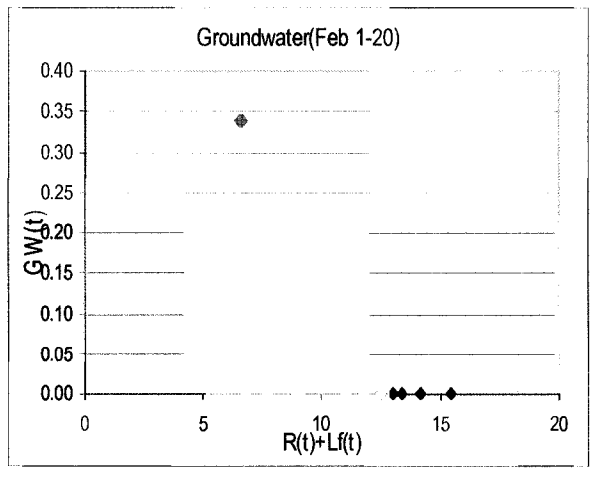
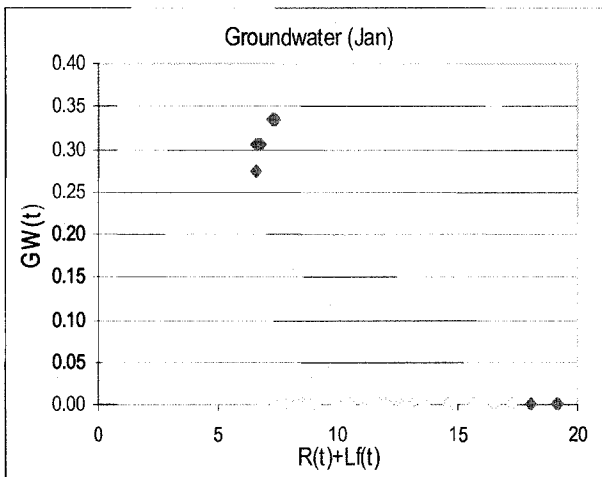
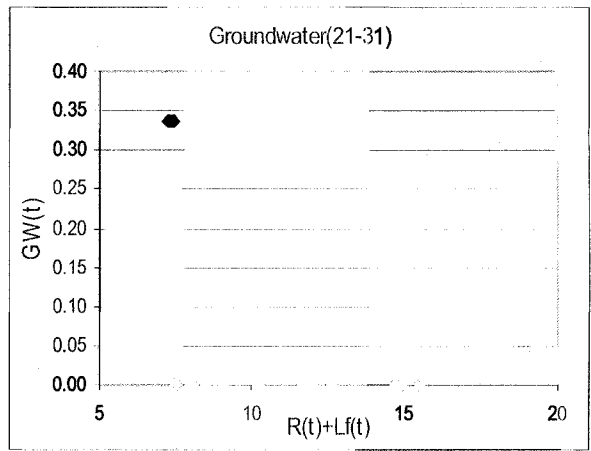
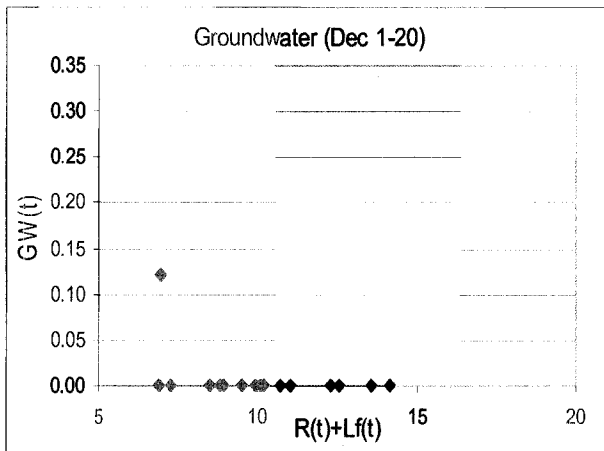
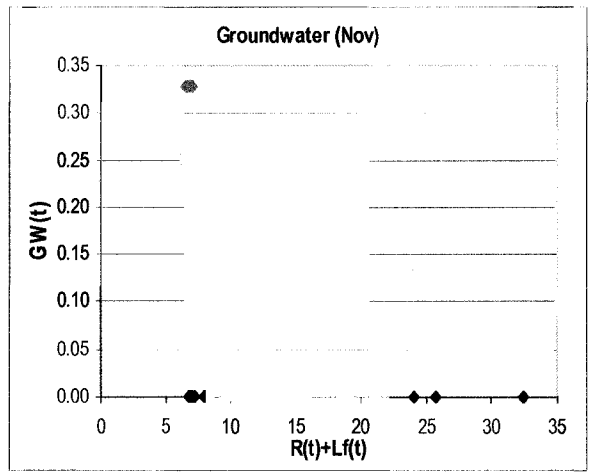
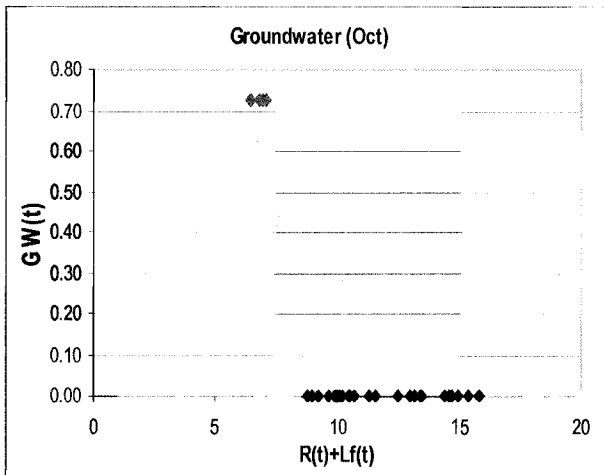
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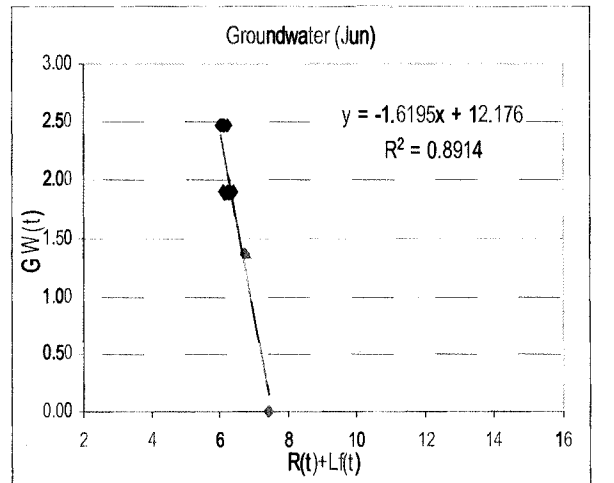
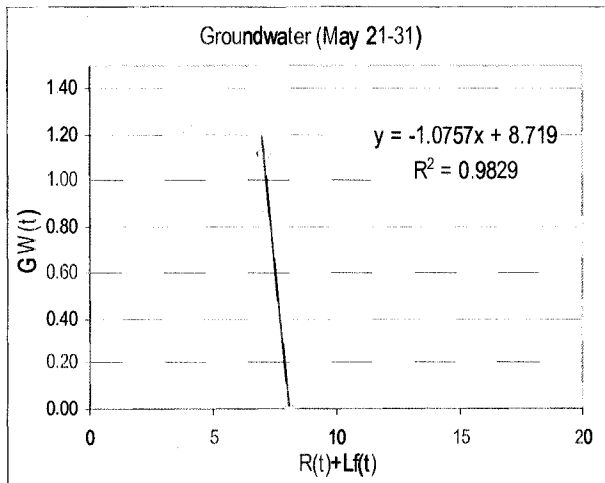
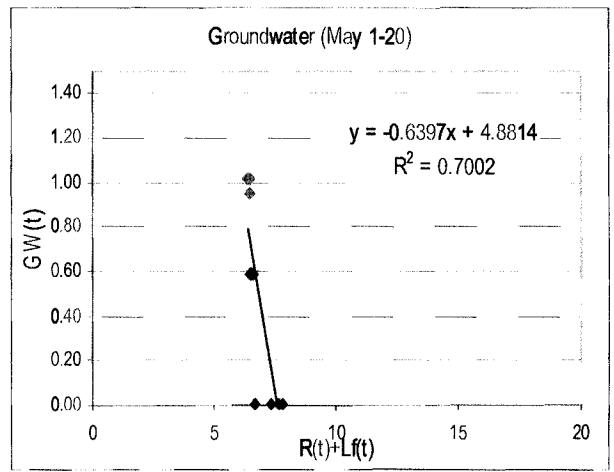
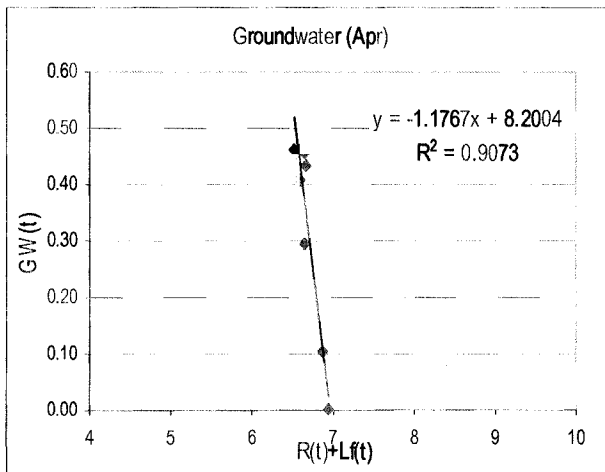
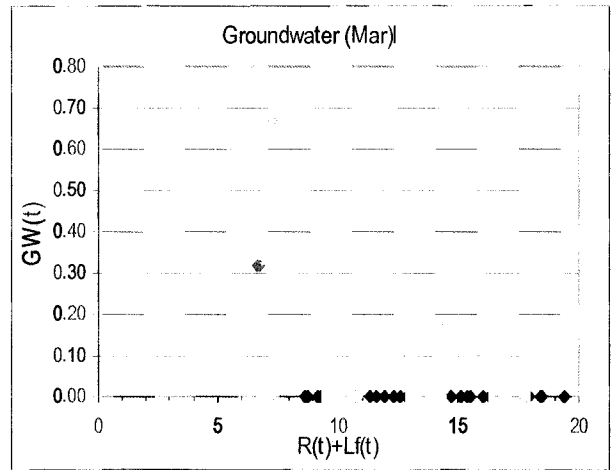
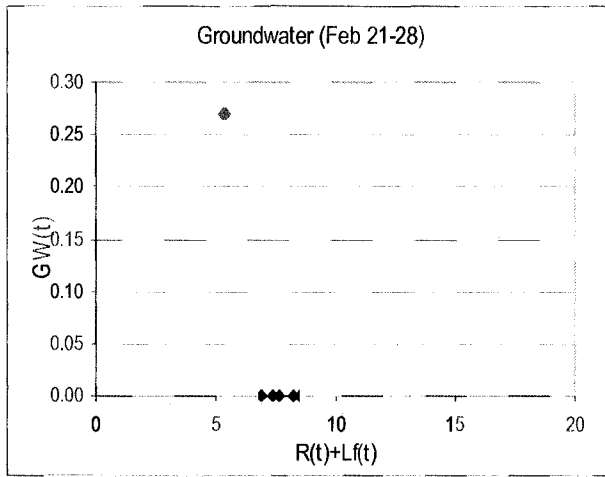
APPENDIX-C : Storage Rule based Operational Guide Curves

