OPTIMAL WATER RESOURCES MANAGEMENT MODEL FOR ASH SHARQIYAH REGION DOMESTIC WATER SUPPLY, OMAN

Said Khamis Mohammed Al-Khamisi

B.Sc., M.Sc.

Thesis Submitted for the degree of

Doctor of Philosophy

Heriot-Watt University

School of the Built Environment

Riccarton, Edinburgh, EH14 4AS

United Kingdom

December 2011

ABSTRACT

Continuously increasing water demand in various sectors is intensifying the water scarcity problem particularly in arid and semi-arid regions like Oman. In many areas of the Sultanate, demand for water far exceeds its current availability. This presents logistical challenges in overcoming this situation or at least keeping the water deficit as low as possible. In Oman, most of the readily accessible fresh groundwater resources have already been extensively developed in order to attempt to meet the increasing demand for water, and any further intensification of groundwater abstraction is therefore not sustainable. Attention has therefore turned to desalination of sea water to supplement the available groundwater resources. Desalination is expensive and energy intensive; hence it cannot realistically be the sole source of drinking water in Oman. Rather, a conjunctive use of groundwater and desalination optimally operated to meet water demands while ensuring the sustainability of the groundwater resources is the best option.

Thus, a numerical simulation model of Ash Sharqiyah Sands Aquifer was developed in this study and used to assess the long-term impacts on piezometric heads of supplying the eight Wilayats of Ash Sharqiyah Region with water from the 29 operational wells located in two regional groundwater fields- the Jaalan and the Al Kamil. The simulation results showed that the existing provision from the two wellfields will be inadequate by the 1st of September 2025 to meet domestic water supply needs without creating excessive drawdown and the cessation of flow in some of the existing operational Aflaj, which are artificial, surface channels that tap and convey by gravity groundwater for diversion into various uses along its route. Supplementing the abstraction from the wellfields with the more costly desalinated water of the Sur Desalination Plant offers the prospect for combating the problem; consequently, a constrained optimization problem was formulated to find the least cost blending of groundwater and desalinated water to meet demands while satisfying various constraints including the need to maintain Aflaj flow. The optimisation revealed increasing contribution of desalination to future total water supply for the Region, as desalination water replaces pumping from wells that affect Aflaj flow, with implications for the project cost. However, significant reduction in the long-term total production cost was achieved by increasing up to 50% the existing pump capacity at the Jaalan, made possible because its associated Aflaj are located upstream of the wellfield and are hence only minimally affected by the current abstractions.

ACKNOWLEDGMENTS

Many people and authorities gave their support and helped me to complete this study. However, when I started to write this statement, it became apparent that it would not be easy to enumerate all of the support I have received throughout my journey to accomplish my research. Therefore, I would like to say that I am indebted to all of them. I would like to start expressing my indebtedness and thankfulness to the government of the Sultanate of Oman for the financial support for this study. My sincere thanks to the officials of the Public Authority for Electricity and Water especially H.E. the Chairman for their continuous support and encouragement and I should not forget to thank the officials of my previous ministry, the Ministry of Regional Municipality and Water Resources, for the initial approval of my study.

I would like to express my sincere recognition and special gratitude to my supervisor, Dr Adebayo Adeloye, for his constructive advice, guidance, assistance and friendship throughout my study. Without his continuous interest, encouragement and frequent fruitful hours of discussion and support, this work would have been significantly more complicated. I am indebted to his tremendous capability to put things into a broader context and to determine the important elements of the study. Also, truthful and exceptional thanks with special acknowledgments to Dr. Ali Gharbi , groundwater expert in the Ministry of Regional Municipality and Water Resources of Oman, for his help, support and encouraging feedback. My gratitude also goes to Dr Ali Al-Maktoomi and Professor Anvar Kacimov from the College of Agricultural and Marine Sciences at Sultan Qaboos University in Oman for providing some of their generous time to advise me with some ideas related to my study. Furthermore, I would like to thank the staff and colleagues at my work in Oman and those in the School of the Built Environment at Heriot-Watt University in Edinburgh, UK, for their support and encouragement.

Words are not enough to express my gratitude to my heartfelt father for continuous prayer for me, to my brothers and sisters for their support and encouragement, to my lovely wife for her patience during those long evenings and weekends I spent on my research rather than with her and for her care of the children during my absence, and finally to my treasured children for their patience during my absence due to the study.

Research Thesis Submission

Name:	Said Khamis Mohammed Al-Khamisi		
School/PGI:	School of the Built Environment		
Version: *(i.e. First, Resubmission, Final)*	Final	Degree Sought	PhD

Declaration

In accordance with the appropriate regulations I hereby submit my thesis and I declare that:

1) the thesis embodies the results of my own work and has been composed by myself
2) where appropriate, I have made acknowledgement of the work of others and have made reference to work carried out in collaboration with other persons
3) the thesis is the correct version of the thesis for submission and is the same version as any electronic versions submitted*.
4) my thesis for the award referred to, deposited in the Heriot-Watt University Library, should be made available for loan or photocopying and be available via the Institutional Repository, subject to such conditions as the Librarian may require
5) I understand that as a student of the University I am required to abide by the Regulations of the University and to conform to its discipline.

* *Please note that it is the responsibility of the candidate to ensure that the correct version of the thesis is submitted.*

Signature of Candidate:		Date:	

Submission

Submitted By *(name in capitals):*	SAID KHAMIS MOHAMMED AL-KHAMISI
Signature of Individual Submitting:	
Date Submitted:	

For Completion in Academic Registry

Received in the Academic Registry by *(name in capitals):*			
Method of Submission *(Handed in to Academic Registry; posted through internal/external mail):*			
E-thesis Submitted (mandatory for final thesis from January 2009)			
Signature:		Date:	

TABLE OF CONTENTS

LISTS OF TABLES

CHAPTER 3

CHAPTER 4

CHAPTER 5

CHAPTER 6

LISTS OF FIGURES

CHAPTER 1

CHAPTER 2

CHAPTER 3

CHAPTER 4

CHAPTER 5

CHAPTER 1

INTRODUCTION

1.1 Statement of the problem

Water is essential for life and very important for different developmental activities for human beings. It is required for irrigation, industrial, commercial, domestic, municipal and many other activities. Continuously increasing water demand in various sectors has intensified the water scarcity problem particularly in arid and semi-arid regions. The Sultanate of Oman (Figure 1.1) lies in an area that has both low rainfall and high potential evaporation. The average annual rainfall throughout most of the country is less than 100mm and evaporation reaches as high as 80% of the rainfall. With these extreme climatic conditions, groundwater is very important in Oman because it is believed to be secure source of clean water supply in the country. Much of the groundwater in Oman is used in irrigated agriculture, exceeding 90% of total consumption (MRMEWR, 2005). Moreover, the steady increase in population and the expansion of agricultural, industrial and tourism activities have all combined to stress the groundwater resources to the limit.

Thus, in many areas of the Sultanate, demand for water exceeds its current availability. This presents logistical challenges in overcoming this situation or at least keeping the water deficit as low as possible. Therefore, the Ministry of Regional Municipalities and Water Resources (MRMWR) has implemented a number of water exploration programmes in various regions of the country during the last thirty years. These groundwater drilling programmes have led to the discovery of a number of groundwater aquifers in different parts of the Sultanate. The Ash Sharqiyah Sands Aquifer, located in the Wadi al Batha Basin of the Ash Sharqiyah Region (Figure 1.3), is one of the most important groundwater discoveries announced in 1996 and since then, many studies and investigations have been conducted on this aquifer so as to fully evaluate its potential as a drinking water supply source (MWR, 1997a).

These comprise several small-scale and large-scale drilling, aquifer testing, geophysical surveying activities, complemented by associated hydrologic, remote sensing, hydro-chemical, borehole logging, monitoring and topographic studies. The integration, interpretation and analysis of the complete data set were completed in January 1997. More on this will be described in Chapter 4.

However, it suffices to states here that the studies led to the discovery of two major aquifer layers in the Ash Sharqiyah Sands Aquifer (see Figure 1.3). The first is the "inactive" aeolianite aquifer, which lies on top of the gravel alluvium and occurs west of the Jaalan townships. It attains a maximum saturated thickness of 100m (MWR, 1997a). The second is the "active" more permeable gravel-rich alluvial aquifer (see Figure 1.3), which also varies in thickness but reaches 160m (MWR, 1997a). It lies within the upper horizons of gravel alluvium that is associated with Wadi al Batha which extends to depths of more than 600m in a largely fault-bounded basin. The water in these aquifer systems is largely fresh, with soluble salt concentrations much below the Omani standards thresholds. For example, the maximum permissible (salinity) limit (MPL) for drinkable water according to the Omani standard is electrical conductivity (EC) value of 2,500µS/cm (MCI, 1978). As shown in Figure 1.3, the EC = 2,500 µS/cm contour for the alluvial aquifer encloses the courses of Wadis al Batha and Bani Khalid and extends southward some distance beneath the aeolianite. The 2,500µS/cm EC contour for the aeolianite aquifer on the other hand encloses a vast area of the north-eastern Ash Sharqiyah Sands area as shown in Figure 1.3. According to the aquifer assessment activities, the fresh water thickness is in excess of 100m from the combined two aquifer systems over an area of approximately 1,000km². The total potable groundwater storage was estimated to be $24x10^9m^3$ comprising $8x10^9m^3$ in the alluvial aquifer and $16x10^9m^3$ in the aeolianite aquifer (MWR, 1997a), which implies an average porosity of 0.24 for the aquifer systems.

With significant development potential of both aquifers afforded by these vast storage quantities, the establishment of a supply scheme to benefit local citizens in the main towns and villages of Wilayats (states) such as at Al Kamil-Al Wafi, Jaalan Bani Bu Hassan and Jaalan Bani Bu Ali of the southern Ash Sharqiyah Region (see Figure 1.2), was started in 1996 with pre-feasibility and feasibility studies (Mott MacDonald, 1997). The purposes of the studies were to identify different water supply options, to identify the best location of the wellfields, to quantify the engineering design and cost of pipeline options from the wellfields and to compare cost/benefits of each option. The scheme envisaged construction of two wellfields that will supply potable water for domestic, commercial and industrial uses over 30 years. In June 1999, a consultancy services agreement was signed to carry out the engineering design and supervise the construction (Dr. Ahmed Abdel Warith & Partners LLC, 1999). Construction started in November 2001 and was completed in February 2004 (GULFAR / SADE consortium, 2001). The commissioning of the project started in April 2004.

However, as a result of intensifying droughts, rapid population growth and the lack of sufficient groundwater resources in the Northern Wilayats of Ash Sharqiyah Region, there is a continuous shortage of domestic water supply, which is threatening the development of the region. Consequently, in order to secure a sustainable, reliable potable water supply for the region, the Omani government decided to expand the existing desalination plant in *"Sur"* (see Figure 1.2) for the sole purpose of supplementing the water available from groundwater sources. This conjunctive use of desalinated water and groundwater, it is hoped, will secure the long-term future water supply situation for most of the Ash Sharqiyah Region. The construction to connect the Sur Desalination Plant output and output of the Ash Sharqiyah wellfields started in 2006 and was completed in 2009. It consisted of the installation of more than 195 km of main water supply pipes of diameters ranging from 150 to 900 mm. Detailed description of the two schemes is explained in Chapter 4. However, in spite of the progress in the construction of the desalination scheme, no clear management strategy has yet been established. The development of such a strategy that will optimise the conjunctive use of groundwater and desalinated water became the focus of this research project.

1.2 Aim and objectives

The aim of this work is to determine the optimum (minimum cost) water management strategy for the conjunctive use of both groundwater and desalinated water for domestic water supply up to the year of 2030 in the Ash Sharqiyah Region of the Sultanate of Oman. The specific objectives are to:

1- Develop a groundwater simulation model to describe the existing conditions of the Ash Sharqiyah Sands Aquifer, and then apply the model for assessing the long-term impacts of current groundwater management strategies at the two existing wellfields.

2- Formulate a constrained optimization model for the conjunctive use of groundwater and desalinated water in the region that will have as its objective the minimization of the total production cost while meeting a number of environmental and physical constraints.

3- Develop a practical and reliable management model to couple the Ash Sharqiyah Sands Aquifer simulation model with the constrained optimization model in order to find optimal, acceptable, sustained water resources management solution to the water supply situation in the region.

4- Investigate through extensive sensitivity studies the impact of variations in various assumptions made on the developed optimal water management strategy for the region.

5- Make recommendations to policy makers and other stakeholders on the best conjunctive use strategy for water recourses development to meet the future Ash Sharqiyah Region domestic water demand.

1.3 Significance of research

This research will combine the use of optimization techniques, hydrogeology, groundwater modelling, and system cost analysis. The distinctiveness of this research is that it not only deals with a normal groundwater simulation modelling but also it will provide water resources managers with a valuable management tool to determine the "optimal" long-term strategy for developing their limited groundwater resources by blending it and the vast sea saline water desalination in such a way that the aggregated cost is minimal. The blending strategy to be developed could be adapted in other arid to semi-arid regions to avoid creating extensive drawdown of aquifers and its consequent negative environmental impact. In particular for Oman is the need to maintain flows in operational *Aflaj*, which are natural systems constructed for taping underground water by gravity. Aflaj can be affected by either reducing their natural flows or drying them out completely which will have harmful effects on the environment by such as soil deterioration, desertification or creating sea water intrusion.

Furthermore, protecting Aflaj is not only to preserve the environment but also Aflaj in Oman are considered an important heritage that illustrate the diligence and determination of the Omani people in building a civilization and enriching global human heritage. This unique water system which dates back more than two thousand years gave a boost to agriculture in Oman. Aflaj system represents a heritage that enabled Omanis to establish an inveterate civilization throughout centuries and provided subsistence for generations who live in harsh climatic and environmental conditions (MRMEWR, 2005). Thus, because of their uniqueness, importance and contribution to water resources in Oman without disturbance to the environment, the UNESCO’s Water Committee decided in July 2006 to include five of the Oman’ aflaj as world heritage sites (MRMWR, 2006). More about this unique water supplying system will be explained in Chapter 3. Protecting these Aflaj by keeping them flowing all the time through appropriate constraints in the management model adds more uniqueness to this research.

1.4 Thesis organization

The thesis is divided into seven chapters. This chapter presents an overview of the research including its aim and objectives, and seeks to demonstrate the broader significance of the work in helping to sustain the natural heritage of Oman while striving to ensure sufficient water resources availability in the Ash Sharqiyah region for a long-time into the future.

Chapter 2 reviews some of the important hydraulic terms which describe the groundwater movement in different types of aquifers as well as Darcy’s Law and the continuity equation, which govern groundwater movement in the transient state. Thiem’s equations at the steady state condition have been reviewed to determine aquifer parameters in both confined and unconfined aquifers. The three-dimension partial differential equation of groundwater flow through a porous media is also discussed. The three-dimension finite-difference formulation for the numerical solution of the equation is described in detail.

The Chapter deals also with the literature review of groundwater modelling and its applications. Example applications of numerical groundwater simulation models and their results are discussed in Section 2.4. Different optimization programmes are discussed in Section 2.5. Finally, management modelling options for coupling simulation model and optimization model in groundwater management studies are discussed in Section 2.6.

Chapter 3 presents an introduction to the Sultanate of Oman and its climate, with a brief description of the water resources and water supply arrangements commonly used across the Sultanate. The Aflaj water system and its important contributions to the water supply infrastructure especially to agriculture sector are also described in detail in this chapter. The Chapter attempts to establish the causes and reasons for the current water deficit nationally and to justify the importance of initiating a strategy of implementing conjunctive use of both desalinated and groundwater resources for domestic water supply in Oman.

Chapter 4 presents the general description of the Ash Sharqiyah study area. It describes the geography and the geological setting of the Wadi al Batha water basin The available data as well an inventory of past studies carried out within the study area are also presented in this chapter. It also includes a brief description of the hydro-geological interpretation of the main aquifer systems within the study area and the exiting Ash Sharqiyah Sands Groundwater Supply Project. The Sur Central Desalination Plant whose output will be blended with the groundwater coming from the existing Ash Sharqiyah Sands wellfields and how the two schemes will be connected are all briefly highlighted at the end of the Chapter.

Chapter 5 covers the simulation model of Ash Sharqiyah Sands Aquifer. It includes the modelling objectives and its approach. The model design which consists of the conceptual model of two layers, model domain and discretization as well as aquifer boundary conditions are all described in this chapter. Recharge and abstraction input data used for the simulation model are also explained in detail in this chapter. The Chapter also includes the results and the discussions of the calibrations and validations for steady state and the transient model as well as the long-term impacts of supplying the eight Wilayats of Ash Sharqiyah Region with water from the two wellfields. The results and the discussions of the sensitivity analysis carried out are also presented at the end of the Chapter.

Chapter 6 describes the optimization model and its results. It presents the results and the discussion of the two water management scenarios investigated and their main driver, which is the need to reduce the current abstractions from the existing wellfields and thus eliminate some of the associated negative environmental impacts, notably the drying up of the Aflaj that derive their flows from the groundwater. The Chapter highlights recommended management scenario. Finally, sensitivity analysis results and discussions on some of the economic factors are presented at the end of the Chapter.

Chapter 7 represents the conclusions of the study and some recommendations for future researches.

Figure 1.1: Oman administrative map showing different governorates and regions as well as the location of the study area (coordinates in metres)

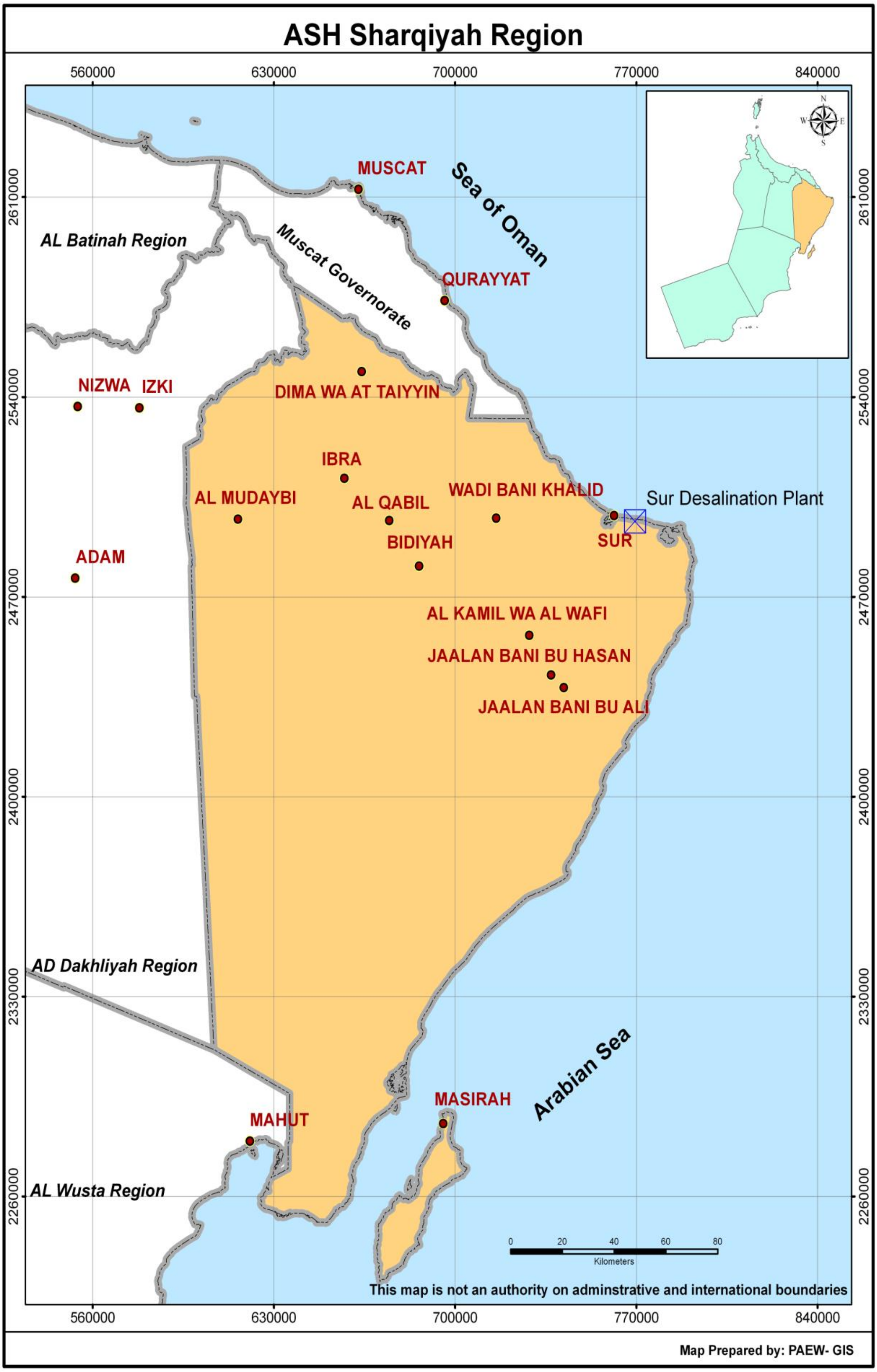


Figure 1.2: Ash Sharqiyah Region map showing the location of the main Wilayats (coordinates in metres)

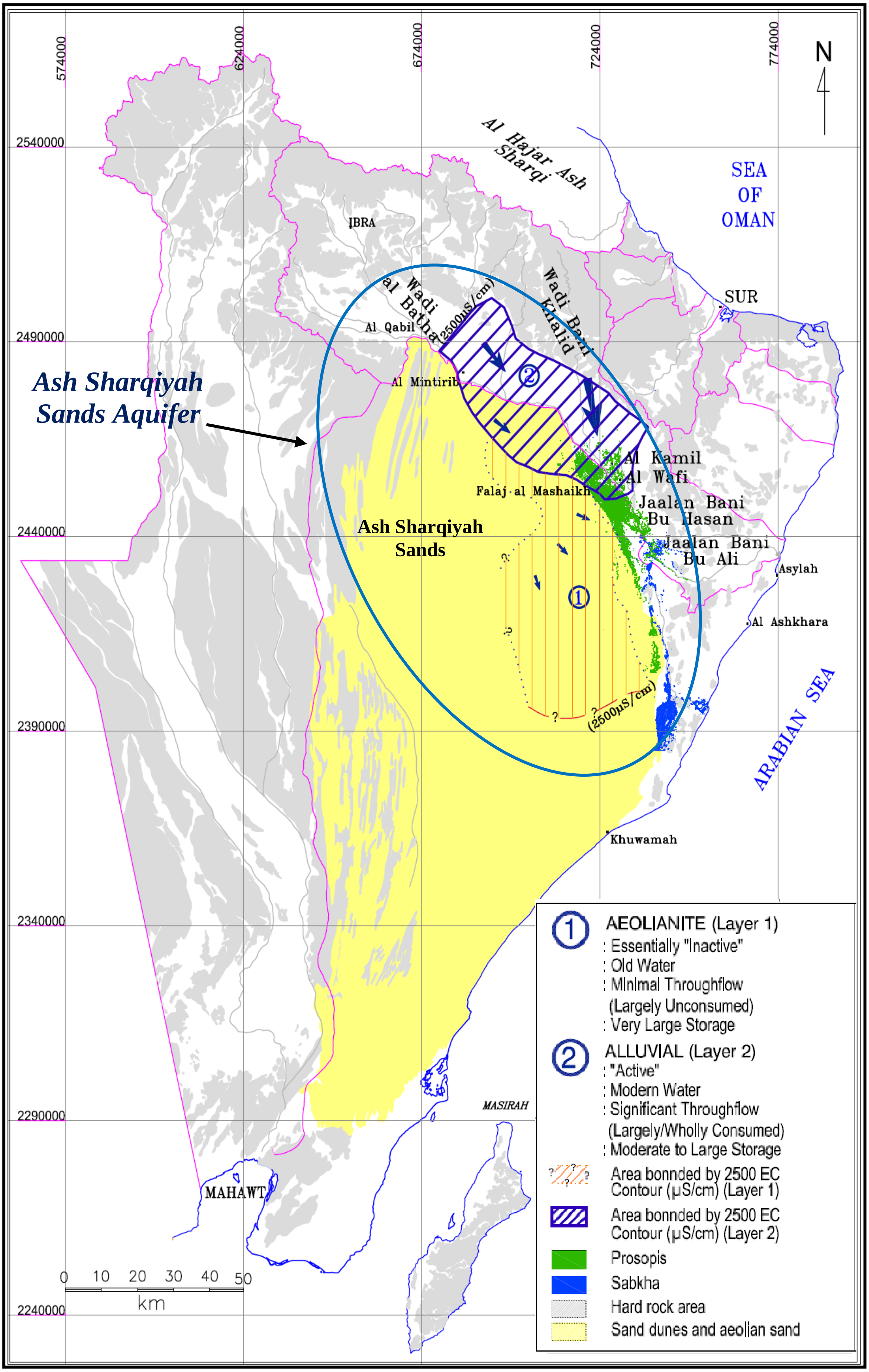


Figure 1.3: The main Aquifer Systems of the Ash Sharqiyah Sands, Wadi al Batha Basin (MWR, 1997a) (coordinates in metres)

CHAPTER 2

LITERATURE REVIEW: SIMULATION AND OPTIMIZATION STUDIES IN GROUNDWATER MANAGEMENT

2.1 Introduction

Water is life and a basic resource required for human existence. It is a vital ingredient for various developmental activities for all humans, animals and plants alike. There is increase in fresh water demand as the world's population continuously grows, which is why water issues have received international attention and is considered one of the most pressing problems that all countries have to deal with. Groundwater has traditionally provided an essential source of clean water because it is readily available in many locations and normally requires little or no treatment apart from precautionary disinfection. The current situation of growing demand-supply imbalance is particularly problematic in arid and semi-arid regions which suffer from low rainfall, high evaporation and long periods of drought and hence water stress. Several factors are responsible for this critical situation, including increasing water demand in various sectors especially for irrigation, insufficiency of groundwater, lack of balance between recharge and discharge and between supply and consumption, the pollution of groundwater sources due to e.g. seawater intrusion and absence of appropriate and integrated water management policies.

Therefore, it is very important to understand groundwater behaviour in order to ensure good groundwater resource management in a particular place. The following sections highlight the literature that describe some of the important hydraulic terms which describe this behaviour.

2.2 Aquifers types

Aquifers are simply defined as underground storage reservoirs and most of them are of large area extent (Todd and Mays, 2005). They usually get their water from natural or artificial recharge. Water flows out of them under the action of gravity or is pumped by wells. Todd and Mays (2005) classified them as confined or unconfined based on the presence or absence of water table, while a leaky aquifer represents a combination of the two types (see Figure 2.1).

Unconfined aquifers are reservoirs in which a water table varies in undulating form and in slope due to recharge and discharge. A special case of an unconfined aquifer contains clay lenses in sedimentary deposits which often have shallow perched water bodies overlying them (Todd, 1980).

Confined aquifers, also known as artesian aquifers, are those in which the water is confined under pressure greater than atmospheric pressure by overlying impermeable strata. The water level in a well penetrating such an aquifer will rise above the bottom of the confining bed as shown by artesian well of Figure 2.1. A confined aquifer gets its recharge from an area where the confining bed rises to the surface (see Figure 2.1). Fluctuations of water in wells penetrating these types of aquifers result from changes in pressure due to recharge or discharge rather than changes in storage. The potentiometric surface of a confined aquifer is defined by Todd (1980) as an imaginary surface coinciding with the hydrostatic pressure level of the water in the aquifer (see Figure 2.1).

Leaky or semi-confined aquifers are common hydraulic features in alluvial, plains, or former lake basins where a permeable stratum is overlain or underlain by semi-confined layer or aquitard (Todd and Mays, 2005). A pumped well in a leaky aquifer removes water by vertical flow through the aquitard into the aquifer and by horizontal flow within the aquifer.

2.3 Groundwater movement

The French hydraulic engineer Henry Darcy (1803-1858) described the movement of fluid through a porous medium (Todd and Mays, 2005). The law was formulated based on the results of experiments on the flow of water through a porous media (sand) (see Figure 2.2). The Darcy's Law was the result of this experiment which can be formulated as follows (Hornberger et al., 1998; Shaw, 2004; Todd and Mays, 2005):

$$v = -K \frac{dh}{dl} \tag{2.1}$$

where, v is the specific discharge (m/day); h is the hydraulic head (m); K is hydraulic conductivity (m/day); and $\frac{dh}{dl}$ is the hydraulic gradient (m/m).

Equation (2.1) is applied when the aquifer is homogeneous and isotropic ($K_x = K_y = K_z$). Otherwise for heterogeneous aquifer and anisotropic conditions (i.e, $K_x \neq K_y \neq K_z$) the equation would be written separately for each of the Cartesian coordinates as follows:

$$v = -K_x \frac{dh}{dx} \tag{2.2}$$

$$v = -K_y \frac{dh}{dy} \tag{2.3}$$

$$v = -K_z \frac{dh}{dz} \tag{2.4}$$

where, K_x, K_y and K_z are hydraulic conductivity in the x, y and z directions respectively.

The negative sign in equations (2.1 – 2.4) means that flow will occur in the direction of decreasing hydraulic head. The flow rate (Q) through the cross sectional area (A) therefore becomes:

$$Q = -KA\frac{dh}{dl} \tag{2.5}$$

Another term much used in analysing the groundwater hydraulic is the *transmissivity* (T) in m²/day, which is calculated by:

$$T = Kb \tag{2.6}$$

where, b is the thickness of the saturated aquifer in metre. It represents the rate of flow per unit width of the aquifer under unit hydraulic gradient.

It is also necessary to define change in state of aquifer when considering water movement in the ground, and for this a description of the storage capacity of the medium is necessary. The *specific storage* (S_s) in m^{-1} is defined by Shaw (2004) as the volume of water that can be released from a unit volume of a saturated aquifer by a unit reduction in hydraulic head, which is calculated by:

$$S = S_s b \tag{2.7}$$

where, S is called the *storativity* or *storage coefficient* and is dimensionless. The storage coefficient is known as the *specific yield* (S_y) in unconfined aquifer.

Groundwater movement in the transient state is governed by both Darcy's Law and the continuity principle (Shaw, 2004). Considering an element of saturated porous sand with sides of length x, y, and z as illustrated on Figure 2.3, and using the principle of continuity, the following equality for first one dimensional water movement can be written for the difference between inflow and outflow as follows:

inflow = outflow + change in storage

$$q_x = (q_x + \frac{\partial q}{\partial x} x) + S_s xyz \frac{\partial h}{\partial t} \tag{2.8}$$

where, q is flow, $\frac{\partial h}{\partial t}$ is the change in head with time and $\frac{\partial q}{\partial x}$ is the change in flow in the x-direction.

Applying Darcy's Law as in equation (2.5):

$$q_x = yz(-K_x \frac{\partial h}{\partial x}) \tag{2.9}$$

Replacing q_x in equation (2.9) with equation (2.8), gives:

$$xyz(-K_x \frac{\partial^2 h}{\partial x^2}) + S_s xyz \frac{\partial h}{\partial t} = 0 \tag{2.10}$$

Re-arranging after dividing by xyz in equitation (2.10):

$$K_x \frac{\partial^2 h}{\partial x^2} = S_s \frac{\partial h}{\partial t} \tag{2.11}$$

For horizontal two-dimensional flow in the $x-y$ plane, equation (2.11) becomes:

$$K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} = S_s \frac{\partial h}{\partial t} \tag{2.12}$$

For three-dimensional flow, equation (2.12) becomes:

$$K_x \frac{\partial^2 h}{\partial x^2} + K_y \frac{\partial^2 h}{\partial y^2} + K_z \frac{\partial^2 h}{\partial z^2} = S_s \frac{\partial h}{\partial t} \tag{2.13}$$

Equation (2.13) is the governing equation for the *transient* flow in three dimensions for anisotropic, heterogeneous aquifer. Its solution gives the hydraulic head $h(x, y, z, t)$ value at any point in a three-dimensional flow field at any time.

For a homogenous, isotropic aquifer where the hydraulic conductivities ($K_x = K_y = K_z = K$) are equal in the three dimensional axes and constant, equation (2.13) becomes:

$$K(\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2}) = S_s \frac{\partial h}{\partial t} \tag{2.14}$$

For the steady state flow condition where $\frac{\partial h}{\partial t} = 0$, equation (2.14) would be reduced to:

$$\frac{\partial^2 h}{\partial x^2} + \frac{\partial^2 h}{\partial y^2} + \frac{\partial^2 h}{\partial z^2} = 0 \tag{2.15}$$

Equation (2.15) is called the Laplace equation assuming Darcy's Law is valid, the aquifer is homogenous and it is isotropic at steady state and with no external stresses. Its solution gives the hydraulic head (h) value at any point in a three-dimensional flow field.

It is generally difficult to find an analytical solution for equations (2.15) and consequently some simplifying assumptions have been made by Thiem in order to obtain an analytical solution at the steady state condition to determine aquifer parameters in both confined and unconfined aquifers (see Shaw, 2004). The common critical assumptions are (Fetter, 2001):

- Darcy's Law is valid
- The aquifer is homogeneous and isotropic, and it assumed to be underlined by infinite extent horizontal confined layer.
- Groundwater flow is horizontal with a constant density and viscosity, and radial towards the well.

For confined aquifers (see Figure 2.4), the relevant expression is:

$$T = \frac{Q}{2\pi(h_1 - h_w)} In\left(\frac{r_1}{r_w}\right) \tag{2.16}$$

where, T is aquifer transmissivity (m²/day); Q is pumping rate (m³/day); h_1 is head at distance r_1 from the pumping well (m); h_w is head at the pumping well (m); and r_w is pumping well radius or radial distance (m).

For unconfined aquifer (see Figure 2.5), the expression can be written as follows:

$$K = \frac{Q}{\pi(h_2^{\ 2} - h_1^{\ 2})} In\left(\frac{r_2}{r_1}\right) \tag{2.17}$$

Where, K is the hydraulic conductivity (m/day); r_1 is distance from head h_1 (m); and r_2 is distance from head h_2 (m).

All of the above analytical solutions for both equations 2.16 and 2.17 have been possible by considering the above assumptions but they do not therefore describe the aquifers exactly in terms of their heterogeneity and anisotropy. Furthermore, the Thiem equation estimates the transmissivity only (and K) but not the storage coefficient, S. Thus, to accommodate such complexities, a numerical solution of equations (2.13) is commonly implemented.

2.4 Numerical simulation of groundwater flow

Groundwater simulation models comprise a set of mathematical equations that describe the state of water and its movement in aquifer systems. Those equations in their general form were introduced in Section 2.3. Once the simulation models effectively calibrated and validated, they are able to simulate groundwater flow, hydraulic heads, and transport of pollutants. They can therefore be used to evaluate groundwater resources, to develop a better understanding of the flow characteristics of aquifers and to predict the impacts of various groundwater management alternatives such as the impacts of pumping and recharge, and saltwater intrusion (Mays and Tung, 1992).

When possible disturbance caused by possible sources (e.g. infiltration and recharge) and sinks (e.g. pumping withdrawal) of water are added into equation (2.13), the partial differential equation of groundwater flow through a porous media for the *transient* flow in three dimensions for anisotropic, heterogeneous aquifer becomes (Anderson and Woessener, 1992):

$$\frac{\partial}{\partial x}\left(K_x \frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y \frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z \frac{\partial h}{\partial z}\right)-W = S_s \frac{\partial h}{\partial t} \qquad (2.18)$$

where, K_x, K_y and K_z are values of the hydraulic conductivities along the x, y and z coordinate axes, (LT^{-1}); h is the potentiometric head (L); W is the volumetric flux per unit volume and represents sources (-) and/or sinks (+) of water (T^{-1}); S_s is the specific storage of the porous medium i.e. the volume of water removed or added to storage per unit volume, per unit change in head (L^{-1}); and t is time (T).

Equation (2.18) together with specification of flow and / or head conditions at the boundaries of an aquifer system and specification of initial-head conditions constitutes a mathematical representation of a groundwater flow system. A solution of equation (2.18), in the analytical sense, is an algebraic expression giving h (x, y, z, t) such that, when the derivatives of h with respect to space and time are substituted into equation (2.18), the equation and its initial and boundary conditions are satisfied. A time-varying head distribution of this nature characterizes the flow system; in those measures both the energy of flow and the volume of water in storage can be used to calculate directions and rates of movement. It is generally common to resort to numerical schemes for its solutions. This is described in the following section.

2.4.1 Three-dimensions finite-difference formulation

A key step in constructing a finite difference solution is the discretization of the domain. Discretization is the description of the aquifer system location in terms of rows, columns and layers. A primary component of this discretization is the node or cell, which represents a specific location in space and time. Solution of the finite difference equations provides the value of the state variable (e.g. heads) at each node.

Figure (2.6) illustrates a three-dimensional hypothetical aquifer system discretized into mesh of blocks called cells. An *i*, *j*, *k* is used as an indexing system, where *i* is the row index, *j* is the column index, and *k* is the layer index. The width of the cells in the row direction is designed as Δr_j, in the column direction as Δc_i and in layer direction as Δv_k. A node is the point within the cell (grid) where the head is calculated. Figure (2.7) illustrates in two dimensions the block-centred formulation. Figure (2.8) illustrates cell ($_{i,\ j,\ k}$) and six adjacent cells: (*i-1*, *j*, *k*); (*i+1*, *j*, *k*); (*i*, *j-1*, *k*); (*i*, *j+1*, *k*); (*i*, *j*, *k-1*) and (*i*, *j*, *k+1*), that are used to derive the finite difference equation for the cell. Figure (2.9) illustrates flow into cell (*i*, *j*, *k*) from cell (i, $j-1$, k). The effective hydraulic conductivity for the entire region between the nodes is donated as $KR_{i,j-1/2,k}$. The subscript (j-1/2) does not relate to a specific point between the nodes but is used to relate to the region from cell (i, $j-1$, k) to cell (*i*, *j*, *k*). Normally, the effective hydraulic conductivity is computed as the harmonic mean between cells. Flows are considered positive if they are entering cell (*i*, *j*, *k*) and the negative sign usually incorporated in Darcy's law has been dropped from all terms.

Following the above conditions, the flow into cell (*i*, *j*, *k*) in the row direction from cell (*i*, *j* -1, *k*) is expressed by Darcy's law as (McDonald and Harbaugh, 1988):

$$q_{i,j-1/2,k} = KR_{i,j-1/2,k}\Delta c_i \Delta v_k \frac{\left(h_{i,j-1,k} - h_{i,j,k}\right)}{\Delta r_{j-1/2}} \tag{2.19}$$

where,

$q_{i,j-1/2,k}$ is the volumetric fluid discharge through the face between cells (*i*, *j*, *k*) and (*i*, *j* -1, *k*) (L^3T^{-1})

$KR_{i,j-1/2,k}$ is the hydraulic conductivity along the row between nodes (*i*, *j*, *k*) and (*i*, *j* -1, *k*) (LT^{-1})

$\Delta c_i \Delta v_k$ is the area of the cell facing normal to the row direction (L^2)

$h_{i,j,k}$ is the head at node *i*, *j*, *k* (*L*)

$h_{i,j-1,k}$ is the head at node *i*, *j* -1, *k* (*L*)

Δr_{j-1l2} is the distance between nodes *i*, *j*, *k* and *i*, *j* -1, *k* (*L*)

Similarly, Darcy's law can be applied to the flow through the remaining five sides of cell (*i*, *j*, *k*) as following:

Flow in the row direction through the face from cell *(i, j, k)* to cell (*i*, *j* +1, *k*)

$$q_{i,j+1/2,k} = KR_{i,j+1/2,k}\Delta c_i \Delta v_k \frac{\left(h_{i,j+1,k} - h_{i,j,k}\right)}{\Delta r_{j+1/2}} \tag{2.20}$$

Flow in the column direction through the forward face from cell *(i, j, k)* to cell (*i+1*, *j*, *k*)

$$q_{i+1/2,j,k} = KC_{i+1/2,j,k}\Delta r_j \Delta v_k \frac{\left(h_{i+1,j,k} - h_{i,j,k}\right)}{\Delta c_{i+1/2}} \tag{2.21}$$

Flow in the column direction through the rear face from cell *(i-1, j, k)* to cell (*i*, *j*, *k*)

$$q_{i-1/2,j,k} = KC_{i-1/2,j,k}\Delta r_j \Delta v_k \frac{\left(h_{i-1,j,k} - h_{i,j,k}\right)}{\Delta c_{i-1/2}} \tag{2.22}$$

Flow in the vertical direction through the bottom face from cell *(i, j, k)* to cell (*i*, *j*, *k+1*)

$$q_{i,j,k+1/2} = KV_{i,j,k+1/2}\Delta r_j \Delta c_i \frac{\left(h_{i,j,k+1} - h_{i,j,k}\right)}{\Delta v_{k+1/2}} \tag{2.23}$$

Flow in the vertical direction through the upper face from *(i, j, k-1)* to cell (*i*, *j*, *k*)

$$q_{i,j,k-1/2} = KV_{i,j,k-1/2}\Delta r_j \Delta c_i \frac{\left(h_{i,j,k-1} - h_{i,j,k}\right)}{\Delta v_{k-1/2}} \tag{2.24}$$

Equations (2.19- 2.24) describe the one-dimensional steady-state flow through each side of cell (*i*, *j*, *k*) in terms of heads, grid dimensions, and hydraulic conductivity. The notation can be simplified by combining the hydraulic conductivity and the grid dimensions into a constant named as a "conductance". The conductance is the product of the effective hydraulic conductivity and cross-sectional area of flow divided by the distance between nodes. Therefore, the conductance $CR_{i,j-1/2,k}$ (L^2/t) in row (*i*) and layer (*k*) between nodes (*i*, *j* -1, *k*) and (*i*, *j*, *k*), can be expressed as:

$$CR_{i,j-1/2,k} = KR_{i,j-1/2,k} \frac{\left(\Delta c_i \Delta v_k\right)}{\Delta r_{j-1/2}} \tag{2.25}$$

Substituting similar conductance expressions as of equation (2.25) into equations (2.19 – 2.24) can give the following equations:

$$q_{i,j-1/2,k} = CR_{i,j-1/2,k}\left(h_{i,j-1,k} - h_{i,j,k}\right) \tag{2.26}$$

$$q_{i,j+1/2,k} = CR_{i,j+1/2,k}\left(h_{i,j+1,k} - h_{i,j,k}\right) \tag{2.27}$$

$$q_{i-1/2,j,k} = CC_{i-1/2,j,k}\left(h_{i-1,j,k} - h_{i,j,k}\right) \tag{2.28}$$

$$q_{i+1/2,j,k} = CC_{i+1/2,j,k}\left(h_{i+1,j,k} - h_{i,j,k}\right) \tag{2.29}$$

$$q_{i,j,k-1/2} = CV_{i,j,k-1/2}\left(h_{i,j,k-1} - h_{i,j,k}\right) \tag{2.30}$$

$$q_{i,j,k+1/2} = CV_{i,j,k+1/2}\left(h_{i,j,k+1} - h_{i,j,k}\right) \tag{2.31}$$

Equations (2.26 – 2.31) represent the flow into cell (*i*, *j*, *k*) from the six adjacent cells.