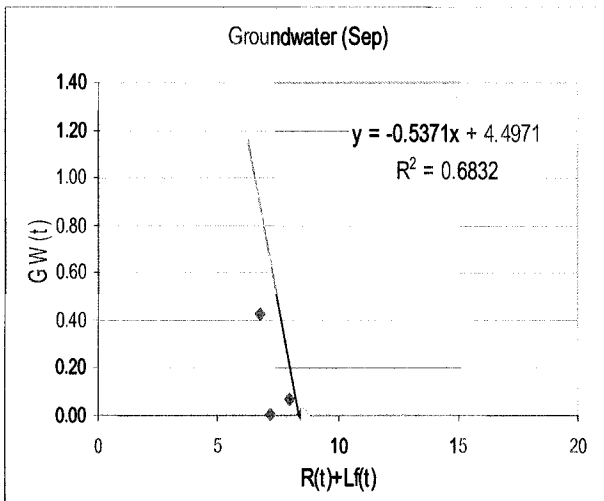
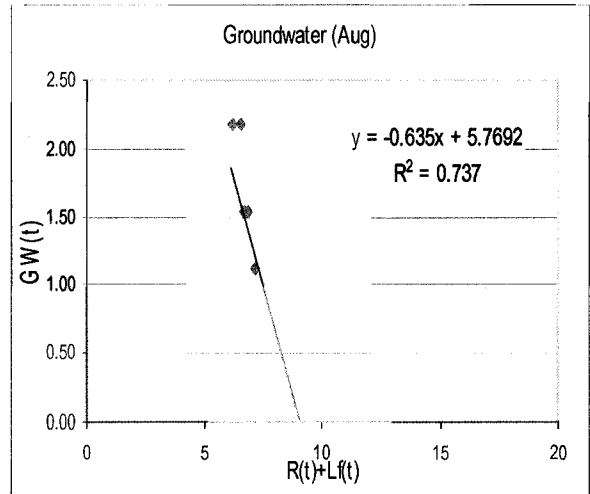
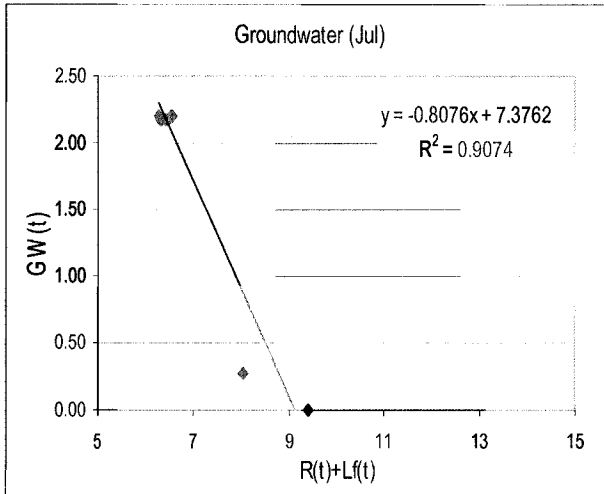












APPENDIX-D Conjunctive Water Management Systems using LINGO

D-1) Stream-Aquifer-Reservoir System

SETS:

POOL: GMX, VAL, INV0;

TIME: SPSUR, DDEF, DSUR, QSM, MINQ, T_MONTH;

MONTHt : R_COEF1, R_COEF2;

MONTHS;

WDT;

DEF;

GW;

SW;

GM;

param : p_mi, p_agri, sw_mi, sw_agri;

TXP(TIME, POOL): IMAX, IMIN, XGEN, XSPIL, INV, INFLO;

PXP(POOL, POOL);

TXG(TIME, GW): QSD, CGW;

TXS(TIME, SW): XWS, CSW, QDEF, QSUR;

TXWD(TIME, WDT): WD, WDEF;

TGM(TIME, GM): GMG;

ENDSETS

DATA:

TIME = 1..864;

POOL = PL1 PL2 PL3 PL4 PL5 PL6 PL7 PL8 PL9;

WDT = WD1 WD2 WD3 WD4 WD5 WD6;

DEF = DEF1 DEF2 DEF3 DEF4 DEF5 DEF6;

MONTHS = 1..36;

MONTHt = 1..36;

param = 1..2;

GW = 1..4;

SW = 1..5;

GM= 1;

ENDDATA

$$\begin{aligned} \text{MIN} = & @\text{SUM}(\text{TIME}(t): \text{QSM}(T)*100) + @\text{SUM}(\text{TIME}(T) : 1*(\text{DDEF}(T)+\text{DSUR}(T)) \\ & + 0.001*\text{SPSUR}(T)) + @\text{SUM}(\text{TXS}(T,P) | P \# \text{GE} \# 2 \# \text{AND} \# P \# \text{LE} \# 5 : \\ & 100*(\text{QDEF}(T,P) + \text{QSUR}(T,P))) + @\text{SUM}(\text{TXS}(T,S) : \text{csw}(T,S)) + \\ & @\text{SUM}(\text{TXG}(T,G) : \text{cgw}(T,G)*1.2) ; \end{aligned}$$

! state equation : Ending storage = beginning storage + inflow - outflow + return flow;

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 1:

INV(T,P) = INV0(P) + INFLO(T,P) - XGEN(T,P) - XSPIL(T,P) - XWS(T,P) ;

WDEF(T,P) = XWS(T,P) - WD(T,P);

XWS(T,P) - WD(T,P) + DDEF(T) - DSUR(T) = 0;

XSPIL(T,P) - SPSUR(T) = 0;

);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 2:

INV(T,6) = INV0(6) + INFLO(T,6) - XGEN(T,6) ;

INV(T,P) = INV0(P) + INFLO(T,2) - XGEN(T,P) - XSPIL(T,P) - XWS(T,P)

+ XGEN(T, 1) + XSPIL(T, 1) ;

XGEN(T,6) + XWS(T,P) <= WD(T,P) ;

WDEF(T,P) = XGEN(T,6) + XWS(T,P) - WD(T,P) ;

WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;

csw(t,p) = sw_mi(1)*xws(t,p) + sw_mi(2);

cgw(t,1) = p_mi(1)*xgen(t,6) + p_mi(2) ; ! cost for M&I;