External flows (sources and stresses) from outside the aquifer into each cell, such as recharge, and flow out of each cell, such as evapotranspiration and well pumping for each individual cell, should also be taken into account. Thus, the total external flow, $QS_{i,j,k}$, for cell (*i*, *j*, *k*) is the combination of source and/or stress terms for an individual cell expressed as:

$$QS_{i,j,k} = p_{i,j,k}h_{i,j,k} + Q_{i,j,k} \tag{2.32}$$

where, $p_{i,j,k}$ and $Q_{i,j,k}$ are constants that describe the individual external sources or stresses of cell (*i*, *j*, *k*), e.g. , $p_{i,j,k}$ is net rainfall and $Q_{i,j,k}$ is abstraction.

Taking into account the flows from the six adjacent cells, as well as the cumulative external flow rate, $QS_{i,j,k}$ of equation (2.32), the continuity equation for cell (i, j, k) can be expressed as:

$$q_{i,j-1/2,k} + q_{i,j+1/2,k} + q_{i-1/2,j,k} + q_{i+1/2,j,k} + q_{i,j,k-1/2} + q_{i,j,k+1/2}$$
$$+ QS_{i,j,k} = SS_{i,j,k} \frac{\Delta h_{i,j,k}}{\Delta t} \Delta r_j \Delta c_i \Delta v_k \qquad (2.33)$$

where,

$SS_{i,j,k}$ is the specific storage of cell (i, j, k) in (L^{-1});

$\frac{\Delta h_{i,j,k}}{\Delta t}$ is a finite difference approximation for the derivative of head with respect to time in (Lt^{-1});

$\Delta r_j \Delta c_i \Delta v_k$ is the volume of cell (i, j, k) in (L^3).

By substituting equations (2.26 – 2.32) into equation (2.33), the finite difference approximation for cell (i, j, k) can be written as:

$$\begin{aligned} & CR_{i,j-1/2,k}\left(h_{i,j-1,k} - h_{i,j,k}\right) + CR_{i,j+1/2,k}\left(h_{i,j+1,k} - h_{i,j,k}\right) \\ & + CC_{i-1/2,j,k}\left(h_{i-1,j,k} - h_{i,j,k}\right) + CC_{i+1/2,j,k}\left(h_{i+1,j,k} - h_{i,j,k}\right) \\ & + CV_{i,j,k-1/2}\left(h_{i,j,k-1} - h_{i,j,k}\right) + CV_{i,j,k+1/2}\left(h_{i,j,k+1} - h_{i,j,k}\right) \\ & + p_{i,j,k} h_{i,j,k} + Q_{i,j,k} = SS_{i,j,k} \frac{\Delta h_{i,j,k}}{\Delta t} \Delta r_j \Delta c_i \Delta v_k \qquad (2.34) \end{aligned}$$

$\frac{\Delta h_{i,j,k}}{\Delta t}$ is the time derivation of the head which could also be expressed in a finite difference form. A backward – difference approximation of this at a given time t_m is:

$$\left(\frac{\Delta h_{i,j,k}}{\Delta t}\right)_m = \left(\frac{h^m{}_{i,j,k} - h^{m-1}{}_{i,j,k}}{t_m - t_{m-1}}\right) \qquad (2.35)$$

where, t_m is the time at which the derivative is being evaluated and t_{m-1} is the proceeding time.

Therefore, the flow terms can be expressed in terms of h^m at time t_m and equation (2.34) can be rewritten in backward-difference form as:

$$\begin{aligned} & CR_{i,j-1/2,k}\left(h^m{}_{i,j-1,k} - h^m{}_{i,j,k}\right) + CR_{i,j+1/2,k}\left(h^m{}_{i,j+1,k} - h^m{}_{i,j,k}\right) \\ & + CC_{i-1/2,j,k}\left(h^m{}_{i-1,j,k} - h^m{}_{i,j,k}\right) + CC_{i+1/2,j,k}\left(h^m{}_{i+1,j,k} - h^m{}_{i,j,k}\right) \\ & + CV_{i,j,k-1/2}\left(h^m{}_{i,j,k-1} - h^m{}_{i,j,k}\right) + CV_{i,j,k+1/2}\left(h^m{}_{i,j,k+1} - h^m{}_{i,j,k}\right) \\ & + p_{i,j,k} h^m{}_{i,j,k} + Q_{i,j,k} = SS_{i,j,k}(\Delta r_j \Delta c_i \Delta v_k)\left(\frac{h^m{}_{i,j,k} - h^{m-1}{}_{i,j,k}}{t_m - t_{m-1}}\right) \end{aligned} \qquad (2.36)$$

Finally, equation (2.36) is a backward-difference equation, which can be used as the basis for a simulation of the partial differential equation of ground water flow, in equation (2.18).

2.4.2 Groundwater simulation studies – some examples

The finite difference formulation presented in equation (2.18) has been implemented in numerous commercial groundwater simulation software packages of which the most commonly used is the modular finite-difference groundwater flow (MODFLOW) (McDonald and Harbaugh, 1988). The MODFLOW model is probably the most frequently used groundwater modelling programme (Winston, 1999). MODFLOW is a computer program that numerically solves one-, two- or three-dimensional groundwater flow equation for a porous medium by using a finite-difference method. The original computer program was developed by McDonald and Harbaugh (1996). MODFLOW-2000, is designed to accommodate the solution of equations in addition to the groundwater flow equation (Harbaugh et al., 2000). It can also simulate steady and non-steady flow in an irregularly shaped flow system in which aquifer layers can be confined, unconfined, or a combination of the two aquifers. The reputation of MODFLOW comes from its power and excellent documentation. It is also friendly to users because it works with many graphical interfaces, such as GMS (EMRL, 2004), MODMAN and GROUNDWATER VISTAS. GMS is a US Department of Defence Groundwater Modelling System**.** It is a comprehensive graphical user environment for performing groundwater simulations. The entire package consists of a graphical user interface and a number of analysis codes such as MODFLOW, MT3DMS, RT3D, SEAM3D, MODPATH, MODAEM, SEEPD2D, FEMWATER etc… (EMRL, 2004).

There are several solver packages in MODFLOW for solving the partial differential equations such as the Preconditioned Conjugate Gradient (PCG2) packages (Hill, 1990) which will be used in this study. It also incorporates automatic calibration to determine aquifer parameters for any degree of heterogeneity. PCG2 package was used in preference to other solvers because PCG2 includes two preconditioning options: the modified incomplete Cholesky preconditioning, which is efficient on scalar computers; and the polynomial preconditioning, which requires less computer storage and, with modifications that depend on the computer used, is most efficient on vector computers (Hill, 1990). PCG2 has also been shown to perform better than other available solvers for many simulation problems (Hill, 1990).

A lot of studies and researchers have been using MODFLOW language in simulation models to solve and address several hydro-geological and groundwater-related problems.

Zhou and Li (2011) provided an excellent recent review of the historical development of regional groundwater flow modelling. They used Death Valley and Great Artesian Basin transient groundwater models as examples to show the application of large scale regional groundwater flow models. However, their models could only be used to give the general overview of regional groundwater flow and could not be used for proper management planning. This is because the basins were discretized with large uniform grid size of 5km x 5km, hence the spatial variations in both the hydrological and geological characteristics had not been taken in considerations especially for these basins with 600 springs and 2300 wells. Therefore, the computed hydraulic head in this model represents the average value over an area of several kilometres and can not be considered as a point value. Furthermore, this large scale model would be better if to be calibrated by automation calibration rather than by trial and error calibration approach adopted in the system.

Numerical simulation modelling by MODFLOW language was also used by the Ministry of Water Resources (MWR) to design and locate the existing two wellfields in the Ash Sharqiyah Sands Aquifer (MWR, 1997i). The study only located potential sites for the wellfields development but it did not simulate the potential effect of the two wellfields which were drilled later in the area during 2002 (GULFAR / SADE consortium, 2001). This research therefore as one of its objective will assess and evaluate the impact of the existing two wellfields on the groundwater aquifer system for the Ash Sharqiyah Sands Aquifer.

Boronina et al. (2003) implemented the simulation model to study the groundwater resources in the Kouris catchment, Cyprus. The catchment has suffered from a scarcity of water resources due to the semi-arid climate. They conducted water balance using a steady state groundwater model in order to find acceptable solutions. While the outcomes of this model have shown serious implications for groundwater in Cyprus, it can not be used to asses the future groundwater conditions as not enough data were available to develop the transient model. Therefore, the usefulness of the outcome of this study for long-term management decision making is limited.

Amsterdam Water Supply Utility has been using MODFLOW to design deep-well recharge systems and to position extraction wells for the groundwater supply in the dune area for many years (Olsthoorn, 1999). The study compared the results by using the analytic element method and the modular three-dimensional finite-difference groundwater flow model (MODFLOW). Each of the two modelling techniques has its own advantages. However, the MODFLOW has extra rewards such as its ability to model transient flow in complex groundwater systems, something which most analytical methods are incapable. In general, as noted ealier, analytical solution of the groundwater flow equation is only valid if the system is grossly over-simplified. The application of MODFOW gave useful information about the hydraulic impact on historical and future groundwater abstraction in Kuwait (Szekely et al., 2000). Many groundwater modelling studies in Oman were executed using MODFLOW software such as Modelling of Groundwater in the Nejd Region (Century Architects, 2007) and Drilling & Aquifer Testing Project in the Western Al Wusta Desert (Geo-Resources, 2005).

Don et al. (2005) used flow model to simulate the groundwater flow and test the environmental impacts of aquifer over-pumping in the south-western Kyushu, Japan. In the study, they coupled MODFLOW and the modular three-dimensional finite difference groundwater solute transport model, MT3D, to simulate groundwater flow hydraulics, land subsidence, and solute transport in the alluvial lowland plain. The simulated results show that subsidence rapidly occurs throughout the area with the central prone in the center part of the plain. Moreover, they concluded that seawater intrusion would be expected along the coast if the current rates of groundwater exploitation continue. The study demonstrated the multi-faceted nature of groundwater investigations that can be achieved by simulation and especially the use of MODFLOW.

Shaki and Adeloye (2007) developed and applied a numerical simulation model of the Murzuq aquifer system in Libya to better understand its hydraulic behaviour and to assess the impact of the water abstractions for irrigating the Irawan irrigation project. Although the study relied on a number of assumptions because of the paucity of data in the Murzuq basin, it nonetheless demonstrated that current abstractions practice from the aquifer was wasteful of water and that effective irrigation of the fields could be achieved by operating a sub-set of the pumps, which will represent a significant saving in water and reverse the downward trend in the trajectory of the water table. The study made a number of recommendations for increasing the availability of data for the Murzuq and indeed other regional aquifer systems in Libya. Abdalla (2008) used a numerical groundwater flow simulation model using MODFLOW to examine groundwater recharge/discharge mechanisms in the regional Central Sudan Rift Basins (CSRB). The decline in groundwater level along a flow path was calculated using Darcy's law to estimate average recharge and evapotranspirative discharge. Steady-state 2D flow modeling used in the study has demonstrated its usefulness as a good tool to evaluate and to understand the hydraulic behaviour of such aquifers.

Seneviratne (2007) used groundwater simulation model within a Geographical Information System (GIS) environment to study the flow in lower part of Walaw Basin in Sri Lanka., Remotely sensed data were used to solve the problem of the lack of in-situ measurements which as remarked earlier was a major issue for the Libyan Murzuq basin reported by Shaki and Adeloye (2007). The study concluded that high recharge was observed in the agricultural area while the discharge most concentrated in the flat area in the lower part of the basin. In this study, an integrated groundwater simulation model incorporating GIS and remote sensing techniques was successfully achieved to establish areas of recharge and discharge in the targeted basin.

Yuan et al. (2011) used a coupled model that simulates interacting surface water and groundwater flow and solute transport processes in these wetlands. The results suggest that the model represents well the interacting surface water and groundwater flow and solute transport processes in the lagoons.

A recent study by Pool et al. (2011) used a numerical flow model (MODFLOW) of the groundwater flow system in the primary aquifers in northern Arizona to simulate interactions between the aquifers, perennial streams, and springs for predevelopment and transient conditions during 1910 through 2005. Results from simulation modeling include the importance of variations in recharge rates throughout the study area and recharge along ephemeral and losing stream reaches in alluvial basins. Also, the groundwater-flow systems in individual basins include the hydrologic influence of geologic structures in some areas and that stream-aquifer interactions along the lower part of the Little Colorado River are an effective control on water level distributions throughout the Little Colorado River Plateau basin. This model is not unique and it needs better information on several aspects of the groundwater flow to reduce uncertainty of the simulated system. Many areas lack documentation of the response of the groundwater system to changes in withdrawals and recharge.

From the above review, it is clear that the MODFLOW model has the capability to simulate different conditions and scenarios in groundwater aquifers, which has clearly fuelled its popularity in groundwater planning and management studies. Therefore, the grid approach of MODFLOW in GMS graphical interfaces has been selected for the simulation model for the Ash Sharqiyah Sands Aquifer in the study of Ash Sharqiyah domestic water supply in using desalinated water and groundwater.

The simulation models are an important management tool because they can be used to investigate different management scenarios. However, a weakness of the simulation models is their inability to determine the optimal aquifer management strategy. Thus, the application of simulation models is limited to the understanding of operation of the aquifer. Also they must be executed repeatedly for different logical guesses to get as close as possible to an optimal solution for a specified objective. Repeated simulation requires much time and money, especially when dealing with large-scale projects. Therefore, a mathematical management model can be achieved to solve the mentioned constraints by combing an optimization model and simulation model. Various optimization models, which have been applied in groundwater management, are described in the next section.

2.5 Optimization models in groundwater management

An optimization model is a mathematical programming tool used to find out the maximum or minimum value of the objective function, usually subject to a number of constraints. It has been used for decision making for many years. Optimization approaches have been implemented in a wide variety of problems solving such as pollutant control, mining and construction dewatering, seawater intrusion and groundwater management. The application of optimization techniques to groundwater flow began in 1970 with the paper by Deninger (Deninger, 1970, Ahlfeld and Mulligan, 2000). Since then, many studies have been carried out using different optimization techniques in groundwater management, including both quality and quantity management.

As regards the application of optimization techniques for the management of groundwater, Das and Datta (2001) present a state of art of the different optimization approachs that have been applied to groundwater management. Specifically, the combined use of simulation and optimization techniques has been demonstrated to be a powerful and useful approach to determine planning and control strategies for groundwater systems (e. g. Katsifarakis et al., 1999; Psilovikos, 1999; Willis and Finney, 1988). In these works, the simulation model component of the management models is generally based upon the partial differential equation of groundwater flow (equation 2.18) and its finite difference solution. Depending upon the processes considered in the management model, either the flow equation, or the solute transport equation, or both, are used in the simulation.

Ahlfeld and Mulligan (2000) provided an excellent review of optimization approaches to groundwater management. An optimization problem, which has a mathematical structure, consists of three key elements. These are the objective function, the constraints and the decision variables. Two types of optimization formulations can be constructed with these elements: unconstrained problems and constrained problems. The unconstrained approaches of the optimisation formulation include only an objective function and decision variables. For design problem, the decision variables describe the controls that are to be designed. The values taken by these variables define the solution of the problem. On the other hand, the constraint elements of the optimisation formulation contain all of the three elements which impose restrictions on the values that can be taken by the decision variables. Therefore, the decision variables might be required to be continuous or integer. Furthermore, upper and lower bounds may be imposed on the value that decision variables may take. Multiple decision variables of the constraint functions may also be defined and bounds on their values may be imposed. The limitation or constraints are derived from managerial considerations and physical behaviours of the system. The general form of a non-linear, constrained optimization is written as:

$$Maximize\ (Minimize)\qquad F(x,y)+b^T x+c^T y \qquad (2.37)$$

$$Subject\ \ to \qquad \begin{aligned} &G(x,y)+A_1 x+A_2 y\le b_1\\ &A_3 x+A_4 y\le b_2\\ &L\le\begin{pmatrix}x\\ y\end{pmatrix}\le u\end{aligned} \qquad (2.38)$$

where, Equation (2.37) is the objective function and Equations (2.38) are the constraints and bounds; the vectors b^T, c^T, b_1, b_2, L, u and the matrices A_1, A_2, A_3, A_4 are constants; F(x,y) is smooth and nonlinear scale functions; G(x ,y) is a vector of smooth nonlinear functions, and x , y (vectors) are the unknown variables.

Optimization methods are generally used to solve problems in which multiple solutions satisfy all of the constraints. The goal is to identify the best solution by some appropriate objective functions. These functions could be maximized or minimized based on the desired application. Das and Datta (2001) provided an excellent review of six programming techniques commonly used by researchers to solve constrained optimization problems in groundwater quality and quantity management models. These are linear programming (LP), nonlinear programming (NLP), mixed-integer programming (MIP), differential dynamic programming (DDP), stochastic programming (SP) and combinatorial optimization (CO). Of these, however, linear programming is the most widely used especially if the problem and its constraints are linear or can be linearised. There are also numerous commercial solvers for linear programming which adds to its general appeal. This wider application of LP formulation can be also attributed to the fact that many water resources management problems can be represented realistically by a linear objective function and set of linear constraints. However, the difficulty occurs when trying to analyse water resources systems using, as objective functions, economic criteria which are basically non-linear functions of the decision variables as is the case with this study. In such cases, NLP is warranted.

Gorelick (1982) used a LP method for maximizing waste disposal. Large-scale management model was formulated as dual linear programming problems so as to reduce the numerical difficulties and computation. The results indicated that waste disposal was enhanced by pulsing rather than maintaining constant disposal rates at various sites. Hallaji and Yazicigil (1996) also used LP technique for optimal management of a coastal aquifer in southern Turkey. They proposed six LP models for steady state and transient state, and one quadratic optimization model for steady state management of the aquifer system. A nonlinear program (NLP) exists when one or more constraints is a nonlinear combination of decision variables. In a nonlinear program, the objective may be a linear or nonlinear combination of decision variables. However, because this model did not specify the lower bounds for well pumping as a constraint, the model provided unrealistic results of system allowing the drawdowns at the coastal nodes to increase.

Mixed-integer programming (MIP) models, an optimization method that combines continuous and discrete variables, have also been used to solve optimization problems with linear objective function and linear constraints in which some of the variables can take only integer values. Psilovikos (1999) compared two optimization methods used in groundwater management, based on LP and MIP. The results obtained from the solution with the two methods agree in terms of the piezometric and water balance constraints. However, the MIP model was more complicated and the feasible region of solutions was more constrained than the LP. Furthermore, Shaki and Adeloye (2007) used a mixed integer-linear programming optimization model to minimize the total water extracted in the Irawan project of Libya, subject to meeting irrigation water demands and other hydraulic constraints. The integer variables determine where pumping was to take place in order to achieve the objective. The outcome of this model is unique because it can incorporate multi-decision variables depending on which critical seasons (e.g. winter or summer) was optimized. The study did not consider the costs (capital and O&M) directly but used the total abstracted water volume at a number of target wells as surrogate of the cost. While this enabled the problem to degenerate into a linear one solvable using LP, it also constituted its major limitation.

Liu et al. (2011) used an optimisation approach with mixed integer linear programming (MILP) model for the integrated management of water resources to two water limited Greek islands - Syros and Paros-Antiparos - including desalinated seawater, wastewater and reclaimed water. The proposed model took into account the subdivided regions in study area, wastewater production, the subsequent localised needs for water use and geographical aspects as well as the integration of potable and non-potable water systems. The optimal water management decisions are obtained by minimising the annualised total capital and operating costs. The decision includes the location of wastewater treatment, desalination, and reclamation plants. This model proposed approach has not incorporated uncertainty issue (e.g. the cost of future development facilities) However, the investigation of efficient solution procedures by using their modeling approach (e.g. decomposition) for tackling large-scale optimization models constitutes a valuable research direction.

In general, LP's are relatively easy to solve but NLP's are difficult and sometimes impossible to solve. As a rule, MIP'S become increasingly difficult to solve as the number of integer variables increases.

Genetic Algorithms (GA), an automated method for creating a working computer program from a high-level problem statement of a problem, has been successfully applied to solve many optimization problems in hydrology and water resources. This technique has an advantage over all classic optimization programming techniques in that it works with a population of possible solutions, whereas the other classic optimization methods work with a single solution (Jain et al, 2005). The significance of GA is in such that it imitates some of the salient features of natural selection and natural genetics in order to find near-optimal solutions in a search space. However, there is no absolute assurance that a genetic algorithm will find a global optimum, which happens very often when the populations have a lot of subjects. An example of using GA method in groundwater optimization is the study by Katsifarakis et al. (1999). They used integrated GAs with a groundwater simulation model to maximize pumping from an aquifer, to minimize cost in water supply development and to minimize cost in aquifer remediation. Their study proved that the proposed combination is very efficient in optimizing the development and protection of groundwater resources. In addition, Prasad and Park (2004) used multi-objective GA for the optimal design of water distribution networks. This model is new and offers promise for finding optimal solutions to complex non-linear optimization problems. Jain el al (2005) employed a real-coded genetic algorithm to the problem of determining the optimal (UPRF) using the historical data from watersheds due to its limited number of decision variables and constraints.

Many studies and researchers have been using optimization models to solve and address several hydro-geological and groundwater-related problems. Moral and Birltles (1983) used the optimization model for groundwater abstraction from a coastal aquifer. They used an analytical model, based on Jacobian elliptic function, to identify feasible wellfield locations and pumping rate for large-scale groundwater abstraction from an unconfined coastal aquifer. Results show that the cheapest wellfield design would be a single large wellfield. This analytical model solution is very simplified to take strategic decision to supply a large city located on an unconfined coastal aquifer. It is more realistic to formulate this problem as NLP that minimizes the cost of water supply from this unconfined aquifer to the proposed large city.

El Harrouni et al. (1996) applied genetic algorithms in their study called groundwater optimization and parameter estimation by genetic algorithm and dual reciprocity boundary element method. They investigated two optimization problems: a pumping management problem in a homogeneous aquifer, and a parameter estimation problem in a heterogeneous aquifer. As noted earlier, genetic algorithms enable complex non-linear optimisation problems to be solved. Both aquifers studied by El Harrouni et al. (1996) were unconfined systems with their inherent non-linearity but the GA formulation applied was able to effectively solve the problem without the usual “curse of dimensionality” that would attend any attempt to solve the same problems by linear programming and embedded coupling method. This clearly demonstrated the promise of these emerging tools in groundwater management studies.

Nabi et al. (2011) used the optimization model to optimize a groundwater monitoring network for a sustainable development of the Maheshwaram Catchment in India. Field observations were combined with a geostatistical analysis to define an optimized monitoring network able to provide sufficient and non-redundant information on key hydro-chemical parameters. The approach is useful to maximize data collection and contributes to better managing the allocation of resources under any budget constraints.

2.6 Groundwater management models

It is clear from Section 2.4 that groundwater system simulation models can simulate the response of the system to a specified management strategy. An optimization, model on the other hand, identifies an optimal management strategy from a set of feasible alternative strategies. The optimization model will inevitably use the numerical approximation of the flow provided by the simulation model as constraints. It is therefore necessary to devise a means of coupling both the simulation and optimization models. This is achieved using a management model.

The coupling approach by most management models is achieved in one of the two ways: the response matrix and the embedding techniques depending upon the physical processes. Gorelick (1983) and Theodossiou, (2004) described these two methods in a comprehensive review of distributed parameters groundwater models.

The response matrix approach is based on the linearity of the system. It allows drawdown induced by one or more wells at any location to be calculated with matrix multiplication, as illustrated in equation (2.39) for a case of three control locations and two pumped wells in a steady-state system:

$$\begin{pmatrix} D_1 \\ D_2 \\ D_3 \end{pmatrix} = \begin{pmatrix} R_{1A} R_{1B} \\ R_{2A} R_{2B} \\ R_{3A} R_{3B} \end{pmatrix} \begin{pmatrix} Q_A \\ Q_B \end{pmatrix} \tag{2.39}$$

where, D_i is drawdown at control location i (1, 2, or 3); Q_{ij} is rate at well j (A or B); and R_{ij} is drawdown response at location i to a unit stress at well j.

Thus, when the response matrix is known, then any pumping rates can be applied to calculate the drawdown (Ahlfeld and Riefler, 1999).

The development of the response matrix uses an external groundwater simulation model to develop the unit responses R_{ij}. In order to generate the unit response matrix, a simulation model is solved several times each with a unit stress (pumping/recharge) or concentration loads at a single node. The assembled unit responses are then used to construct the response matrix, which is included in the management model. The response matrix works on the principles of superposition; thus, it is applicable only when the system is linear and the boundary conditions are homogeneous. Superposition is not valid when the governing equation is non-linear, as is the case for unconfined flow simulation, or when the boundary conditions are non-linear (Theodossiou, 2004).

Motz et al. (2001) constructed simulation and optimization groundwater models to manage the seawater intrusion in the Goksu Delta at Silifke in Turkey. Optimization model involved maximizing the total pumping rate subject to hydraulic and environmental constraints. The response of the aquifer system was linked to the optimization model by means of the response matrix method, implying a purely confined aquifer system, although this information was not made explicit in the study.

On the other hand, the embedding approach incorporates the equations of the simulation model directly into the optimization problem to be solved. In the embedding method, the finite difference forms of the governing groundwater flow equation are directly incorporated as part of the constraints. Some of the unknown groundwater variables, such as hydraulic heads and source rates, may become decision variables in the optimization problem.

Peralta and Datta (1990) optimized sustained yield planning for a 3,200 square miles area over 300 cells using the embedded method. They noted that when large numbers of pumping cells are used and steady state management policies are desired, the embedding technique requires less computer memory and processing time than the response matrix approach. It is not based on the principle of superposition and thus it has a wider range of application. For nonlinear systems, the response matrix approach is not applicable and use of embedding technique becomes necessary. However, the time step used in the embedding approach for non-linear transient problems may require a larger number of variables and constraints for accuracy of the solution. In highly non-linear problems such as those involving density dependent transport models, where the response matrix approach is not applicable, a management model even for a small study area may become dimensionally large (Das and Datta, 2001).

Management objectives must be selected in order to develop groundwater management models. These objectives involve not only geologic and hydrologic considerations but also other considerations such as economic, legal, political and financial aspects. Identifying the least-cost management strategy, to meet specified hydraulic and water quality restrictions in an aquifer, is one of many examples of groundwater management models. Excellent reviews on the types of groundwater management models and their applications are made by Gorelick (1983), Wills and Yeh (1987), and Yeh (1992). Todd and Mays (2005) classified groundwater management models into two basic categories: hydraulic management models and policy evaluation models. The first category models are aimed at managing pumping and recharge. The second type of models can consider the economics of water allocations.

Many researchers have reported the use of embedding technique and (or) response matrix approach in conjunction with combined simulation-optimization groundwater management models. They have been developed for a variety of applications, such as restoration of contaminated groundwater, control of aquifer hydraulics, allocation of ground and surface water resources, and evaluation of groundwater policies. Some good example in the use of management models in groundwater planning and decision making have been reviewed.

Aguado and Remson (1976) tried embedded technique in two-dimensional artesian aquifers. Solutions of LP models are used to determine optimal well distributions and pumping rates to meet given management objectives for a hypothetical unsteady state problem and for a steady-state field problem. As noted previously, embedded technique is time consuming and can readily suffer from the curse of dimensionality, which is why it is surprising that the authors have used the approach for coupling their management and simulation models. As a purely artesian aquifer, the relationship between head and discharge is linear which is a necessary and sufficient condition for applying the simpler, and quicker response matrix method. Nonetheless, this wider application of LP formulation can be also attributed to the fact that many water resources management problems can be represented realistically be a linear objective function and set of linear constraints.

Tung and Koltermann (1985) used embedding method to compare two basic approaches for solving groundwater management models. In the first approach, the multi-period groundwater model was considered as a whole and was solved all at once. But in the second approach each time period is executed separately, beginning with the first time period. The final head from the previous period is used as the initial head for the next period. They observed that using the second stepwise approach gives the same answer as the first multi-period approach, when the objective is to maximize the sum of heads, However, the two approaches give different answer, when the objective is to minimize pumping. They also concluded that the second approach requires less computer memory compared with the first large scale approach. The model approaches could be useful to be considered when formulating the management model for this study because it is also used the embedding method, which is appropriate where the aquifer is predominantly unconfined.

Pezeshk et al. (1994) used a nonlinear optimization model to minimize pumping costs for both a wellfield and a main water-supply distribution system. Considerations were given to individual well losses, pump efficiencies, and the hydraulic losses in the pipe network. As usual, this required the coupling of a simulation model with an optimization model and the resulting coupled system was solved using the general nonlinear optimization program MINOS. The optimization technique (MINOS) was clearly demonstrated in the study as a very useful tool for solving constrained nonlinear optimization problems, such as those being encountered in the current research study.

Takahashi and Peralta (1995) used the management model to find the optimal perennial yield planning for complex multilayer nonlinear aquifers. Embedding (EM) simulation/optimization modelling procedures was implemented. This approaches satisfactorily addressed the nonlinearities posed by over 2000 piecewise-linear constraints for evapotranspiration, discharge from flowing wells, drain discharge, and vertical interlayer flow reduction due to desaturation of a confined aquifer. The model deals with a confined aquifer; therefore, response matrix approach would be more appropriate to use rather than using the embedded optimization technique especially when linear constraints were implemented in this study.

Hallaj and Yazicigil (1996) constructed seven groundwater models to provide decision makers with optimal management policies to aid planning and operating of a coastal aquifer in southern Turkey threatened by saltwater intrusion. Their objective was to optimize development and operation of the Erzin plain groundwater system while minimizing the potential impacts of seawater intrusion. A finite element simulation model that was linked to linear and quadratic optimization models using the response matrix approach represented the hydraulic response of the aquifer system. Five of these models were developed for steady-state conditions, whereas the remaining two models were developed for the transient conditions. Modelling results indicated that significant increase in total aquifer yield were possible with controlled drawdown so that the infringement of saltwater is prevented. The model deals with an unconfined aquifer; therefore, embedded approach with finite-difference method would be more appropriate to use rather than using the response matrix with finite-element method.

McPhee and Yeh (2004) demonstrated the use of groundwater simulation and optimization model to construct a decision support system for solving a groundwater problem associated with the Upper San Pedro River Basin, located in south-eastern Arizona, USA. The case was treated as multi-objective optimization problem in which environment objectives are explicitly considered by minimizing the magnitude and extent of drawdown within a pre-specified region. The management model aim was to define a set of best groundwater pumping and recharge policies in a basin where groundwater is the main supply source. The management model provided two important kind of information. First, used the payoff matrix which allows decision makers to know what the best and worse values of objectives considered are. Second, the tradeoffs were quantified, therefore providing direction in terms of desirable and attainable management policies. The problem stated in this model uses a mixed-integer nonlinear programming problem which is extremely difficult to solve and, in general, global optimality can not be guaranteed. Therefore, it would be more realistic if embedded approach was used to link the simulation and optimization models rather than linearizing the problem to use MILP.

Safavi et al. (2009) used the management model to focuses on the simulation-optimization for conjunctive use of surface water and groundwater on a basin-wide scale, the Najafabad plain in west-central Iran with low precipitation and high potential of evapotranspiration. The purpose of their management model was to minimize shortages in meeting irrigation demands for three irrigation systems subject to constraints on the control of cumulative drawdown of the underlying water table and maximum capacity of surface irrigation systems. Results of the proposed model demonstrate the importance of the conjunctive use approach for planning the management of water resources in semiarid regions. The model simulation-optimization for conjunctive use will help in formulating the management model of the current study to meet the steady increase in domestic water demands by blinding the limited available groundwater with the costly desalinated water in Oman.

Wagner and Gannett (2010) constructed a groundwater management decision model for the upper Klamath Basin located in Oregon and California, USA, to couple groundwater simulation with optimization in order to identify strategies that best meet the resource allocation goals of the basin. The model is set to meet the complex set of goals and constraints associated with groundwater uses in the basin such as water demands for wildlife habitat and irrigation. The formulated groundwater management model has an extensive set of constraints such as limit the reduction in groundwater discharge to streams and lakes, seasonal, long-term drawdown and imposes geographic and seasonal demands on groundwater pumping. This model approach is very unique in formulating groundwater management model with an extensive set of constraints. This approach could be useful when formulating the constrained management modelling of this research.

The management model in this study used GAMS (General Algebraic Modelling System) software tool. GAMS is a high language program for formulating models with concise algebraic statements that are easily read by modellers and computers alike, easily modified, and easily moved from one environment to another. It is independent of the solution algorithms of specific solvers. Linear, nonlinear, integer, and mixed integer problem can be solved with GAMS (Brooke et al., 1988 and Vigerrske, 2009). It is such flexibility that has made GAMS so popular for solving groundwater management problems. For example, Gharbi (1991) used GAMS code to study the optimal groundwater quantity and quality management with application to the Salt Lake Valley in USA. The outcomes showed that GAMS code could successfully compute the pumping values which represent an optimal sustained yield pumping strategy and that computed strategies are very stable with respect to assumptions. Gordu et al. (2001) also used the GAMS software to develop an optimization model to manage the supplemental use of groundwater in a costal aquifer subjected to saltwater intrusion in the Goksu Delta at Silifke in Turkey. . The results showed that the predicted hydraulic heads by the simulation model matched the observed very closely. However, due to nonlinear effects, the correlation between the predicted and observed chloride concentrations was not as good. Since the over-riding factor dictating the influx of saline water in salt water intrusion problems is the drawdown (i.e. the hydraulic head), failure to accurately simulate the salt concentration was seen less of a problem for the objective of the optimization, as long as the hydraulic heads have been simulated well. Vieira et al. (2011) recently constructed a management models that coupled simulation and optimization models aimed at helping water utilities determine the best way to operate large-scale multisource water-supply systems. The operation of the systems is optimized taking into account the main planning objectives that include reducing operating costs, satisfying demand, delivering water of appropriate quality, and not prompting the use of emergency sources. The model is a highly nonlinear programming problem and is solved with the GAMS code using the MINOS algorithm. This model approach is unique in term of using a highly nonlinear programming similar to optimization problem of management problem for this research. However, Vieira et al. have used the matrix response approach to couple the simulation and the optimization models rather than using the embedded method, which is more applicable for water table aquifers found in the Ash Sharqiyah study area.

Thus in the above studies , it has been repeatedly demonstrated that the GAMS software is highly versatile for solving highly nonlinear constrained optimization problems similar to that being investigated in the current research. This consideration has influenced the decision to use the GAMS for the management model of the current research.

2.7 Summary

This Chapter reviewed some of the important hydraulic terms which describe the groundwater movement in different types of aquifers. Darcy's Law and the continuity equation, which govern groundwater movement, were used to derive the three-dimensional flow equations for both steady and unsteady states conditions. The three-dimension finite-difference formulation for solving the partial differential equation of groundwater flow was described in detailed.

The Chapter has also reviewed the literature of simulation and optimization techniques in groundwater modelling and management, ranging from the optimal location of pumped wells, to determining optimal pumping strategy that minimizes pumping cost, to assessing the long term impacts of pumping strategies on the sustainability of groundwater systems. Some of the other reviewed optimization problems in this Chapter have been driven by water quality concerns, especially in coastal aquifers where the objective has been determining pumping strategies that avert salt water intrusion. It is clear from the review that for any groundwater optimization problem to be feasible, a simulation model that can accurately describe the response of the system to hydraulic stresses, i.e. pumping, must be available.The need to have such simulation tools has engaged modellers for a long time and there now exist proven and tested commercially available software tools such as the MODFLOW that was applied in this work. Optimization is commonly formulated as linear programming, non-linear programming, dynamic programming but more recently, formulations using evolutionary programming such as Genetic Algorithms have been applied with varying degrees of success as discussed in the chapter. While linear programming is useful and the most widely used in general optimization problems, it is limited by the fact that both the objective function and the constraint equations of the problem must be linear (or linearisable) in the decision variables. For groundwater systems, especially unconfined aquifers, this is often not the case, thus making it necessary to apply non-linear optimization.

A further aspect of groundwater optimization that became evident from the review relates to the coupling of the simulation model and the optimzation. This is important since although the optimzation model is finding optimal solutions to aquifer properties, e.g. the head as it is affected by the pumping, the relationship between these properties can only be known by the simulation model. The finite difference formulation of the flow governing equations thus forms constraint equations for the optimization model. The groundwater simulation model therefore can be coupled with an optimization formulation by a computer code groundwater management model to find optimal management objectives while satisfying all the constraints.

As reviewed in the chapter, one of two well-known methods can be used to implement the coupling: the response matrix and the embedding techniques. The response matrix method assumes a linear relationship between flow and head, which is valid for confined aquifer systems but not so for unconfined systems. Where the response matrix method is applicable, however, the resulting solution of the optimization is much faster since evaluation of the heads will only be required at target wells only. The embedded method, as the name implies, embeds the flow governing equation for the entire modelled domain directly as constraints in the optimization. Consequently, the flow equation must be solved at all the finite difference grid nodes, making the optimization a much slower process than the response matrix approach. Embedded approach can also suffer from the curse of dimensionality especially for large domains and fine solution mesh. However, as emphasized in the review, the proper analysis of unconfined systems can only be done using this embedded approach.

Finally as was the case with groundwater flow simulation models, there are also commercially available management models for coupling simulation and optimization models to tackle groundwater management problems. The choice of the GAMS (General Algebraic Modelling System) utilized in the current research was informed by its flexibility by being independent of the solution algorithms of specific solvers and thus supports linear, non-linear, integer, mixed integer, etc. optimization formulations.