);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 3:

INV(T,7) = INV0(7) + INFLO(T,7) - XGEN(T,7) ;

INV(T,P) = INV0(P) - XGEN(T,P) - XSPIL(T,P) - XWS(T,P) + XGEN(T, 2) +

XSPIL(T, 2) ;

XGEN(T,7) + XWS(T,P) <= WD(T,P) ;

WDEF(T,P) = XGEN(T,7) + XWS(T,P) - WD(T,P) ;

WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;

csw(t,p) = sw_agri(1)*xws(t,p) + sw_agri(2);

cgw(t,2) = p_agri(1)*xgen(t,7) + p_agri(2) ; ! cost for Agri;

);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 4:

INV(T,8) = INV0(8) + INFLO(T,8) - XGEN(T,8) ;

INV(T,P) = INV0(P) + INFLO(T,3) - XGEN(T,P) - XSPIL(T,P) - XWS(T,P)

+ XGEN(T, 3) + XSPIL(T, 3) ;

XGEN(T,8) + XWS(T,P) <= WD(T,P) ;

WDEF(T,P) = XGEN(T,8) + XWS(T,P) - WD(T,P) ;

WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;

csw(t,p) = sw_mi(1)*xws(t,p) + sw_mi(2);

cgw(t,3) = p_mi(1)*xgen(t,8) + p_mi(2) ;

);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 5:

INV(T,9) = INV0(9) + INFLO(T,9) - XGEN(T,9) ;

INV(T,P) = INV0(P) - XGEN(T,P) - XSPIL(T,P) - XWS(T,P) + XGEN(T, 4) +

XSPIL(T, 4) ;

XGEN(T,9) + XWS(T,P) <= WD(T,P) ;

```

WDEF(T,P) = XGEN(T,9) + XWS(T,P) - WD(T,P) ;
WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;
csw(t,p) = sw_agri(1)*xws(t,p) + sw_agri(2);
cgw(t,4) = p_agri(1)*xgen(t,9) + p_agri(2);
);
! minimum requirement flow downstream of reservoir ;
@FOR( TXWD(T,S)| T #LE#2 #AND# S #EQ# 6:
  MINQ(T) = XGEN(T,5) + XSPIL(T,5) + 0.6*(XGEN(T,8) + XWS(T,4)) +
  0.3*(XGEN(T,9) + XWS(T,5)) ;
  wdef(t,6) = MINQ(T) - WD(T,S);
  MINQ(T) - WD(T,S) + QSM(T) >= 0;
);
@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 1:
  INV(T,P) = INV(T-1,P) + INFLO(T,P) - XGEN(T,P) - XSPIL(T,P) - XWS(T,P) ;
  WDEF(T,P) = XWS(T,P) - WD(T,P);
  XWS(T,P) - WD(T,P) + DDEF(T) - DSUR(T) = 0;
  XSPIL(T,P) - SPSUR(T) = 0;
);
@FOR( TXP(T,P)| T #EQ# 864 #AND# P #GE# 6:
  INV(T,6) >= 30 ;
  INV(T,7) >= 30 ;
  INV(T,8) >= 30 ;
  INV(T,9) >= 30 ;
);

! initial streamflow depletion values ;
@FOR( TXG(T,G)| T #LT# 36 #AND# G #LE# 4:
  QSD(T,G) = 0.0 ;
);
@FOR( TXP(T,P)| T #GE# 36 #AND# P #EQ# 6:
  QSD(T,1) = @SUM(MONTHS(I)| I #GT# 1 #and# I #LT# 36 : XGEN(T - I + 1, P)*
  R_COEF2(I)) ;
);
@FOR( TXP(T,P)| T #GE# 36 #AND# P #EQ# 7:
  QSD(T,2) = @SUM(MONTHS(I)| I #GT# 1 #and# I #LT# 36 : XGEN(T - I + 1, P)*
  R_COEF2(I)) ;
);
@FOR( TXP(T,P)| T #GE# 36 #AND# P #EQ# 8:
  QSD(T,3) = @SUM(MONTHS(I)| I #GT# 1 #and# I #LT# 36 : XGEN(T - I + 1, P)*
  R_COEF2(I)) ;
);
@FOR( TXP(T,P)| T #GE# 36 #AND# P #EQ# 9:
  QSD(T,4) = @SUM(MONTHS(I)| I #GT# 1 #and# I #LT# 36 : XGEN(T - I + 1, P)*
  R_COEF2(I)) ;
);

```

! dummy reservoir state equation from 2nd period for #2 reservoir;

```
@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 2:
  INV(T,6) = INV(T-1,6) + INFLO(T,6) - XGEN(T,6) ;
  INV(T,P) = INV(T-1,P) + INFLO(T,P) + XGEN(T,1) + XSPIL(T,1)
    - XGEN(T,P) - XSPIL(T,P) - XWS(T,P) - xgen(t,6)*r_coef2(1) - QSD(T,1) ;
  XGEN(T,6) + XWS(T,P) <= WD(T,P) ;
  WDEF(T,P) = XGEN(T,6) + XWS(T,P) - WD(T,P) ;
  WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;
  csw(t,p) = sw_mi(1)*xws(t,p) + sw_mi(2);
  cgw(t,1) = p_mi(1)*xgen(t,6) + p_mi(2) ; ! cost for M&I;
);
```