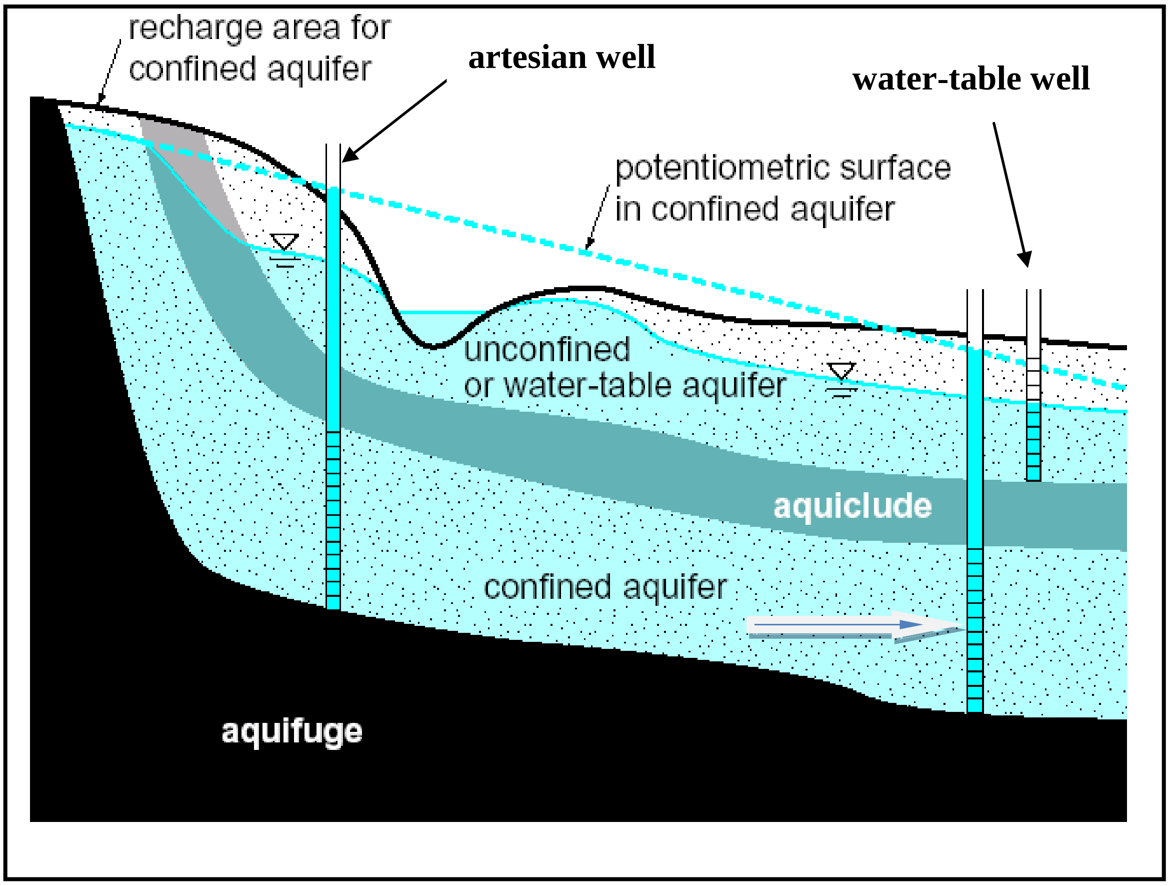


Figure 2.1: Diagram showing types of aquifers and subsurface water distribution (Adopted from Hornberger et al., 1998)

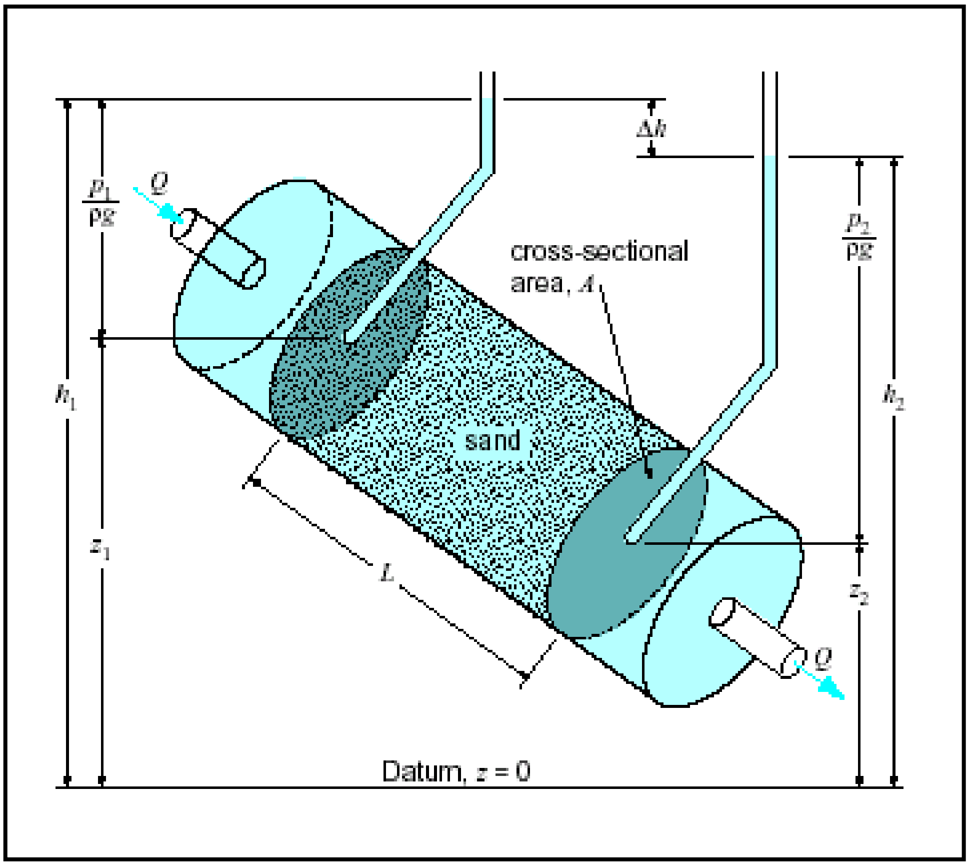


Figure 2.2: Diagram showing generalization of Darcy's column (Adopted from Hornberger et al., 1998)

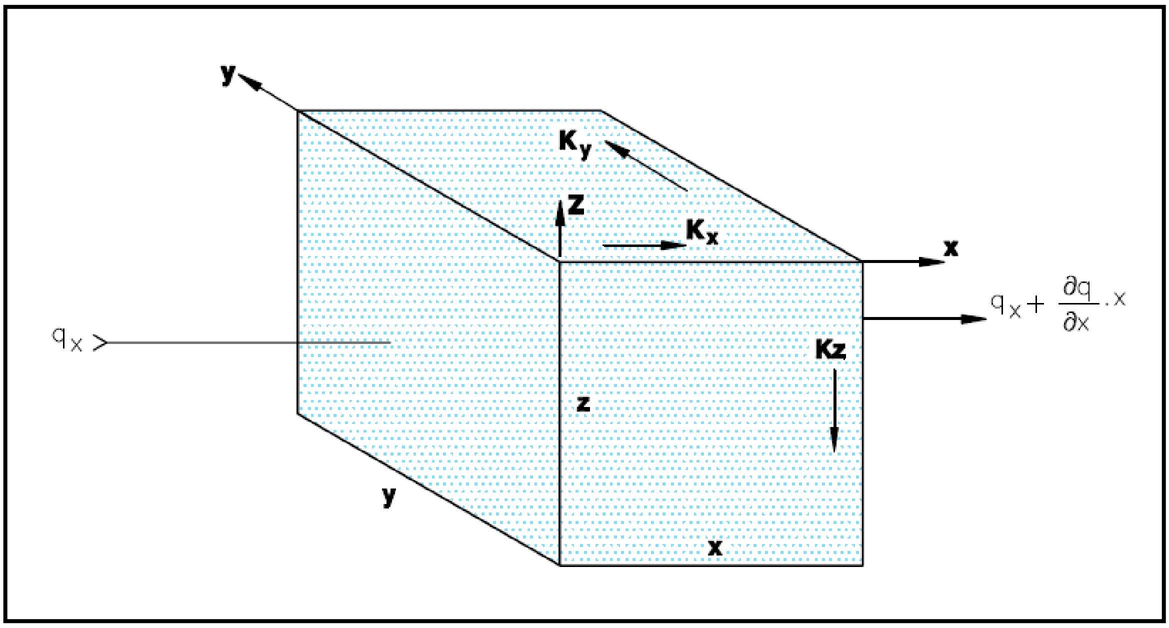


Figure 2.3: Diagram showing element of saturated porus sand (Adopted from Shaw, 2004)

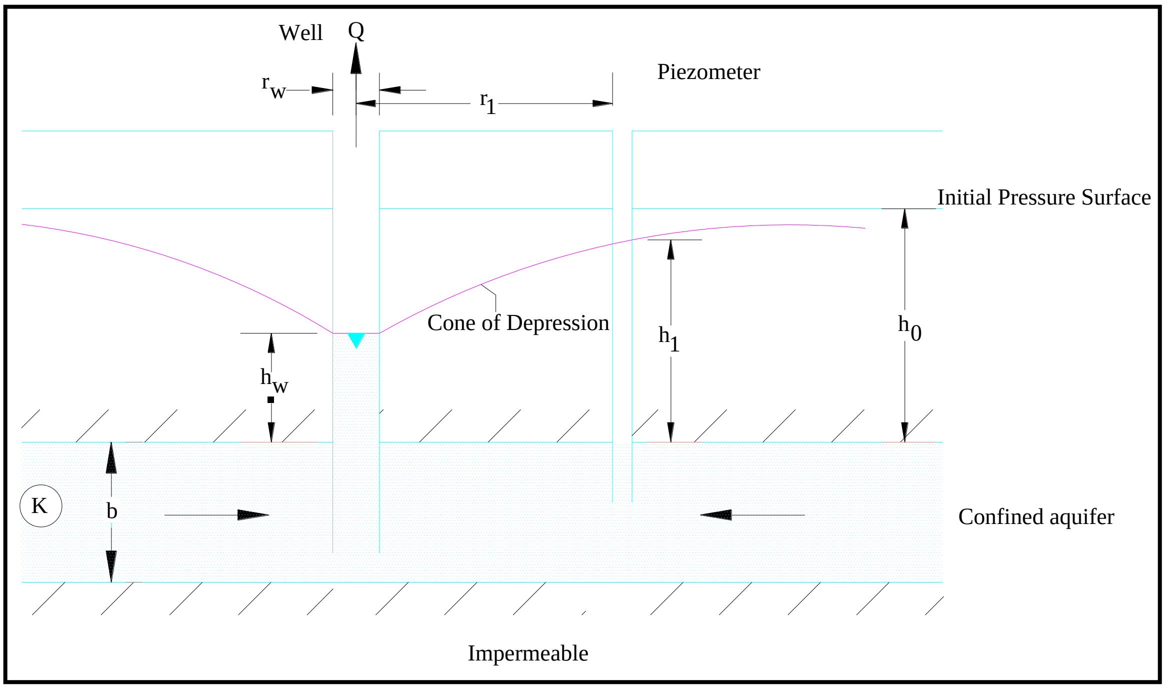


Figure 2.4: Steady flow to a single well in a confined aquifer (Adopted from Shaw, 2004)

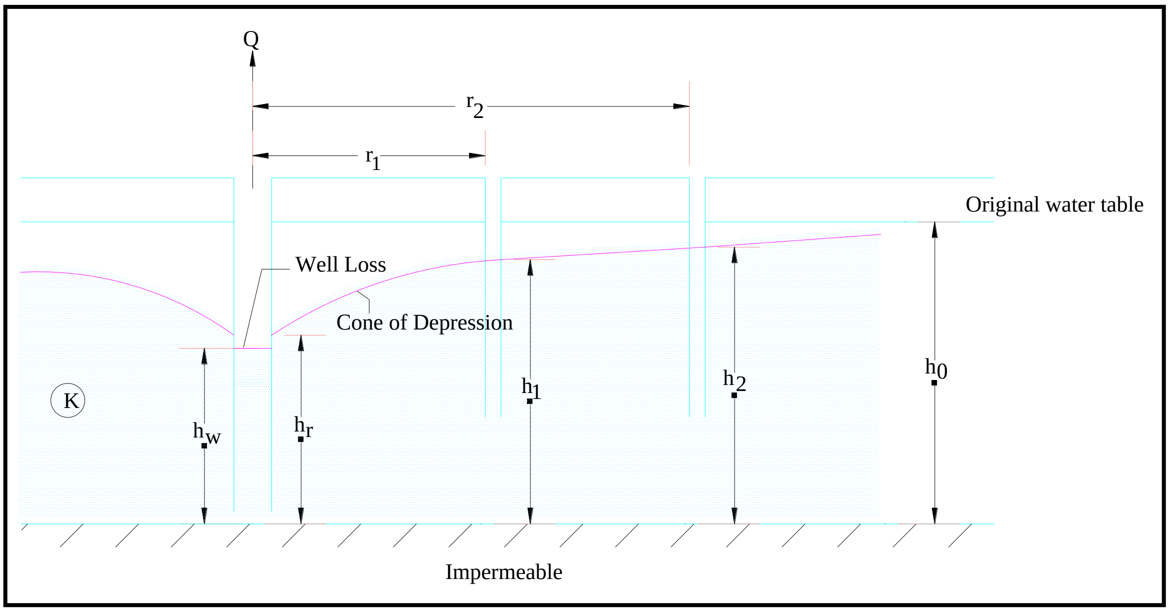


Figure 2.5: Steady flow into a well in an unconfined aquifer (Adopted from Shaw, 2004)

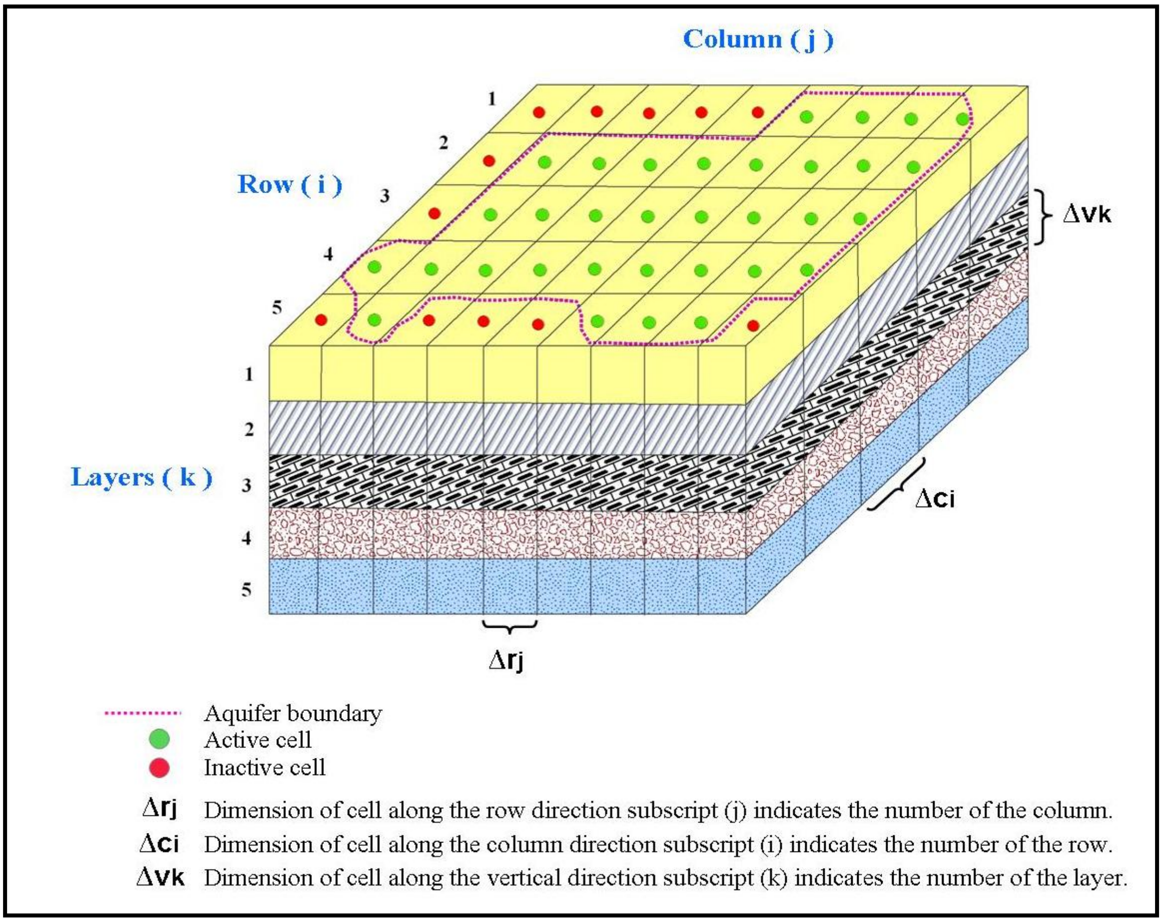


Figure 2.6: A discretized hypothetical aquifer system (after McDonald and Harbaugh, 1988)

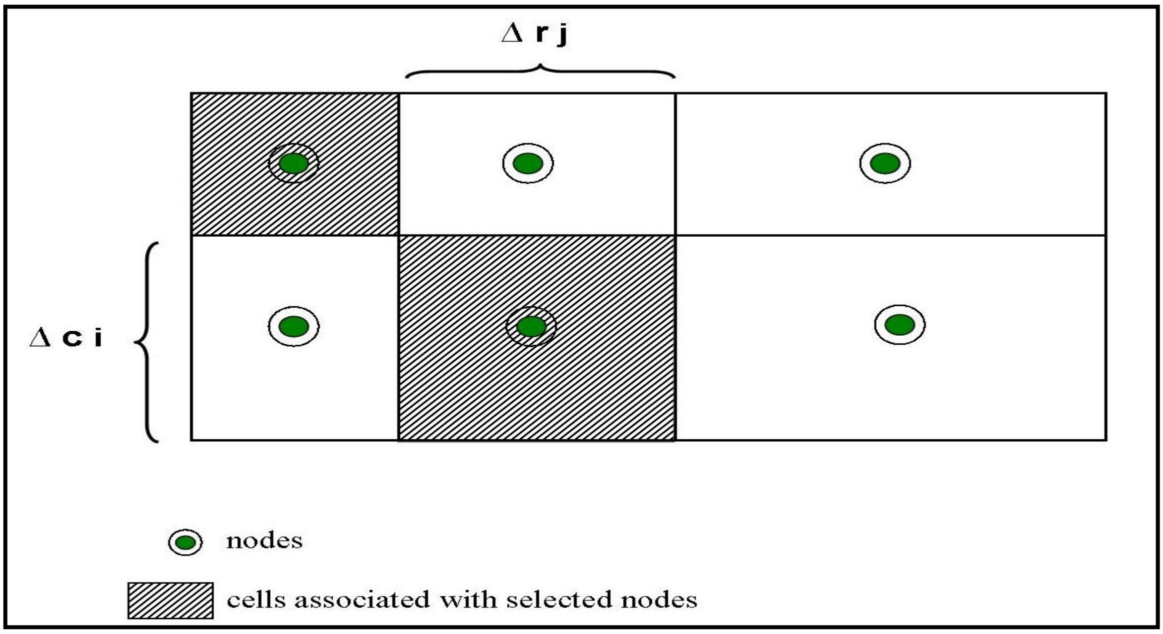


Figure 2.7: A block-centred grid system (after McDonald and Harbaugh, 1988)

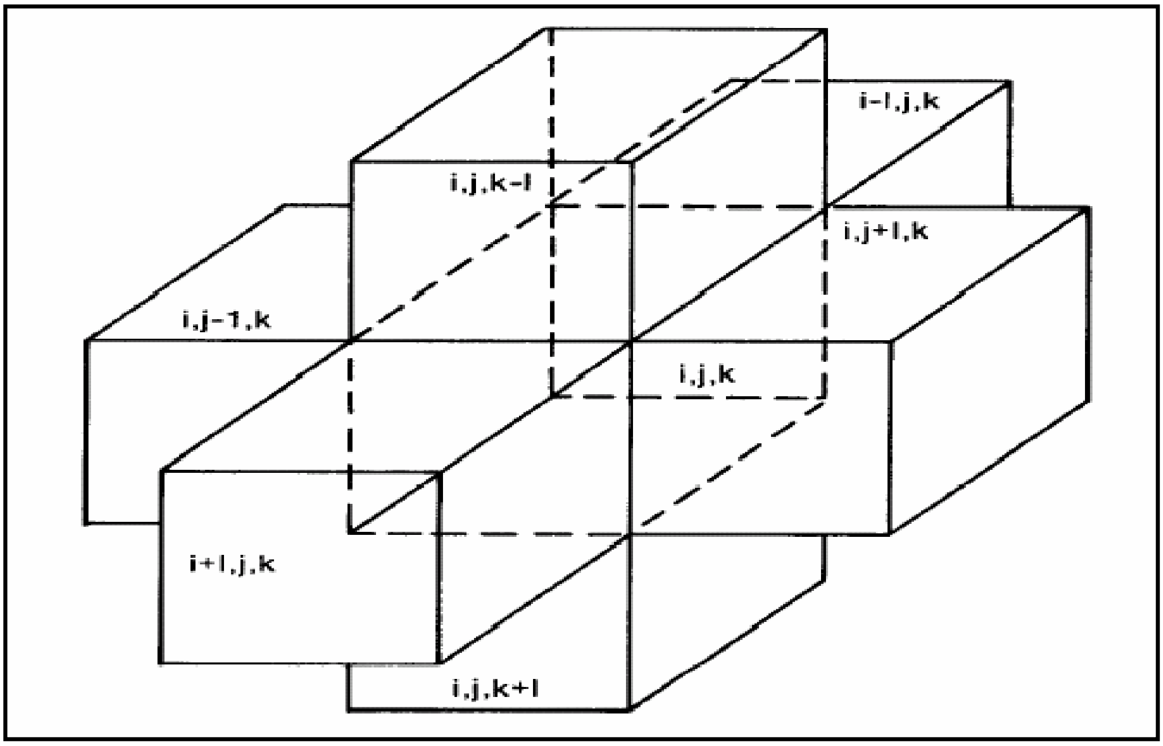


Figure 2.8: Cell *i*, *j*, *k* **and indices for the six adjacent cells** (after McDonald and Harbaugh, 1988)

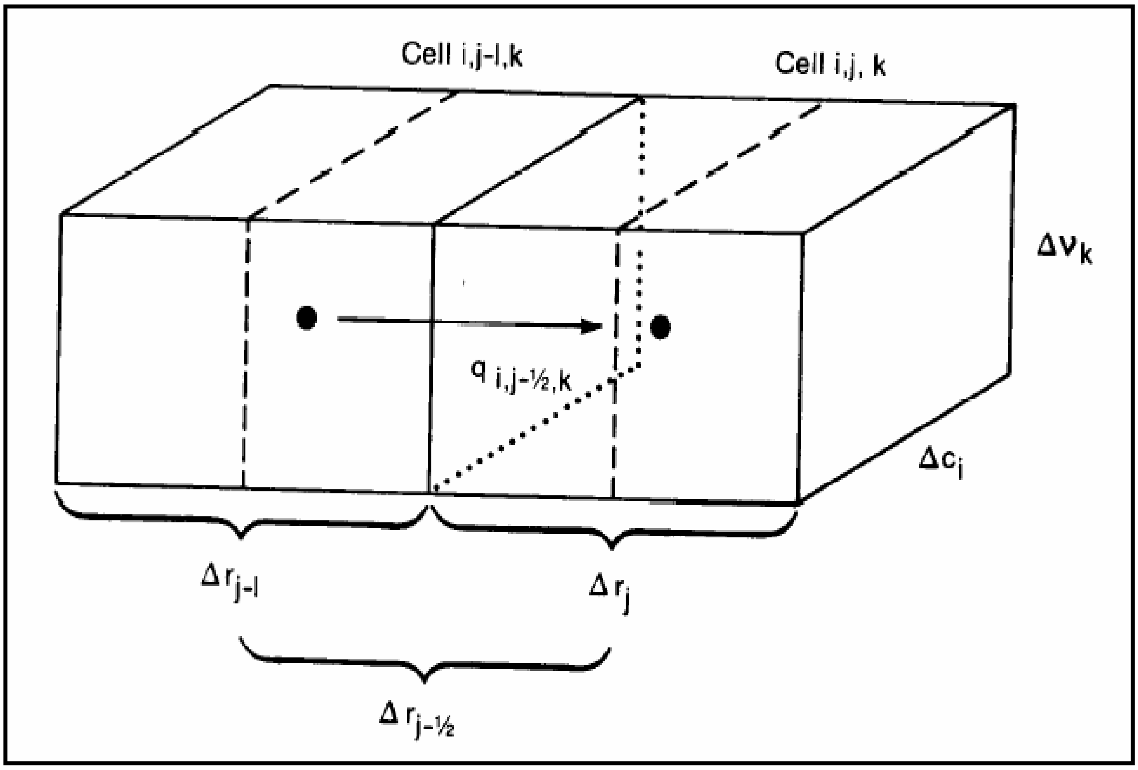


Figure 2.9: Flow into cell *i*, *j*, *k* **from cell** *i*, *j* −1, *k* (after McDonald and Harbaugh, 1988)

CHAPTER 3

WATER RESOURCES OF THE SULTANATE OF OMAN

3.1 Introduction

This chapter presents an introduction to the Sultanate of Oman and its climate, with a brief description of the water resources and water supply arrangements commonly used across the Sultanate. Such a review of water resources and the water structures in Oman is essential in order to understand the reasons for the current water deficit nationally and to justify the importance of implementing conjunctive use of both desalinated and groundwater resources for domestic water supply.

3.2 General information about Oman

The Sultanate of Oman is located in the south-east of the Arabian Peninsula. It has a total area of about 309,500km^2 and a total population of 2,340,815 according to the 2003 Census (MONE, 2003). About 15 % of the country is mountainous with the highest peak reaching up to 3050m above sea level. Geologically, Oman is an open book of the finest geological examples because most of its past is clearly visible on the surface (Clarke, 1990). As can be seen in Figure 1.1, Oman is bordered on the east by the Oman Sea, on the southwest by Yemen, on the west by Saudi Arabia, on the north by the Arabian Gulf and on the northwest by the United Arab Emirates. Oman is divided into four governorates (named *muhafazah* in Arabic), and five regions (called *mintaqah* in Arabic) (see Table 3.1). The governorate and region are similar in terms of services provided to the citizens but politically, the governorates are superior.

The annual demographic growth rate according to the 2003 Census is 2.79% and the average population density is 7.6 inhabitants/ km². Muscat is the capital city with an area of 3,900 km² (1.26% of the total area), 632,073 inhabitants (27% of the total population) and 162.1 inhabitants/ km² in 2003. By comparison, Ash Sharqiyah, where the current study area is located, has an area of 36,400 km² (11.76% of the total area), 313,761 inhabitants (13.40% of the total population) and 8.6 inhabitants/ km² as per the latest 2003 Census (see Table 3.1).

Generally, the Sultanate of Oman is characterized by hot, dry summers and mild winters. Summer begins in May and ends by October with an average temperature range of between 24°C to 37°C, except for the highest mountain peak where it is milder, i.e. 17-23°C. The winter season extends from November to April with an average temperature range of between 20°C to 26°C. Temperatures at the highest mountain peak ranges between 6-14°C in the winter months (Chebaane, 1996). The average annual rainfall throughout most of Oman is relatively low; less than 100mm, but in mountain areas rainfall can reach up to 350mm. In volume units, the average annual amount of rain falling on Oman is estimated to be about $19{,}250\text{x}10^6\text{m}^3$. Of this total, some 80% is evaporated leaving approximately $3{,}850\text{x}10^6\text{m}^3$ as effective rainfall of which 25% is run-off to the sea, and the remaining 75% is direct infiltration to groundwater (MRMEWR 2005). Therefore, Oman is considered as arid to semi-arid country because it lies in an area that has both the low rainfall and high potential evaporation. Droughts of two or three years' duration are common in Oman as demonstrated by the long-term rainfall record for Muscat since 1895 (see Figure 3.1 (a)). Also, it is indicated from the 5-year rainfall moving average trend (see Figure 3.1 (b)) that rainfall hydrological cycle occurred nearly every seven years, i.e. every seven years of above rainfall average is followed by at least seven drought years or bellow rainfall average.

3.3 Water resources and water structures in Oman

In many areas of the Sultanate, demand for water exceeds its availability. The total national water deficit is estimated at $378x10^6 m^3$/year in 2001 (see Table 3.2). Rainfall, springs and Aflaj, which are considered as surface water, contribute about 35% of the total water resources. On the other hand, groundwater constitutes about 65% of available water resources (Binnie and Partners, 2000). The steady increase in population and the expansion of agricultural, industrial and tourism activities constitute a heavy burden on the water balance to the extent that water resources abstraction in some areas, notably in Al Batinah coast and Salalah coast of Dhofar (see Figure 1.1), has largely exceeded the rate of groundwater recharge. The situation has led to a continuous lowering of the water table and pollution by saline intrusion from the sea. Not all of the water used in Oman is metered. Consequently, records of water used have to be estimated from indirect measurements. The completion of the National Well Inventory in 1993 and National Aflaj Inventory in 1997, for the first time enabled derivation of reasonable estimates for water use throughout the country. The vast majority of water withdrawn (92%) is consumed for agricultural purposes (MWR, 2000). As noted previously, this derives mostly from groundwater but there are contributions from other surface water sources such as Ghaili Aflaj, Birkats and dams as will be explained in the following sections.

3.3.1 Aflaj

Aflaj (plural of *Flaj*) are mostly systems constructed for taping underground water. They are conduits which are dug in the ground to convey water by gravity from higher elevations to lower areas. Aflaj are man-made structures and many have been carrying water for hundreds, if not thousands, of years. Some Aflaj were constructed 150 years ago in northern Ash Sharqiyah Region such as those in Ibra – Mudharib area (Al Shaqsi, 1996). According to National Aflaj Inventory Project, commenced in 1997, there are 4,112 Aflaj, of which nationally 3,017 of them are operational (MRMEWR, 2001). There are 1,095 Aflaj have not being operational due to lack of continuous maintains and drought. Table 3.3 shows the number of Aflaj in different Governorates / Regions of the Sultanate. It can be noted from the Table that Al Wusta Region and two Governorates of Dhofar and Musandam do not have any recorded Flaj. Because of their uniqueness, importance and contribution to water resources in Oman without disturbance to the environment, UNESCO's Water Committee decided in July 2006 to include five of the Oman' Aflaj as world heritage sites (MRMWR, 2006). These are Al-Khatmeen, Al-Malki, Daris, Al-Jeela and Al-Muyasser. The government of Oman is providing the necessary support and state-of-art technology for the maintenance and renewal of Aflaj which are considered one of the main sources of irrigation water in the Sultanate.

Traditionally, Aflaj's water was used for drinking, domestic uses and irrigation. The discharge of the Flaj is related to several factors such as rainfall intensity and frequency, topography and geology, infiltration into alluvium and lateral formation, and hydro-geological properties of the formations wherein the groundwater is stored (Al Shaqsi, 1996). The sizes of Omani Aflaj vary from a Flaj that serves one or two families to those that cater for thousands of residents (MRMWR, 2006). Hence, small Flaj can be managed by one person named *Wakel* who handles all of administrative work on daily or annual basis, while large Flaj require the partnership of all the locals.

Nowadays, most of the Aflaj's water is used for irrigation. The annual total volume of water withdrawal of groundwater in Oman for agricultural purposes is estimated to be $1{,}131 \times 10^6 m^3$. Aflaj provide about 34% of the agriculture water consumption, which is equivalent to approximately 33% of the total water use in Oman (MRMEWR, 2001).

There are three types of Aflaj in Oman: Daudi, Ghaili and Aini as shown in Figure 3.2 and described below. Their classification depends on whether they utilize shallow or deep groundwater. The first two types are widespread in Oman, but the third type can be found in only few places and originate from groundwater springs (see Figure 3.2c).

Daudi Flaj consists of an underground tunnel often of tens of km in length at depth reaching tens of metres at the source of water (mother well) (see Figure 3.2a). These Aflaj constitute about 23% of the total number of Aflaj existing in Oman. Daudi Aflaj are usually perennial in nature in which flow is available throughout the year. This type of Aflaj provides high discharges reaching up to 2000 l/s as the case of Flaj Dares in Ad Dakhiliyah Region.

Unlike other types of Aflaj, *Ghaili Flaj* consists of a surface channel reaching a depth of three to four metres and collects water from a wadi channel after periods of continuous rainfall (see Figure 3.2b). The discharge of such Aflaj increases instantly after rainfall events and decreases rapidly once the rain stops and remains dry during drought periods. The lengths of these Aflaj vary between 500 to 2000 metres. The width of Aflaj channel depends on the quantity of water which can be collected from the wadi. This type constitutes about 49% of the total Aflaj in Oman. The Ghaili Aflaj are highly localized, but some can reach tens of kilometres in length along wadis.

Aini Flaj is the third type of Aflaj**.** This type of Aflaj is fed directly from groundwater springs. Some of the springs, as described in the next section, are thermal, i.e. having warm water. Lengths of these Aflaj are short, extending from 100 to 200 metres. The number of such Aflaj is 28% of the total Aflaj in Oman.

3.3.2 Springs

A *spring* is a natural discharge point of groundwater at the surface of the ground. There are several hundred springs in Oman and most of them are located in the mountainous areas. They vary according to their discharge, temperature and water quality. Chebaane (1994) classified the springs in Oman as hot or cold springs; however, both have played an important role in the settlement of people in communities as their water has served as a local source of water for drinking, and irrigation. Some of them are known for their attractive landscape and therapeutic effects.

Springs in Oman discharge their water from limestone rocks or from ophiolites. The limestone springs flow through fracture and faults, and usually provide good quality water. Many of these are found in the Dhofar Governorate. Ain Razat is one of these springs and is famous for its high water yield that can reach up to 200 l/s for crop irrigation. On the other hand, the less permeable ophiolite springs discharge low quantity water and many of them yield alkaline water. Many of these springs exist in Ad Dakhiliyah, Ad Dhahirah and Al Batinah regions. These are more than 225 developed springs distributed all over governorates and regions of the Sultanate (MWR, 2000).

3.3.3 Birkats

Birkat (or pool in English) is an excavated chamber or naturally occurring hollow structure used to collect rain water. They are only found in the mountainous area of Musandam Governorate with the absence of surface water flows and limited aquifer potential due to the topographical, hydrological and hydrogeological conditions of the area. There, traditionally, the utilization of Birkats has been vital for the survival and development of many remote settlements because they have been the only source of water to meet domestic and livestock requirements. When these Birkats became empty due to no rain, the people in the area used to walk for long distance to find dug traditional wells.

An inventory of Birkats was undertaken by the Ministry of Regional Municipalities Environment and Water Resources between February and July 2001 (Geo-Resources, 2001). A total of 967 Birkats were located in 385 inventoried locations of which 80% were found operational. Manmade (dug) Birkats are 86% of the total with a total storage capacity of approximately 78,000 m^3. Individual capacity ranges from less than one cubic meter to 2,540 m^3, but about 79% are small with capacity less than 100m^3 (Geo-Resources, 2001). The natural Birkats are openings in structures like openings, joints and fissures in hard rocks. The storage capacity of this type is less than 500m^3. These days the primary use of water from Birkats is for livestock. Nowadays, the use of Birkat's water for domestic purposes is limited because the people moved close to the coastal areas where the government is supplying them with desalinated water.

3.3.4 Dams

One of the purposes for constructing dams in Oman is to benefit from the wadi flow otherwise lost to the sea or the desert. Annual run-off is estimated at $963x10^6 m^3$ (MRMEWR 2005) which can be harnessed and used to artificially recharge groundwater aquifers. There are three main types of dam built in the country: recharge dams, storage dams and flood protection dams.

A *recharge dam* is constructed across an alluvial channel to capture water during floods. The stored and clarified water is released slowly to infiltrate thick alluvium downstream of the dam as illustrated in Figure 3.3. Recharge mainly occurs downstream of the dam and not in the reservoir itself because the reservoir bed becomes quickly sealed by silt

(MRMEWR, 2006a). One of the important benefits of artificially recharging groundwater in this way is the reduction of sea water intrusion in the coastal areas, especially on the Al Batinah plain. Al Khawd Dam was the first such dam constructed in the Sultanate in 1985. It is located in Muscat Governorate with storage capacity of $11.6x10^6m^3$. Table 3.4 contains details of the other recharge dams subsequently constructed in various regions of the Sultanate.

Storage dams are limited in Oman due to low rain run-off (5% of rainfall) and high rain evaporation losses (80% of rainfall). However, a number of small storage dams have been established in high elevated remote areas of Al Jabal Al Akhdar (green mountain), located in Ad Dakhiliya region, to provide water supply to isolated scattered communities. There with relatively low temperatures (6-20°) it is feasible to construct such kind of dams. The storage capacities of these dams vary from $240m^3$ to $10,200m^3$. There are now more than 60 small storage dams in the Sultanate with approximate total storage capacity of $750,000m^3$.

Moreover, Wadi Dayqah Dam is considered as the largest storage dam in the Sultanate. It is situated across Wadi Dayqah, in Wilayat Qurayat of Muscat Governorate. This Wadi have perennial water flow reaching up to $260x10^6m^3$ in 1997. The average annual flow of Wadi Dayqah during the wetter months is about $60x10^6m^3$ (MRMEWR, 2006b). Technical evaluations of the hydrology, hydrogeology and geology of the dam area were completed in 1993 which revealed that it is capable of yielding approximately $35x10^6m^3$/year. This will provide $20x10^6m^3$/year domestic supply to the Capital area of Muscat and $15x10^6m^3$/year for both domestic and irrigation water supplies to Wilayat Qurayat (MRMEWR, 2006b). The dam construction started in 2006 and was completed in 2010 with $100x10^6m^3$ reservoir storage capacity.

Although an arid country, floods in the Sultanate frequently occur and cause property damage and at times, loss of lives. Some Wadis in Oman are capable within minutes to provide flood peaks of the order of $20m^3/s/km^2$ due to a combination of high, steep sloping mountains and highly localised rainfall (Chebaane, 1996). Therefore, flood protection dams are very important for protecting life and property. Their water also contributes to graoundwater recharge especially at the beginning of their commission and before their reservoir beds become quickly sealed by silt. Additionally, they act as recharge dams when floods stop by releasing their collected water slowly. After the destruction caused by the cyclone “Gonu” in June 2007, the government launched a very ambitious programme for the construction of flood protection dams on major Wadis upstream of populated areas. More than five of these dams had been constructed until middle of year 2011 across the Sultanate.

3.3.5 Groundwater

Groundwater has traditionally provided the major source of clean water in Oman because it is readily available in many locations and requires little or no treatment. It constitutes about 65% of available water resources in Oman (Binnie and Partners, 2000). Groundwater is exploited almost up to the maximum in some areas, which has led in some areas to continuous lowering of water tables and sea water intrusion. Available data indicate that abstraction from groundwater in the Sultanate exceeds national recharge by 25% (Binnie and Partners, 2000). Therefore, the Ministry of Regional Municipalities, Environment and Water Resources has implemented a number of water exploration programmes in various regions of the country during the last thirty years. Water assessment activities resulted in the discovery of several important aquifers in various area of the Sultanate. Table 3.5 presents the most significant groundwater aquifers in Oman (MRMEWR, 2007). Ash Sharqiyah Sands Aquifer is one of the most important groundwater discoveries announced in 1996 by His Majesty Sultan Qaboos Bin Said, the Sultan of Oman. His Majesty the Sultan instructed that all necessary studies and investigation should be conducted to benefit from this drinking water supply. Chapter 4 will highlight the gathered information about this discovery as it is considered to be part of this research.

3.3.6 Desalinated seawater

Desalinated seawater has become an essential contributor to water supplies where fresh water resources are limited or unavailable. Generally, seawater is used only where sources of fresh water are not economically viable, or where there are constraints on pumping from groundwater as the case of Ash Sharqiyah Sands groundwater supply project. The total cost of the desalinated water in Oman including treatment (PH correction, hardness, etc) is considered to be in the range of 0.700-0.755 Rail Omani (RO)/m^3 (equivalent to \$(1.81-1.96)/m^3) from an efficient plant. The US Dollar (\$) is equivalent to 0.386 Rail Omani. It is still expensive compared to the cost of groundwater. The cost of production from a wellfield, at a reasonable distance from the point of demand, is estimated at 0.200-0.250 RO/m^3 (Binnie and Partners, 2000); i.e. about a third of desalination cost.

Al Ghubra desalination plant in Muscat Governorate was the first such plant in Oman and commenced operation in 1976 with an annual production capacity of $10x10^6m^3$ of water. Since then, it has been upgraded to produce an average of $52x10^6m^3$/year to supply about 90% of the Capital Muscat (Binnie and Partners, 2000). Barka desalination plant in south Al Batinah Region has been in operation since 2005 producing $20x10^6m^3$/year, primarily for Muscat water supply, south Al Batinah and Ad Dakhiliya. Sohar desalination plant in north Al Batinah Region has also started operation as of December 2007. It produces approximately $55x10^6m^3$/year, primarily for north Al Batinah and Al Buraimi Governorate water supply. A connection was completed in 2009 between the Sohar and Barka schemes and during emergencies, the Sohar plant will also be able to supply water to south Al Batinah and the Capital Muscat. One more big desalination plant at Sur in Southern Ash Sharqiyah Region (Figure 1.2) has been commissioned since 2010. It produces approximately $29x10^6m^3$/year ($80,000m^3$/day), to supply water to Ash Sharqiyah Region. Its water will be managed by this study along with the groundwater of Ash Sharqiyah Sands Aquifer to supply Ash Sharqiyah Region water demands up to 2030. Furthermore, there are many small units installed both on the coast using sea water, and in the interior using brackish water. As of 2010, desalination plants provide more than 35% of the potable water supplied nationally.

3.3.7 Treated wastewater

Generally, reuse of wastewater in Oman is uncommon as most of the domestic wastewater is disposed of through septic tanks. In the Capital Muscat, however, there are collection and treatment systems for some 25% of the population (MWR, 2000). Muscat Municipality has extended its sewage collection and treatment system to generate around $25.5x10^6m^3$/year of effluent since 2006, eventually increasing to an approximately $100x10^6m^3$/year by 2030 (Binnie and Partners, 2000). Currently treated wastewater is being used very effectively for irrigating ornamental and greening plants in some urban areas such as the Capital Muscat.

In Salalah city of Dhofar Governorates (see Figure 1.1), a major wastewater treatment and re-injection scheme has been commissioned. Approximately up to 80% of the Salalah city has been connected to the scheme since August 2003**.** The total scheme capacity is currently about of $20,000m^3$/day ($7.3x10^6m^3$/year) with future extension to double the current capacity (Binnie and Partners, 2000). The chlorinated tertiary level treated wastewater is injected as recharge water through tube-wells drilled parallel to the coast in an attempt to push back the existing sea water intrusion or at least stabilize the interface.

In addition, wastewater treatment plants are being installed and commissioned in the main towns in various governorates and regions of the Sultanate to benefit from renewable resources and to protect the groundwater from contamination. Similarly, there are plants for the industrial estates. In the near future, there will be considerable potential for increasing the use of treated wastewater, particularly for aquifer recharge and irrigation as more advanced wastewater treatment systems are constructed nationally.

3.4 Water conservation

Over the last decades the increasing demand for water has exerted great pressure on the fresh water bodies of the country. It is therefore essential to save every drop of water, build an awareness of, and continual concern about, water conservation into every aspect of life. The government of Oman is actively encouraging wise water use and water conservation by offering financial incentives to agricultural activities using modern, water-saving irrigation systems, industries that practice water reuse since they contribute directly to water-demand management and reduce effluents. Additionally, legislation has recently been passed that makes it compulsory for new housing, commercial and industrial estates to be fitted with water-saving devices, including rainwater harvesting devices, as well as comply with new drainage concept of zero increase in peak flow from developed areas when compared with the pre-development conditions (MRMWR, 2010).

3.4.1 Improvement of irrigation efficiency

Agricultural water use accounts for approximately 90% of the total water use of the country (Binnie and Partners, 2000); consequently to achieve any significant saving in water, it is essential for agricultural users to fully participate in conservation and management measures. Although the Ministry of Agriculture and Fisheries (MAF) is making efforts to introduce modern irrigation techniques, the traditional flood system remains the most common irrigation technique, which in comparison to sprinklers and sub-surface drip irrigation systems is notoriously wasteful of water (Shaki & Adeloye, 2006). Factors militating against the ready uptake of modern methods by Omani farmers include lack of technical know-how for operating the systems and the high cost of the equipment. In order to encourage farmers to take up the new techniques, the MAF has approved a financial subsidy for purchasing the equipment. A sliding scale is used with small scale farmers (less than 10 feddans or 4.2 ha) receiving up to 75 % of the capital outlay as subsidy. 50 feddans for Medium-scale schemes receive 50% while large scale farmers with holdings of 21 ha or more receiving 25% subsidy (MRMEWR, 2005). This intervention by the government is gradually making a difference in the uptake of modern irrigation systems especially among small scale farmers that make up the majority of farmers in the country (MRMWR, 2010).