! dummy reservoir state equation from 2nd period for #3 reservoir;

```
@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 3:
  INV(T,7) = INV(T-1,7) + INFLO(T,7) - XGEN(T,7) ;
  INV(T,P) = INV(T-1,P) + XGEN(T,2) + XSPIL(T,2) - XGEN(T,P) - XSPIL(T,P)
    - XWS(T,P) - xgen(t,7)*r_coef2(1) - QSD(T,2) ;
  XGEN(T,7) + XWS(T,P) <= WD(T,P) ;
  WDEF(T,P) = XGEN(T,7) + XWS(T,P) - WD(T,P) ;
  WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;
  csw(t,p) = sw_agri(1)*xws(t,p) + sw_agri(2);
  cgw(t,2) = p_agri(1)*xgen(t,7) + p_agri(2); ! cost for Agri;
);
```

```
@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 4:
  INV(T,8) = INV(T-1,8) + INFLO(T,8) - XGEN(T,8) ;
  INV(T,P) = INV(T-1,P) + INFLO(T,3) + XGEN(T,3) + XSPIL(T,3) - XGEN(T,P) -
  XSPIL(T,P) - XWS(T,P) - xgen(t,8)*r_coef2(1) - QSD(T,3) + 0.6*(XGEN(T,6) +
  XWS(T,2)) + 0.3*0.7*(XGEN(T,7) + XWS(T,3)) + 0.3*0.2*(XGEN(T-1,7) +
  XWS(T-1,3)) + 0.3*0.1*(XGEN(T-2,7) + XWS(T-2,3)) ;
  XGEN(T,8) + XWS(T,P) <= WD(T,P);
  WDEF(T,P) = XGEN(T,8) + XWS(T,P) - WD(T,P) ;
  WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;
  csw(t,p) = sw_mi(1)*xws(t,p) + sw_mi(2);
  cgw(t,3) = p_mi(1)*xgen(t,8) + p_mi(2) ;
);
```

! dummy reservoir state equation from 2nd period for #5 reservoir;

```
@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 5:
  INV(T,9) = INV(T-1,9) + INFLO(T,9) - XGEN(T,9) ;
  INV(T,P) = INV(T-1,P) + XGEN(T,4) + XSPIL(T,4) - XGEN(T,5) - XSPIL(T,5)
    - XWS(T,P) - xgen(t,9)*r_coef2(1) - QSD(T,4) ;
  XGEN(T,9) + XWS(T,P) <= WD(T,P) ;
  WDEF(T,P) = XGEN(T,9) + XWS(T,P) - WD(T,P) ;
  WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;
```

```

csw(t,p) = sw_agri(1)*xws(t,p) + sw_agri(2);
cgw(t,4) = p_agri(1)*xgen(t,9) + p_agri(2);
);

! minimum requirement flow downstream of reservoir ;
@FOR( TXWD(T,S)| T #GT#2 #AND# S #EQ# 6:
  MINQ(T) = XGEN(T,5) + XSPIL(T,5) + 0.6*(XGEN(T,8) + XWS(T,4))
    + 0.3*0.7*(XGEN(T,9) + XWS(T,5)) + 0.3*0.2*(XGEN(T-1,9) + XWS(T-1,5))
    + 0.3*0.1*(XGEN(T-2,9) + XWS(T-2,5));
  wdef(t,6) = MINQ(T) - WD(T,S);
  MINQ(T) - WD(T,S) + QSM(T) >= 0;
);

! Bounds. Cannot store more in a pool than capacity.
@FOR(TXP(T,P)| P #EQ# 1 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0.0, XSPIL(T,P), 500);
  @BND( 2.0, XGEN(T,P), GMG(t,1));
  @BND( 0.0, XWS(T,P), 17.0);
);
@FOR(TXP(T,P)| P #EQ# 1 :
  @free( wdef(t,p));
);
@FOR(TXP(T,P)| P #GE# 2 #AND# P #LE# 5 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0, XGEN(T,P), 500);
  @BND( 0.0, XSPIL(T,P), 500);
  @free( wdef(t,p));
);
@FOR(TXP(T,P)| P #EQ# 6 :
  @free( wdef(t,p));
);
@FOR(TXP(T,P)| P #GE# 6 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0.0, XGEN(T,P), GMX(P));
);

```

D-2) Single Well Stream-Aquifer System

SETS:

POOL: GMX, VAL, INV0;
TIME: SPSUR, DDEF, DSUR, QSM, MINQ, T_MONTH, wd_down, wd_min; !
MIN_INSTREAM_FLOW, JAN TO DEC (12);
MONTHt : R_COEF1, R_COEF2, R_COEF3, R_COEF4; ! distance from the stream
with 50m, 100m, 200m, 500m ;
MONTHS;
WDT;
DEF;
GW;
SW;
GM;
param : p_mi, p_agri, sw_mi, sw_agri;
TXP(TIME, POOL): IMAX, IMIN, XGEN, XSPIL, INV, INFLO;
PXP(POOL, POOL);
TXG(TIME, GW): QSD, CGW;
TXS(TIME, SW): XWS, CSW, QDEF, QSUR;
TXWD(TIME, WDT): WD, WDEF;
TGM(TIME, GM): GMG;

ENDSETS

DATA:

TIME = 1..864;
POOL = PL1 PL2 PL3 PL4 PL5 PL6 PL7 PL8 PL9;
WDT = WD1 WD2 WD3 WD4 WD5 WD6;
DEF = DEF1 DEF2 DEF3 DEF4 DEF5 DEF6;
MONTHS = 1..36;
MONTHt = 1..36;
param = 1..2;
GW = 1..4;
SW = 1..5;
GM= 1;
ENDDATA

! Response coefficient values of streamflow depletion for 12 months;

! R_COEF : jan feb mar apr may jun jul aug sep oct nov dec;

! Minimize the value of water deficit, Dam spill, Groundwater release, and withdrawal and pumping cost;

MIN = @SUM(TIME(t): QSM(T)*100) +
 @SUM(TXS(T,P)| P #EQ#5 : 1*(QDEF(T,P) + QSUR(T,P))) +
 @SUM(TXS(T,S) : csw(T,S)) +
 @SUM(TXG(T,G) : cgw(T,G)*1.0);

! state equation;