In parallel with the incentives to farmers to the save water by encouraging them to adopt modern irrigation system, recent studies have also revealed that water quotas could be established for all wells according to the type and size of cropped area (Aldar Consultancy, 2006). In fact a study undertaken by the Ministry of Regional Municipalities and Water Resources revealed that there is a very high discrepancy between the quantities of water actually used for production and the theoretical amount required based on the crop type, the soil characteristics, etc. The problem is that most of the water is wasted and not put to productive use because of the wild flooding irrigation method commonly used as described before. The application of the water can also be better timed, e.g. applying irrigation water during the day when evaporation is very high is wasteful of water; much less water will be needed if application is done in the evenings (Aldar Consultancy, 2006). However, without a system of quota that restricts the amount of water that can be abstracted, it is very unlikely that farmers will be inclined to adopt such practical approach for saving water. It can therefore be argued that there is the possibility of water saving if strict regulations are imposed. This water rationing does not have to be limited to agricultural production alone; indeed, given the increase use of water in industrial and commercial activities in Oman in recent years, extend the same policy to industrial water use will be necessary.

3.4.2 Rehabilitation of water distribution networks

The Public Authority of Electricity and Water works on minimizing the Non -Revenue water resulting from, illegal connections, non-working meters, under registering of meters, errors in reading meters, damages to water lines and leakage through the distribution networks (MRMWR, 2010). The Authority has launched several projects to reduce the non-revenue water. Some of them have been implemented and others are under implementation, the most important are:

- Training the engineers and technicians on specialized programs intended for leak detection, reading errors and estimation of non-revenue water.
- Replacement of the defected meters.
- Monitoring the non-revenue water through modern (SCADA) systems.
- Implementation of (GIS) for asset management and inventory.
- Purchase of water leakage detecting equipments

- Carrying out consultancy studies to identify and manage leaks based district metering systems.
- Renewing and upgrading water distribution networks.

3.4.3 Water saving devices

A study by the Ministry of Regional Municipalities and Water Resources has revealed that there is the possibility of water saving using appropriate water saving devices, e.g. short flush toilet flapper, water saver low flow shower head and sink faucet aerator, which can be installed in touristic facilities commercial, private and government buildings (Space Designers International, 2006).

3.5 Summary

This chapter introduced the background to the water resources of the Sultanate of Oman. It presented an introduction to the Sultanate of Oman and its climate, with a brief description of the water resources and water supply arrangements commonly used across the Sultanate. It also included a review of water resources and the different water structures, water demand/supply balance in the country. Aflaj water system and its importance contribution to the water supply especially to agriculture sector were described in detail. Other important elements of water resources including groundwater, desalinated water and treated wastewater were also reviewed. Finally, the chapter ends with some water conservation measures that have been implemented in Oman in order to build an awareness and continual concern about water conservation. The chapter attempted to establish the causes and reasons for the current water deficit nationally.

Table 3.1: Governorates and regions of Oman with their areas and population (2003 Census, MONE)

Governorates / Regions	Area (km^2)	Population (inhabitants)	Population density (inhabitants/ km^2)
Muscat •	3,900	632,073	162.1
Dhofar •	99,300	215,960	2.2
Musandam •	1,800	28,378	15.8
Al Batinah	12,500	653,505	52.3
Ad Dakhiliya	31,900	267,140	8.4
Ad Dhahirah /Al Buraimi	44,000	207,015	4.7
Ash Sharqiyah	36,400	313,761	8.6
Al Wusta	79,700	22,983	0.3
Oman	**309,500**	**2,340,815**	**7.6**

• **Governorate**

Al Buraimi was established as a governorate in 2006. It used to be part of Ad Dhahirah

Table 3.2: Oman water balance (Binnie and Partners, 2000)

Governorates / Regions	Available ($10^6 m^3$)	Demand ($10^6 m^3$)	Deficit	
			($10^6 m^3$)	(%)
Muscat	18	22	4	18
Dhofar	74	104	30	29
Musandam	154	219	65	30
Al Batinah	586	766	180	23
Ad Dakhiliya	117	144	27	19
Ad Dhahirah /Al Buraimi	86	151	65	43
Ash Sharqiyah	229	236	7	3
Al Wusta	3	3	0	0
Total	**1267**	**1645**	**378**	**23**

Table 3.3: Aflaj distribution in Oman

MWR (2000) National Aflaj inventory report, Ministry of Water Resources

Governorates / Regions	Aflaj type			Total Aflaj	Operational Aflaj
	Daudi	Ghayli	Ainy		
Muscat	25	84	130	239	173
Al Batinah	193	925	443	1561	1209
Ad Dakhiliya	279	275	196	750	501
Ad Dhahirah /Al Buraimi	152	419	145	716	473
Ash Sharqiyah	318	290	238	846	661
Total	**967**	**1993**	**1152**	**4112**	**3017**

Table 3.4: Recharge dams in Oman (MRMEWR, 2006a)

No	Name	Location	Capacity ($10^6 m^3$)	Length (m)	Max Height (m)	Year Completed
1	Khawd	Seeb	11.6	5100	11	1985
2	Hilti / Salahi	Sohar	0.55	9063	4.5	1985
3	Quryat	Bahla	0.13	1620	5.3	1986
4	Khasab	Khasab	16	830	23.0	1986
5	Shariya	Khasab	1.50	740	9.2	1986
6	Mawa	Khasab	1.40	820	8.0	1986
7	AL-Jizi	Sohar	5.4	1234	20.4	1989
8	Tanuf	Nizwa	0.68	135	17.0	1989
9	Ghul	Hamra	0.45	415	7.6	1989
10	Kabir	Ibri	0.50	2664	8.9	1990
11	Ma'awil	Braka	10.00	7500	8.3	1991
12	Fulayj	Sur	0.78	530	7.5	1991
13	Fara	Rustaq	0.60	638	12.0	1992
14	Fulayj (Halban)	Halban	3.70	4500	7.7	1992
15	AL-Taww	Barka	5.10	3000	7.7	1992
16	Sahalnawt	Salalah	6.4	3315	21.8	1993
17	Ahin	Saham	6.80	5640	8.0	1994
18	Hawasinah	Kabura	3.70	5900	6.8	1995
19	ALAla-1	Bahla	0.04	185	4.5	1996
20	Al Ruhbah	Bahla	0.05	190	5.5	1996
21	Muaydin	Nizwa	2.50	3365	10.2	2002
22	Mistal1	Nakal	0.18	955	N/A	2004
23	Mistal1	Nakal	0.07	381	N/A	2004
24	Bani Kharus	Musannah	5.00	7300	6.2	2004
25	A'Sarooj	Mudha	1.35	16.8	25.5	2004
26	Sahtan-1	Rustaq	0.04	210	5.8	2006
27	Sahtan-2	Rustaq	0.07	172	8.9	2006
28	Al Awabi	Al-Awabi	0.29	130	6.5	2006
29	Al Khab	Diba	2.80	500	17.4	2006
30	Thumaid	BidBid	0.10	48	7.7	2006
31	Al Guwaif	Biraimi	0.42	500	17.4	2006

N/A: not available

Table 3.5: Significant groundwater aquifers in Oman (MRMEWR, 2007)

Aquifer Name	**Location**	**Storage ($10^6 m^3$)**
Nejd	Dhofar	5,000
Al Masarrat	Ad Dhahirah	19,500
Ash Sharqiyah Sands	Ash Sharqiyah	12,000
Wadi Al Ma’awil	South Al Batinah	100
West Al Wusta	Dhofar	1,000
Wadi Rawnab	Al Wusta	100

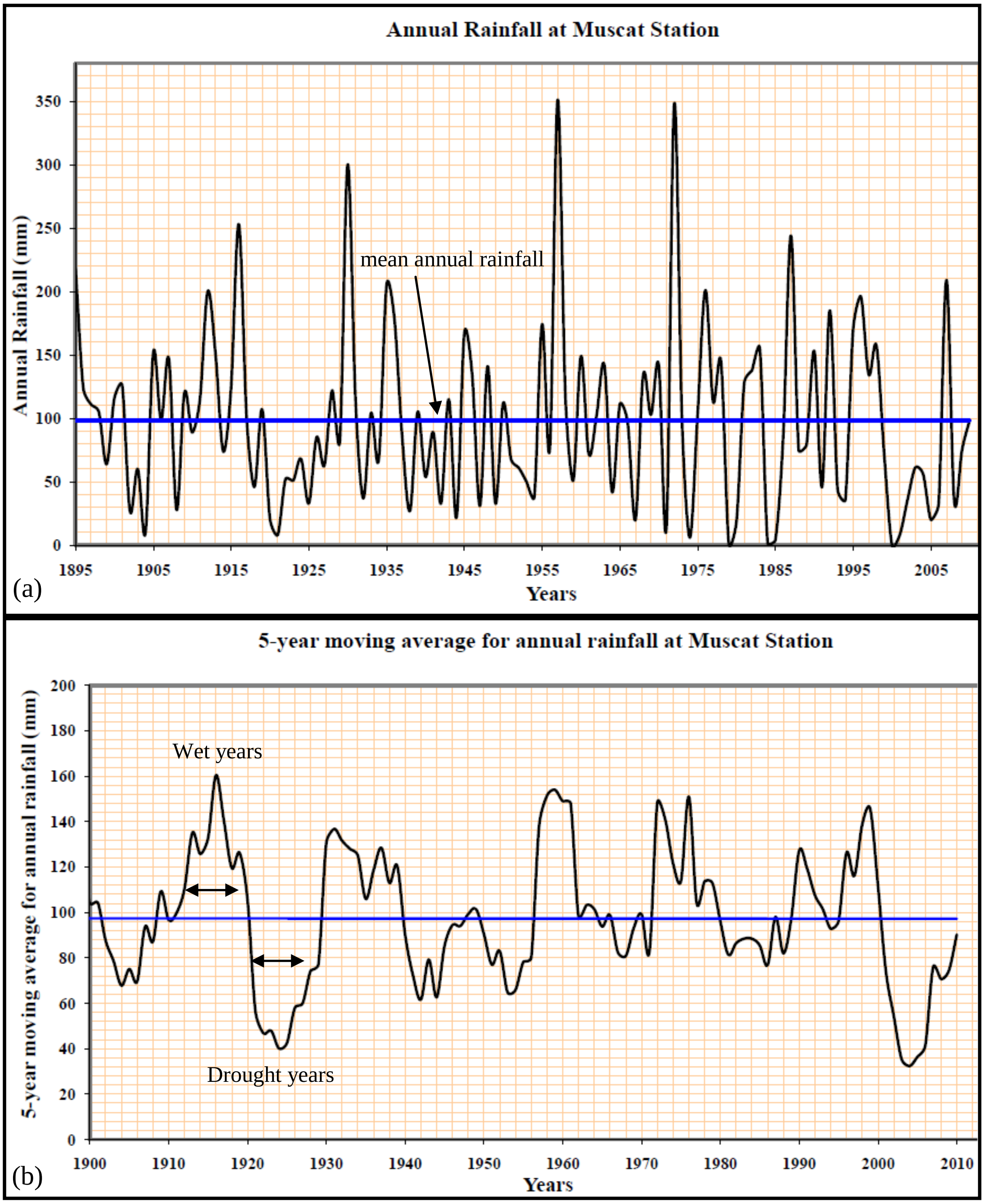
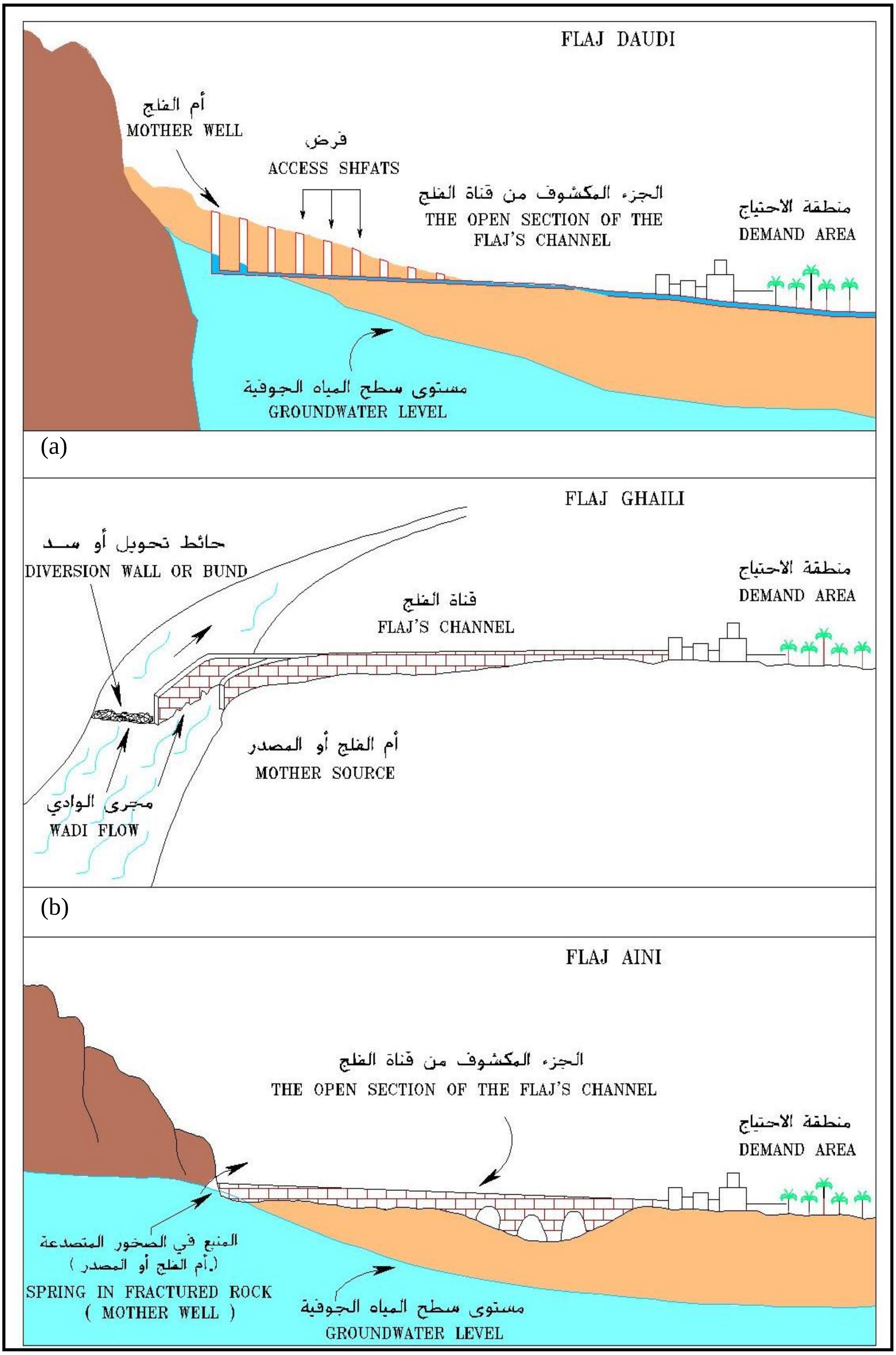


Figure 3.1: Variability of Muscat rainfall 1895-2010



(c)

Figure 3.2: The three types of Aflaj in Oman: (a) Daudi, (b) Ghayli and (c) Ani (MREWR, 2001)

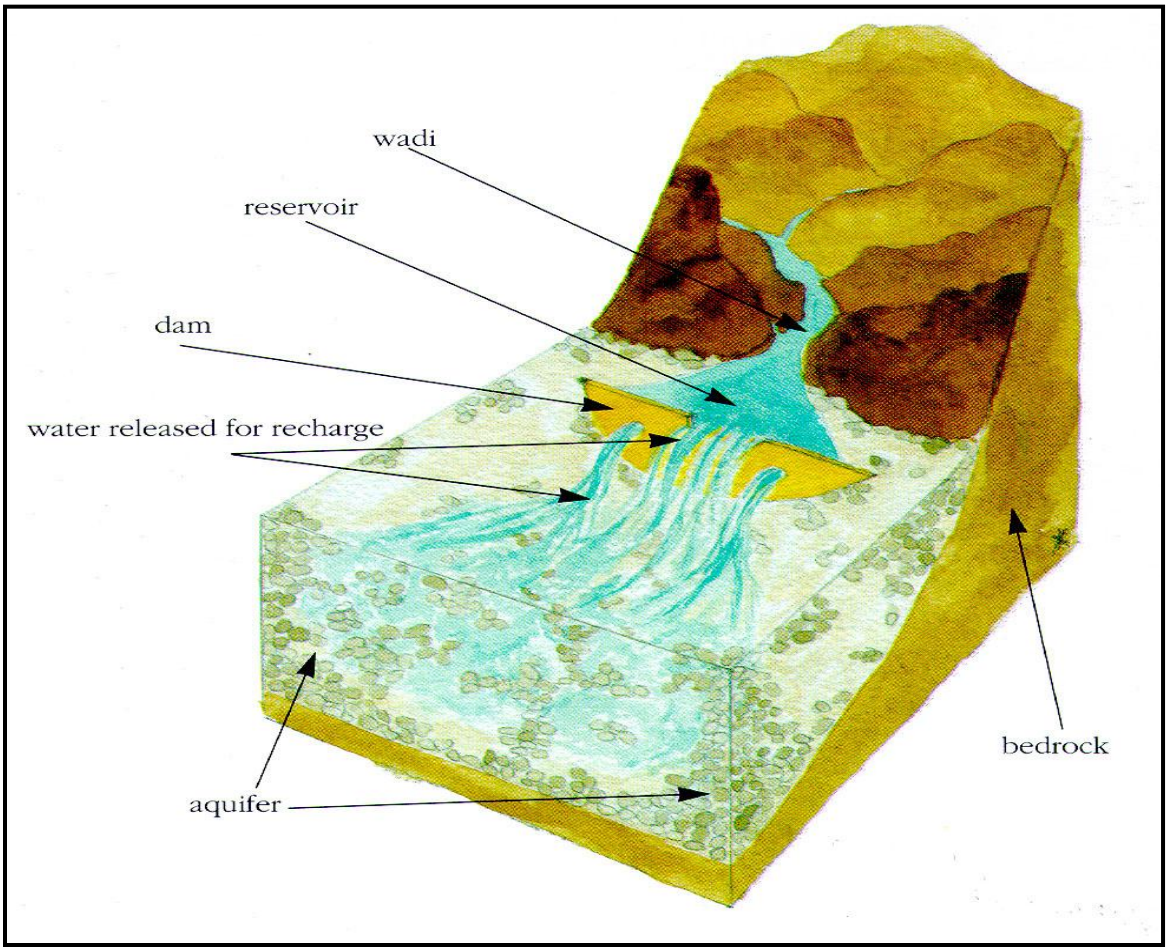


Figure 3.3: Schematic of a recharge dam (MWR, 1998)

CHAPTER 4

CHARACTERISTICS OF STUDY AREA

4.1 Introduction

This chapter presents the general description of the study area. Since the current study is about the Ash Sharqiyah Sands Aquifer which is located in Wadi al Batha Basin, the Chapter starts with the geography of Wadi al Batha Basin and its geological setting. A brief description of the hydro-geological interpretation of the two deposits (alluvial and aeolianite) within the study area is also discussed. Data collected for the groundwater simulation model are also presented. This is then followed by summarizing the previous studies carried out within the study area. The existing Ash Sharqiyah Sands Groundwater Supply Project, which is considered the major groundwater source for domestic water supply in southern Ash Sharqiyah Region, is also described in this Chapter. This is followed by describing the Sur Central Desalination Plant in which its water will be mixed with groundwater coming from the existed Ash Sharqiyah Sands groundwater supply scheme. Finally, water demand projection criterions up to the year of 2030 are discussed for the eight targeted Wilayats of the Sharqiyah Region.

4.2 Geography of the study area

The groundwater resources of the study area (Wadi al Batha Basin) are naturally recharged by high intensity, low frequency, rainfall events that generate runoff in the hard-rock mountain catchments of what is called Al Hajar Ash Sharqi (see Figure 1.3), which rise to 2,200mamsl in Wadi Bani Khalid (MWR, 1997a). Up to 80% of run-off evaporates, but a proportion infiltrates and drains south and south-east into the alluvial plain of Wadi al Batha Basin and Al Sharqiyah Sands. In the north-west of the basin, there are several Wadis that converge to form the main Wadi al Batha drainage plain, which later joins the run-off flowing from Wadi Bani Khalid and other Wadis. Wadi al Batha storm water is forced to flow eastwards by the northern edge of Ash Sharqiyah Sands body to continue its journey to the Arabian Sea (see Figure 1.3).

Historically, hand-dug wells and Aflaj have been the main sources of water to supply the agriculture along Wadi al Batha, which is dominated by dates, fruit trees (mainly limes), grasses, and winter vegetables. The main towns in the study area are considered as major sites of traditional agricultural development including Al Kamil, Al Wafi, Jaalan Bani Bu Hassan and Jaalan Bani Bu Ali (see Figure 1.3).

As shown in Figure 1.3, in Wadi al Batha, just north of Al Kamil to Jaalan Bani Bu Ali, occurs the largest natural woodland forest in the Sultanate. The variable density ‘Prosopis Belt’ extends over an area approximately 85km long and 20km wide (Brown, 1988) and is a significant groundwater consumer, as will be explained later.

To the south of Wadi al Batha is located Ash Sharqiyah Sands (see Figure 1.3), which is roughly triangular in shape. Its maximum width is about 100km wide (east to west) and 200km long (north to south), and covers an area of approximately 12,000km^2 (Warren, 1988a). To the east, the sands are bounded by a discontinuous area of sabkhas (wet salty soil) and the Arabian Sea (see Figures 1.3). Part of the Sands is included in the Ash Sharqiyah Sands Aquifer boundary. The Sands mostly consists of longitudinal dunes with relief between swales and crests of up to 100m. Dune crests are nearly parallel and are generally one to two kilometres apart and can persist for many tens of kilometres in length. Generally, scrub vegetation and swales plants in these sands are scattered on slopes and crest of the dunes (Warren, 1988b). Scarcity of wells and difficulty of access have constrained the development of the Ash Sharqiyah Sands. However, small sparse settlements can be found between the dunes.

4.3 Geological setting of the study area

The simplified geology of the study area is shown on Figure 4.1 (MWR, 1997a). This simplified regional geological map was constructed with the aid of geological maps compiled for the Ministry of Minerals and Petroleum (BRGM, 1992).

The eastern Oman Mountains, which are located around the northern and eastern margins of the Ash Sharqiyah Sands Aquifer, are formed from eight distinct classifications of rocks (Glennie, 1988; MWR, 1997a). These rocks from older to younger comprise:

(i) Pre-Cambrian gneiss and schists of Jabal Jaalan.

(ii) The Huqf (Cambrian) and Haima (Cambro-Ordovician) Group in Ash Sharqiyah Sands. A seismic reflector in Haima, referred to as the Intra Haima, is an important horizon extending laterally below the Sands. These comprise formations which alternate between carbonates and silicaclastics deposited in shallow-marine to terrestrial environment.

(iii) The Hajar Super Group, which is a thick sequence of Late Permian to Late Cretaceous shallow marine carbonates together with Early Jurassic silicaclastics. These rocks are exposed in an isolated outcrop 15km north of Al Kamil at the edge of Jabal al Hajor Ash Sharqi.

(iv) The Hawasina, which are deep marine sedimentary rocks (radiolarian chests and silicified carbonate turbidites) of the same general age as the Hajar Super Group. These rocks are exposed to the west of the study area on the edge of the northern Ash

Sharqiyah Sands, to the north along the edge of Jabal al Hajor Ash Sharqi, and between Jabal Jaalan and the coast.

(v) The Metamorphic Sole, which is an irregular contact of metamorphic rocks located between the Hawasina and lower portion of the Samail Nappe. The lower part consists of quartzites, schist and marble, and the upper part amphibolites. These rocks do not appear in the study area but do occur some 30km north of the study area (north-west of Mintirib in Wadi Dohir.

(vi)The Samial Nappe, which comprises a thick slice of former oceanic crust and mantle that now overlies the Hawasina and Metamorphic Sole. These rocks do not extend into the study area, but are exposed on the southern mountain front between ad Dahir and Zilaft.

(vii) Shallow-marine and terrestrial Tertiary formations, which may lie unconformably on all other rock units (a-f) above. Fars Group which comprise Miocene-Pliocene marine and terrestrial conglomerates, silts, clays, mudstones, limestones, sands and gravels have been cut by a number of borehole beneath the Ash Sharqiyah Sands These data together with information obtained from the TDEM and Seismic surveys indicate a deepening of the Fars, from west to east, beneath the study area (MWR, 1997a).

(viii) Quaternary alluvium is found throughout the study area in alluvial fans, terraces, wadi channels, and beneath the Ash Sharqiyah Sands and varies in thickness from a few meters to more than 100m. It consists of gravels, sands, and clays, with variable carbonate cementation. It is the main source of potable water in the region.

4.3.1 Alluvial deposits

In the central part of the study area, Tertiary and Quaternary alluvial deposition (see units g and h in the Section above) was heavily influenced by the formation of a basinal depression, probably formed during the late Oligocene to Early Miocene. The alluvial deposits represent the erosional products of Oman mountains area (up-lifted during the late Oligocene to early Miocene) transported southward by intermittent fluvial action under varying climatic conditions (Maizels and Anderson, 1988). Alluvial deposition in Wadi al Batha Basin is bounded by normal faults in the north, east and south as shown in Figures 4.1, 4.2 & 4.3. Creep has occurred on the faults during the latter part of the Tertiary and possibly also during the Quaternary ages due to the large thickness of alluvium, aeolianite and sands, which has caused subsidence in the basin. The base of the alluvium could be block faulted bedrock (MWR, 1997a). The area of alluvial deposition, however, is not entirely bounded by faulting. The edge of the alluvium is sedimentary east of Al Kamil and Jaalan Bani Bu Hassan, where the deposits gradually thin to zero at the foot of Jabal Jaalan. In the north-west of the study area, the approximate edge of the basin lies between Mintirib and Hawiyah. To the north-west, the alluvium is less than 50m thick and to the south-east the alluvium thickens rapidly to greater than 100m.

4.3.2 Aeolianite and sand deposits

Aeolianite and sand deposits occur on top of the alluvium (see Figures 4.2 and 4.3). Gardner (1988) and Glennie (1988) recognize three major aeolian sequences:

1. A strongly cemented older aeolianite sequence with low primary porosity, the pores being infilled with low Mg-calcite cement and halite. This sequence is composed predominantly of allochem material containing shell fragments, forams, peloids, and algae, and an insignificant terrigeneous quartzose component. This supports the suggestion of Glennie (1970), that the bulk of the aeolianite had originated by deflation of the near shore areas and continental shelf during periods of low sea level.

2. A coastal or younger aeolianite which is only loosely cemented and porous, retaining an important shelly carbonate component with a strong terriginous component.
3. A sequence of looser linear mega-dunes and meso-dunes, which are large north-south trending linear dunes being 1-2 km wide and ranging from 50 to 100 m high. The sands consist predominantly of quartz and carbonate. Pye (1984) suggested that the mega-dune were deflated from an exposed coastal plain during lower sea-level during glacial time, an origin similar to that suggested for the older aeolianites.

The existing Ash Sharqiyah Sands groundwater supply project provides water from both the alluvium and aeolianite.

4.4 Data collection

The main sources of data have been the various technical reports obtained from the Ministry of Regional Municipalities, Environment and Water Resources (MRMEWR) previously named as the Ministry of Water Resources (MWR) in Oman. Furthermore, technical reports on Sur Central Desalination Plant and the North Ash Sharqiyah Desalinated Water Supply Scheme were obtained from the Ministry of National Economy and the Public Authority for Electricity and Water. Apart from these formal reports, large amounts of unpublished data were also made available with the assistance of officials of these Ministries. The following data were collected and analysed by the Ministry of Water Resources during the exploration and the assessment of the aquifer in 1997(MWR, 1997a):

(a) - General information data such as longitude, latitude, and topographical elevation. This information is used to site the locations of wells, Aflaj and woodland (prosopis belts) in the study area, and helped to describe the study area.

(b) - Lithological descriptions of main formations to be used in drawing the layers and to know which of the layers is water bearing and which are not (MWR, 1997b). This information also helped to locate the cross sections and match the depth of the layer between the wells.

(c) – Drilling and Geophysical data, such as Time Domain Electromagnetic (TDEM) and seismic surveys were investigated (MWR, 1997c). These surveys are required to delineate the extent and geometry of the aquifers, their thicknesses and top/bottom of each layer as presented in Tables 4.1 and 4.2. Borehole logs were collected from 71 wells in the study area during the exploration drilling in 1997 (MWR, 1997d). They are needed to define the two aquifers: aeolianite and alluvium.

(d) – Hydro-geological data such as pumping test data, which were used to prove the sustainability of well yields and to determine initial values of aquifer properties, such as hydraulic conductivity, transmissivity and specific yields. Static water level data were used to draw the contour maps of the water levels and subsequently to establish the general direction of the flow in study area which is from northwest to southeast direction with horizontal hydraulic gradients across most of the area range from 0.0054 to 0.00063 (MWR, 1997e). The steepest gradients occur in the north-west.

The aquifer tests performed comprised short four-hour constant rate (carried out on 33 wells) and longer one-day to seven-day constant rate tests (MWR, 1997e) and the results were used to calculate transmissivity (T) and, specific yield (Sy) for the two layers as shown in Tables 4.3 and 4.4. Sites selected for longer term tests were chosen based on a review of the initial tests.

Step tests were used to find maximum discharge rates and to determine well efficiencies. Well efficiency is the ratio of the theoretical drawdown in a well divided by the actual drawdown obtained from the well completion test data. The efficiency of wells completed into the aeolianite was better (averaged 92%) compared to the wells completed in the alluvium (averaged 56%), (MWR, 1997e). The small well efficiency of the alluvium may be considered to be due to poor well completion.

Table 4.3 presents the safe yields of the wells, which ranged between 0.7 and 23 l/s for the aeolianite. T values ranged from 15 m^2/day to 1898 m^2/day and the average Sy is 0.16. The safe yield in a basin can be defined as a fixed quantity of water that can be withdrawn basically limited to the average annual basin recharge (Todd, 1980). On the other hand, Table 4.3 indicates that the yields of the alluvium wells were much higher, ranging from just less than 1 l/s and up to 84 l/sec. T vales ranged from 15m^2/day to 12,500m^3/day and Sy values ranged from 0.0023 to 0.31, also much higher in the alluvium aquifer, than the overlying aeolianite aquifer.

(e) - Groundwater quality data and the results of its analysis for potability assessment were also obtained (MWR, 1997f). These were compared with the maximum permissible limit (MPL) of the Omani Standards for Drinking Water (MCI, 1978). The maximum permissible limit (MPL) is 1,500mg/l for TDS which corresponds to EC value of 2,239µS/cm. EC limit of 2,500µS/cm is nominated for general delineation of potable water (see Figure 1.3). As noted previously, the EC = 2,500 µS/cm contour encloses the courses of Wadis al Batha and Bani Khalid and extends southward some distance beneath the aeolianite aquifer. The EC = 2,500 µS/cm contour for the aeolianite aquifer encloses a vast area of the north-eastern Ash Sharqiyah Sands area.

4.5 Summary of previous studies carried out within the study area

4.5.1 Hydrologic studies

During the 1970's and in the course of various water resources studies, more than 13 rainfall stations were installed (MWR, 1997g). A further 6 rainfall stations were installed in the period 1982-84. Further 39 rainfall stations were installed in 1993-94 (MWR, 1997g). Figure 4.4 shows isohyets of the average annual rainfall for the 23-year period 1975-1997 and key gauging stations at Wadi al Batha Catchment. The hydrological analysis of the Wadi al Batha Basin was carried out by the Surface Water Department of the Ministry of Water Resources (MWR) in 1997 (MWR, 1997g). The main results are summarised in the following sub-sections.

4.5.1.1 Meteorological data

Sur Meteorological Station in the Ash Sharqiyah Region, which was installed in 1974 at the coast, is likely to be more representative of conditions in the study area. There, the monthly mean temperature ranges from 22°C in January to 34°C in June with the average annual temperature is 28.4°C. The range of mean monthly maximum daily temperatures is 26.2°C in January to 40.3°C in June (MWR, 1997a). These temperatures follow much the same pattern as other locations in northern Oman.

The monthly mean of average daily humidity range of 50% to 70%, the least humid months being April to July, and the annual average sunshine is 9.6 hours (MWR, 1997a).

The monthly mean wind speeds range from 2.1 m/s in November to 4.7 m/s in July with dominance northerly winds during winter (November to April) and southerly winds in summer (May to October), (MWR, 1997a).

The Sur annual potential evapotranspiration rate is 3.3m, which is higher rate compared to the other existing meteorological stations in Oman (MWR, 1997a). That is because of the higher wind speeds recorded at the coast. Therefore, the overall potential evapotranspiration rate for the study area is likely to be closer to the annual rate of 2.9 m for Seeb station, which is not effected by higher wind speeds as of Sur (MWR, 1997a).

Average annual rainfall in the 33-year period, 1975 to 2007 for Ash Sharqiyah varies from 80 mm to 190 mm. Isohyets constructed in Figure 4.4 show that annual rainfall in the northern part of the study area can be expected to exceed 125 mm and in much of the Ash Sharqiyah Sands is likely to be less than 75 mm (MWR, 1997g). The three wettest months are February to April which account for more than 60% of the total annual rainfall and less relatively wet period is July and August (MWR, 1997g).

4.5.1.2 Wadi flows

There are 13 usable Wadi gauging stations within Wadi al Batha. Eight of these gauges are located in the middle catchment within or close to the main study area. All of these gauging stations are transducer type within ± 5% accuracy. They used to be additional several peak discharge gauges to report Wadi peak flow at each particular Wadi flow (MWR, 1997g). The average annual recorded inflows amount to approximately $18.3 \times 10^6 m^3$/year. The average annual flow from Wadi Bani Khalid accounts for the largest flows amounting to some $11.3 \times 10^6 m^3$/year, representing more than 61% of the total measured flows of Wadi al Batha (MWR, 1997g). The distribution of Wadi flow recharge input will be explained in Chapter 5.

4.5.1.3 Aflaj flows

Aflaj data, together with sample hydrographs have been collected in the study area of Wadi al Batha basin since 1982 (MWR, 1997g). Of the 74 Aflaj in Wadi al Batha basin, for which records exist, 31 fall within the main study area and on the edge of the Ash Sharqiyah sands. Average annual Flaj flow of the 31 Aflaj between 1982 and 1997 was approximately $33.4x10^6m^3$/year. This decreased between 1998 and 2007 to $24.9x10^6m^3$/year as presented in Table 4.5. The declining flows in some Aflaj are not only the results of drought and lack of maintains but the most likely explanation is that groundwater levels in these areas have been adversely affected by increased groundwater abstractions.

4.5.2 Remote sensing studies

The technology of remote sensing has been applied to assist a number of project studies within the study area. Three basic research activities were undertaken each focused on a major component of the regional water balance (MWR, 1997h);

- Regional vegetation analysis including differentiation of Aflaj- and non-Aflaj (wells) -fed agriculture.
- Evaluation of the extent and density of natural woodlands (prosopis cineraria) in the study area. The results indicated that the prosopis forest is a significant consumer of water and covers about 13,000 hectares (approximately 85 km long and 20 km wide) and the total number of prosopis trees is about 555,000 trees.

- Estimation of the area covered by Sabka (salty water) in the lower catchment, south of Jaalan Buni Bu Ali. It indicated 7.4 km^2 of "active" Sabka in the study area. Difficulties in estimating the total area of Sabka were encountered because of the nature of the "inactive" Sabka, which are often covered by sand/gravel making them difficult to distinguish from their surroundings. Outflow to Sabka is estimated $8x10^6 m^3$/year.

4.5.3 Water use studies

4.5.3.1 Agriculture irrigated by Aflaj

The Remote Sensing Section in the Ministry of Water Resources (MWR) calculated the type of agriculture and agricultural areas irrigated by Aflaj in the study area during 1995 and 1996. The net water demands were calculated from the evapotranspiration requirements of different types of crops with allowances made for leaching and irrigation efficiency (MWR, 1997h). The results of these calculations showed that $28x10^6 m^3$/year was the net irrigation water demands in the area served by Aflaj. This estimate can be compared with that derived from average annual Aflaj flow in the area which is estimated at $33.4x10^6 m^3$/year between 1982 and 1997 and had decreased between 1998 and 2007 to $24.9x10^6 m^3$/year, as discussed above in Section 4.5.1.3. For modelling purposes, an average of $24x10^6 m^3$/year will be assumed in the current study because not all of the Aflaj flow is used for irrigation.

4.5.3.2 Agriculture irrigated by wells

The National Well Inventory undertook an inventory of all wells in the Sultanate, including the main study area. The information collected during 1995 for this inventory included data on each well's location, physical dimensions, water-level, discharge, chemical quality and details of water use. The total net water demand for well-watered agriculture in the main study area is estimated at approximately $23.5 x 10^6 m^3$/year by using the Penman-Monteith Equation (MWR, 1997a). The main agriculture crops are Date Palms, limes, bananas, mangoes, alfalfa and grasses.

4.5.3.3 Water used by prosopis forests

The prosopis forests in the study area act as phreatophytic consumers of groundwater and are hydro-logically very significant when considering water balances. The prosopis receives groundwater flow from the Ash Sharqiyah Sands, from a west northwest direction. The annual net water demands for the prosopis were calculated from the evapotranspiration by the Remote Sensing Section in MWR. The total annual water consumption for 530,351 trees of prosopis in an area of 12,213.3 hectares north of Jaalan Bani Bu Ali was calculated to be approximately $47 x 10^6 m^3$/year and approximately $2 x 10^6 m^3$/year for the 24,614 trees of prosopis in an area of 673.6 hectares south of Jaalan Bani Bu Ali (MWR, 1997a).

4.5.3.4 Domestic, industrial, commercial and other municipal water demand

The National Well Inventory calculated the domestic, industrial, commercial, livestock and other municipal annual water demands. Household water demand was estimated by assuming as conservative per capita demand of 80 l/day. This per capital usage seems to be low because the water distribution network did not exist in 1995, hence the water supply was provided by tankers. However, it is more realistic to use 97 litre/capita/day in the management model than what was estimated in 1995 by the National Well Inventory because it is estimated base on several previous studies done in Oman (Parsons

International & Co LLC (2005). Industrial, commercial and other municipal demands were calculated by estimating the annual discharge from wells used for these sectors. The total water demand for the mentioned sectors was estimated to be in order of $6.7x10^6 m^3$/year (MWR, 1997a).

4.6 Ash Sharqiyah Sands groundwater supply scheme

Prior to the project implementation in 2004, the number of residential buildings supplied with potable water in the Ash Sharqiyah region was 27,953 buildings, which represented 84.4% of the total number of buildings. This is some 75.4% of the total population. The water was provided to the buildings by privately owned water tankers from “fixed tanker" points at every town. The residents also use Aflaj for potable water, in addition to its use in the irrigation of agricultural lands. In addition to the tanker points and Aflaj, some private wells inside individual properties are used as non-potable water sources.

To improve the water supply situation in Wilayats of Al Kamil and Al Wafi, Jaalan Bani Bu Hassan and Jaalan Bani Bu Ali, the government of Oman under the supervision of the Ministry of Water Resources (MWR) decided to execute the Ash Sharqiyah Sands Groundwater Supply Project. The construction started on 20 November 2001 and it was completed on 28 February 2004 (GULFAR / SADE consortium, 2001). The main project's objectives were to provide these three Wilayats with potable water from the new discovery of Ash Sharqiyah Sands Aquifer for domestic, commercial & industrial uses which meet Omani Drinking Water Standard, and to create new investments opportunities for the region. Approximately 79,000 people benefited from the project in 2004, requiring about a million cubic metres of domestic water. This water supply system is monitored and controlled by state of the art instrumentation and advanced control system (SCADA) which accurately measures abstractions, flows and water quality at the production wells, pumping stations, and transmission and distribution pipelines. In addition, a new system of using pre-paid water credit cards was installed at tanker filling stations in order to control the selling of water and to minimize water losses. Tankers have been used to serve the remote villages which were not covered by the water distribution networks. Figure 4.5 is a schematic illustration of the main components of the project.

Two wellfields were constructed in the first phase with 29 production wells and 19 monitoring wells. Eight production wells in the northern Al Kamil Wellfield supply the towns of Al Kamil and Al Wafi with distributed water network. The remaining 21 production wells of southern Jaalan Wellfield supply the Wilayats of Jaalan Bani Bu Hassan, Jaalan Bani Bu Ali also with distributed water network. Smaller communities continued to be supplied by tankers from 13 filling stations in these towns or along the transmission lines. In this phase, 115km-transmission pipelines with diameters ranging between 200mm and 800mm as well as the laying of more than 500km of water distribution networks pipelines of 100 to 400mm diameters. Two pumping stations, three storage reservoirs with varying capacities from $3330m^3$ to $12300m^3$, 12 elevated tanks with varying capacities from $25m^3$ to $1300m^3$ were also constructed. Power plants, water treatment facilities, administrative offices, maintenance workshops, water testing laboratory were also included in this phase.

To develop the coastal areas of Jaalan Bani Bu Ali, the construction of the second phase started on February 2004 and completed on January 2006. It provides potable water from the same wells of Jaalan Wellfield to the towns of Al Sowaih, Al Bander Al Jadeed, Al Haddah, Al Rowais, Al Khabbah, Al Daffah and Wadi Sal along the coast (see Figure 4.5).

4.7 Sur Central Desalination Plant

The original reverse osmosis (RO) desalination plant of 4,500m³/day capacity was constructed in 1993 to supply Sur town in south Ash Sharqiyah with potable water. The plant was extended a couple of times and as of mid 2008, the plant started to produce 12,000m³/day of desalinated water from 13 beach wells. The strategy of the Government of the Sultanate of Oman is now to use sustainable, reliable sources for domestic water supply and this can only be achieved through desalinated seawater. To realize this strategy, in 2007 the government decided to construct a new reverse osmosis plant at Sur Wilayat, with an initial capacity of 80,000m³/day by 2009 and to develop the project through private sector participation as a Build Own Operate (BOO) scheme (Mot MacDonald, 2007). The plant has been in operation since early 2010.