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 1:
 INV(T,P) = INV0(P) + INFLO(T,P) - XGEN(T,P) - XSPIL(T,P);
);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 2:
 INV(T,P) = INV0(P) + INFLO(T,2) - XGEN(T,P) - XSPIL(T,P)
 + XGEN(T, 1) + XSPIL(T, 1);
);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 3:
 INV(T,P) = INV0(P) - XGEN(T,P) - XSPIL(T,P) + XGEN(T, 2) + XSPIL(T, 2);
);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 4:
 INV(T,P) = INV0(P) + INFLO(T,3) - XGEN(T,P) - XSPIL(T,P)
 + XGEN(T, 3) + XSPIL(T, 3);
);

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 5:
 INV(T,P) = INV0(P) - XGEN(T,P) - XSPIL(T,P) - XWS(T,P) + XGEN(T, 4) +
 XSPIL(T, 4);
 INV(T,6) = INV0(6) + INFLO(T,6) - XGEN(T,6);

XGEN(T,6) + XWS(T,P) <= WD_down(T);
 WDEF(T,P) = XGEN(T,6) + XWS(T,P) - WD_down(T);
 WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;
 csw(t,p) = sw_agri(1)*xws(t,p) + sw_agri(2);
 cgw(t,1) = p_agri(1)*xgen(t,6) + p_agri(2);
);

! minimum requirement flow downstream of reservoir + XSPIL(T,8);

@FOR(TXWD(T,S)| T #LE#2 #AND# S #EQ# 6:

MINQ(T) = XGEN(T,5) + XSPIL(T,5) + 0.3*(XGEN(T,6) + XWS(T,5));
 wdef(t,6) = MINQ(T) - WD_min(T);
 MINQ(T) - WD_min(T) + QSM(T) >= 0;

);

! Ending storage;

@FOR(TXP(T,P)| T #GT# 2 #AND# P #EQ# 1:

INV(T,P) = INV(T-1,P) + INFLO(T,P) - XGEN(T,P) - XSPIL(T,P) ;
);

! end storage of GW 6, 7, 8, 9 site ;

@FOR(TXP(T,P)| T #EQ# 864 #AND# P #GE# 6:

INV(T,6) >= 30 ;
);

! Streamflow depletion of sector 2, 3, 4, 5 due to the groundwater pumping;

! initial streamflow depletion values ;

@FOR(TXG(T,G)| T #LT# 36 #AND# G #LE# 4:

QSD(T,G) = 0.0 ;
);

@FOR(TXP(T,P)| T #GE# 36 #AND# P #EQ# 6:

QSD(T,1) = @SUM(MONTHS(I)| I #Ge# 1 #and# I #LT# 37 : XGEN(T - I + 1, P)*
R_COEF1(I)) ;
);

! dummy reservoir state equation from 2nd period for #2 reservoir;

@FOR(TXP(T,P)| T #GT# 2 #AND# P #EQ# 2:

INV(T,P) = INV(T-1,P) + INFLO(T,P) + XGEN(T,1) + XSPIL(T,1)
- XGEN(T,P) - XSPIL(T,P) ;
);

! dummy reservoir state equation from 2nd period for #3 reservoir;

@FOR(TXP(T,P)| T #GT# 2 #AND# P #EQ# 3:

INV(T,P) = INV(T-1,P) + XGEN(T,2) + XSPIL(T,2) - XGEN(T,P) - XSPIL(T,P);
);

! dummy reservoir state equation from 2nd period for #4 reservoir ;

@FOR(TXP(T,P)| T #GT# 2 #AND# P #EQ# 4:

INV(T,P) = INV(T-1,p) + INFLO(T,3) + XGEN(T,3) + XSPIL(T,3) - XGEN(T,P) -
XSPIL(T,P);
);

! dummy reservoir state equation from 2nd period for #5 reservoir;

@FOR(TXP(T,P)| T #GT# 2 #AND# P #EQ# 5:

INV(T,P) = INV(T-1,P) + XGEN(T,4) + XSPIL(T,4) - XGEN(T,p) - XSPIL(T,p)
- XWS(T,P) - (QSD(T,1)) ;

INV(T,6) = INV(T-1,6) + INFLO(T,6) - XGEN(T,6) ;

XGEN(T,6) + XWS(T,P) <= WD_down(T) ;

WDEF(T,P) = XGEN(T,6) + XWS(T,P) - WD_down(T) ;

WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;

csw(t,p) = sw_agri(1)*xws(t,p) + sw_agri(2);

```

cgw(t,1) = p_agri(1)*xgen(t,6) + p_agri(2);
);

! minimum requirement flow downstream of reservoir ;
@FOR( TXWD(T,S)| T #GT#2 #AND# S #EQ# 6:
  MINQ(T) = XGEN(T,5) + XSPIL(T,5)
    + 0.3*0.7*(XGEN(T,6) + XWS(T,5))
    + 0.3*0.2*(XGEN(T-1,6) + XWS(T-1,5))
    + 0.3*0.1*(XGEN(T-2,6) + XWS(T-2,5));
  wdef(t,6) = MINQ(T) - WD_min(T) ;
  MINQ(T) - WD_min(T) + QSM(T) >= 0 ;
);

! Boundary ;

@FOR(TXP(T,P)| P #EQ# 1 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0.0, XSPIL(T,P), 1000);
);

@FOR(TXP(T,P)| P #EQ# 1 :
  @free( wdef(t,p));
);

@FOR(TXP(T,P)| P #GE# 2 #AND# P #LE# 5 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0, XGEN(T,P), 1500);
  @BND( 0.0, XSPIL(T,P), 1500);
  @free( wdef(t,p));
);

@FOR(TXP(T,P)| P #EQ# 6 :
  @free( wdef(t,p));
);

@FOR(TXP(T,P)| P #GE# 6 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0.0, XGEN(T,P), GMX(P));
);

```

D-3) Multiple Well Stream-Aquifer System

SETS:

POOL: GMX, VAL, INV0;
TIME: SPSUR, DDEF, DSUR, QSM, MINQ, T_MONTH, wd_down, wd_min; !
MIN_INSTREAM_FLOW, JAN TO DEC (12);
MONTHt : R_COEF1, R_COEF2, R_COEF3, R_COEF4; ! distance from the stream
with 50m, 100m, 200m, 500m ;
MONTHS;
WDT;
DEF;
GW;
SW;
GM;
param : p_mi, p_agri, sw_mi, sw_agri;
TXP(TIME, POOL): IMAX, IMIN, XGEN, XSPIL, INV, INFLO;
PXP(POOL, POOL);
TXG(TIME, GW): QSD, CGW;
TXS(TIME, SW): XWS, CSW, QDEF, QSUR;
TXWD(TIME, WDT): WD, WDEF;
TGM(TIME, GM): GMG;

ENDSETS

DATA:

TIME = 1..864;
POOL = PL1 PL2 PL3 PL4 PL5 PL6 PL7 PL8 PL9;
WDT = WD1 WD2 WD3 WD4 WD5 WD6;
DEF = DEF1 DEF2 DEF3 DEF4 DEF5 DEF6;
MONTHS = 1..36;
MONTHt = 1..36;
param = 1..2;
GW = 1..4;
SW = 1..5;
GM= 1;
ENDDATA

! Minimize the value of water deficit, Dam spill, Groundwater release, and withdrawal and pumping cost;

$$\begin{aligned} \text{MIN} = & @\text{SUM}(\text{TIME}(t): \text{QSM}(T)*100) + \\ & @\text{SUM}(\text{TXS}(T,P) | P \# \text{EQ}\#5 : 1*(\text{QDEF}(T,P) + \text{QSUR}(T,P))) + \\ & @\text{SUM}(\text{TXS}(T,S) : \text{csw}(T,S)) + @\text{SUM}(\text{TXG}(T,G) : \text{cgw}(T,G)*1) ; \end{aligned}$$

! state equation : Ending storage = beginning storage + inflow - outflow + return flow;

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 1:

$$\text{INV}(T,P) = \text{INV0}(P) + \text{INFLO}(T,P) - \text{XGEN}(T,P) - \text{XSPIL}(T,P);$$

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 2:

$$\text{INV}(T,P) = \text{INV0}(P) + \text{INFLO}(T,2) - \text{XGEN}(T,P) - \text{XSPIL}(T,P) \\ + \text{XGEN}(T, 1) + \text{XSPIL}(T, 1);$$

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 3:

$$\text{INV}(T,P) = \text{INV0}(P) - \text{XGEN}(T,P) - \text{XSPIL}(T,P) + \text{XGEN}(T, 2) + \text{XSPIL}(T, 2);$$

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 4:

$$\text{INV}(T,P) = \text{INV0}(P) + \text{INFLO}(T,3) - \text{XGEN}(T,P) - \text{XSPIL}(T,P) \\ + \text{XGEN}(T, 3) + \text{XSPIL}(T, 3);$$

@FOR(TXP(T,P)| T #LE# 2 #AND# P #EQ# 5:

$$\text{INV}(T,P) = \text{INV0}(P) - \text{XGEN}(T,P) - \text{XSPIL}(T,P) - \text{XWS}(T,P) + \text{XGEN}(T, 4) + \\ \text{XSPIL}(T, 4); \\ \text{INV}(T,6) = \text{INV0}(6) + \text{INFLO}(T,6) - \text{XGEN}(T,6); \\ \text{INV}(T,7) = \text{INV0}(7) + \text{INFLO}(T,7) - \text{XGEN}(T,7); \\ \text{INV}(T,8) = \text{INV0}(8) + \text{INFLO}(T,8) - \text{XGEN}(T,8); \\ \text{INV}(T,9) = \text{INV0}(9) + \text{INFLO}(T,9) - \text{XGEN}(T,9);$$

$$\text{XGEN}(T,6) + \text{XGEN}(T,7) + \text{XGEN}(T,8) + \text{XGEN}(T,9) + \text{XWS}(T,P) \leq \\ \text{WD_down}(T);$$

$$\text{WDEF}(T,P) = \text{XGEN}(T,6) + \text{XGEN}(T,7) + \text{XGEN}(T,8) + \text{XGEN}(T,9) + \text{XWS}(T,P) - \\ \text{WD_down}(T);$$

$$\text{WDEF}(T,P) + \text{QDEF}(T,P) - \text{QSUR}(T,P) = 0;$$

$$\text{csw}(t,p) = \text{sw_agri}(1)*\text{xws}(t,p) + \text{sw_agri}(2);$$

$$\text{cgw}(t,1) = \text{p_agri}(1)*\text{xgen}(t,6) + \text{p_agri}(2);$$

$$\text{cgw}(t,2) = \text{p_agri}(1)*\text{xgen}(t,7) + \text{p_agri}(2);$$

$$\text{cgw}(t,3) = \text{p_agri}(1)*\text{xgen}(t,8) + \text{p_agri}(2);$$

$$\text{cgw}(t,4) = \text{p_agri}(1)*\text{xgen}(t,9) + \text{p_agri}(2);$$

);

! minimum requirement flow downstream of reservoir + XSPIL(T,8);