Before the Sur desalination project has been implemented, however, the government decided to connect the existing groundwater Ash Sharqiyah Sands water supply scheme to supply the Wilayats of Bidiyah, Al Qabil and Ibra of northern Ash Sharqiyah. In spite of the progress in the construction of the desalination scheme, no clear management strategy has yet been established.

As explained in the introduction Chapter 1, this study will be used as a management tool for using groundwater and or desalinated water to supply the eight main Wilayats in Ash Sharqiyah Region of Al Kamil and Al Wafi, Jaalan Bani Bu Hassan, Jaalan Bani Bu Ali, Bidiyah, Al Qabil, Ibra and Al Mudaybi, which are suffering from severs water shortage. Sur will remain dependent on desalinated water, but the project will have the facility for potable water to be pumped to supply Sur as emergency water supply from Ash Sharqiyah Sands groundwater wellfields unless the management model of this study recommended otherwise. Figure 4.6 illustrates the layout of the two schemes to use conjunctive domestic water supply of both groundwater and desalinated water to meet Ash Sharqiyah water demands up to 2030.

4.8 Water demand assessment

As mentioned earlier, the main objective of connecting the Sur desalinated water scheme with the Ash Sharqiyah Sands groundwater scheme is to provide a comprehensive water supply scheme for the eight Wilayats of Ash Sharqiyah Region. This requires a good knowledge of the water demands requirements to be effectively done.

Based on the Ministry of National Economy (MONE) population growth forecasts and design criteria done by Parsons International & Co LLC (the designer consultant), the water demand assessment for these eight Wilayats was projected to 2030 as explained briefly in the next sections ((MONE (2003) & (Parsons International & Co LLC (2005)).

4.8.1 Projected population

The released results of the 2003 Census shows an annual growth rate of 1.27% to 2.52% in the project areas between 1993 and 2003, with an average of 1.94%. A uniform growth rate was applied for the entire project area. Based on the above published MONE growth rate, the projected population of all eight Wilayats of the study area in Ash Sharqiyah region will be 337,193; 384,975; 432,766; 484,451 and 542,520 inhabitants in 2010, 2015, 2020, 2025 and 2030 respectively as given in Table 4.6.

4.8.2 Supplied criteria and projected water demand

All settlements with a population greater than 1,000 inhabitants according to the 1993 Census, and adjacent villages are supplied. In addition, some settlements along the route of the transmission pipelines are also included. The calculations of the projected water demands per capita (in litre/day) for distribution networks were based on criteria presented in Table 4.7. In the case of supply by tanker, the average daily demand has been determined as 97 litre/capita/day (Parsons International & Co LLC (2005). This per capita demand is more realistic than what was estimated in 1995 by the National Well Inventory, i.e. 80 litre/day.

The above per capita demands were used to calculate the projected water demand to 2030 for the eight Wilayats of the study area in Ash Sharqiyah region. Up to 2010, the two water schemes will provide water supply to at least 75% of the population in the eight Wilayats until the Sur Desalination Plant starts functioning. It is assumed that this will be expanded to 90% coverage in 2015 as more distribution networks commence, with a future increase to 95% by 2025 as all of the eight Wilayats will be covered more distribution networks . It is assumed that 2% of the covered population will receive water by tanker in the future for the small scattered communities. The gradually staged increase in water supply coverage after 2010 is due to supply limitation. On this basis, the total projected water demand in the eight Wilayats has been determined to be $20.5x10^6m^3$, $26.8x10^6m^3$, $31.8x10^6m^3$ and $35.6x10^6m^3$ in 2015, 2020, 2025 and 2030 respectively as given in Table 4.8.

4.9 Summary

This chapter described the geography and the geological setting of the study area. Alluvial and its overlaying aeolianite deposits are the two distinctive deposits that dominate in the study area. Drilling and Geophysical data were presented to delineate the extent and geometry of the two aquifer layers, their thicknesses and top/bottom of each layer. Hydrological data such as pumping test data were discussed. The general direction of the flow in study area was found to be from northwest to southeast direction with horizontal hydraulic gradients across most of the area ranging from 0.0054 to 0.00063. The steepest gradients occur in the north-west. The efficiency of wells completed into the aeolianite was better (averaged 92%) compared to those wells completed in the alluvium (averaged 56%). A review of the hydro-geological and water use studies carried out within the study area was also presented.

Two groundwater wellfields of 29 wells have been operational since 2004 as part of the Ash Sharqiyah Sands Groundwater Supply Project to supply the southern Wilayats of Ash Sharqiyah Region. However, to implement the government optimum water management strategy, the existing Ash Sharqiyah Sands water supply scheme was connected with the Sur Desalination Plant by the end of 2009 to facilitate the conjunctive use of both groundwater and desalinated water for supplying the eight Wilayats of the Ash Sharqiyah Region. The total projected water demands in the eight Wilayats have been determined to be $20.5x10^6m^3$, $26.8x10^6m^3$, $31.8x10^6m^3$ and $35.6x10^6m^3$ in 2015, 2020, 2025 and 2030 respectively.

Table 4.1: Aeolianite aquifer (layer 1) assessment drilling boreholes data in 1997 (MWR, 1997b)

Well No.	Easting (m)	Northing (m)	Top (mamsl)	Bottom (mamsl)	Water table (mamsl)	Thickness (m)	Aquifer thickness (m)
WAB132	692526	2483980	293.6	284.6	-	9.0	0
WAB190A	695331	2482695	286.8	285.7	-	1.1	0
WAB214	679269	2481352	330.5	329.5	-	1.0	0
WAB208A	688796	2476743	278.4	264.4	-	14.0	0
WAB003	683096	2468802	346.5	222.3	238.2	124.2	15.9
WAB247	691764	2470968	291.2	218.2	224.1	73.0	5.9
WAB229	695843	2470798	275.3	222.3	-	53.0	0
WAB215	698126	2471640	285.1	230.1	231.6	55.0	1.5
WAB195	708682	2472336	217.5	209.6	-	7.9	0
WAB216	703079	2468995	248.8	192.8	-	56.0	0
WAB248	692493	2465992	317.2	201.2	209.1	116.0	7.9
WAB196	711888	2466072	205.4	174.1	-	31.3	0
WAB217	704705	2464277	262.4	166.4	181.7	96.0	15.3
WAB198	710392	2461420	222.0	146.0	169.5	76.0	23.5
WAB224	701935	2456835	280.3	133.3	181.6	147.0	48.3
WAB174	717450	2457280	171.6	134.6	155.8	37.0	21.2
WAB241	714746	2454633	228.3	107.3	158.9	121.0	51.6
EW1	723630	2456220	158.8	138.3	145.0	20.5	6.7
W-A	710140	2473030	222.0	214.0	-	7.9	0
EW3	716730	2464120	189.6	177.6	-	12.0	0
WAB218A	710499	2449984	235.4	92.4	164.6	143.0	72.2
EW2	728330	2450040	139.0	127.0	-	12.0	0
W3	691530	2444966	258.2	150.2	200.7	108.0	50.5
WAB226A	705075	2446553	259.6	78.6	174.2	181.0	95.6
WAB231	717268	2448193	209.6	71.6	149.2	138.0	77.6
WAB114A	732417	2448980	125.1	115.1	117.2	10.0	2.1
WAB221	711115	2444772	241.5	60.5	163.0	181.0	102.5
WAB200	721471	2442003	191.8	58.8	137.1	133.0	78.3
W5A	728340	2441430	133.2	63.2	117.6	70.0	54.4
WAB222	709934	2436812	232.0	70.0	163.5	162.0	93.5
WAB232	718838	2437855	218.5	55.5	140.9	163.0	85.4
WAB116A	731520	2438464	119.2	59.2	108.3	60.0	49.1
WAB202	723070	2435142	201.0	51.0	131.8	150.0	80.8
W8A	727428	2436779	148.5	36.5	117.1	112.0	80.6
WAB204	727911	2433072	151.0	41.0	114.4	110.0	73.4
EW5	735200	2434620	102.8	48.8	99.6	54.0	50.8
WAB228A	705901	2430012	222.9	77.9	168.3	145.0	90.4
WAB235	714737	2430272	206.6	56.6	152.5	150.0	95.9
WAB236	725342	2429674	183.3	43.3	123.8	140.0	80.5
EW4	734256	2430763	104.5	49.5	99.9	55.0	50.4
WAB238A	725517	2424395	176.1	36.1	125.0	140.0	88.9
W9	730783	2420546	134.4	50.0	110.7	84.4	60.7
WAB244	725264	2417508	170.2	36.6	123.1	133.6	86.5
WAB203	723060	2435139	201.0	51.0	130.0	150.0	79.0
WAB227	705076	2446546	259.5	78.5	174.2	181.0	95.8
W-7	733560	2400274	81.9	15.9	56.2	66.0	40.3
WAB220	710490	2450002	235.6	92.6	164.6	143.0	72.0
WAB230	715012	2454469	221.7	109.7	158.4	112.0	48.7
WAB240	715035	2454719	227.7	103.7	158.6	124.0	54.9
WAB242	714894	2454419	225.4	107.4	158.5	118.0	51.1
W-5B	728340	2441430	132.6	64.6	117.9	68.0	53.3
W-8B	727431	2436836	150.7	50.7	118.1	100.0	67.5

Table 4.2: Alluvium aquifer (layer 2) assessment drilling boreholes data in 1997 (MWR, 1997b)

Well No.	Easting (m)	Northing (m)	Top (mamsl)	Bottom (mamsl)	Water table (mamsl)	Aquifer thickness (m)
WAB132	692526	2483980	284.6	134.6	265.3	130.7
WAB190A	695331	2482695	285.7	135.7	252.2	116.5
WAB214	679269	2481352	329.5	179.5	294.7	115.2
WAB208A	688796	2476743	264.4	114.4	244.3	129.9
WAB003	683096	2468802	222.3	72.3	238.2	150.0
WAB247	691764	2470968	218.2	68.2	224.1	150.0
WAB229	695843	2470798	222.3	72.3	213.7	141.4
WAB215	698126	2471640	230.1	80.1	231.6	150.0
WAB195	708682	2472336	209.6	59.6	174.2	114.6
WAB216	703079	2468995	192.8	42.8	192.3	149.5
WAB248	692493	2465992	201.2	51.2	209.1	150.0
WAB196	711888	2466072	174.1	24.1	167.7	143.6
WAB217	704705	2464277	166.4	16.4	181.7	150.0
WAB198	710392	2461420	146.1	-3.9	169.5	150.0
WAB224	701935	2456835	133.3	-16.7	181.6	150.0
WAB174	717450	2457280	134.6	-15.4	155.8	150.0
WAB241	714746	2454633	107.3	-42.7	158.9	150.0
EW1	723630	2456220	138.3	-11.7	145.0	150.0
W-A	710140	2473030	214.0	64.0	168.4	104.4
EW3	716730	2464120	177.6	27.6	161.9	134.3
WAB218A	710499	2449984	92.4	-57.6	164.6	150.0
EW2	728330	2450040	127.0	-23.0	126.7	149.7
W3	691530	2444966	150.2	0.2	200.7	150.0
WAB226A	705075	2446553	78.6	-71.4	174.2	150.0
WAB231	717268	2448193	71.6	-78.4	149.2	150.0
WAB114A	732417	2448980	115.1	-34.9	117.2	150.0
WAB221	711115	2444772	60.5	-89.5	163.0	150.0
WAB200	721471	2442003	58.8	-91.2	137.1	150.0
W5A	728340	2441430	63.2	-86.8	117.6	150.0
WAB222	709934	2436812	70.0	-80.0	163.5	150.0
WAB232	718838	2437855	55.5	-94.5	140.9	150.0
WAB116A	731520	2438464	59.2	-90.8	108.3	150.0
WAB202	723070	2435142	51.0	-99.0	131.8	150.0
W8A	727428	2436779	36.5	-113.5	117.1	150.0
WAB204	727911	2433072	41.0	-109.0	114.4	150.0
EW5	735200	2434620	48.8	-101.2	99.6	150.0
WAB228A	705901	2430012	77.9	-72.1	168.3	150.0
WAB235	714737	2430272	56.6	-93.4	152.5	150.0
WAB236	725342	2429674	43.3	-106.7	123.8	150.0
EW4	734256	2430763	49.5	-100.5	99.9	150.0
WAB238A	725517	2424395	36.1	-113.9	125.0	150.0
W9	730783	2420546	50.0	-100.0	110.7	150.0
WAB244	725264	2417508	36.6	-113.4	123.1	150.0
WAB106	724953	2476591	231.5	81.5	166.5	85.0
WAB110	714916	2473442	229.9	79.9	165.9	85.9
WAB111B	717475	2457264	168.8	18.8	155.9	137.1
WAB167	714933	2473424	230.2	80.2	165.6	85.4
WAB170	724940	2476556	232.3	82.3	165.5	83.2
WAB179B	718060	2468323	197.8	47.8	164.0	116.2
WAB183	722826	2469988	203.1	53.1	164.6	111.4
WAB199	710405	2461419	222.2	72.2	169.5	97.3
WAB220	710490	2450002	92.6	-57.4	164.6	150.0

Table 4.2: Continue

Well No.	Easting (m)	Northing (m)	Top (mamsl)	Bottom (mamsl)	Water table (mamsl)	Aquifer thickness (m)
WAB230	715012	2454469	109.7	-40.3	158.4	150.0
WAB233B	715029	2454459	221.0	71.0	158.4	87.3
WAB240	715035	2454719	103.7	-46.3	158.6	150.0
WAB242	714894	2454419	107.4	-42.6	156.5	150.0
WAB246B	727905	2433054	150.9	0.9	114.4	113.5
KWEH1	727980	2456370	152.1	2.1	140.0	137.9
KWEH4	724339	2461486	170.3	20.3	156.4	136.0
KWTW1	727980	2456370	152.1	2.1	134.2	132.1
KWTW2	725160	2461957	171.0	21.0	158.5	137.5
KWTW3	728600	2456900	151.4	1.4	141.5	140.1
KWTW4	724339	2461486	170.4	20.4	157.3	137.0
NE-02	692060	2478940	269.2	119.2	242.6	123.3
TPW1	724217	2462300	173.5	23.5	156.4	132.9
TPW2	724102	2462679	174.8	24.8	156.9	132.1
TPW3	724570	2461020	167.7	17.7	148.9	131.2
TPW4	725083	2460917	166.2	16.2	154.2	138.0
TPW5	725261	2460620	164.8	14.8	154.8	140.0
TPW6	724147	2462553	174.0	24.0	156.6	132.7
W-2	709803	2455342	231.6	81.6	168.7	87.1
W-5B	728340	2441430	64.6	-85.4	117.9	150.0
W-8B	727431	2436836	50.7	-99.3	118.1	150.0
WAB240	715035	2454719	103.7	-46.3	158.6	150.0
WAB242	714894	2454419	107.4	-42.6	156.5	150.0
WAB246B	727905	2433054	150.9	0.9	114.4	113.5
KWEH1	727980	2456370	152.1	2.1	140.0	137.9
KWEH4	724339	2461486	170.3	20.3	156.4	136.0
KWTW1	727980	2456370	152.1	2.1	134.2	132.1
KWTW2	725160	2461957	171.0	21.0	158.5	137.5
KWTW3	728600	2456900	151.4	1.4	141.5	140.1
KWTW4	724339	2461486	170.4	20.4	157.3	137.0
NE-02	692060	2478940	269.2	119.2	242.6	123.3
TPW1	724217	2462300	173.5	23.5	156.4	132.9
TPW2	724102	2462679	174.8	24.8	156.9	132.1
TPW3	724570	2461020	167.7	17.7	148.9	131.2
TPW4	725083	2460917	166.2	16.2	154.2	138.0
TPW5	725261	2460620	164.8	14.8	154.8	140.0
TPW6	724147	2462553	174.0	24.0	156.6	132.7
W-2	709803	2455342	231.6	81.6	168.7	87.1
W-5B	728340	2441430	64.6	-85.4	117.9	150.0
W-8B	727431	2436836	50.7	-99.3	118.1	150.0

Table 4.3: Summary of Aeolianite aquifer (layer 1) parameters (MWR, 1997e)

Exploration Well ID	EC (µS/cm)	Yield (l/s)	T (m^2/day)	specific yield (Sy)
WAB201	1370	8	575	N/A
WAB203	1620	9	791	N/A
WAB205	1019	14.6	1440	N/A
WAB218B	N/A	N/A	230	0.14
WAB219	960	4.5	15	N/A
WAB223	1273	11	414	N/A
WAB225	1138	6	1898	N/A
WAB226B	N/A	4.5	390	0.134
WAB227	6550	9.8	615	N/A
WAB233C	2500	N/A	398	0.15
WAB234	1292	11.6	718	N/A
WAB235	1793	11.5	337	N/A
WAB237	1118	11.2	1610	N/A
WAB238B	1520	12.8	339	0.311
WAB239	2000	10	883	N/A
WAB244	1134	10.5	443	N/A
WAB246C	2870	0.7	277	0.072
BBBA1	1820	23	256	N/A
BBBA2	2050	23	700	N/A
BBBA3	2450	22.9	250	N/A
BBBA4	2370	22.3	353	N/A
W-7	1220	20.3	1190	N/A
W-9	1850	15.7	843	N/A
Mean	**1758**	**12.5**	**651**	**0.161**

N/A: not available

Table 4.4: Summary of Alluvium aquifer (layer 2) parameters (MWR, 1997e)

Exploration Well ID	EC (µS/cm)	Yield (l/s)	K (m/day)	specific yield (Sy)
WAB106	881	3.6	3.9	N/A
WAB110	840	9.4	46.2	N/A
WAB111B	N/A	N/A	23	0.1
WAB116B	2750	N/A	7.3	0.31
WAB166	706	19.7	36.7	N/A
WAB167	815	21	45.3	0.06
WAB168	801	7.9	62.3	0.06
WAB169A	939	5.9	3.6	N/A
WAB170	855	5.5	4.6	N/A
WAB171	862	3.7	4.4	N/A
WAB172	1122	N/A	3.6	N/A
WAB173	825	3.1	4.1	N/A
WAB174	1069	84	21.3	0.1
WAB175A	1080	34.9	20.7	0.005
WAB179B	886	N/A	48.5	N/A
WAB180	843	49	52.7	N/A
WAB181	847	29.6	52.3	N/A
WAB182	629	15.4	66.7	N/A
WAB183	639	45	68.7	0.02
WAB184A	631	8.5	70	0.035
WAB184B	542	N/A	70	0.035
WAB198	1200	26	43.2	N/A
WAB199	1215	49.9	14.8	N/A
WAB200	3310	13	7.7	N/A
WAB202	2750	5.2	0.2	N/A
WAB204	1760	24.7	2.6	N/A
WAB220	2000	55	0.4	N/A
WAB221	3870	16	N/A	N/A
WAB224	2710	13.7	4.0	N/A
WAB229	988	22.6	83.3	N/A
WAB230	1750	24.1	33.6	N/A
WAB231	2200	14	4.9	N/A
WAB233B	1829	3.35	0.2	N/A
WAB236	2330	4.9	0.1	N/A
WAB240	1570	60.9	49.5	N/A
WAB241	1720	73	27.9	N/A
WAB242	1800	71.7	62.1	N/A
WAB246B	1817	1	0.5	N/A
WAB263	868	22.6	34.4	N/A
BAT034	N/A	9.7	N/A	N/A
EW-1	748	24.2	1.8	N/A
EW-2	1269	24.2	N/A	N/A
EW-3	752	26	15.2	N/A
EW-4	2350	25	N/A	N/A

N/A: not available

Table 4.4: Continue

Exploration Well ID	EC (µS/cm)	Yield (l/s)	K (m/day)	specific yield (Sy)
JE-1	2000	26	19.0	N/A
JE-2	1726	38	27.3	N/A
KWEH1	2150	N/A	3.5	0.052
KWEH4	675	N/A	8.7	0.002
KWTW1	2450	22.3	3.5	0.052
KWTW2	705	26.6	7.2	N/A
KWTW3	745	26.9	6.8	N/A
KWTW4	800	26.3	8.7	0.002
NE-02	640	4	0.5	N/A
NE-03	483	10	1.0	N/A
TPW1	N/A	25	20.1	N/A
TPW2	N/A	23.4	16.3	N/A
TPW3	N/A	8.4	3.5	N/A
TPW4	N/A	11.2	4.9	N/A
TPW5	N/A	13.3	6.7	N/A
TPW6	N/A	9.5	6.7	N/A
W-2	1920	14.3	N/A	N/A
W-5B	1940	22.2	12	0.08
W-5CP1B	1810	N/A	10	0.23
W-5CP2A	2070	N/A	6.7	0.17
W-8B	2260	19	7.3	N/A
Mean	**1434**	**23.1**	**21.4**	**0.082**

N/A: not available

Table 4.5: Average annual flows and electrical conductivity of Flaj (MWR, 1997b)

Area	Site ID	Local Name	Easting (m)	Northing (m)	Avg. Flow 1982 - 1997 (l/s)	Avg. Flow 1998 - 2007 (l/s)	Avg. EC (µS/cm)
AL GHABI	FK884379AB	Al Gahis	684731	2483903	18	15	742
	FV789854AB	Al Wasil	679412	2489330	53	20	1053
	FV873997AB	Arraka+	683914	2480566	21	18	1434
	FV876918AB	Matawa+	685897	2480252	18	15	1336
	FV882149AB	Shahik	683137	2482491	47	35	1803
	FV884191AB	Mintirib	685104	2481711	73	40	1006
	FV886254AB	Al Ghabi	686683	2483169	69	50	1127
	FV970666AB	Hawiya	690879	2477400	57	38	566
BANI KHALID	GV189726AB	Sabt	719280	2487960	47	37	930
MASHAIKH	GV242938AB	Mashaikh +	722640	2450360	14	20	1392
	GV243865AB	Faghri	724010	2448970	27	20	931
	GV244841AB	Hilal	724660	2448860	11	11	781
AL WAFI	GV256890AB	Al Kamil	726770	2458560	48	20	635
	GV259362AB	Al Wafi	729430	2454090	87	55	602
BANI BU HASAN	GK343401AB	Awlad	733019	2444156	9	15	2649
	GK344186AB	Mashraf	734850	2441690	15	15	4521
	GK344388AB	Igeriah	734818	2443882	38	35	2019
	GK344482AB	Souquia	734812	2444200	50	26	1566
	GK344537AB	Mahyul	734300	2445700	53	40	1011
	GK345279AB	Sharqui+	735790	2442990	25	5	2148
	GV343592AB	Buwered	733980	2445410	35	15	1219
	GV344527AB	Minjred	734620	2445900	72	52	733
BANI BU ALI	GK338487AB	Rahian	738828	2434743	12	14	16284
	GK338529AB	Gaderan	738239	2435925	45	70	9264
	GK338799AB	Hamad	738970	2437900	14	16	4075
	GK338799BB	Jadid	738938	2437907	8	11	4275
	GK338860AB	Zwaeid	738640	2438010	11	13	5823
	GK339729AB	Balhiss	739238	2437905	25	27	3614
	GV034532AB	Flayaj	740250	2435690	12	12	6974
	GV338561AB	Asah	739000	2436300	23	23	3339
	GV339683AB	Adhahir	739490	2435950	19	7	4226

Notes: + Flaj support

Table 4.6: Projected population of the targeted Wilayats based on MONE growth rate

Wilayat	Actual Population		Projected Population				
	1993	2003	2010	2015	2020	2025	2030
Sur	53381	66587	80614	92038	103463	115820	129703
Kamil/ Wafi	16712	20166	24414	27874	31334	35076	39281
JBB Hassan	21878	25717	31135	35546	39959	44731	50093
JBB Ali	39715	50916	61642	70377	79114	88562	99178
Bidiyah	15136	17784	21530	24581	27633	30933	34641
Al Qabil	11957	13564	16421	18748	21076	23593	26421
Ibra	19964	24619	29805	34029	38253	42822	47954
Mudaybi	51192	59167	71631	81782	91934	102914	115249
Total	**229935**	**278520**	**337193**	**384975**	**432766**	**484451**	**542520**

Table 4.7: Projected water demands per capita (in litre/day) for distribution networks (after Parsons Intern. & Co LLC, 2005)

Year	2003	2010	2020	2030
Domestic	110	117	127	130
Non-domestic (20% of the domestic**);** includes commercial, industrial and institutional demand	22	23	25	26
Sub-total	132	140	152	156
Non--revenue water (25% of sub-total); includes water losses through leakage, water use for fire fighting, illegal connections, etc	33	35	38	39
Average daily demand	165	175	190	195

Table 4.8: Projected water demand of the targeted Wilayats (m³/day)
(after Parsons Intern. & Co LLC, 2005)

Wilayat	Year	2015	2020	2025	2030
Sur	Distribution	14,095	17,384	21,027	23,547
	Tankers	218	180	213	239
	Sub-total	14,313	17,564	21,240	23,785
Al Kamil/Al Wafi	Distribution	4,499	5,265	6,033	6,756
	Tankers	49	55	61	68
	Sub-total	4,548	5,320	6,094	6,824
Jaalan Bani Bu Hassan	Distribution	5,737	6,714	7,693	8,615
	Tankers	62	70	78	87
	Sub-total	5,799	6,784	7,771	8,702
Jaalan Bani Bu Ali	Distribution	11,359	13,293	15,232	17,058
	Tankers	123	138	154	173
	Sub-total	11,482	13,431	15,386	17,231
Bidiyah	Distribution	3,447	4,643	5,616	6,289
	Tankers	57	48	57	64
	Sub-total	3,504	4,691	5,673	6,353
Al Qabil	Distribution	1,635	3,541	4,283	4,797
	Tankers	14	37	43	49
	Sub-total	1,648	3,578	4,327	4,845
Ibra	Distribution	4,964	6,427	7,774	8,706
	Tankers	64	67	79	88
	Sub-total	5,028	6,494	7,853	8,794
Al Mudaybi	Distribution	9,666	15,447	18,684	20,923
	Tankers	230	160	189	212
	Sub-total	9,897	15,607	18,873	21,135
TOTAL	**(m³/day)**	**56,218**	**73,469**	**87,215**	**97,669**

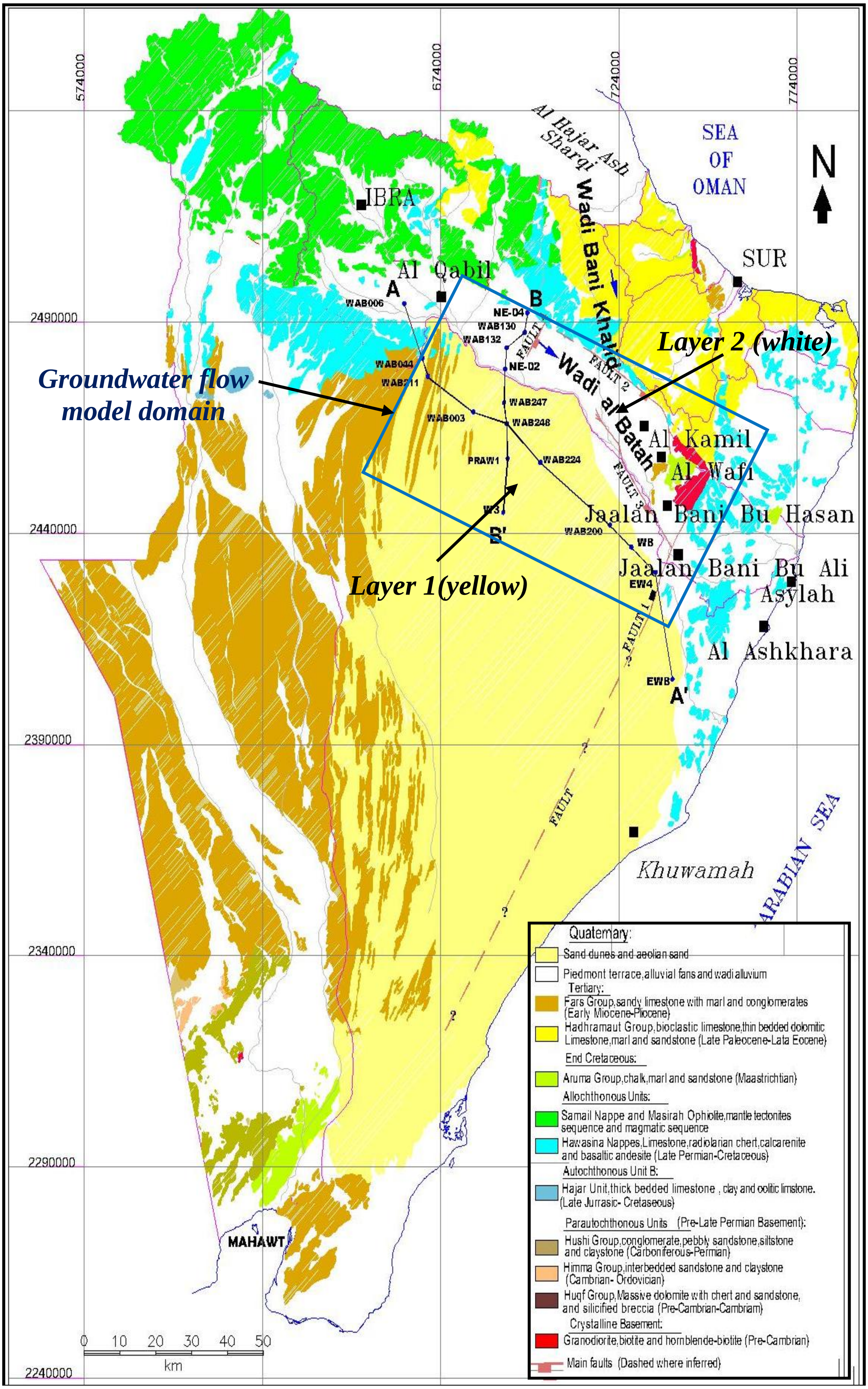


Figure 4.1: Simplified regional geology of the study area showing the model domain and the extent of the two layers (MWR, 1997a)

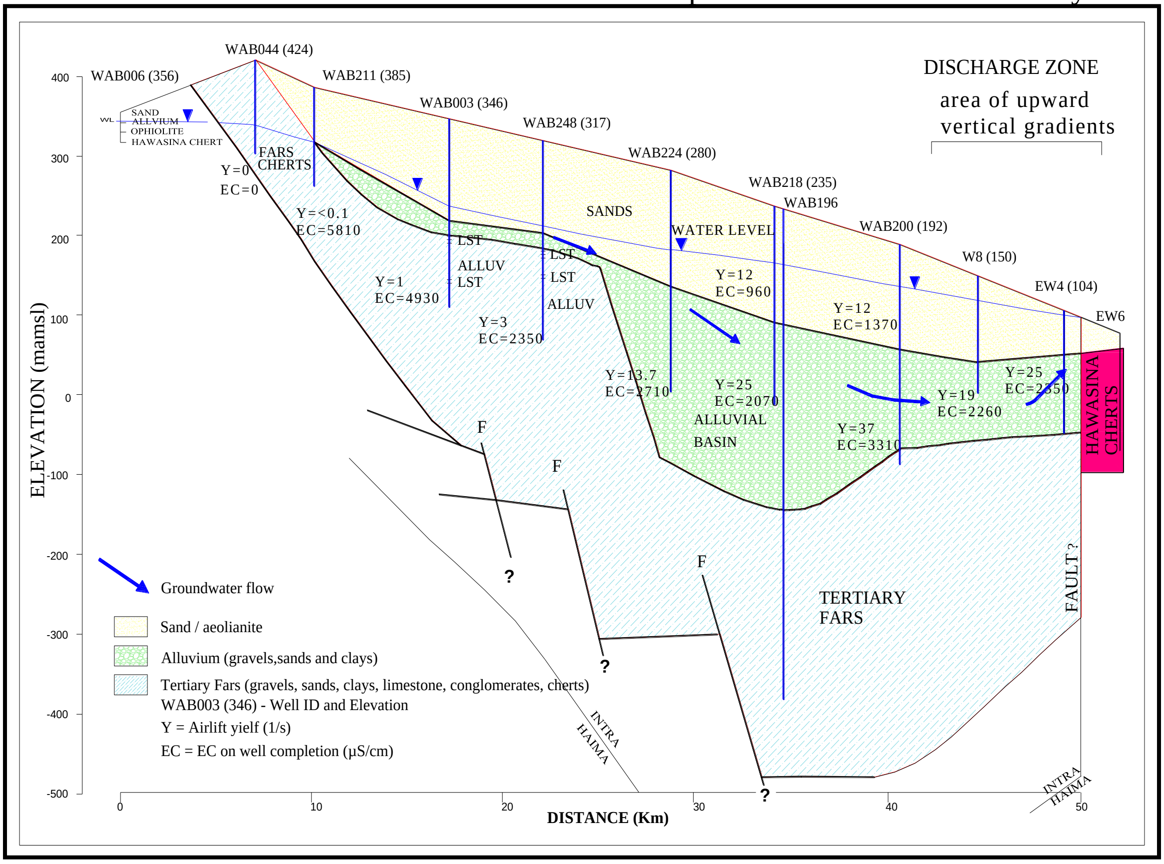


Figure 4.2: Northwest – Southeast cross-section (A-A') in Figure 4.1 in the study area (MWR, 1997)

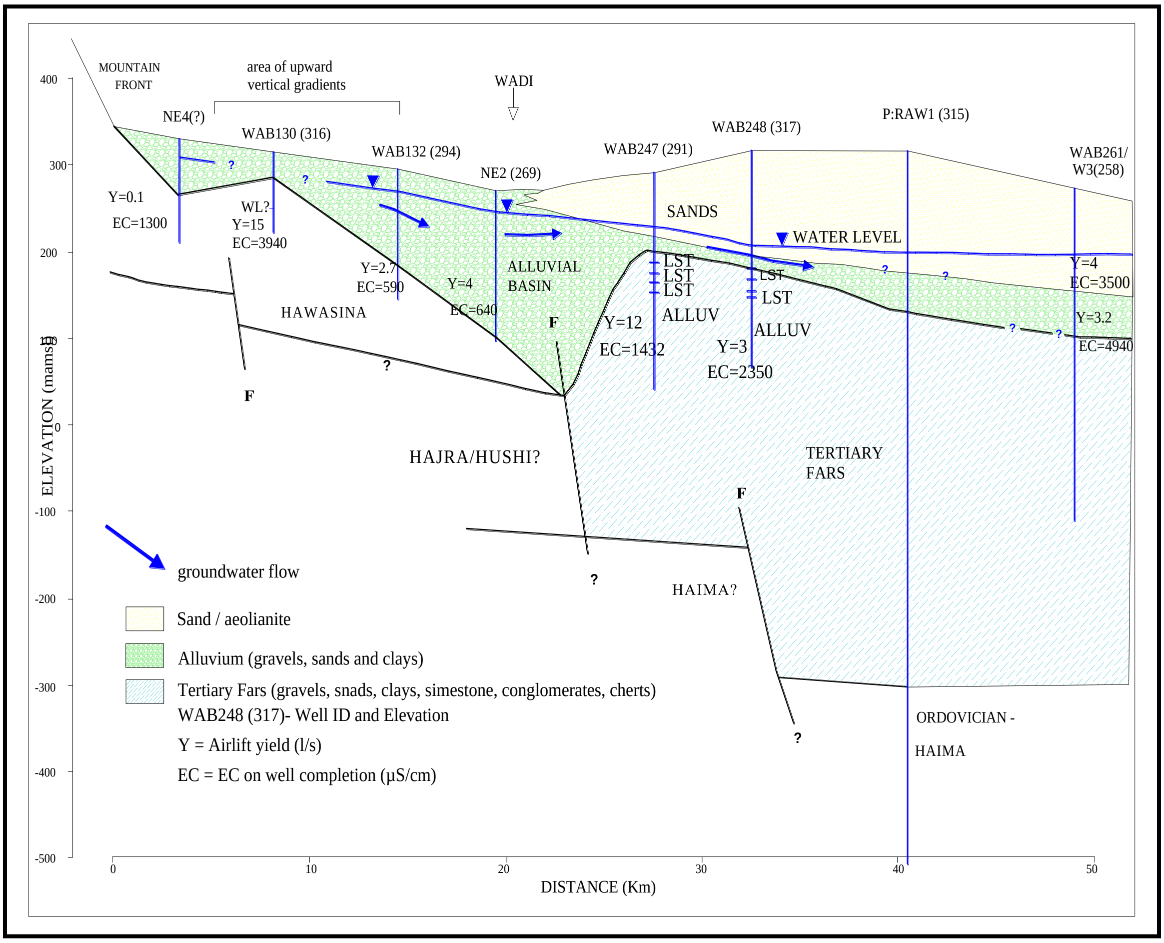


Figure 4.3: North – South cross-section (B-B') in Figure 4.1 in the study area (MWR, 1997**)**

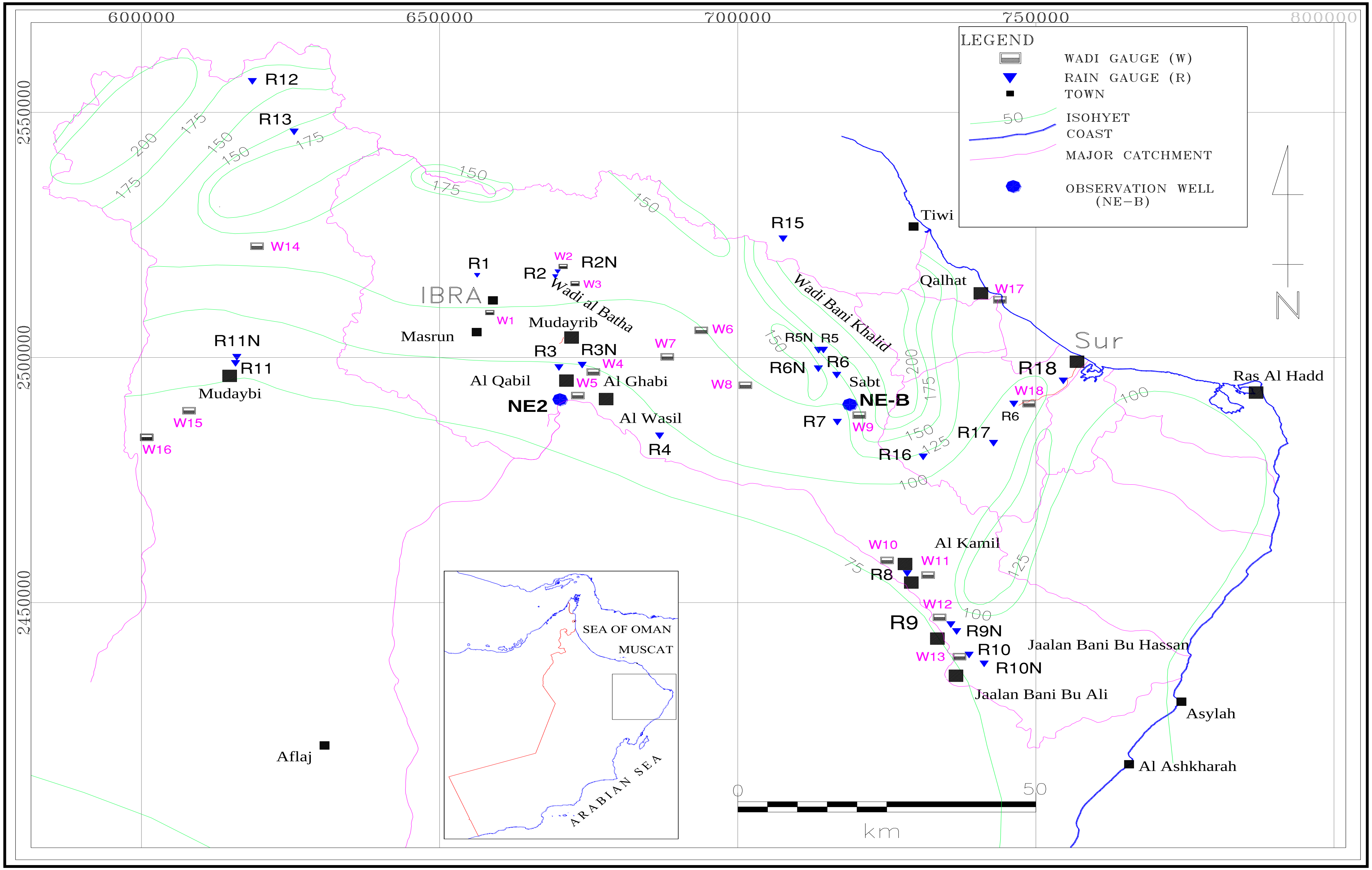


Figure 4.4: Isohyets of average annual rainfall (1975-1997) and key gauging stations in Wadi al Batha Catchment (MWR, 1997g)

Figure 4.5: Map showing the main components of the Ash Sharqiyah Sands Groundwater Scheme (Dr. Ahmed Abdel Warith & Partners LLC, 1999)

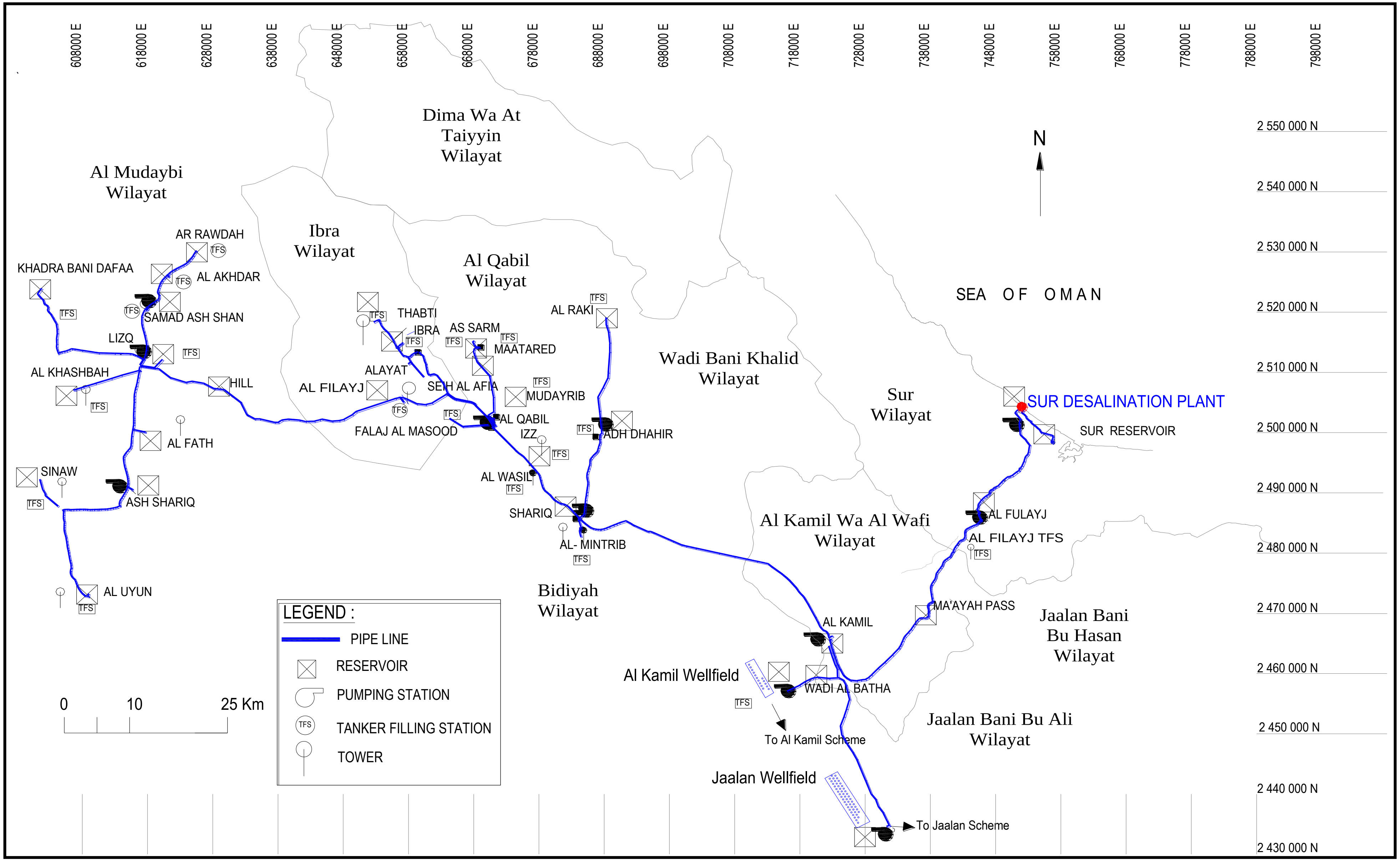


Figure 4.6: Schematic layout to use conjunctive Ash Sharqiyah Region domestic water supply of both groundwater and desalinated water (after Parsons Intern. & Co LLC, 2000), coordinates in metres.

CHAPTER 5

SIMULATION MODEL OF ASH SHARQIYAH SANDS AQUIFER

5.1 Introduction

In Chapter 2 examples of the use of groundwater simulation models to investigate the effects of different groundwater management scenarios in various parts of the world were presented. The simulation approach is widely used to find an acceptable scenario close to real aquifer's behaviour. However, it should be kept in mind that groundwater management solutions are as good as the skill of the model. Therefore, the model should be calibrated, verified and validated to ensure that it is valid for the present system before it can be used to predict the aquifer's behaviour in the future. As explained in Chapter 2, the models are constructed of mathematical equations which describe the physical laws that govern groundwater flow in saturated porous media. A model must include the hydro-geologic area of interest, the boundary and its conditions and the parameters of the aquifers. The construction of a MODFLOW (McDonald and Harbaugh, 1988) model for the Ash Sharqiyah Sands aquifer system and the simulation results form the subject of this chapter.

5.2 Modelling approach

Groundwater modelling consists of data collection, development of a conceptual model and development of a mathematical model. These three phases are interlinked. The data collection exercise carried out for this study is as presented in Chapter 4.

5.2.1 Conceptual model for Ash Sharqiyah Sands Aquifer

Construction of conceptual models is essential to understand the way in which systems are put together and work. A conceptual model can be defined as a synthesis of how a real system behaves, based on qualitative and quantitative analysis of data (SEPA, 2006). A groundwater conceptual model defines the extent of the study area, the hydro-geological conditions and flows at the boundaries of the area, identifies all the water-dependent features in the area, and the limitations of the current conceptual understanding and the major sources of uncertainty (SEPA, 2006). In general, the accuracy of the numerical simulation model depends on how well the conceptual model represents the real aquifer behaviour. Therefore, it is good practice that the simplified conceptual model for the groundwater system should be first constructed. If the simulation model is verified and becomes valid for the present system, then it can be used to predict the system's behaviour in the future assuming that the system is stationary in terms of parameter values and model structure.

Based on the geologic and hydro-geologic information reviewed previously in Chapter 4, the aquifer system is considered as unconfined and semi-confined layers for numerical modelling purposes. These are referred to as layer 1 (or the aeolianite) and layer 2 (or the alluvium) respectively and their extents are shown in Figure 1.3. The conceptual model is shown in Figure 5.1 and the extent of the model is shown in Figure 4.1.

Drilling and Geophysical data, such as Time Domain Electromagnetic (TDEM) and seismic surveys were used to delineate the extent and geometry of the aquifers, their thicknesses and top/bottom of each layer as discussed in Chapter 4 and presented in Tables 4.1 and 4.2 (MWR, 1997c). Borehole logs were also collected from 71 wells in the study area during the exploration drilling in 1997 to help to define the two layers and to identify these aquifers: aeolianite and alluvium (MWR, 1997d). After identifying the layers, they were then prepared in a compatible format to be used in the Groundwater Modelling System (GMS) software model. Borehole elevation and geophysical data of the two layers have been kriged, contoured and assigned cell value using the GMS Software.

Figures 5.2 and 5.3 show the contour maps of the two layers' elevations. Figure 5.2 (a) presents the top of layer 1 which represents the ground surface; as can be seen, the ground surface elevation slopes from the north-west to south-east. It ranges from 440 to 50masl. Figure 5.2 (b) represents the bottom elevation of layer 1 and ranges from 250 to 40masl. It also slopes from the north-west to south-east. This direction agrees with general direction of the groundwater slope.

Figure 5.3 (a) presents the top of layer 2 which represents the ground surface when it is not overlaid by layer 1, otherwise, it is the base of layer1 as illustrated in Figure 4.3. It ranges from 320 to 40masl. It slopes from north to south-east. Figure 5.3 (b) represents the bottom elevation of layer 2 and ranges from 170 to -110masl and it has the same slope as the top elevation.

In the model only the top elevation is needed for layer 1; then the GMS programme considers the bottom elevation of layer 1 as the top elevation of layer 2 except when layer two is exposed at the ground surface. In this case, the elevations for the scatter points have been adjusted so that the bottom of layer 1 extends above the layer 2 on the right side of the model.

5.2.2 Model domain and discretization

The modelled domain is shown in Figure 5.4 together with the locations of the two wellfields as well as the locations of the monitored wells used to calibrate the steady state model. As noted earlier, there are eight wells producing from Al Kamil Wellfield, four wells each producing from layer 1 & 2, and 21 wells producing from Jaalan Wellfield all of them producing from layer1 (see Tables 5.1 and 5.2). A comparison of Figure 4.1 and the study area in the same figure will reveal that the model domain used is smaller than the study area because of insufficient data outside the model domain. The model domain has also been rotated to coincide with the predominant south-eastern direction of regional groundwater flow in order to reduce numerical dispersion. The domain covers an area of 4675km^2 (85km by 55km). For the finite difference schematization, the modelled area was discretized into a square grid of 500 m spacing, comprising 170 rows and 110 columns, which were refined by two to become (182 columns and 118 rows) with a square grid of 250 m spacing at the stress area (wellfields) as presented in Figure 5.5. The first cell starts at 667880m E 2452555m N. Three factors were considered in determining the grid size: production well spacing (500m), computational efficiency and proper representation of available data.

As stated above, the modelled area was discretized into square grids of 500 m spacing, which were refined to a square grid of 250 m spacing at the stress areas (wellfields). The numerical finite difference solution adopted assumes that the hydraulic head is uniform within a given grid square. Whilst this is not a major problem in grid cells where there are no external stresses (i.e. well abstractions), it may not accurately describe the rapid drawdown caused by turbulence and well losses in the proximity of the pumped wells. To better model such effects, a much finer mesh, typically with spacing of the order of the diameter of the pumped well, would be required. However, this will cause the computation time to increase astronomically and may run the risk of causing instability of the numerical solution scheme. It is precisely to avoid such problems that a relatively coarse time interval of four months was adopted for the discretisation in the time domain for the unsteady state simulations. For the broad objective of developing an optimal, conjunctive groundwater-seawater desalination use strategy as implemented in the current study, such a “lumped” approach involving relatively coarse spatial and temporal discretisation scales should suffice. Nonetheless, a recommendation to investigate this assumption will be included in the suggestions for further work at the end of the thesis.

5.2.3 Aquifer boundary conditions

The aquifer boundary conditions define where water enters or leaves the model domain and in what quantity. Thus, the boundaries should represent real physical boundaries such as surface water bodies like river, drain, stream and sea. The upper boundary of layer 1 is chosen to be the water table in the aeolianite. The extent of layer 1 and water table contours as of March 1997 are shown in Figure 5.6.

Constant head cells are assigned values of -1 as recommended in the GMS software guide (EMRL, 2004). These cells have been set along the eastern edge of layer 1 beyond Fault 1 in order to reflect the existing continuous constant head at the adjacent *sabkha* areas (see Figure 5.6). All other boundaries for layer 1 are treated as no flow boundaries (i.e. inactive) where cells are assigned values of 0, since water levels in this layer are not considered to be sustained by flows across model boundaries. This is conservative because the water table contours do suggest some minor inflow across the southern and northern boundaries and these flows could be expected to increase as pumping occurs.

On the other hand, the extent of layer 2 and the potentiometric surface contours are shown on Figure 5.7. Elevation data prepared for the base of layer 1 have been merged with borehole and geophysics survey data to prepare a composite surface for the top of the alluvium (layer 2). As discussed earlier, this layer is complex and for modelling purpose it has been assigned a uniform thickness of 150m.

Constant head boundaries conditions, with cell values of -1, have been imposed upon layer 2 because it is reliant upon substantial through flow across the western and northern model boundaries and to a much lesser extent, the southern boundary also, is an important source of recharge for layer 2, to sustain flow (MWR, 1997a). Water levels fluctuate generally over a few meters due to Wadi recharge areas but the available data do not suggest a pronounced long term rise or decline in the water levels for the region as a whole (MWR, 1997a). Therefore, constant head boundary condition was acceptable to generate average sustaining boundary flows for layer 2 as shown in Figure 5.7.

A general head boundary condition (GHB) has been created on the Wadi al Batha channel where it crosses Fault 1 in order to reflect the existing flows leaving or entering the model via Wadi al Batha channel alluvium (see Figure 5.7). GHB is a generic form of the head dependent boundary normally used along the edge of the model to allow ground water to flow into or out of the model under the regional gradient. The function of the GHB package in the MODFLOW is mathematically similar to that of the river, drain and evapotranspiration package, in that flow into or out of a cell (i, j, k) from an external source is provided in proportion to the difference between the head in the cell, $h_{i,j,k}$, and the head assigned to the external source, $hb_{i,j,k}$ (McDonald and Harbaugh 1988). The difference between the GHB and the drain and rivers, however, is that the drain boundary condition will allow only water to be removed from the system. In additional, the river boundary condition also limits the amount of water injected into the aquifer (McDonald and Harbaugh 1988).

5.3 Recharge and abstraction input data

A comprehensive hydrological analysis for the Wadi al Batha was done during Ash Sharqiyah Sands Aquifer assessment activities and discussed in detail in Chapter 4 (MWR, 1997g). The data provide a detailed analysis of rainfall, Wadi flow and Aflaj flow for the study area and is considered the primary source for the hydrometric data used in the model. Also, National Well Inventory Project survey data for the study area have been reprocessed to provide the total, annual, net water demand by private wells, Aflaj in each model grid cell (MWR, 1996). The estimate of water evapotranspiration by prosopis trees as discussed in Chapter 4 was also used in the model as water abstraction.

Wadi flow losses are considered a major recharge component for layer 2. The mean Wadi annual flow over 15 years from 1983 to 1997 record was used for the steady state calibration (see Table 5.3 for more details). Furthermore, the mean annual precipitation (MAP) for the model domain has been approximated as the average of the MAP recorded of 87 mm/year (see Table 5.4 for more details), (MWR, 1997g). It is not considered necessary to divide the model domain into precipitation zones because as the rainfall data shown in Table 5.4 will reveal, there has been very low spatial variability in the recorded rainfall by the five rainfall stations in the area. Therefore, mean annual recharge and abstraction over the mentioned 15 years were used as model input for the steady state calibration (see Table 5.5). The distribution of recharge input for the two layers is illustrated in Figure 5.8. A briefly description of these input data is as follows:

5.3.1 Recharge data

Direct infiltration of precipitation is the main recharge source for the aeolianite (layer 1). The hydrochemistry study of Wadi al Batha estimated a value of 2% of the MAP as direct recharge (MWR, 1996b). This value has been adopted in the model. In addition, the small area of this layer not covered by dunes in the north east has been allocated 5% MAP recharge. This is because the water level here is relatively shallow and this area also receives a small amount of Flaj return flows ($1.55 \times 10^6 m^3$/year) from the Mashaikh traditional farming area (MWR, 1996b & see Figure 5.8). Generally, the water table of the aeolianite is stable and shows little or no direct response to normal rainfall events compared to the water table for the alluvium.

On the other hand, the sources of recharge for the alluvium (layer 2) are considered to be direct infiltration of precipitation, infiltration of Wadi flow and infiltration of Aflaj return flows (see Figure 5.8). The hydrochemistry of Wadi al Batha Study estimated a value of 5% of the MAP, which is equivalent to $4.09 \times 10^6 m^3$/year, as direct recharge (MWR, 1996b). Therefore, this value has been adopted in the model for this layer. Average annual Wadi flow has been measured at approximately $18.3 \times 10^6 m^3$/year (see Table 5.5) and its recharge distribution is presented in Figure 5.8. The third recharge source for layer 2 is Aflaj return flows (see Figure 5.8). This recharge component is approximately $5.65 \times 10^6 m^3$/year and concentrated within the traditional Aflaj watered irrigation areas including irrigation distribution system losses and a leaching component (MWR, 1997d). The distribution of Aflaj irrigated area has been determined by remote sensing (MWR, 1997h). The total estimated return flows determined has been allocated evenly across all model cells that occur within Aflaj irrigated area for layer 1 and layer 2 ($5.65 \times 10^6 m^3$/year).

5.3.2 Abstraction data

The sources of groundwater abstraction from both the aeolianite and the alluvium are private wells, Aflaj and prosopis evapotranspiration. More than 90% of total abstraction occurs between Al Kamil and Bani Bu Ali (MWR, 1997a). While the alluvium supports most of the private well and Aflaj abstractions, the aeolianite is the primary source for the prosopis belt.

National well inventory (MWR, 1996a) for private wells have been apportioned across the model grid and the resulting cell total has been assigned to each model grid cell in the appropriate layer. The average annual extraction rate has been determined for each Flaj mother well and assigned to a model cell in the appropriate layer. It is important to note that flows in several Aflaj are declining. However, these Aflaj are likely to be provided with support wells and will probably maintain their present extraction levels. The area of prosopis has been determined for each grid cell by remote sensing supported by ground truthing (MWR, 1997h). This indicated that the total area of prosopis within the model domain is approximately 84.5 km^2. The estimated total groundwater extraction by prosopis evapotranspiration in the model domain is $27.6x10^6 m^3$/year (MWR, 1997a). An estimate of the average annual extraction for each square metre of prosopis has been derived by dividing the estimated total extraction (by prosopis vegetation), by the total area of prosopis. This figure was then used to estimate individual cell extraction rate (i.e. area of prosopis in each cell multiplied by average annual extraction per square metre). The total abstraction of $71.74x10^6 m^3$/year was used as input to the model for layer 1 and layer 2 as presented in Table 5.5.

5.4 Steady state simulation model

Once the field has been discretized, the model grids must be initialised. This involves assigning starting values of the hydraulic parameters, namely the hydraulic conductivity (K) and specific yield (Sy) (see Tables 4.3 and 4.4).

5.4.1 Steady state calibration

Most of the uncertainties in predicting real aquifer behaviours are due to lack of adequate data for assessing the hydraulic parameters of the aquifer and its spatial variability (SEPA, 2006). In order for a groundwater model to be used in any type of predictive mole, it must be demonstrated that the model can successfully simulate observed aquifer behaviour; where significant differences exist, it is an indication that the initial parameter estimates are inadequate and that more reliable estimates must be obtained by a formal calibration of the model. The best way to know whether or not these parameters accurately reflect the true behaviour of the aquifer is to compare the computed heads with the observed heads. If this is not satisfactory, then the only way to obtain representative aquifer parameters is by calibrating the model, i.e. during which aquifer parameters are varied until the simulated heads are acceptably close to those observed. Calibration is the process wherein certain parameters of the model such as recharge and hydraulic conductivity are altered in a systematic fashion and the model is repeatedly run until the computed solution matches field-observed values within an acceptable level of accuracy.

Calibration begins by choosing the calibration targets and determining the ranges for all potential parameters that can be adjusted during calibration. The model calibration process adjusts model parameters from their initial values until the calibration goal which involves the minimization of an objective function is achieved. The objective function is the sum of squares of the residuals, i.e.

$$J = \min \sum_{i=1}^{N} (h^i{}_o - h^i{}_s)^2 \qquad (5.1)$$

where, $h^i{}_o$ is the observed head at target site *I*; $h^i{}_s$ is the simulated head at target site *I*; and N is the total number of target wells.

GMS software provides a number of criteria for testing the adequacy of the calibration. These can be quantitative and semi quantitative.

Quantitative calibration criteria test the runs statistic value to ensure that the residual are random and the correlation between ordered weighted residuals and normal order statistics to ensure that they are normally distributed. The non-parametric runs test can be described as follows (Adeloye and Montaseri, 2002):

Let the objective be to test whether the data sample (in this case the residuals) Y_i, i =1,….,n is random based on the runs of the data with respect to the median of the observation. The procedure is therefore as follows:

1. Determine the median of the observation. To do this, sort the sample in increasing order of magnitude such that $y_1 \le y_2 \le \ldots \le y_n$. Then for an integer k, such that n = 2k (even) or n = 2k+1 (odd), the sample median denoted by $\hat{y}_{0.5}$:

$$\hat{y}_{0.5} = \begin{cases} y_{k+1} & \text{for } n = 2k+1 \\ 0.5(y_k + y_{k+1}) & \text{for } n = 2k \end{cases} \qquad (5.2)$$

2. Examine each data item in turn to see whether or not it exceeds the median. If a data item exceeds the median, then this is a success case (replaced by letter S) but if it does not exceed the median, it is a failure case (denoted by letter F). Cases that are exactly equal to the median are excluded.
3. Count the number successes and denote this by n_1; similarly denote the number failures by n_2. In general, $n = n_1 + n_2$ except where some of the values are omitted as explained in step 2 above.
4. Determine the total number of runs in the data. A run is a continuous sequence of S's until it is interrupted by an F and vice versa. Let the total number of runs be denoted by R.
5. Compute the test statistic

$$z = \frac{R - \left(\frac{2n_1n_2}{n_1+n_2} + 1\right)}{\sqrt{\frac{2n_1n_2(2n_1n_2 - n_1 - n_2)}{(n_1+n_2)^2(n_1+n_2-1)}}} \tag{5.3}$$

6. Under the null Hypothesis H_o that the sequence of S's and F's is random, z has a standard normal distribution. Hence obtain critical values of the standard normal distribution for the chosen significance level α and denote these by $\pm z_{\alpha/2}$
7. Compare the z obtained in step 5 (see equation (5.3)) with the critical values $\pm z_{\alpha/2}$. Reject H_o if $z < -z_{\alpha/2}$ or $z > z_{\alpha/2}$. In general the critical z vales are tabulated in standard statistical textbooks but for $z_{\alpha/2}$= 1.96, 1.65 and 1.28 for the 5%, 10%, and 20% significance level respectively. Since the run statistic (-0.0887) in Table 5.9 is higher than -1.96, then we do not have any statistical evidence to reject the null hypothesis. The residuals can therefore be considered to be random at the 5% level

To test for normality of the residual, it is necessary to test the statistical significance of the correlation between the residuals and the normal order statistics. Consider the null hypothesis

H_o: $R = 0$, against the alternative hypothesis

H_1: $R \neq 0$,

where R is the correlation coefficient. The appropriate test statistic for these hypotheses is (Montgomery and Runger, 2003, page 402):

$$t_o = \frac{R\sqrt{n-2}}{\sqrt{1-R^2}} \tag{5.4}$$

which has the t distribution with n-2 degrees of freedom if Ho is true. Therefore, the null hypothesis will be rejected if the calculated $t_o < -t_{\alpha/2,n-2}$ or $t_o > t_{\alpha/2,n-2}$.

The sample size for the example in Table 5.9 is 61; hence the corresponding critical value of the t-statistic at the 5% level, $t_{0.025,\ 59} = 1.96$. Also from the results in Table 5.9, the estimated correlation coefficient $R = 0.985$. Thus,

$$t_o = \frac{R\sqrt{n-2}}{\sqrt{1-R^2}} = 0.985\sqrt{\frac{61-2}{1-0.985^2}} = 43.85$$

which is very much greater than 1.96. Hence there is evidence to reject the null hypothesis at the 5% level, in other words we can accept the alternative hypothesis that the residuals are normally distributed at the 5% level.

Semi quantitative calibration criteria involve ensuring that:

1. Parameters adjusted during calibration should be consistent with field measured values.

2. Groundwater flow direction in the key area of the site should be matched by the model.
3. Important hydrological features such as groundwater structures, shapes and divides should be replicated by the model.

The GMS software provides a number of automated calibration tools as well as a trial and error method to iteratively adjust model parameters until the model computed values match the field observed values to an acceptable level of agreement. Model calibration can be done either manually or by using automated methods. The common practice is to use both methods.

In many cases, calibration can be achieved much more rapidly with an inverse model. The GMS software contains an interface to three inverse models similar to the use of equation 5.1: MODFLOW 200 PES process, PEST and UCODE. An inverse model is an internal process which is MODFLOW 200 PES process or an external utility (PEST and UCODE) that automates the parameter estimation process (EMRL, 2004). It systematically adjusts a user-defined set of input parameters until the difference between the computed and observed values of heads is minimized. MODFLOW 200 PES process was used in this study because layer 2 encompasses a diverse mix of deposits such as conglomerate, limestone, mudstone and siltstone. It also has locality variation in lithology. Therefore, PES process calibration would be the best to represent smaller K zones variations for this complex and heterogamous layer. This involved identifying polygonal zones of hydraulic conductivity, making the zones as parameters, and assigning a starting value for each zone. The PES Process will then adjust the K values assigned to the zones as it attempts to minimize the residual error between computed versus observed heads and flows. Bahremand and De Smedt (2008) used a model-independent parameter estimator, PEST, in their study titled Distributed Hydrological Modelling and Sensitivity

Analysis in Torysa Watershed, Slovakia. The results of this study demonstrated that the use of combining a GIS-based hydrological model with PEST can produce calibrated parameters that are physically sensible.

Using 42 control points (see Table 4.4 of Chapter 4), the hydraulic conductivity (K) distribution for layer 2 was derived based on field pumping test transmissivity values obtained from the exploration well drilling data using an average saturated thickness of 150 m (MWR, 1997e). This layer is much more complex as it encompasses a diverse mix of deposits such as conglomerate, limestone, mudstone and siltstone. Therefore, the K data were subdivided into smaller zones and adjusted as illustrated in Figure 5.9a by initial trial and error calibration method. In doing this, the mentioned K control points were used as guide to establish the reasonable initial starting K distribution zones compatible with the regional generated contours heads from the exploration wells in 1997 (see Figure 5.7) and the existing observation wells in these zones. Then automatic steady state calibration for these polygonal zones was carried out to determine more accurate estimates of the hydraulic parameters. Figure 5.9 (a- and b) shows the initial hydraulic conductivity values (0.3 – 65 m/day) and the final calibrated K values (0.55 – 554 m/day) for layer 2 (alluvium) respectively. This large difference in K is reflecting the heterogeneity deposits of this layer (mudstone to conglomerate).

On the other hand, the available data of the layer 1 and the homogeneity in this layer suggest that parameter zonation for K is not essential (MWR, 1997e). Therefore, an average K value of 4 m/day has been adopted for the aeolianite in the model which was acceptable value after the calibration.

Sixty one observation wells were selected to carry out the calibration process; 21 and 40 observation wells were used for layer 1 and layer 2 respectively. In order to evaluate the calibration performance, the comparison between the observed and simulated heads is reported in Tables 5.7 and 5.8 for layer 1 and layer 2 respectively. The differences between the observed and simulated heads are maximum two metres with exception of well WAB236 showing 3.1 m and 3.2 m for layer 1 and layer 2 respectively. This close difference indicates that there is good overall agreement between observed and simulated water levels throughout the model domain.

Furthermore, the software automation calibration statistics reported a value of 0.985 for the correlation between weighted residuals and normal order statistics, which is greater than 0.963 (the critical value for the correlation at the 5% significance level). This means that one can accept the hypothesis that the weighted residuals are independent and normally distributed at the 5% significance level. The calibration summary statistics are presented in Table 5.9.

5.4.2 Steady state water budget

The steady state water budget of the aquifer simulation shows that $2.65814 \times 10^5 m^3$/day as total inflow. This total inflow is subdivided into 92503m^3/day as recharge and 173311m^3/day from the surrounding boundary (171771m^3/day from the constant head and 1540m^3/day from the general head boundary). However, the total outflow was $2.65814 \times 10^5 m^3$/day, made up of 196561m^3/day as discharge from private wells, Aflaj and prosopis, and 69253m^3/day from the surrounding boundary (68734m^3/day from the constant head and 519m^3/day from the general head boundary). The difference between the total inflow and the total outflow is zero m^3/day which presents zero percent discrepancy. This indicates that the model works perfectly. The model also shows more flow enters the model from the surrounding boundary to compensate for the discharge than the flow which exits the model.

5.5 Transient model

Even though the model seemed to be performing well at the steady state, practical groundwater management modelling applications are used in the transient state which involves decision making over time. To guarantee that the model performs adequately at dynamic state, transient state calibration alike to the steady state has to be carried out.

5.5.1 Transient model calibration

Carrying out transient state calibration requires storativity (S) or specific yield (Sy), water level record versus time data and associated recharge and pumping rates. S/Sy data were available from the aquifer assessment pumping test data (MWR, 1997e). The water level records data and associated recharge and pumping rates were available for various times between 1997 and 2008 (11 years). Details of these data are presented in Tables 5.10–5.12. Four observation wells (see Figure 5.10) were used to calibrate the transient model for layer 1, as they have continuous water record during this period, while six observation wells (see Figure 5.11) were chosen to calibrate the transient model for layer 2. These observation wells were distributed to cover different part of the model domain in order to achieve representative model, but more of these observation wells were located closer to the two production well fields in order to monitor the water drawdown due to increase in pumping rate with time.

In the transient modelling, the modelling duration is divided into stress periods, defined as the time when the pumping is active. The length of the stress period in the study is four months which is equivalent to the summer period. Each stress period is considered as one time step (four months long) as there is not much variation in input and output parameters such as recharge and discharge if each time stress was considered as one month. The hot summer season starts from the beginning of May until the end of August every year. The abstraction rates during the summer season were found almost equal to the combined rate during the other eight months of the year. Therefore, each year is divided into three stress periods of four months long each. These periods are from January to April, from May to August and from September to December every year. The calibration started from January 1997 until December 2007 making the total stress periods to be 30 over this time. All boundary conditions and parameters specified in the steady state calibration were kept unchanged for the initialization of the model grid for the transient model.

Every four months water levels recorded at the observation wells: WAB11A, WAB116A, W5C and EW5 between 1997 and December 2006, were used to calibrate layer 1 (see Table 5.10). On the other hand, the water level recorded for the same period at the observation wells: EW1, EW2, EW3, NE-02, WAB238A and TPW2, were used to calibrate layer 2 (see Table 5.11). The mean value of 0.082 was used as initial specific yield (Sy) for layer 2 (see Table 4.4 of Chapter 4). However, it was not practical to use one value for this heterogeneous layer. Therefore, same distribution zones used for the hydraulic conductivity values in the steady state modelling was adopted in calibrating the specific yield values for layer 2. Figure 5.12 presents the calibrated specific yield zones for layer 2. On the other hand, the mean Sy value of 0.161 has been adopted for the aeolianite in the model because of the homogeneity of this layer.

The mean recharge of every four months (stress period) presented in Table 5.12 is used during the calibration. The abstraction rates, which were used initially, were calculated from the data collected from the National Well Inventory carried during 1993 (MWR, 1996a) and from the data collected from the Aflaj Inventory Project carried during 1997 (MRMEWR, 2001). Annual abstraction rate reached up to approximately $91x10^6 m^3$ at the beginning of 2007 as presented in Table 5.12. This increase in discharge rate is due to agriculture expansion and increase in domestic water demand as 29 production wells from the two Ash Sharqiyah groundwater wellfields have started in operation since January 2004 as the production data presented in Tables 5.13 and 5.13 (MRMEWR, 2004-2008).

Figures 5.15 - 5.19 compare the observation and prediction heads in some of the observation wells WAB111A and W5C of the aeolianite (layer 1), and EW3, TPW2 and WAB238A of the alluvium (layer 2). Wells WAB111A, EW3 and TPW2 show immediate heads decline because they are located close to private agriculture wells, unlike the distant wells W5C and WAB238A which show stable heads until the two wellfields began operation in 2004. Since then, as revealed by Figures 5.16 and 5.19, the heads in these wells have shown continuous rapid decline because they are located close to those wellfields.

In general, when the simulated heads are over or under predicted at a particular period of time in some of the observation wells such as in wells EW3 and TPW2 respectively, that could be due to heavy or less abstraction in the area occurs. That may be because of high or less water demand than assumed in that area at that particular time, which is possible given that most of the wells are owned by the citizens (private wells) which poses several logistical challenges in accurately measuring and recording the abstraction. The values of the pumping rates that were used in this modelling were obtained from the records of the National Well Inventory in 1993 and hence are the best available.

5.5.2 Transient model validation

The calibrated transient model was run for 10 years, from the beginning of January 1997 until the end of December 2006, with each year consisting of three stress periods. Each stress period consists of four months leading to 30 stress periods. Then the model should be validated to ensure that it is able to predict heads data not used in its calibration. Therefore, the model validation was run for two years, from the beginning of January 2007 until the end of December 2008 with six stress periods each four months long. The observation date, and the mean recharge and abstraction input which were used for the model validation are presented on Table 5.12. These observation data were collected from the data base of the Ministry of Regional Municipalities and Water Resources.

The observed and simulated heads in observation wells for layer 1 are compared in Figure 5.13. Superimposed on the scatter plots are the trend lines and their associated R^2. In general, the R^2 values are above 0.6, implying that over 60% of the observed variability in the head was explained by the model. The only exception was in observation well W5C for which only 33% of the observed variability was explained. This relatively poor performance at well W5C could be attributed to the steadiness of the head in this well because the effects due to the pumping were yet to manifest in this well during the calibration and validation periods (se Figure 5.16). A similar behaviour was repeated in observation wells in layer 2 as shown by the plots in Figure 5.14. For these wells, R^2 values were generally much higher than for layer 1; indeed, the least performing well in layer 2 (i.e. WAB238A) recorded a much higher R^2 of 0.39 than the 0.33 recorded for W5C in layer 1. Again as was the case for well W5C, there was a delayed response in well WAB238A to the pumping during the transient calibration and validation period and hence the head had been relatively steady in this observation well in comparison to the other observation wells in layer 2. The delayed head responses in these two wells could be due to being so distance from the pumping in the relatively short period of the calibration and validation compared to head responses at later periods of the simulation as shown in Figures 5.16 and 5.19.

Figures 5.15-5.19 also show the validation results for observation wells WAB111A and W5C of layer 1, and EW3, WAB238A and TPW2 of layer 2. In all of these observation wells, the modelled heads validated reasonably well and matched the observed value with a maximum difference of approximately (+/-) 0.2m with exception to 0.5m for well TPW2 in Figure 5.18. This big difference could be due to over estimation of the recharge or under estimation of the abstraction at that particular time period. In fact, the simulated and observed heads in wells W5C and WAB238A overlie each other as presented in Figures 5.16 and 5.19.

5.5.3 Transient water budget

The total transient water budget of the aquifer simulation for the 33 years from the beginning of 1997 to the end of 2030 shows that 4708.67x10^6m³ as total inflow. This total inflow is subdivided into 1080.26x10^6m³ as recharge, 1236.41x10^6m³ from the storage and 2392x10^6m³ from the surrounding boundary (2380.9x10^6m³ from the constant head and 11.1x10^6m³ from the general head boundary). However, the total outflow was 4708.85x10^6m³which came as 3525.35x10^6m³ a discharge from private wells, Aflaj and prosopis as well as 394.58x10^6m³from the storage and 788.92x10^6m³ from the surrounding boundary (783.29x10^6m³from the constant head and 5.63x10^6m³from the general head boundary). The difference between the total inflow and the total outflow is -0.16x10^6 m³ which presents zero percent discrepancy. This indicates that the model works perfectly. The model also shows more flow enters the model from the surrounding boundary and from the storage to compensate for the discharge than the flow which exits the model from these parameters.

5.6 Results and discussions of the impacts of long-term abstractions on Ash Sharqiyah Sands aquifers system

Once the calibration and verification of the model have been done successfully the groundwater simulation model can be used to assess the long-term implications of continued abstractions at the two operational wellfields on groundwater conditions in the aquifers of Ash Sharqiyah Sands. Of special importance and significance is the effect of pumping on the operational Aflaj in the study area to avoid their drying out after 2008 when the entire eight Wilayats of Ash Sharqiyah Region will be supplied with water from this existing groundwater water supply scheme. Thus, the model will be used to determine limits on pumping to avoid excessive drawdown and any negative impact on these operational Aflaj and environment.

Therefore, the simulation model was run for 33 years from the beginning of 1997 to the end of 2030. This period of time consists of 102 stress periods of four months each. Table 5.17 presents the mean recharge and abstraction rates used in the simulation transient model for these stresses after the once used in the calibration and the validation. Recharge data from the beginning of 1997 to the end of 2008, were obtained as field data base from the Ministry of Regional Municipalities and Water Resources. The long-term rainfall record for Oman since 1895 has indicated that the rainfall pattern approximately repeats itself every seven years (Chebaane, 1996). Therefore, the same recharge data events every seven years were adopted as recharge inputted in the model starting from 2009. On the other hand, the values of the pumping rates, that were used in this model, were obtained from the records of the National Well Inventory in 1993 plus the metered production data of the 29 production wells from the two Ash Sharqiyah groundwater wellfields starting from January 2004 up to end of December 2008 as presented in Tables 5.13 and 5.14 (MRMEWR, 2004-2008). However, the eight Wilayats projected domestic water demands have been modelled as extra abstraction starting 2009 as presented in Table 5.16 and illustrated in Figure 5.20. These demands were assumed to be delivered equally by each of the existing operational 29 wells from the two Ash Sharqiyah groundwater wellfields. Mean recharge and abstraction rates used in the simulation transient model up to 2030, are presented in Table 5.16.

Figure 5.21 shows the heads just at the beginning of operation of Al Kamil in April 2004, while Figure 5.22 represents drawdown due to mandatory pumping on the simulated heads at the end of 2030 to deliver the required domestic water demand at the eight operational wells of Al Kamil Wellfield. Figure 5.23 shows the drawdown reaching its maximum of approximately 12 m at the wellfield at the end of 2030. It is clear from this Figure that drawdown will not effect the production from all of the eight operational wells as it will be above the pump installation depth as presented in Table 5.1 assuming rainfall and other hydrological conditions within the basin remain as assumed. The shallowest pump was installed at 135.2 masl in well KP-5 and the deepest one was installed at 97.2 masl in well KP-15, where the simulated heads at the end of 2030 in all of the eight production wells are above 145 masl as illustrated in Figure 5.22.

On the other hand, Figure 5.24 shows the heads just at the beginning operation of Jaalan Wellfield in April 2004, whereas Figure 5.25 represents drawdown due to mandatory pumping on the simulated heads at the end of 2030 to deliver the required domestic water demand at the twenty one operational wells of Jaalan Wellfield. Figure 5.26 shows the drawdown reaching its maximum approximately 55 m at the wellfield at the end of 2030. Unlike Al Kamil Wellfield, it is clear from this Figure that drawdown will effect the production from 16 operational wells out of 21 wells as it will be below the pump depths as presented in Table 5.2. Considering all assumptions in the simulation model will be valid to 2030, the production wells which are predicted to be dried out are JP-1, JP-2, JP-

6, JP-7, JP-20, JP-21, JP-22, JP-23, JP-24, JP-26, JP-39, JP-39A, JP-40, JP-41, JP-43 and JP-44. Other five production wells will not be affected by the drawdown. These wells are JP-3, JP-20A, JP-25, JP-45 and JP-46. As there are more production wells in Jaalan Wellfield, it is very clear that the drawdown is more and very distinguished in the area of Jaalan Wellfield compared to the one of Al Kamil Wellfield (see Figures 5.22 and 5.25) . It must to be acknowledged that in reality actual heads at the pumping wells or at the Aflaj mother wells might be slightly different compared to the simulated heads due to cone of depression at the pumping wells. This is because the calculated heads are approximations of the heads at the four nodes of grid cells in which the well is located.

Therefore, it is necessary to develop a practical and reliable optimization model in order to find the optimum pumping scenarios from the existing production wells or more future operational wells to provide the required domestic water demands for Ash Sharqiyah Region up to 2030 without drying the existing operational wells or the Aflaj. These Aflaj Mashaikh, Faghri, Hilal, Minjired, Mahyul, Bailhiss, Al Kamil and Al Wafi, which are shown in Figure 5.27, are a very important groundwater source for both domestic and irrigation uses in the area. The first three aflaj deliver water from aeolianite (layer 1), and the other five produce water from alluvium (layer 2) as shown in Table 5.18.

One of the important purposes of the simulation model was to determine limits on pumping to avoid excessive drawdown and any negative impact on Aflaj and environment. Therefore, it is essential to avoid drying out any one of the above eight Aflaj. The results of the simulation model have shown that three of these eight Aflaj will be dried out before the end of 2030 as presented in Figures 5.28, 5.29 and 5.30. Those are Mashaikh, Faghri and Bailhiss respectively. Flaj Mashaikh is expected to be dried out on 1st September 2025, Flaj Faghri on 1st September 2030 and Flaj Bailhiss on 1st January 2025 as presented in Table 5.19. These Aflaj are located either downstream or near the wellfields. For instance Mashaikh and Faghri are the nearest to the Al Kamil Wellfield and located downstream of it. Flaj Bailhiss is located downstream of Jaalan Wellfield and it is close to the intensive private agriculture wells. It is therefore not surprising that it is predicted to have the most severe drawdown (ca. 17m) and the earliest to dry out (see Table 5.19). The simulated head at Flaj Hilal is predicted to be only a metre above the base of its mother well (see Figure 5.31). Therefore, it is essential to determine technically and financially the optimum water management strategy for the conjunctive use of domestic water supply of both groundwater and desalinated water to meet the domestic water supply needs for the eight Wilayats of Ash Sharqiyah Region up to 2030 without excessive drawdown. This scenario will be investigated in the next Chapter 6. The water levels at the mother wells of most of the upstream Aflaj is expected to be five to 11 metres above the base of their mother wells such as Flaj Minjired, Flaj Mahyul, Flaj Al Kamil and Flaj Al Wafi, as illustrated in Figures 5.32, 5.33, 5.34 and 5.35 respectively and presented in Table 5.19.

5.7 Results and discussions of the sensitivity analysis

As the hydrological and hydro-geological parameters assumptions are subjected to uncertainty, sensitivity analysis is performed. A sensitivity analysis is the process of varying model input parameters and assumptions over a reasonable range and observing the relative change in the model response (Mandle, 2002). The parameter values will be varied over an acceptable range that reflects the aquifer system and observing the relative change in model response. Typically, the observed change in hydraulic head, flow rate or contaminant transport are noted (Mandle, 2002). Commonly, if a small change in a parameter is found to produce a relatively large change in the results, then the model is considered sensitive to this parameter and more effort should be devoted to improving its determination. The two essential input parameters that are tested are the recharge and abstraction rates. Also, the boundary conditions assumptions are tested for both layers. The hydraulic conductivity and specific yield input parameter were also examined as they were used in the simulation model based on automatic calibration determination. In all sensitivity analysis runs for the steady state, only the parameter of interest is changed, others are kept constant. Comparison of results from each run and the corresponding calibrated results will indicate how sensitive the model is to the tested parameter.

5.7.1 Sensitivity analysis of the recharge rates

It is necessary to carry out the sensitivity analysis on the recharge rates as there are possible variations in rainfalls and Wadi flows, and hence infiltration into the basin due to climate changes. The proportional percentages decrease (increase) of the recharge rates were chosen to vary as -10%, -20%, +10% and +20% because there were not expected to vary more than that as there were sufficient coverage of gauging stations distributed nicely all over the study area as mentioned in Section 4.5 of Chapter 4. Table 5.20 and 5.20 show the sensitivity analysis with recharge rate as varied parameter for layer 1 and layer 2 respectively, and its investigated effect on calibrated head in some of the observation wells. The percentage variation on heads in layer 1 varied in the observation wells from -0.1% to - 0.6%, -0.1% to -1.2%, 0.1% to 0.6%, 0.1% to 1.2%, when the recharge rates varied to -10%, -20%, +10% and +20% respectively. On the other hand, the percentage variation on heads in layer 2 varied in the observation wells from zero to - 0.6%, zero to -1.2%, zero to 0.6% and zero to 1.2% when the recharge rates varied to -10%, -20%, +10% and +20% respectively. Therefore, it is clear that the simulation model is relatively not sensitive to the range varied in recharge rate indicating that the values used in the simulation model were relatively accurate and an incertitude within -20% to 20% will not affect the results of the model.