@FOR(TXWD(T,S)| T #LE#2 #AND# S #EQ# 6:

$$\text{MINQ}(T) = \text{XGEN}(T,5) + \text{XSPIL}(T,5) + 0.3*(\text{XGEN}(T,6) + \text{XGEN}(T,7) \\ + \text{XGEN}(T,8) + \text{XGEN}(T,9) + \text{XWS}(T,5));$$

```

wdef(t,6) = MINQ(T) - WD_min(T) ;
MINQ(T) - WD_min(T) + QSM(T) >= 0 ;

);

! Ending storage = beginning storage + inflow from outside world + ;
! inflow from upstream pools - outflow for gen and spill + return flow;

@FOR( TXP(T,P) | T #GT# 2 #AND# P #EQ# 1:
    INV(T,P) = INV(T-1,P) + INFLO(T,P) - XGEN(T,P) - XSPIL(T,P) ;
);

! end storage of GW 6, 7, 8, 9 site ;
@FOR( TXP(T,P) | T #EQ# 864 #AND# P #GE# 6:
    INV(T,6) >= 30 ;
    INV(T,7) >= 30 ;
    INV(T,8) >= 30 ;
    INV(T,9) >= 30 ;
);
!

! Streamflow depletion of sector 2, 3, 4, 5 due to the groundwater pumping;
! initial streamflow depletion values ;

@FOR( TXG(T,G) | T #LT# 36 #AND# G #LE# 4:
    QSD(T,G) = 0.0 ;
);

@FOR( TXP(T,P) | T #GE# 36 #AND# P #EQ# 6:
    QSD(T,1) = @SUM(MONTHS(I) | I #Ge# 1 #and# I #LT# 37 : XGEN(T - I + 1, P)*
R_COEF1(I)) ;
);

@FOR( TXP(T,P) | T #GE# 36 #AND# P #EQ# 7:
    QSD(T,2) = @SUM(MONTHS(I) | I #Ge# 1 #and# I #LT# 37 : XGEN(T - I + 1, P)*
R_COEF2(I)) ;
);

@FOR( TXP(T,P) | T #GE# 36 #AND# P #EQ# 8:
    QSD(T,3) = @SUM(MONTHS(I) | I #Ge# 1 #and# I #LT# 37 : XGEN(T - I + 1, P)*
R_COEF3(I)) ;
);

@FOR( TXP(T,P) | T #GE# 36 #AND# P #EQ# 9:
    QSD(T,4) = @SUM(MONTHS(I) | I #Ge# 1 #and# I #LT# 37 : XGEN(T - I + 1, P)*
R_COEF4(I)) ;
);

```

```

);

! dummy reservoir state equation from 2nd period for #2 reservoir;

@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 2:
  INV(T,P) = INV(T-1,P) + INFLO(T,P) + XGEN(T,1) + XSPIL(T,1)
    - XGEN(T,P) - XSPIL(T,P) ;
);

! dummy reservoir state equation from 2nd period for #3 reservoir;

@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 3:
  INV(T,P) = INV(T-1,P) + XGEN(T,2) + XSPIL(T,2) - XGEN(T,P) - XSPIL(T,P);
);

! dummy reservoir state equation from 2nd period for #4 reservoir ;

@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 4:
  INV(T,P) = INV(T-1,p) + INFLO(T,3) + XGEN(T,3) + XSPIL(T,3) - XGEN(T,P) -
    XSPIL(T,P);
);

@FOR( TXP(T,P)| T #GT# 2 #AND# P #EQ# 5:
  INV(T,P) = INV(T-1,P) + XGEN(T,4) + XSPIL(T,4) - XGEN(T,p) - XSPIL(T,p)
    - XWS(T,P)
    - ( QSD(T,1) + QSD(T,2) + QSD(T,3) + QSD(T,4) );

  INV(T,6) = INV(T-1,6) + INFLO(T,6) - XGEN(T,6) ;
  INV(T,7) = INV(T-1,7) + INFLO(T,7) - XGEN(T,7) ;
  INV(T,8) = INV(T-1,8) + INFLO(T,8) - XGEN(T,8) ;
  INV(T,9) = INV(T-1,9) + INFLO(T,9) - XGEN(T,9) ;

  XGEN(T,6) + XGEN(T,7) + XGEN(T,8) + XGEN(T,9) + XWS(T,P) <=
  WD_down(T) ;
  WDEF(T,P) = XGEN(T,6) + XGEN(T,7) + XGEN(T,8) + XGEN(T,9) + XWS(T,P) -
  WD_down(T) ;
  WDEF(T,P) + QDEF(T,P) - QSUR(T,P) = 0;

  csw(t,p) = sw_agri(1)*xws(t,p) + sw_agri(2);
  cgw(t,1) = p_agri(1)*xgen(t,6) + p_agri(2);
  cgw(t,2) = p_agri(1)*xgen(t,7) + p_agri(2);
  cgw(t,3) = p_agri(1)*xgen(t,8) + p_agri(2);
  cgw(t,4) = p_agri(1)*xgen(t,9) + p_agri(2);
);

! minimum requirement flow downstream of reservoir ;

```

```

@FOR( TXWD(T,S)| T #GT#2 #AND# S #EQ# 6:
  MINQ(T) = XGEN(T,5) + XSPIL(T,5)
    + 0.3*0.7*(XGEN(T,6) + XGEN(T,7) + XGEN(T,8) + XGEN(T,9) + XWS(T,5))
    + 0.3*0.2*(XGEN(T-1,6) + XGEN(T-1,7) + XGEN(T-1,8) + XGEN(T-
      1,9) + XWS(T-1,5)) + 0.3*0.1*(XGEN(T-2,6) + XGEN(T-2,7) +
      XGEN(T-2,8) + XGEN(T-2,9) + XWS(T-2,5)) ;
  wdef(t,6) = MINQ(T) - WD_min(T) ;
  MINQ(T) - WD_min(T) + QSM(T) >= 0 ;
);

! Boundary ;
@FOR(TXP(T,P)| P #EQ# 1 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0.0, XSPIL(T,P), 1000);
);

@FOR(TXP(T,P)| P #EQ# 1 :
  @free( wdef(t,p));
);

@FOR(TXP(T,P)| P #GE# 2 #AND# P #LE# 5 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0, XGEN(T,P), 1500);
  @BND( 0.0, XSPIL(T,P), 1500);
  @free( wdef(t,p));
);

@FOR(TXP(T,P)| P #EQ# 6 :
  @free( wdef(t,p));
);

@FOR(TXP(T,P)| P #GE# 6 :
  @BND( IMIN(T,P), INV(T,P), IMAX(T,P));
  @BND( 0.0, XGEN(T,P), GMX(P));
);

```