5.7.2 Sensitivity analysis of the abstraction rates

It is also necessary to carry out the sensitivity analysis on the abstraction rates as there are uncertainties of prosopis consumptive uses and uncertainties of the private wells abstractions. The proportional percentages decrease (increase) of the abstraction rates were also chosen for compression to vary as -10%, -20%, +10% and +20%. The changes in abstraction were applied to the total abstraction to meet water demands for domestic water supply as well as prosopis and private wells. Tables 5.22 and 5.23 show the sensitivity analysis with abstraction rate as varied parameter for layer 1 and layer 2 respectively, and its investigated effect on calibrated head in some of the observation wells. The percentage variation on heads in layer 1 varied in the observation wells from 0.1% to 3.3%, 0.1% to 6.5%, -0.1% to -3.2%, -0.1% to -6.4%, when the abstraction rates varied to -10%, -20%, +10% and +20% respectively. On the other hand, the percentage variation on heads in layer 2 varied in the observation wells from zero to 3.3%, zero to 6.5%, zero to -3.3% and zero to -6.4% when the abstraction rates varied to -10%, -20%, +10% and +20% respectively. Therefore, the simulation model is slightly sensitive to the +/-20% range varied in abstraction.

5.7.3 Sensitivity analysis of boundary conditions

The effects of changing the chosen boundary conditions for both layers were tested. The boundary condition for layer 1 was changed from being constant heads to be general head boundary. Table 5.24 presents the percentage variation on calibrated hydraulic heads in some of the observation wells of layer 1. The hydraulic heads in all of these observation wells varied less than 2.5% with exception to well EW5 which showed 4.6% because it is located close to the general head boundary. Similarly, the boundary condition for layer 2 was changed from being constant heads to be general head boundary. Table 5.25 shows the percentage variation on calibrated hydraulic heads in some of the observation wells of layer 2. The hydraulic heads in all of these observation wells varied from zero to -1%. This indicates that the simulation model is relatively not sensitive to the boundary condition especially for layer 2 because the boundary condition is located far away from the pumping wellfields.

5.7.4 Sensitivity analysis of the hydraulic conductivity

Two types of investigations were carried out to test the sensitivity analysis of the hydraulic conductivity. These were the effect of a percentage change of hydraulic conductivity as varied parameter on calibrated head in some of the observation wells and its effect on the steady state water budget. The proportional percentages decrease (increase) of hydraulic conductivity were chosen to vary as -10%, -20%, +10% and +20% because there were more than 23 and more than 42 hydraulic conductivity control points distributed nicely all over the study area for layer 1 (aeolianite) and layer 2 (alluvium) respectively as explained in detailed in Chapter 5. Therefore, it is not expected to vary more than -/+ (10% or 20%) even when one calibrated value was up to 800% bigger than initial value as shown in Figure 5.9. That is because the calibrated hydraulic conductivities are better indicators of the true hydraulic conditions in the aquifer than the initial estimates of this parameter. Thus, for the purpose of the sensitivity analysis, the variations applied to the K were based on the calibrated rather than on the initial values of the K.

Tables 5.26 and 5.27 show the sensitivity analysis with hydraulic conductivity as varied parameter for layer 1 and layer 2 respectively, and its investigated effect on calibrated head in some of the observation wells. The percentage variation on heads in layer 1 varied in the observation wells from -0.1% to 0.8%, -0.2 to 1.7%, -0.8% to 1.0% and -1.5% to

0.2% when the hydraulic conductivities varied to -10%, -20%, +10% and +20% respectively. One the hand, the percentage variation on heads in layer 2 varied in the observation wells from -0.1% to 0.9%, -0.2% to 1.8%, -0.8% to 0.1% and -1.5% to 0.2% when the hydraulic conductivities varied to -10%, -20%, +10% and +20% respectively. Furthermore, Table 5.28 presents the effect of a percentage change of hydraulic conductivity as varied parameter on the steady state water budget. Here, the percentage variation on the steady state water budget varied by -0.5%, -1.0%, 0.5%, and 1.0% when the hydraulic conductivities varied to -10%, -20%, +10% and +20% respectively. Therefore, it is clear that the simulation model is relatively not sensitive to the range varied hydraulic conductivity indicating that the values used in the simulation model were relatively accurate and an incertitude within -20% to 20% will not affect the results of the model.

5.7.5 Sensitivity analysis of the specific yield

Sensitivity analysis was also carried out to test for the specific yield as it was inputted in the model based on automatic calibration results. Table 5.29 presents the results of the sensitivity analysis with specific yield as varied parameter and its investigated effect on transient simulated head at the end of 2030 at the mother wells of the eight targeted Aflaj in study area. Similarly, proportional percentages decreases (increases) of specific yield were chosen to vary as -10%, -20%, +10% and +20%. In all cases and for an increase or decrease within same tested percentage of the specific yield the results are almost the same variation values of heads but with deeper or shallower heads respectively at the same Flaj’s mother well. These Aflaj which are located close and downstream of the two wellfields show more variations in head than these which are located upstream of the wellfields such as Flaj Al Kamil and Flaj Al Wafi (Table 5.29). In general, the changes in head relatively very minimal vary from zero to 0.5m, and heads decrease or increase with decreasing or increasing in specific yield percentages. Therefore, it can be concluded that the simulation model is also relatively not sensitive to the specific yield indicating again that the values used in the simulation model were relatively accurate and an incertitude within -20% to 20% will not affect the results of the model.

5.7.6 Effects of sensitivity analysis on the model water balance

Table 5.30 shows the effects of the previously discussed sensitivity analysis on the model water balance. It is clear from the Table that the inflow and the outflow from the model boundaries have a greater influence on the model behaviour than changes in the recharge, boundary conditions, hydraulic conductivity or specific yield. For example, the effects of increased abstraction on the inflow and outflow from the model boundary are very noticeable, with the former decreasing by up to 14% and the latter increasing by 22% when the abstraction was decreased by 20%. These compare with the 16% increase in inflow and 14% decrease outflow when the abstraction was decreased by 20%. The (-/+ 20%) variations in the recharge rates produce (+7%) and (-7%) changes in the inflow from the model boundary while the same variation percentage in the recharge rates result in (-9%) and (10%) changes in the outflow from the model boundary respectively. These recorded changes or sensitivities are much larger than those obtained when the hydraulic characteristics were changed. For example, when the hydraulic conductivity was decreased by 20%, only a slight change in the inflow (-2%) from the model boundary was recorded; the corresponding change in the boundary outflow was -4% On the other hand, when the hydraulic conductivity was increased by 20%, the inflow from the model boundary changed by (+1%) and the outflow changed by (+4%). The variation by (-20%) in the specific yield produces (-10%) change inflow from the model storage and only (+1%) change in the inflow from the model boundary, while it results in (+2%) change outflow from the model storage and only (-3%) change in the outflow from the model boundary. In contrast, the variation by (+20%) in the specific yield produces (+3%) change inflow from the model storage and only (-1%) change in the inflow from the model boundary, while it results in no change in outflow from the model storage and only (+2%) change in the outflow from the model boundary.

Finally, changing the boundary condition from being constant head boundary condition to be general head boundary condition resulted in a 5% increase in the inflow from the model boundary and +16% change in the outflow from the model. By nature, a general head boundary condition allows the flow to either to enter or leave the model domain depending on the direction of hydraulic gradient at the boundary, i.e. a higher head within the domain relative to outside it at boundary will allow water to move out of the domain whereas the opposite will happen if at the boundary, the head within the domain is lower than outside the domain. The fact that overall, in proportional terms, more water actually flowed out of the model domain than into it for the general head boundary condition is a reflection of the highly dynamic way in which hydraulic conditions can change during the simulation, which may not be captured with a constant head assumption at the boundary.

5.8 Summary

This Chapter described the simulation model of Ash Sharqiyah Sands Aquifer. The model of a uniform square grid of 500 m spacing, comprising 170 rows and 110 columns was developed for the unconfined layer 1 and semi-confined layer 2 of the aquifer. The MODFLOW model was run in the steady state mode via the commercial GMS-software. Polygon zonation distributions were used successfully in implementing the automatic steady state calibration for the hydraulic parameters (K and Sy) of the heterogeneous layer 2 using 42 control points. Sixty one observation wells were selected to carry out the calibration process; 21 and 40 observation wells were used for layer 1 and layer 2 respectively. The comparison between the observed and simulated heads was reported to be maximum two metres. Four observation wells, covering different parts of the model domain and close to the two operational wellfields, were used to calibrate the transient model for the aeolianite (layer 1), as they have continuous water record during this period, while six observation wells were chosen to calibrate the transient model for the alluvium (layer 2). The model was also validated to ensure that it was able to predict heads data not used in its calibration. In all of these observation wells, the modelled heads validated reasonably well and matched the observed value with a maximum difference of approximately (+/-) 0.2m.

Once the calibration and verification of the model had been done successfully, the groundwater simulation model was then used to assess the long-term implications of continued abstractions at the two operational wellfields on groundwater conditions in the aquifers of Ash Sharqiyah Sands to meet the domestic water supply to Ash Sharqiyah Region up to 2030. It was found that the existing operational 29 wells of the two groundwater wellfields will not be capable by 1st September 2025 to meet the domestic water supply needs for the eight Wilayats of Ash Sharqiyah Region without creating extensive drawdown and causing negative impact on existing operational Aflaj and the environment. Therefore, it was essential to develop a practical and reliable optimization model in order to determine the optimum pumping scenarios from the existing production wells as well as from Sur Desalination Plant to meet the increasing domestic water supply needs for the eight Wilayats of Ash Sharqiyah Region without drying out the existing 29 operational wells and insuring a minimum flow in the existing Aflaj. This scenario will be investigated in detailed in the next optimization Chapter 6.

It was found that the simulation model was relatively not sensitive within -20% to 20% to the recharge rate, hydraulic conductivity and specific yield indicating that the values used in the simulation model were determined very accurately. The model was also found to be relatively not sensitive to the boundary condition especially for layer 2 when its boundary condition was changed from being constant heads to be general head for both layers. However, the simulation model is sensitive (-/+6.4%) to the +/-20% variation in abstraction. It was also found that the inflow and the outflow from model boundaries as influenced by the abstractions have a greater influence on the model behaviour than changes to the recharge, boundary conditions, hydraulic conductivity or specific yield.

Table 5.1: Al Kamil Production Wellfield

Well No.	Easting (m)	Northing (m)	Ground elevation (masl)	Well base (masl)	Pump location (masl)	Pump capacity (m^3/day)	Layer No.
KP-1	714439.84	2454924.92	228.52	36.52	113.52	2592	2
KP-2	714038.29	2455196.93	235.16	118.16	134.16	864	1
KP-3	713624.33	2455477.34	245.40	121.40	134.40	346	1
KP-4	713210.36	2455757.76	261.39	115.39	124.39	691	1
KP-5	712796.39	2456038.17	253.20	103.20	135.20	1210	1
KP-13	714525.69	2455470.69	213.20	13.20	97.20	864	2
KP-14	714111.72	2455751.10	224.23	24.23	120.23	2592	2
KP-15	713697.76	2456031.52	219.48	19.48	121.48	1555	2

Table 5.2: Jaalan Production Wellfield

Well No. *	Easting (m)	Northing (m)	Ground elevation (masl)	Well base (masl)	Pump location (masl)	Pump Capacity (m^3/day)
JP-1	727646.8	2433314	169.20	47.20	88.20	864
JP-2	727392.2	2433756	172.36	37.36	84.36	1555
JP-3	727147.6	2434181	166.30	37.30	73.30	1210
JP-6	726398.9	2435480	201.58	66.58	86.58	346
JP-7	726149.3	2435914	204.20	43.20	88.20	432
JP-20	727955.3	2433780	153.30	28.30	88.30	1555
JP-20A	728157.9	2433320	153.00	33.00	76.00	691
JP-21	727705.7	2434214	155.47	47.47	91.47	1296
JP-22	727456.1	2434647	163.98	50.98	88.98	1296
JP-23	727159.5	2435053	159.69	39.69	84.69	1296
JP-24	726957.0	2435513	164.23	39.23	72.23	1037
JP-25	726707.4	2435947	169.44	44.44	71.44	691
JP-26	726457.8	2436380	185.68	50.68	85.68	778
JP-39	728263.7	2434247	166.88	34.88	75.88	691
JP-39A	728458.8	2433799	155.07	35.07	90.07	432
JP-40	728016.6	2434676	162.94	40.94	88.94	518
JP-41	727764.6	2435113	163.04	43.04	88.04	1080
JP-43	727265.4	2435980	158.73	0	91.73	518
JP-44	727015.8	2436413	158.16	0	98.16	1728
JP-45	726766.3	2436846	158.00	0	81.00	1555
JP-46	726516.7	2437279	160.51	0	86.51	1555

* All of these wells are producing from layer 1

Table 5.3: Recorded monthly wadi flow data for Wadi al Batha in 10^6m^3 from 19983 to 1997 (MWR, 1997g)

Wadi	Area Km	Record period	Mean monthly flow for period of record (10^6m^3)												Year 10^6m^3
			Oc	N	Dec	Ja	Fe	Ma	Ap	Ma	Jun	Jul	Au	S	
Ibra	684	83-97	0.1	0	0.1	0.0	0.8	0.4	1.3	0.2	0.1	0.5	0.3	0	3.7
Haju	243	93-97	0.0	0	0.2	0.0	0.2	0.0	0.0	0.0	0.0	0.0	0.0	0	0.5
Niba	347	93-97	0.1	0	0.3	0.0	0.2	0.0	0.1	0.0	0.1	0.6	0.0	0	1.4
Qabil	811	93-97	0.4	0	1.4	0.0	0.5	0.3	0.2	0.3	0.3	0.3	0.0	0	3.7
Taym	797	93-97	0.0	0	0.1	0.0	0.0	0.0	0.0	0.0	0.0	0.1	0.0	0	0.3
Suq	107	93-97	0.0	0	0.1	0.0	0.1	0.0	0.0	0.0	0.0	0.1	0.0	0	0.3
Zahir	403	93-97	0.0	0	0.2	0.0	0.0	0.0	0.2	0.2	0.0	0.2	0.0	0	0.8
Jhool	149	93-97	0.0	0	0.2	0.0	0.0	0.1	0.0	0.0	0.0	0.0	0.0	0	0.3
B Khali	368	83-97	0.0	0	0.4	0.1	2.5	2.9	3.1	1.3	0.3	0.7	1.2	0	12.6
Bath aKha	4472	91-97	0.0	0	0.3	0.1	0.0	0.2	0.5	0.1	0.0	0.0	0.0	0	1.3
Didu	372	91-97	0.0	0	0.0	0.0	0.0	0.0	0.1	0.0	0.1	0.0	0.0	0	0.2
Bath BBH	4916	91-97	0.0	0	0.5	0.0	0.0	0.2	0.7	0.2	0.8	0.0	0.0	0	2.4
Bath BBA	5059	91-97	0.0	0	0.2	0.0	0.0	0.2	0.6	0.1	0.1	0.0	0.0	0	1.2

Table 5.4: Recorded main monthly Rainfalls data for Wadi al Batha in (mm) from 1976 to 1997 (MWR, 1997g)

Station	Elevation (m)	Mean monthly flow for period of record (10^6m^3)												Year (mm)
		Oc	No	De	Ja	Feb	Mar	Ap	Ma	Jun	Jul	Au	S	
Ibra	425	3	3	7	8	14	15	1	5	5	10	1	2	95
Dariz	325	3	0	4	6	13	16	1	3	3	13	9	2	89
Ghabi	280	1	0	4	8	16	18	2	4	3	7	8	2	91
JBB	120	0	2	2	5	17	19	2	4	2	7	1	1	81
JBB	125	0	1	4	6	19	14	2	4	4	4	3	0	81
Mean annual precipitation (MAP) in (mm)														87

Table 5.5: Mean annual recharge and abstraction rates used in the steady state model ($10^6 m^3$/year)

	Layer 1	Layer 2	Total for the two layers
Recharge			
Direct infiltration	**4.17**	**4.09**	**8.26**
Wadi flow recharge	**0**	**18.31**	**18.31**
Aflaj return flow recharge	**1.55**	**5.65**	**7.20**
Total recharge	**5.72**	**28.05**	**33.77**
Abstraction	**22.12**	**49.62**	**71.74**

Table 5.6: Initial and the calibrated aquifer hydraulic parameters used in the model

	Initial value		Calibrated value	
	Layer 1	**Layer 2**	**Layer 1**	**Layer 2**
Hydraulic Conductivity (m/day)	**4**	**(0. 3 – 65)**	**4**	**(0.55 – 554)**
Specific Yield (Sy)	**0.16**	**0.082**	**0.16**	**(0.0023-0.11)**

Table 5.7 Steady state calibration summary for layer 1

Observation Well ID	Easting (m)	Northing (m)	Observed head (masl)	Computed Head (masl)	Residual head (m)
WAB003	683096	2468802	239.0	238.3	-0.7
WAB198	710392	2461420	170.0	168.9	-1.1
WAB224	701935	2456835	181.5	183.2	1.6
WAB174	717450	2457280	155.8	155.9	0.1
EW2	728330	2450040	127.8	127.5	-0.3
W3	691530	2444966	201.0	201.2	0.2
WAB226A	705075	2446553	174.2	174.2	-0.1
WAB231	717268	2448193	149.2	149.5	0.3
WAB221	711115	2444772	163.0	162.8	-0.2
WAB200	721471	2442003	137.1	135.6	-1.5
W5C	728340	2441430	117.8	117.5	-0.2
WAB222	709934	2436812	163.5	163.9	0.3
WAB232	718838	2437855	140.8	142.6	1.8
WAB116A	731520	2438464	108.0	106.9	-1.1
WAB202	723070	2435142	130.0	131.7	1.7
EW5	735200	2434620	101.0	101.1	0.1
WAB236	725342	2429674	123.8	127.0	3.1
WAB238A	725517	2424395	125.1	127.1	2.0
WAB220	710490	2450002	164.9	163.8	-1.1
WAB230	715012	2454469	158.3	158.0	-0.3
W-2	709803	2455342	169.0	167.2	-1.8

Table 5.8 Steady state calibration summary for layer 2

Observation Well ID	Easting (m)	Northing (m)	Observed head (masl)	Computed head (masl)	Residual head (m)
WAB003	683096	2468802	238.0	238.3	0.3
WAB216	703079	2468995	192.3	191.8	-0.5
WAB198	710392	2461420	169.5	168.9	-0.6
WAB224	701935	2456835	181.6	183.2	1.6
WAB174	717450	2457280	155.7	156.0	0.2
EW1	723630	2456220	145.0	144.6	-0.4
EW2	728330	2450040	126.7	127.5	0.8
W3	691530	2444966	200.8	201.2	0.4
WAB226A	705075	2446553	176.2	174.2	-2.0
WAB231	717268	2448193	149.2	149.5	0.3
WAB114A	732417	2448980	117.2	117.0	-0.2
WAB200	721471	2442003	137.1	135.5	-1.6
W5C	728340	2441430	118.9	117.6	-1.3
WAB222	709934	2436812	163.5	163.9	0.4
WAB232	718838	2437855	141.2	142.6	1.4
WAB116A	731520	2438464	108.6	107.0	-1.6
WAB202	723070	2435142	131.8	131.7	0.0
W8A	727428	2436779	118.1	119.5	1.4
WAB204	727911	2433072	118.0	119.8	1.7
EW5	735200	2434620	100.5	101.1	0.6
WAB236	725342	2429674	123.8	127.0	3.2
WAB238A	725517	2424395	127.0	127.1	0.1
WAB220	710490	2450002	164.7	163.8	-0.9
WAB230	715012	2454469	158.5	158.0	-0.5
W-2	709803	2455342	168.7	167.2	-1.5
WAB208A	688796	2476743	244.3	245.3	1.0
WAB247	691764	2470968	224.1	223.2	-0.8
WAB195	708682	2472336	174.7	174.0	-0.7
W-A	710140	2473030	169.2	170.4	1.2
WAB110	714916	2473442	165.9	166.1	0.3
WAB170	724940	2476556	165.5	165.3	-0.2
WAB179B	718060	2468323	164.0	165.5	1.5
WAB183	722826	2469988	164.6	165.2	0.6
KWTW4	724339	2461486	154.9	154.9	0.0
NE-02	692060	2478940	245.2	244.3	-0.9
TPW1	724217	2462300	156.5	156.5	0.0
NE-B	718980	2478680	165.8	166.7	0.9
EW3	716730	2464120	162.7	163.7	1.0
NE-5B	719280	2473850	165.5	165.8	0.3
TPW2	724010	2462760	156.9	157.6	0.7

Table 5.9: Steady state calibration summary statistics

Statistics	**Value**
Mean residual	-0.2784
Mean absolute residual	0.9267
Root mean squared residual	1.1972
Maximum weighted residual	4
Minimum weighted residual	-6.28
Mean weighted residual	-0.5456
Mean absolute weighted residual	1.8163
Root mean squared weighted residual	2.3464
Sum of squared weighted residual	335.8446
Number of residuals ≥ 0	26
Number of residuals < 0	35
Number of runs in 61 observations	30
Run statistic value	- 0.0887
Correlation between ordered weighted residuals and normal order statistics for observations	0.985

Table 5.10: Water table (masl) recorded at layer 1 observation wells used to calibrate and to validate the transient model (Ministry of Regional Municipalities and Water Resources data base)

	WAB111A	WAB116A	W5C	EW5
1/5/1997			117.5	101.2
1/9/1997			117.5	101.2
1/1/1998			117.5	101.3
1/5/1998			117.5	101.2
1/9/1998	155.8	107.5	117.5	101.1
1/1/1999	155.8	107.8	117.5	101.1
1/5/1999	155.9	107.9	117.5	101.2
1/9/1999	156.0	107.8	117.5	101.2
1/1/2000	156.0	107.7	117.5	101.2
1/5/2000	156.0	108.0	117.5	101.2
1/9/2000	155.8	107.6	117.5	101.2
1/1/2001	155.8	107.8	117.5	101.2
1/5/2001	155.8	107.8	117.5	101.2
1/9/2001	155.8	108.0	117.4	101.3
1/1/2002	155.5	108.1	117.5	101.3
1/5/2002	155.5	108.1	117.5	101.3
1/9/2002	155.3	108.2	117.5	101.3
1/1/2003	155.3	107.9	117.5	101.4
1/5/2003	155.4	108.2	117.5	101.4
1/9/2003	155.4	108.2	117.5	101.4
1/1/2004	155.4	108.3	117.4	101.5
1/5/2004	155.4	108.2	117.4	101.8
1/9/2004	155.0	108.2	117.5	101.7
1/1/2005	154.9	108.0	117.5	101.7
1/5/2005	154.9	108.4	117.6	101.6
1/9/2005	154.9	108.3	117.6	101.7
1/1/2006	154.9	108.4		101.5
1/5/2006	154.9	108.3	117.6	
1/9/2006		108.2	117.5	101.5
1/1/2007	**154.7 V**	**108.0 V**		**101.6 V**
1/5/2007	**154.6 V**	**108.3 V**	**117.5 V**	**101.6 V**
1/9/2007	**154.5 V**	**108.3 V**	**117.4 V**	**101.7 V**
1/1/2008	**154.6 V**	**108.0 V**	**117.3 V**	**101.6 V**
1/5/2008	**154.6 V**	**108.2 V**	**117.2 V**	**101.6 V**
1/9/2008	**154.5 V**	**108.2 V**	**117.0 V**	**101.6 V**
1/1/2009	**154.4 V**	**108.1 V**	**117.0 V**	**101.6 V**

V: data used for validation

Table 5.11: Water table (masl) records at some observation wells for layer 2 used to calibrate and to validate the transient model (Ministry of Regional Municipalities and Water Resources data base)

	EW1	EW2	EW3	NE-02	WAB238 A	TPW2
1/5/1997	144.9	126.82	162.95	246.32	127.05	157.40
1/9/1997	144.7	126.70	163.27	245.68	127.10	157.47
1/1/1998	144.8	126.80	163.49	245.90	127.04	158.21
1/5/1998	144.8	126.78	163.73	245.65	127.07	158.12
1/9/1998	144.7	126.62	163.75	245.06	127.04	158.02
1/1/1999	144.7	126.58	163.71	244.67	127.05	157.98
1/5/1999	144.9	126.73	163.86	245.30	127.06	158.22
1/9/1999	144.7	126.56	163.78	245.10	127.04	158.06
1/1/2000	144.7	126.49	163.66		127.04	157.93
1/5/2000	144.6	126.43	163.55		127.02	157.51
1/9/2000	144.5	126.35	163.39		127.03	157.30
1/1/2001	144.5	126.26	163.26		127.01	157.21
1/5/2001	144.5	126.20	163.17		127.02	156.89
1/9/2001	144.4	126.14	162.99	243.38	127.03	156.72
1/1/2002	144.3	126.01	162.90	243.03	127.05	156.65
1/5/2002	144.2	125.97	162.75	243.00	127.06	156.3
1/9/2002	144.2	125.87	162.66	242.9	127.07	156.07
1/1/2003	144.1	125.77	162.60	242.85	127.06	155.96
1/5/2003	144	125.72	162.47	242.78	127.06	155.59
1/9/2003	143.9	125.58	162.40	242.76	127.07	155.73
1/1/2004	143.8	125.57	162.31	242.73	127.06	155.39
1/5/2004	143.7	125.45	162.29	242.72	126.91	155.31
1/9/2004	143.5	125.38	162.14	242.53	126.90	155.26
1/1/2005	143.4	125.39	162.14		126.98	155.40
1/5/2005	143.3	125.35	162.08	243.23	127.02	155.45
1/9/2005	143.2	125.22	162.07	243.24	126.98	155.29
1/1/2006	143.1	125.20	162.02	243.21	126.97	155.25
1/5/2006		125.12	162.00	243.50	126.96	154.96
1/9/2006		125.05			126.98	
1/1/2007	**142.6 V**		**162.1 V**	**243.12 V**	**126.99 V**	**155.16 V**
1/5/2007	**142.7 V**	**125.08 V**	**162.0 V**	**242.82 V**	**126.99 V**	**155.14 V**
1/9/2007	**142.8 V**	**125.08 V**	**162.1 V**	**244.05 V**	**127.05 V**	**156.00 V**
1/1/2008	**142.8 V**	**125.04 V**	**162.9 V**	**243.68 V**		**156.50 V**
1/5/2008	**142.8 V**	**124.93 V**	**162.9 V**	**243.20 V**		**156.36 V**
1/9/2008	**142.7 V**	**124.88 V**	**162.7 V**		**126.90 V**	**156.15 V**
1/1/2009	**142.7 V**	**124.85 V**	**162.5 V**		**126.92 V**	**155.99 V**

V: data used for validation

Table 5.12: Mean recharge and abstraction used in different time steps to calibrate and to validate the transient model

Stress no.	Date	Recharge ($10^6 m^3$)	Abstraction ($10^6 m^3$)
1	Jan-April 1997	69.6	21.2
2	May-August 1997	10.3	33.6
3	Sept-Dec 1997	27.1	21.5
	total	**107.0**	**76.3**
4	Jan-April 1998	10.5	21.1
5	May-August 1998	7.5	33.6
6	Sept-Dec 1998	5.2	21.5
	total	**23.2**	**76.3**
7	Jan-April 1999	13.8	21.1
8	May-August 1999	5.3	33.5
9	Sept-Dec 1999	2.4	21.5
	total	**21.5**	**76.1**
10	Jan-April 2000	9.3	21.3
11	May-August 2000	7.4	33.5
12	Sept-Dec 2000	6.2	21.5
	total	**22.9**	**76.2**
13	Jan-April 2001	5.1	21.1
14	May-August 2001	4.8	33.5
15	Sept-Dec 2001	5.4	26.2
	total	**15.3**	**80.8**
16	Jan-April 2002	2.4	26.2
17	May-August 2002	2.4	38.6
18	Sept-Dec 2002	5.1	26.6
	total	**9.9**	**91.3**
19	Jan-April 2003	11.6	26.1
20	May-August 2003	26.4	37.8
21	Sept-Dec 2003	2.4	25.7
	total	**40.4**	**89.7**
22	Jan-April 2004	8.9	26.2
23	May-August 2004	6.1	38.2
24	Sept-Dec 2004	4.8	26.2
	total	**19.7**	**90.6**
25	Jan-April 2005	8.3	25.4
26	May-August 2005	5.2	38.4
27	Sept-Dec 2005	2.4	26.3
	total	**15.9**	**90.2**
28	Jan-April 2006	12.7	25.8
29	May-August 2006	15.4	38.4
30	Sept-Dec 2006	5.7	26.5
	total	**33.8**	**90.7**
31	Jan-April 2007 **(V)**	7.0	26.0
32	May-August 2007 **(V)**	43.2	38.5
33	Sept-Dec 2007 **(V)**	27.1	26.7
	Total	**77.3**	**91.2**
34	Jan-April 2008 **(V)**	10.6	26.6
35	May-August 2008 **(V)**	7.5	39.2
36	Sept-Dec 2008 **(V)**	5.2	27.4
	Total	**23.3**	**93.2**

(V): data used for validation

Table 5.13: Al Kamil Wellfield actual average production (m^3/day) used in different time steps to calibrate and validate the transient model (MRMEWR, 2004-2008)

Date	KP-1	KP-2	KP-3	KP-4	KP-5	KP-13	KP-14	KP-15	total ($10^6 m^3$)
Jan-April 2004	202	194	107	43	64	47	132	80	0.11
May-Aug 2004	8	47	38	20	3	3	8	5	0.02
Sep- Dec 2004	4	125	34	63	0	6	23	12	0.03
Jan-April 2005	127	383	37	216	0	149	372	233	0.18
May-Aug 2005	400	394	111	246	46	316	1051	164	0.34
Sep- Dec 2005	26	289	89	189	11	235	783	10	0.20
Jan-April 2006	92	220	71	155	39	196	651	46	0.18
May-Aug 2006	484	156	44	111	204	148	494	242	0.23
Sep- Dec 2006	494	163	58	6	218	157	525	257	0.23
Jan-April 2007	0	190	69	146	256	157	635	258	0.21
May-Aug 2007	0	248	91	186	334	63	822	396	0.26
Sep- Dec 2007	0	281	101	210	361	0	894	439	0.28
Jan-April 2008	701	255	24	202	349	177	852	416	0.36
May-Aug 2008	1298	455	170	325	580	431	1463	708	0.67
Sep- Dec 2008	1551	519	186	349	643	477	2047	760	0.80

Table 5.14: Jaalan Wellfield actual average production (m^3/day) used in different time steps to calibrate and validate the transient model (MRMEWR, 2004-2008)

Date	JP-1	JP-2	JP-3	JP-6	JP-7	JP-20	JP-20a	JP-21	JP-22	JP-23	JP-24
Jan-April 2004	340	571	117	204	61	148	60	111	164	177	189
May-Aug 2004	150	472	338	49	53	105	73	297	220	412	40
Sep- Dec 2004	67	456	345	41	52	317	77	262	323	434	30
Jan-April 2005	58	404	342	21	37	505	55	381	342	428	22
May-Aug 2005	229	425	247	109	146	491	220	366	331	416	364
Sep- Dec 2005	197	392	268	90	128	455	162	338	308	385	333
Jan-April 2006	170	360	268	96	109	390	173	289	263	326	285
May-Aug 2006	241	479	359	129	153	511	230	379	349	431	377
Sep- Dec 2006	258	414	384	137	163	556	245	403	374	310	405
Jan-April 2007	247	495	337	132	155	532	222	384	358	413	338
May-Aug 2007	283	457	421	162	187	675	297	485	440	474	440
Sep- Dec 2007	306	618	456	161	199	669	221	481	438	403	479
Jan-April 2008	303	612	450	175	214	727	1	544	449	465	514
May-Aug 2008	287	765	561	218	258	878	0	626	560	666	625
Sep- Dec 2008	392	752	580	216	255	852	32	620	547	649	618
Date	**JP-25**	**JP-26**	**JP-39**	**JP-39a**	**JP-40**	**JP-41**	**JP-43**	**JP-44**	**JP-45**	**JP-46**	**total ($10^6 m^3$)**
Jan-April 2004	885	137	64	86	50	110	54	220	6	155	0.5
May-Aug 2004	17	64	14	12	49	22	10	37	0	32	0.3
Sep- Dec 2004	6	5	10	12	48	21	4	10	1	12	0.3
Jan-April 2005	3	11	12	11	30	17	0	32	1	7	0.3
May-Aug 2005	21	212	198	138	79	226	133	180	2	37	0.6
Sep- Dec 2005	251	199	190	125	125	202	134	216	0	31	0.5
Jan-April 2006	214	27	163	103	107	193	115	462	0	47	0.5
May-Aug 2006	285	167	217	133	143	254	154	579	0	5	0.7
Sep- Dec 2006	304	251	233	147	152	209	162	655	0	0	0.7
Jan-April 2007	289	239	224	141	146	259	156	470	0	0	0.7
May-Aug 2007	370	111	282	182	179	316	175	782	0	42	0.8
Sep- Dec 2007	366	248	281	181	182	318	197	699	0	56	0.8
Jan-April 2008	393	346	305	197	197	345	213	750	0	154	0.9
May-Aug 2008	482	398	368	233	234	393	256	993	0	80	1.1
Sep- Dec 2008	477	395	367	242	231	403	252	1037	0	143	1.1

Table 5.15: Projected domestic water demand in the study area Wilayats (m^3/day) used in the simulation model (after Parsons Intern. & Co LLC, 2005)

Wilayat	Sur	Kamil Wafi	JBB Hassan	JBB Ali	Bidiyah	Qabil	Ibra	Mudaybi	TOTAL	TOTAL
Year	**Sub-total**	**Sub-total**	**Sub-total**	**Sub-total**	**Sub-total**	**Sub-total**	**Sub-total**	**Sub-total**	**(m^3/day)**	**($10^6 m^3$/year)**
2009	12260	3699	4718	9550	2900	1342	4098	8123	*46689*	**17.0**
2010	12602	3802	4849	9816	2981	1379	4212	8350	*47991*	**17.5**
2011	12954	3908	4984	10090	3064	1417	4329	8582	*49329*	**18.0**
2012	13315	4018	5124	10372	3149	1457	4450	8822	*50706*	**18.5**
2013	13687	4130	5266	10661	3237	1498	4574	9068	*52121*	**19.0**
2014	14069	4245	5413	10958	3327	1539	4702	9321	*53575*	**19.6**
2015	14313	4548	5799	11482	3504	1648	5028	9897	*56218*	**20.5**
2016	15832	4795	6114	12106	4228	3225	5853	14067	*66220*	**24.2**
2017	16210	4909	6260	12395	4329	3302	5993	14404	*67802*	**24.7**
2018	16597	5027	6410	12691	4433	3381	6137	14748	*69423*	**25.3**
2019	16994	5147	6563	12995	4539	3462	6283	15100	*71082*	**25.9**
2020	17564	5320	6784	13431	4691	3578	6494	15607	*73469*	**26.8**
2021	18333	5552	7081	14019	4896	3735	6778	16290	*76684*	**28.0**
2022	18183	5507	7023	13904	4856	3704	6723	16157	*76056*	**27.8**
2023	18598	5632	7183	14221	4967	3788	6876	16525	*77790*	**28.4**
2024	19616	5941	7576	15000	5239	3996	7253	17430	*82050*	**29.9**
2025	21240	6094	7771	15386	5673	4327	7853	18873	*87215*	**31.8**
2026	21726	6359	8110	16057	5796	4421	8024	19285	*89779*	**32.8**
2027	22224	6505	8296	16424	5929	4522	8208	19726	*91835*	**33.5**
2028	22733	6654	8486	16801	6065	4626	8396	20178	*93938*	**34.3**
2029	23253	6814	8690	17204	6204	4732	8588	20640	*96125*	**35.1**
2030	23785	6824	8702	17231	6353	4845	8794	21135	*97669*	**35.6**

Table 5.16: Annual recharge and abstraction rates used in the transient simulation model to predict heads up to end of 2030

Year	Recharge ($10^6 m^3$)	Abstraction ($10^6 m^3$)
1997	107.0	76.3
1998	23.2	76.2
1999	21.5	76.1
2000	22.9	76.3
2001	15.3	80.8
2002	9.9	91.4
2003	40.4	89.6
2004	19.8	90.6
2005	15.9	90.1
2006	33.8	90.7
2007	77.3	91.2
2008	23.3	93.2
2009	10.0	105.1
2010	40.4	105.6
2011	19.7	106.1
2012	15.9	106.8
2013	33.8	107.0
2014	77.3	107.4
2015	23.2	108.3
2016	10.0	112.1
2017	40.4	112.4
2018	19.7	113.0
2019	15.9	113.6
2020	33.9	114.7
2021	77.3	115.6
2022	23.1	115.4
2023	10.0	116.0
2024	40.5	117.7
2025	19.7	119.1
2026	15.9	120.1
2027	33.8	120.7
2028	77.3	121.2
2029	23.2	122.2
2030	10.0	122.7

Table 5.17: Mean recharge and abstraction used in the simulation transient model

Stress no.	Date	Recharge (10^6m^3)	Abstraction (10^6m^3)	Stress no.	Date	Recharge (10^6m^3)	Abstract (10^6m^3)
37	Jan-April 2009	2.4	30.7	70	Jan-April 2020	12.7	34.1
38	May-August	2.5	43.2	71	May-August	15.4	46.2
39	Sept-Dec 2009	5.1	31.2	72	Sept-Dec 2020	5.7	34.4
	Total	**10.0**	**105.1**		**Total**	**33.8**	**114.7**
40	Jan-April 2010	11.6	30.9	73	Jan-April 2021	7.0	34.2
41	May-August	26.4	43.3	74	May-August	43.2	46.6
42	Sept-Dec 2010	2.4	31.4	75	Sept-Dec 2021	27.1	34.8
	Total	**40.4**	**105.6**		**Total**	**77.3**	**115.6**
43	Jan-April 2011	8.8	31.0	76	Jan-April 2022	10.5	34.2
44	May-August	6.1	43.5	77	May-August	7.5	46.5
45	Sept-Dec 2011	4.8	31.6	78	Sept-Dec 2022	5.2	34.7
	Total	**19.7**	**106.1**		**total**	**23.2**	**115.4**
46	Jan-April 2012	8.3	31.5	79	Jan-April 2023	2.4	34.4
47	May-August	5.2	43.6	80	May-August	2.5	46.7
48	Sept-Dec 2012	2.4	31.7	81	Sept-Dec 2023	5.1	34.9
	Total	**15.9**	**106.8**		**total**	**10.0**	**116.0**
49	Jan-April 2013	12.7	31.4	82	Jan-April 2024	11.6	35.2
50	May-August	15.4	43.7	83	May-August	26.4	47.1
51	Sept-Dec 2013	5.7	31.9	84	Sept-Dec 2024	2.4	35.4
	Total	**33.8**	**107.0**		**total**	**40.4**	**117.7**
52	Jan-April 2014	7.0	31.5	85	Jan-April 2025	8.8	35.4
53	May-August	43.2	43.9	86	May-August	6.1	47.7
54	Sept-Dec 2014	27.1	32.0	87	Sept-Dec 2025	4.8	36.0
	Total	**77.3**	**107.4**		**total**	**19.7**	**119.1**
55	Jan-April 2015	10.5	31.8	88	Jan-April 2026	8.3	35.8
56	May-August	7.5	44.2	89	May-August	5.2	48.0
57	Sept-Dec 2015	5.2	32.3	90	Sept-Dec 2026	2.4	36.3
	Total	**23.2**	**108.3**		**total**	**15.9**	**120.1**
58	Jan-April 2016	2.4	33.3	91	Jan-April 2027	12.7	36.0
59	May-August	2.5	45.3	92	May-August	15.4	48.1
60	Sept-Dec 2016	5.1	33.5	93	Sept-Dec 2027	5.7	36.6
	Total	**10.0**	**112.1**		**total**	**33.8**	**120.7**
61	Jan-April 2017	11.6	33.2	94	Jan-April 2028	7.0	36.5
62	May-August	26.4	45.5	95	May-August	43.2	47.9
63	Sept-Dec 2017	2.4	33.7	96	Sept-Dec 2028	27.1	36.8
	Total	**40.4**	**112.4**		**total**	**77.3**	**121.2**
64	Jan-April 2018	8.8	33.4	97	Jan-April 2029	10.5	36.5
65	May-August	6.1	45.7	98	May-August	7.5	48.6
66	Sept-Dec 2018	4.8	33.9	99	Sept-Dec 2029	5.2	37.1
	Total	**19.7**	**113.0**		**total**	**23.2**	**122.2**
67	Jan-April 2019	8.3	33.6	100	Jan-April 2030	2.4	36.6
68	May-August	5.2	45.9	101	May-August	2.5	48.8
69	Sept-Dec 2019	2.4	34.1	102	Sept-Dec 2030	5.1	37.3
	Total	**15.9**	**113.6**		**total**	**10.0**	**122.7**

Table 5.18: The targeted eight Aflaj to be protected from drying out

Flaj's name	Wilayat	E	N	Depth of mother well (masl)	Layer
Mashaikh	JBB Hassan	718950	2455350	147.0	1
Faghri	JBB Hassan	720372	2455391	144.0	1
Hilal	JBB Hassan	720489	2456249	144.0	1
Minjired	JBB Hassan	730796	2450861	118.0	2
Mahyul	JBB Hassan	730981	2449873	110.0	2
Bailhiss	JBB Ali	738023	2439140	76.0	2
Al Kamil	Al Kamil /Al Wafi	718025	2463415	152.0	2
Al Wafi	Al Kamil / Al Wafi	723415	2459157	139.0	2

JBB: Jaalan Bani Bu

Table 5.19: Drawdown at the end of 2030 at the mother wells of the targeted Aflaj to be protected from drying out

Flaj's name	Water level in May 1997 (masl)	Water level at the end of 2030 (masl)	Drawdown (m)	Base of mother well (masl)	Remarks
Mashaikh	151.7	146.3	5.3	147.0	**Dried on 1/9/2025**
Faghri	149.0	144.1	4.9	144.0	**Nearly dried on 1/9/2030**
Hilal	150.0	145.1	4.9	144.0	**1.1 m above**
Minjired	128.0	123.2	4.8	118.0	**5.2 m above**
Mahyul	126.3	121.0	5.3	110.0	**11 m above**
Bailhiss	91.6	75.0	16.6	76.0	**Dried on 1/1/2025**
Al Kamil	162.0	159.0	3.0	152.0	**7 m above**
Al Wafi	151.6	147.1	4.5	139.0	**8.1 m above**

Table 5.20: Sensitivity analysis of hydraulic head (%) to changes in recharge rate for layer 1 (base recharge rate = 5.72 x10^6m³/year)

Observation well id	Calibrated head used in the simulation model (masl)	Proportional change in recharge rate (%)			
		-10%	-20%	+10%	+20%
WAB003	238.3	-0.1	-0.3	+0.1	+0.3
WAB198	168.9	-0.2	-0.3	+0.2	+0.3
WAB224	183.2	-0.2	-0.4	+0.2	+0.4
WAB174	155.9	-0.2	-0.3	+0.2	+0.4
EW2	127.5	-0.3	-0.6	+0.4	+0.7
W3	201.2	-0.1	-0.1	+0.1	+0.1
WAB226A	174.2	-0.2	-0.3	+0.1	+0.3
WAB231	149.5	-0.2	-0.5	+0.3	+0.5
WAB221	162.8	-0.2	-0.4	+0.1	+0.3
WAB200	135.6	-0.4	-0.7	+0.3	+0.6
W5C	117.5	-0.5	-1.0	+0.5	+1.0
WAB222	163.9	-0.1	-0.2	+0.1	+0.2
WAB232	142.6	-0.2	-0.4	+0.2	+0.5
WAB116A	106.9	-0.6	-1.2	+0.6	+1.2
WAB202	131.7	-0.2	-0.5	+0.3	+0.6
EW5	101.1	-0.5	-1.0	+0.5	+1.0
WAB236	127.0	-0.2	-0.4	+0.2	+0.4
WAB238A	127.1	-0.1	-0.2	+0.1	+0.1
WAB220	163.8	-0.2	-0.4	+0.2	+0.4
WAB230	158.0	-0.2	-0.4	+0.2	+0.4
W-2	167.2	-0.2	-0.3	+0.2	+0.4

(+): head rises **(-): head drops**

Table 5.21: Sensitivity analysis of hydraulic head (%) to changes in recharge rate for layer 2 (base recharge rate = 28.05 x10^6m³/year)

Observation well id	Calibrated head used in the simulation model (masl)	Proportional change in recharge rate (%)			
		-10%	-20%	+10%	+20%
WAB003	238.3	-0.1	-0.3	+0.1	+0.3
WAB216	191.8	-0.3	-0.6	+0.3	+0.5
WAB198	168.9	-0.2	-0.3	+0.2	+0.3
WAB224	183.2	-0.2	-0.4	+0.2	+0.4
WAB174	156.0	-0.2	-0.4	+0.2	+0.3
EW1	144.6	-0.2	-0.4	+0.2	+0.5
EW2	127.5	-0.3	-0.6	+0.4	+0.7
W3	201.2	-0.0	-0.1	+0.1	+0.1
WAB226A	174.2	-0.2	-0.3	+0.1	+0.3
WAB231	149.5	-0.2	-0.5	+0.3	+0.5
WAB114A	117.0	-0.4	-0.7	+0.3	+0.7
WAB200	135.5	-0.3	-0.7	+0.3	+0.7
W5C	117.6	-0.5	-1.0	+0.5	+1.0
WAB222	163.9	-0.1	-0.2	+0.1	+0.2
WAB232	142.6	-0.2	-0.4	+0.2	+0.5
WAB116A	107.0	-0.6	-1.2	+0.6	+1.2
WAB202	131.7	-0.2	-0.5	+0.3	+0.6
W8A	119.5	-0.4	-0.8	+0.4	+0.8
WAB204	119.8	-0.3	-0.7	+0.3	+0.6
EW5	101.1	-0.5	-1.0	+0.5	+1.0
WAB236	127.0	-0.2	-0.4	+0.2	+0.4
WAB238A	127.1	-0.1	-0.2	+0.1	+0.1
WAB220	163.8	-0.2	-0.4	+0.2	+0.4
WAB230	158.0	-0.2	-0.4	+0.2	+0.4
W-2	167.2	-0.2	-0.3	+0.2	+0.4
WAB208A	245.3	-0.4	-0.7	+0.4	+0.7
WAB247	223.2	-0.3	-0.6	+0.3	+0.6
WAB195	174.0	-0.2	-0.3	+0.1	+0.3
W-A	170.4	-0.1	-0.2	+0.1	+0.2
WAB110	166.1	0.0	0.0	+0.1	+0.1
WAB170	165.3	0.0	0.0	0.0	0.0
WAB179B	165.5	0.0	-0.1	0.0	+0.1
WAB183	165.2	0.0	0.0	0.0	+0.1
KWTW4	154.9	-0.2	-0.3	+0.1	+0.3
NE-02	244.3	-0.4	-0.9	+0.4	+0.8
TPW1	156.5	-0.1	-0.2	+0.1	+0.3
NE-B	166.7	0.0	0.0	0.0	0.0
EW3	163.7	-0.1	-0.2	+0.1	+0.2
NE-5B	165.8	0.0	0.0	0.0	+0.1
TPW2	157.6	-0.1	-0.2	+0.1	+0.2

(+): head rises **(-): head drops**

Table 5.22: Sensitivity analysis of hydraulic head (%) to changes in abstraction rate for layer 1 (base abstraction rate = 22.12 x10^6m³/year)

Observation well id	Calibrated head used in the simulation model (masl)	Proportional change in abstraction rate (%)			
		-10%	-20%	+10%	+20%
WAB003	238.3	+0.1	+0.1	-0.1	-0.1
WAB198	168.9	+0.4	+0.7	-0.3	-0.7
WAB224	183.2	+0.3	+0.6	-0.3	-0.6
WAB174	155.9	+0.8	+1.6	-0.7	-1.5
EW2	127.5	+2.4	+4.8	-2.4	-4.8
W3	201.2	+0.1	+0.1	-0.1	-0.1
WAB226A	174.2	+0.3	+0.7	-0.4	-0.7
WAB231	149.5	+1.2	+2.3	-1.1	-2.1
WAB221	162.8	+0.6	+1.2	-0.6	-1.1
WAB200	135.6	+1.7	+3.3	-1.6	-3.1
W5C	117.5	+2.9	+5.7	-2.7	-5.4
WAB222	163.9	+0.3	+0.6	-0.3	-0.6
WAB232	142.6	+1.0	+2.0	-0.9	-1.8
WAB116A	106.9	+3.3	+6.5	-3.2	-6.4
WAB202	131.7	+1.3	+2.5	-1.1	-2.3
EW5	101.1	+2.5	+5.0	-2.5	-4.9
WAB236	127.0	+0.8	+1.7	-0.8	-1.6
WAB238A	127.1	+0.3	+0.7	-0.4	-0.7
WAB220	163.8	+0.7	+1.3	-0.6	-1.1
WAB230	158.0	+0.8	+1.6	-0.7	-1.4
W-2	167.2	+0.6	+1.1	-0.5	-0.9

(+): head rises **(-): head drops**

Table 5.23: Sensitivity analysis of hydraulic head (%) to changes in abstraction rate for layer 2 (base abstraction rate = 49.62 x10^6m³/year)

Observation well id	Calibrated head used in the simulation model (masl)	Proportional change in abstraction rate (%)			
		-10%	-20%	+10%	+20%
WAB003	238.3	+0.1	+0.1	-0.1	-0.1
WAB216	191.8	+0.2	+0.4	-0.2	-0.3
WAB198	168.9	+0.4	+0.7	-0.3	-0.7
WAB224	183.2	+0.3	+0.6	-0.3	-0.6
WAB174	156.0	+0.8	+1.6	-0.8	-1.5
EW1	144.6	+1.4	+2.7	-1.3	-2.4
EW2	127.5	+2.4	+4.8	-2.1	-4.1
W3	201.2	+0.1	+0.1	-0.1	-0.1
WAB226A	174.2	+0.4	+0.7	-0.4	-0.7
WAB231	149.5	+1.2	+2.3	-1.1	-2.1
WAB114A	117.0	+3.0	+6.0	-2.9	-5.8
WAB200	135.5	+1.7	+3.4	-1.6	-3.1
W5C	117.6	+2.8	+5.6	-2.8	-5.4
WAB222	163.9	+0.3	+0.6	-0.3	-0.6
WAB232	142.6	+1.0	+2.0	-0.9	-1.8
WAB116A	107.0	+3.3	+6.5	-3.3	-6.4
WAB202	131.7	+1.3	+2.5	-1.1	-2.3
W8A	119.5	+2.1	+4.2	-2.1	-4.0
WAB204	119.8	+1.5	+3.0	-1.5	-2.9
EW5	101.1	+2.5	+5.0	-2.5	-4.9
WAB236	127.0	+0.8	+1.7	-0.8	-1.6
WAB238A	127.1	+0.3	+0.7	-0.4	-0.7
WAB220	163.8	+0.6	+1.3	-0.6	-1.2
WAB230	158.0	+0.8	+1.6	-0.7	-1.4
W-2	167.2	+0.6	+1.1	-0.5	-0.9
WAB208A	245.3	+0.2	+0.4	-0.2	-0.4
WAB247	223.2	+0.2	+0.3	-0.1	-0.3
WAB195	174.0	+0.1	+0.2	-0.1	-0.2
W-A	170.4	0.0	+0.1	-0.1	-0.1
WAB110	166.1	+0.1	+0.1	0.0	0.0
WAB170	165.3	0.0	0.0	0.0	0.0
WAB179B	165.5	0.0	+0.1	-0.1	-0.1
WAB183	165.2	+0.1	+0.1	0.0	-0.1
KWTW4	154.9	+0.7	+1.3	-0.7	-1.3
NE-02	244.3	+0.2	+0.5	-0.2	-0.5
TPW1	156.5	+0.6	+1.2	-0.5	-1.1
NE-B	166.7	0.0	0.0	0.0	0.0
EW3	163.7	+0.2	+0.5	-0.2	-0.4
NE-5B	165.8	0.0	+0.1	0.0	-0.1
TPW2	157.6	+0.5	+1.0	-0.5	-0.9

(+): head rises **(-): head drops**

Table 5.24: Sensitivity analysis of hydraulic head (%) to change the aquifer boundary condition for layer 1 from being constant head to be general head

Observation well id	Calibrated head used in the simulation model (masl)	Change in hydraulic head (m)
WAB003	238.3	0.0
WAB198	168.9	0.1
WAB224	183.2	0.0
WAB174	155.9	0.1
EW2	127.5	0.4
W3	201.2	0.0
WAB226A	174.2	0.1
WAB231	149.5	0.3
WAB221	162.8	0.1
WAB200	135.6	0.6
W5C	117.5	1.3
WAB222	163.9	0.1
WAB232	142.6	0.6
WAB116A	106.9	2.5
WAB202	131.7	1.2
EW5	101.1	4.6
WAB236	127.0	1.8
WAB238A	127.1	1.3
WAB220	163.8	0.2
WAB230	158.0	0.2
W-2	167.2	0.1

(+): head rises **(-): head drops**

Table 5.25: Sensitivity analysis of hydraulic head (%) to change the aquifer boundary condition for layer 2 from being constant head to be general head

Observation well id	Calibrated head used in the simulation model (masl)	Change in hydraulic head (m)
WAB003	238.3	-0.6
WAB216	191.8	-0.2
WAB198	168.9	-0.1
WAB224	183.2	-0.3
WAB174	156.0	-0.2
EW1	144.6	-0.3
EW2	127.5	-0.6
W3	201.2	-0.1
WAB226A	174.2	-0.3
WAB231	149.5	-0.3
WAB114A	117.0	-1.0
WAB200	135.5	-0.4
W5C	117.6	-0.7
WAB222	163.9	-0.3
WAB232	142.6	-0.4
WAB116A	107.0	-0.7
WAB202	131.7	-0.4
W8A	119.5	-0.6
WAB204	119.8	-0.5
EW5	101.1	-0.5
WAB236	127.0	-0.5
WAB238A	127.1	-0.5
WAB220	163.8	-0.3
WAB230	158.0	-0.2
W-2	167.2	-0.2
WAB208A	245.3	-0.5
WAB247	223.2	-0.4
WAB195	174.0	0.0
W-A	170.4	0.0
WAB110	166.1	0.0
WAB170	165.3	-0.1
WAB179B	165.5	0.0
WAB183	165.2	0.0
KWTW4	154.9	-0.1
NE-02	244.3	-0.4
TPW1	156.5	-0.1
NE-B	166.7	0.0
EW3	163.7	0.0
NE-5B	165.8	0.0
TPW2	157.6	-0.1

(+): head rises **(-): head drops**

Table 5.26: Sensitivity analysis of hydraulic head (%) to changes in hydraulic conductivity for layer 1

Observation well id	Calibrated head used in the simulation model (masl)	Proportional change in hydraulic conductivity (%)			
		-10%	-20%	+10%	+20%
WAB003	238.3	-0.1	-0.2	+0.1	+0.2
WAB198	168.9	0.0	+0.1	-0.1	-0.1
WAB224	183.2	0.0	+0.1	0.0	0.0
WAB174	155.9	+0.1	+0.1	-0.1	-0.2
EW2	127.5	+0.2	+0.4	-0.2	-0.4
W3	201.2	-0.1	-0.1	0.0	+0.1
WAB226A	174.2	0.0	0.0	0.0	0.0
WAB231	149.5	+0.2	+0.4	-0.2	-0.4
WAB221	162.8	0.0	0.0	0.0	0.0
WAB200	135.6	+0.4	+0.8	-0.3	-0.6
W5C	117.5	+0.6	+1.2	-0.6	-1.1
WAB222	163.9	0.0	-0.1	+0.1	+0.1
WAB232	142.6	+0.2	+0.4	-0.2	-0.3
WAB116A	106.9	+0.8	+1.7	-0.8	-1.5
WAB202	131.7	+0.2	+0.6	-0.3	-0.5
EW5	101.1	+0.7	+1.5	-0.7	-1.3
WAB236	127.0	+0.2	+0.4	-0.2	-0.4
WAB238A	127.1	+0.1	+0.2	0.0	-0.1
WAB220	163.8	+0.1	+0.2	-0.1	-0.2
WAB230	158.0	+0.1	+0.2	-0.1	-0.2
W-2	167.2	0.0	+0.1	-0.1	-0.2

(+): head rises **(-): head drops**

Table 5.27: Sensitivity analysis of hydraulic head (%) to changes in hydraulic conductivity for layer 2

Observation well id	Calibrated head used in the simulation model (masl)	Proportional change in hydraulic conductivity (%)			
		-10%	-20%	+10%	+20%
WAB003	238.3	-0.1	-0.2	+0.1	+0.2
WAB216	191.8	0.0	-0.1	0.0	+0.1
WAB198	168.9	0.0	+0.1	-0.1	-0.1
WAB224	183.2	0.0	+0.1	0.0	0.0
WAB174	156.0	+0.1	+0.2	-0.1	-0.1
EW1	144.6	+0.1	+0.2	-0.1	-0.2
EW2	127.5	+0.2	+0.4	-0.2	-0.4
W3	201.2	-0.1	-0.1	0.0	+0.1
WAB226A	174.2	0.0	0.0	0.0	0.0
WAB231	149.5	+0.2	+0.4	-0.2	-0.4
WAB114A	117.0	+0.3	+0.5	-0.2	-0.4
WAB200	135.5	+0.4	+0.8	-0.4	-0.7
W5C	117.6	+0.6	+1.2	-0.5	-1.1
WAB222	163.9	-0.1	-0.2	+0.1	+0.1
WAB232	142.6	+0.2	+0.4	-0.2	-0.3
WAB116A	107.0	+0.9	+1.8	-0.8	-1.5
WAB202	131.7	+0.3	+0.6	-0.3	-0.5
W8A	119.5	+0.6	+1.2	-0.5	-1.0
WAB204	119.8	+0.5	+0.9	-0.4	-0.7
EW5	101.1	+0.7	+1.5	-0.7	-1.3
WAB236	127.0	+0.2	+0.4	-0.2	-0.3
WAB238A	127.1	+0.1	+0.2	0.0	-0.1
WAB220	163.8	+0.1	+0.2	-0.1	-0.2
WAB230	158.0	+0.1	+0.2	-0.1	-0.2
W-2	167.2	+0.1	+0.1	-0.1	-0.2
WAB208A	245.3	0.0	-0.1	0.0	+0.1
WAB247	223.2	-0.1	-0.3	+0.1	+0.2
WAB195	174.0	0.0	0.0	0.0	0.0
W-A	170.4	0.0	0.0	0.0	0.0
WAB110	166.1	0.0	0.0	0.0	0.0
WAB170	165.3	0.0	0.0	0.0	0.0
WAB179B	165.5	0.0	0.0	0.0	0.0
WAB183	165.2	0.0	0.0	0.0	0.0
KWTW4	154.9	+0.1	+0.1	0.0	-0.1
NE-02	244.3	0.0	-0.1	0.0	+0.1
TPW1	156.5	0.0	+0.1	0.0	-0.1
NE-B	166.7	0.0	0.0	0.0	0.0
EW3	163.7	0.0	0.0	0.0	0.0
NE-5B	165.8	0.0	0.0	0.0	0.0
TPW2	157.6	0.0	+0.1	0.0	-0.1

(+): head rises **(-): head drops**

Table 5.28: Sensitivity analysis of the steady state water budget (%) to changes in hydraulic conductivity for layer 1 and layer 2

Calibrated steady state water budget (flow-in = flow-out) (m^3/day)	Proportional change in hydraulic conductivity (%)			
	-10%	-20%	+10%	+20%
2.6558×10^5	-0.5	-1.0	0.5	1.0

Table 5.29: Sensitivity analysis of hydraulic head (%) at the end of 2030 at the mother wells of the targeted Aflaj to changes in specific yield

Flaj	Simulated head at Flaj mother well (masl)	Proportional change in specific yield (%)			
		-10%	-20%	+10%	+20%
Mashaikh	146.3	-0.3	-0.5	0.2	0.4
Faghri	144.1	-0.2	-0.4	0.2	0.4
Hilal	145.1	-0.2	-0.4	0.1	0.3
Minjired	123.2	-0.1	-0.3	0.1	0.2
Mahyul	121.0	-0.1	-0.3	0.1	0.3
Bailhiss	75.0	-0.4	-0.5	0.3	0.5
Al Kamil	146.3	-0.1	-0.1	0.0	0.1
Al Wafi	144.1	-0.1	-0.2	0.1	0.1

(+): head rises (-): head drops

Table 5.30: Effects of sensitivity analysis on the model water balance

	Inflow (m^3/day)				**Outflow (m^3/day)**			
Variation	inflow from storage	Inflow from model boundary	inflow change from Storage (%)	inflow change from model boundary (%)	outflow from storage	outflow from model boundary	outflow change from Storage (%)	outflow change from model boundary (%)
Recharge (-10%)	0	173311	0	3	0	69253	0	-5
Recharge (-20%)	0	173311	0	7	0	69253	0	-9
Recharge (+10%)	0	173311	0	-3	0	69253	0	5
Recharge (+20%)	0	173311	0	-7	0	69253	0	10
Abstraction (-10%)	0	173311	0	-8	0	69253	0	10
Abstraction (-20%)	0	173311	0	-14	0	69253	0	22
Abstraction (+10%)	0	173311	0	8	0	69253	0	-8
Abstraction (+20%)	0	173311	0	16	0	69253	0	-14
K (-10%)	0	173311	0	-1	0	69253	0	-2
K (-20%)	0	173311	0	-2	0	69253	0	-4
K (+10%)	0	173311	0	1	0	69253	0	2
K (+20%)	0	173311	0	1	0	69253	0	4
Sy (-10%)	112093	221718	-2	1	23574	46317	0	-1
Sy (-20%)	112093	221718	-10	1	23574	46317	2	-3
Sy (+10%)	112093	221718	2	0	23574	46317	0	1
Sy (+20%)	112093	221718	3	-1	23574	46317	0	2
change in boundary condition	0	173311	0	5	0	69253	0	16

(-): decrease
(+): increase

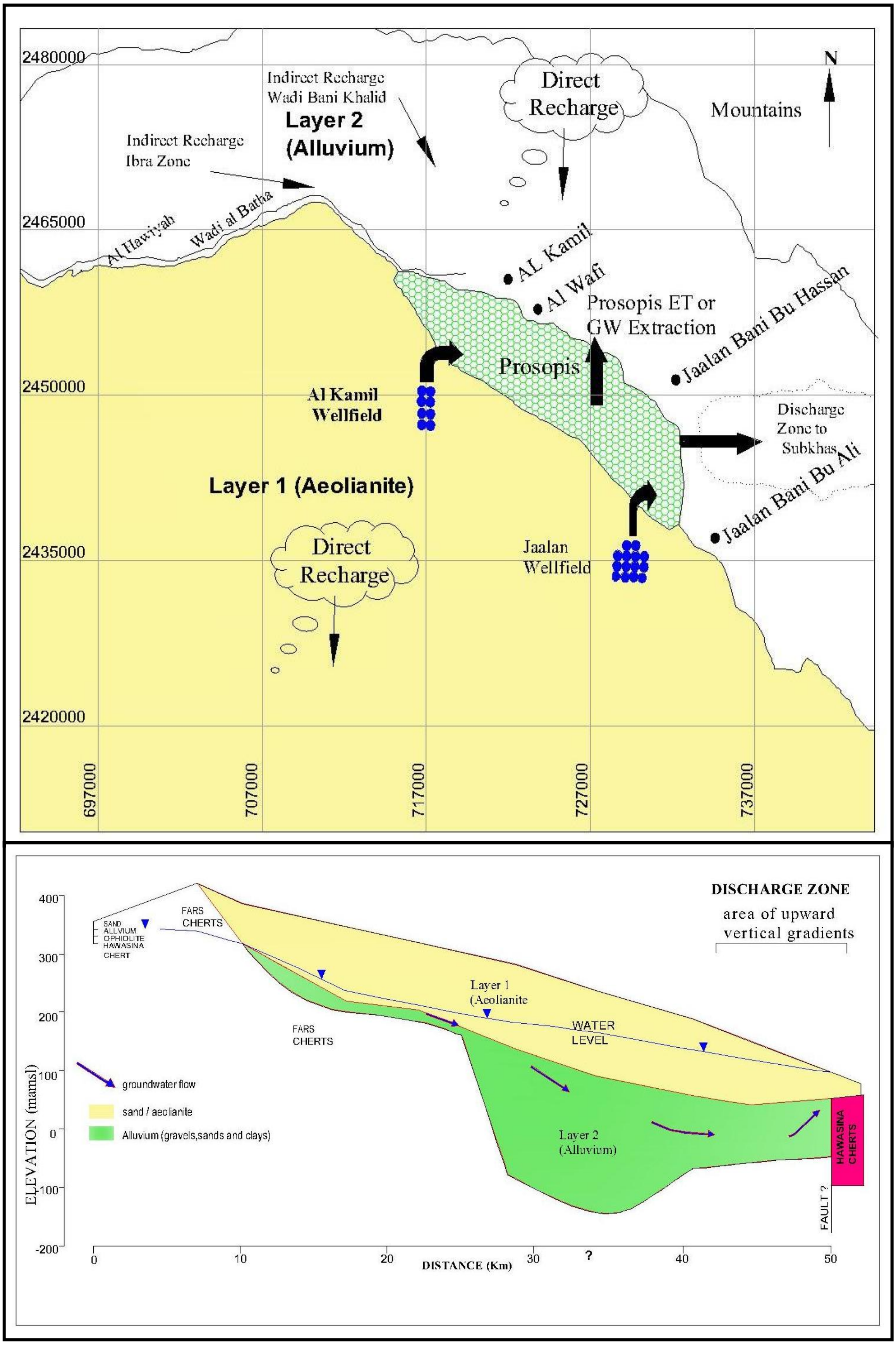


Figure 5.1: Conceptual model of the simulated area (northwest-southeast cross-section)

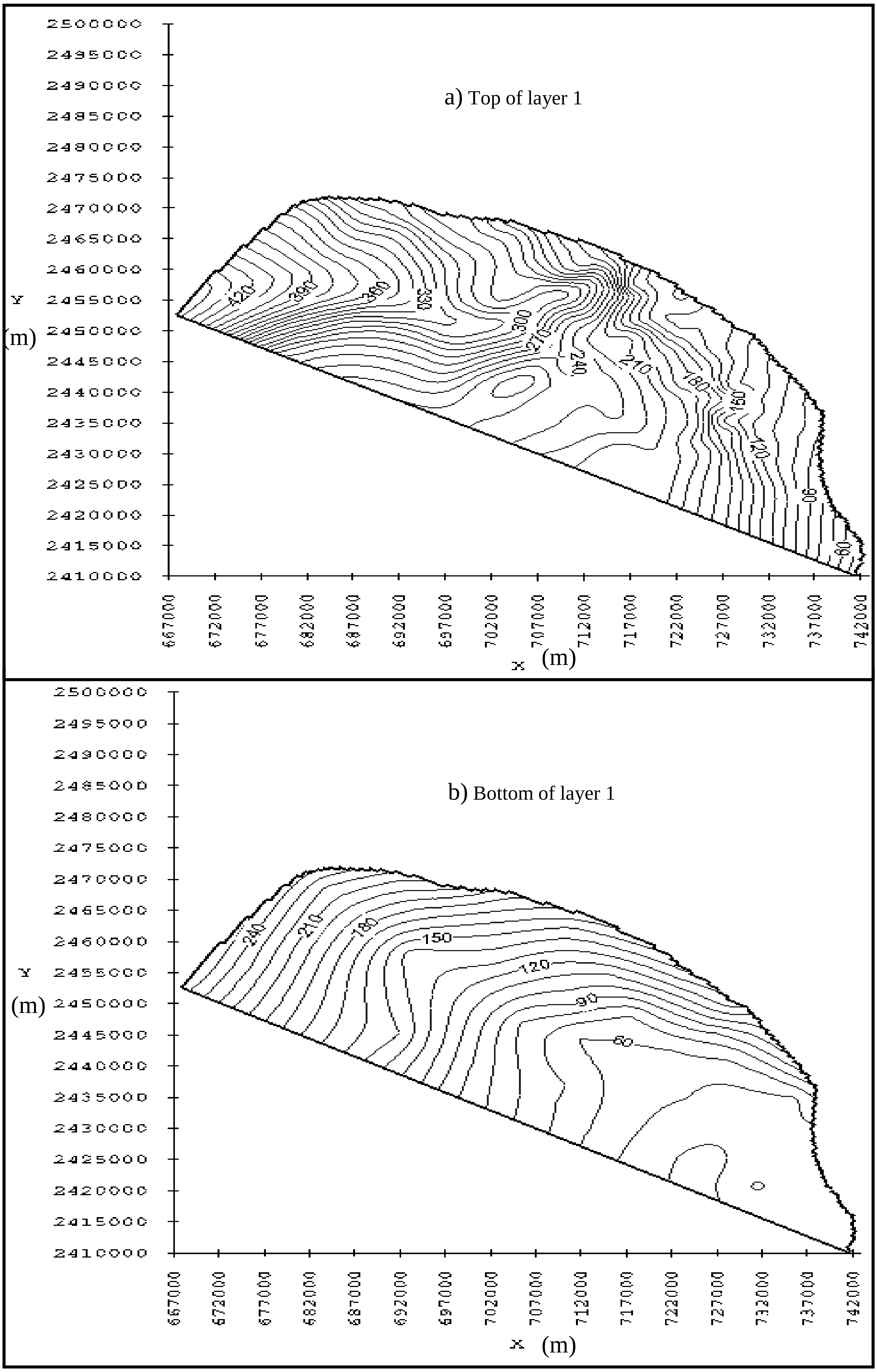


Figure 5.2: Contours showing top and bottom of layer 1 (Aeolianite)

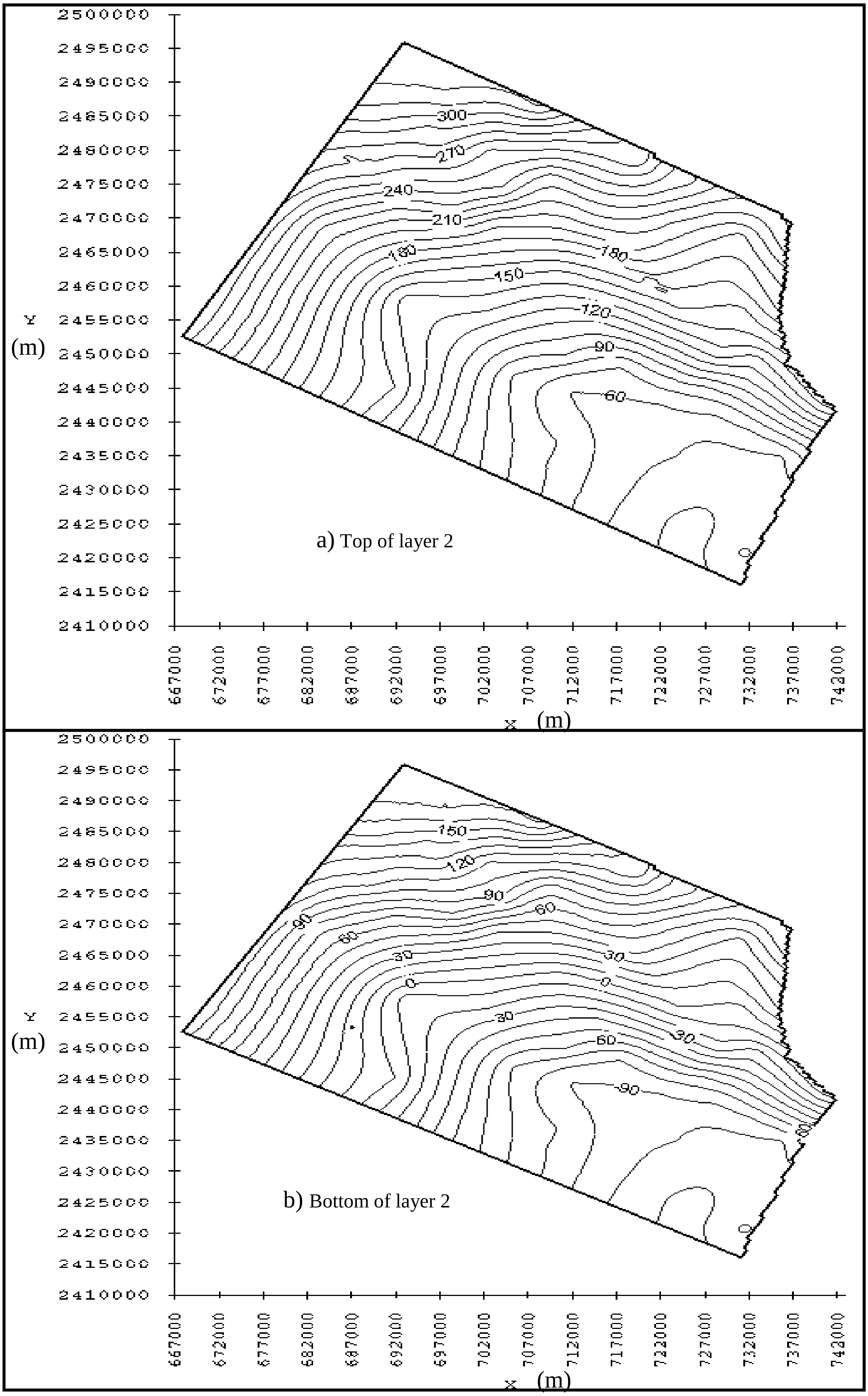


Figure 5.3: Contours showing top and bottom of layer 2 (Alluvium)

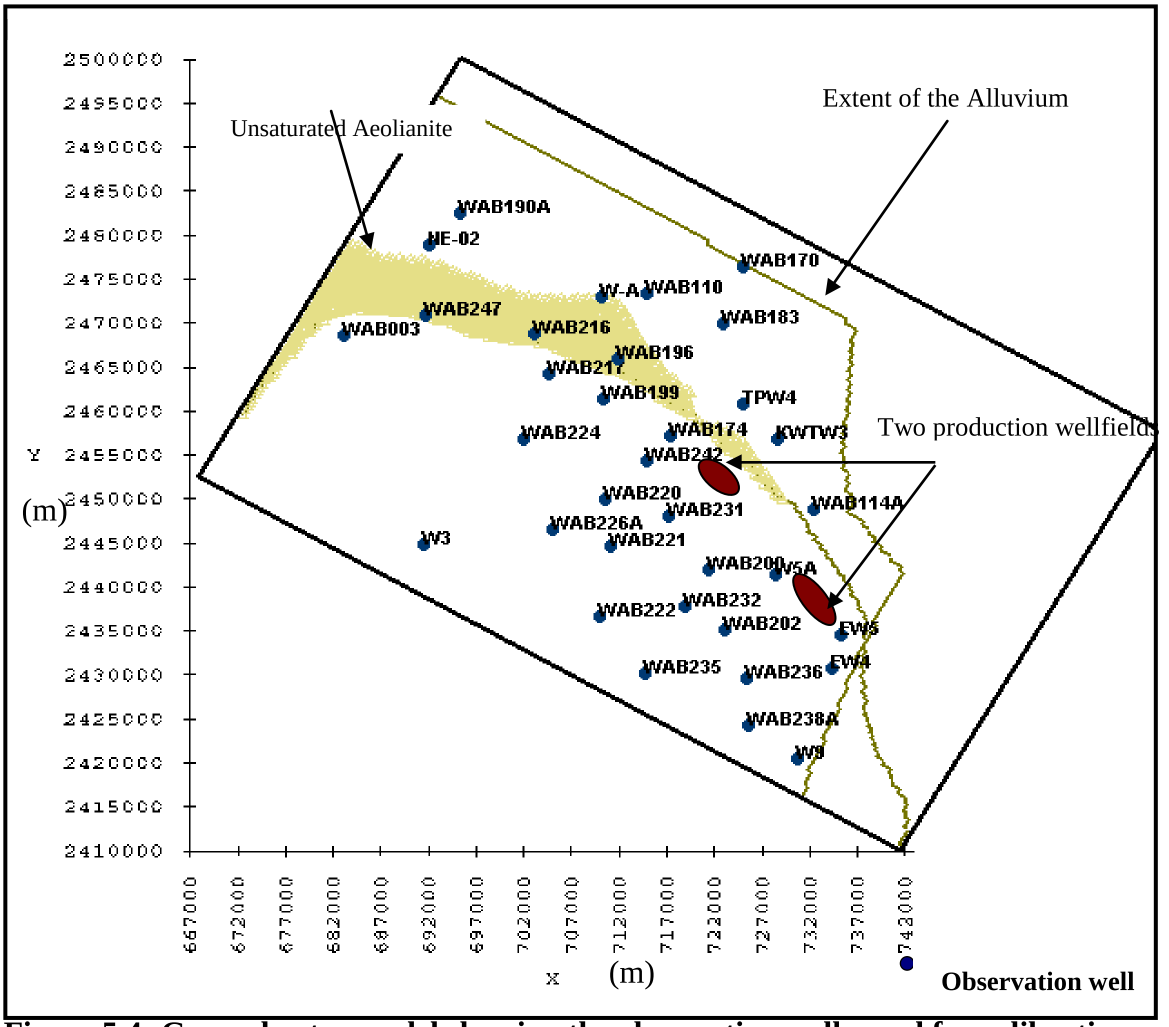


Figure 5.4: Groundwater model showing the observation wells used for calibration

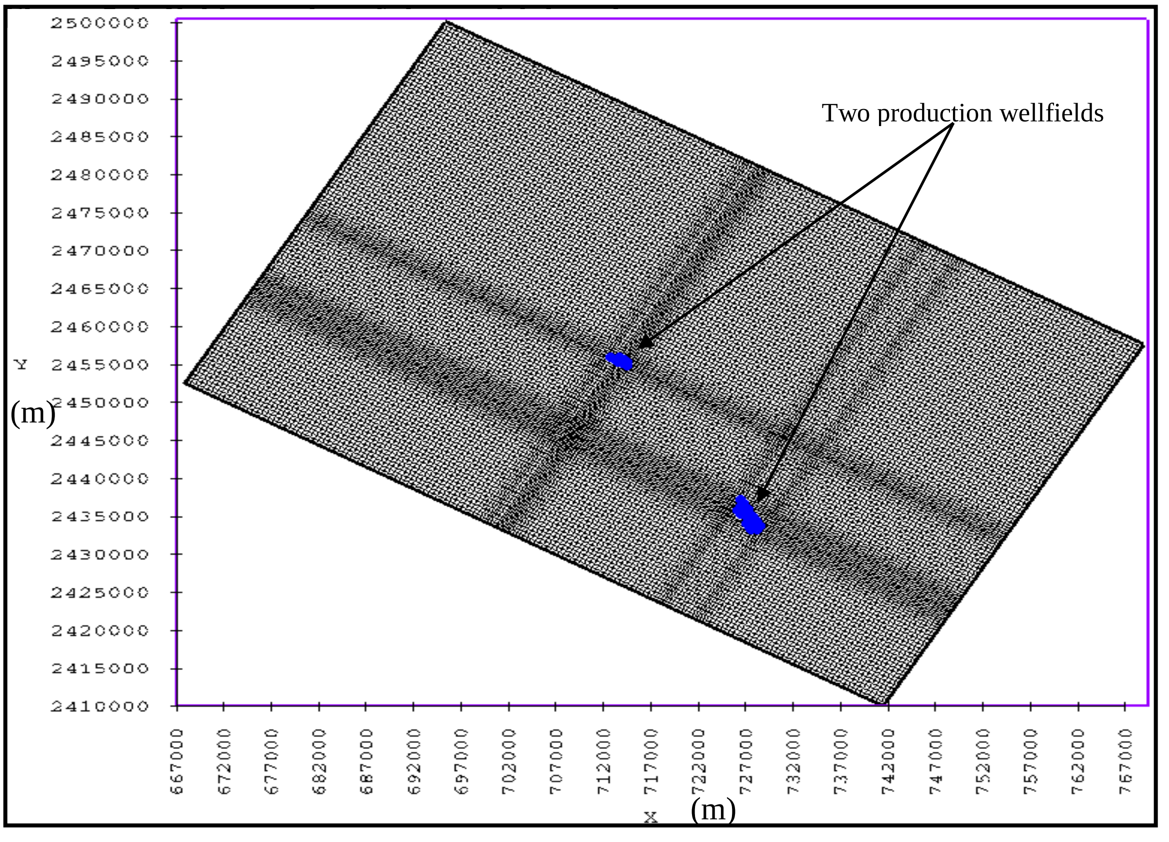


Figure 5.5: Grids spacing of the model domain showing grid cells refine at wellfields

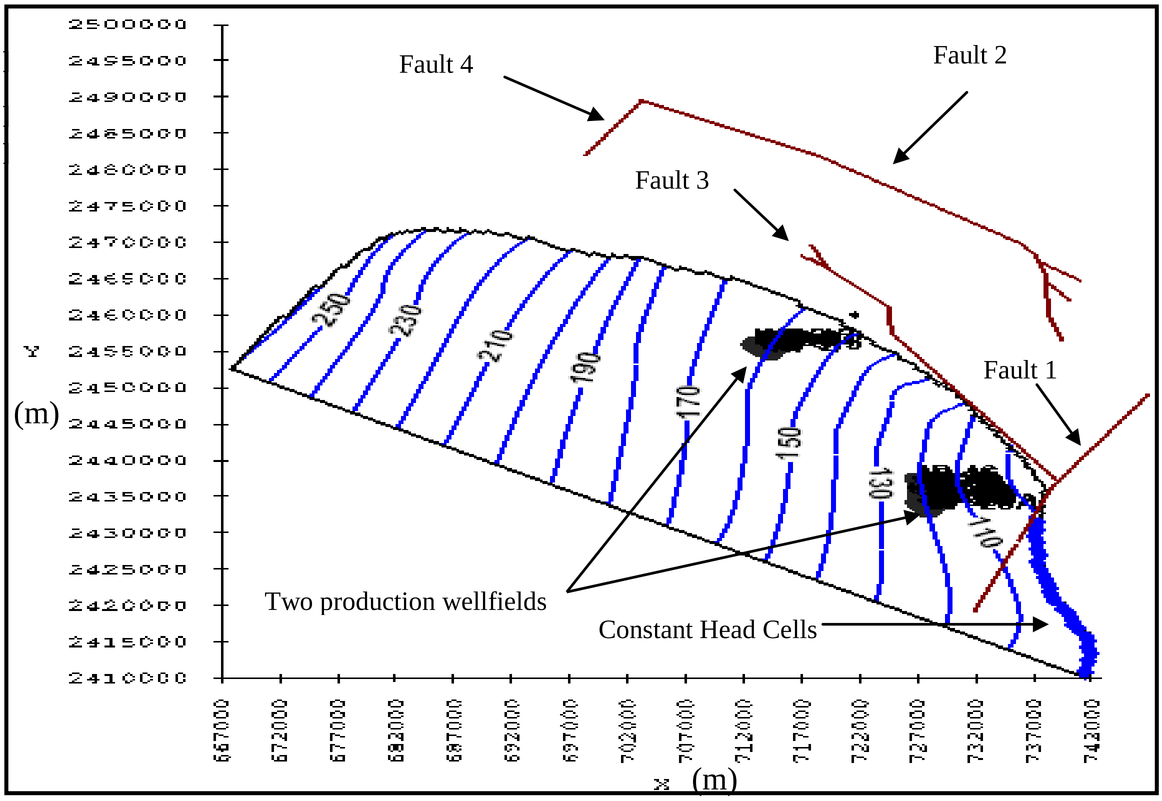


Figure 5.6: Model boundary conditions for layer 1 with water heads in 1997

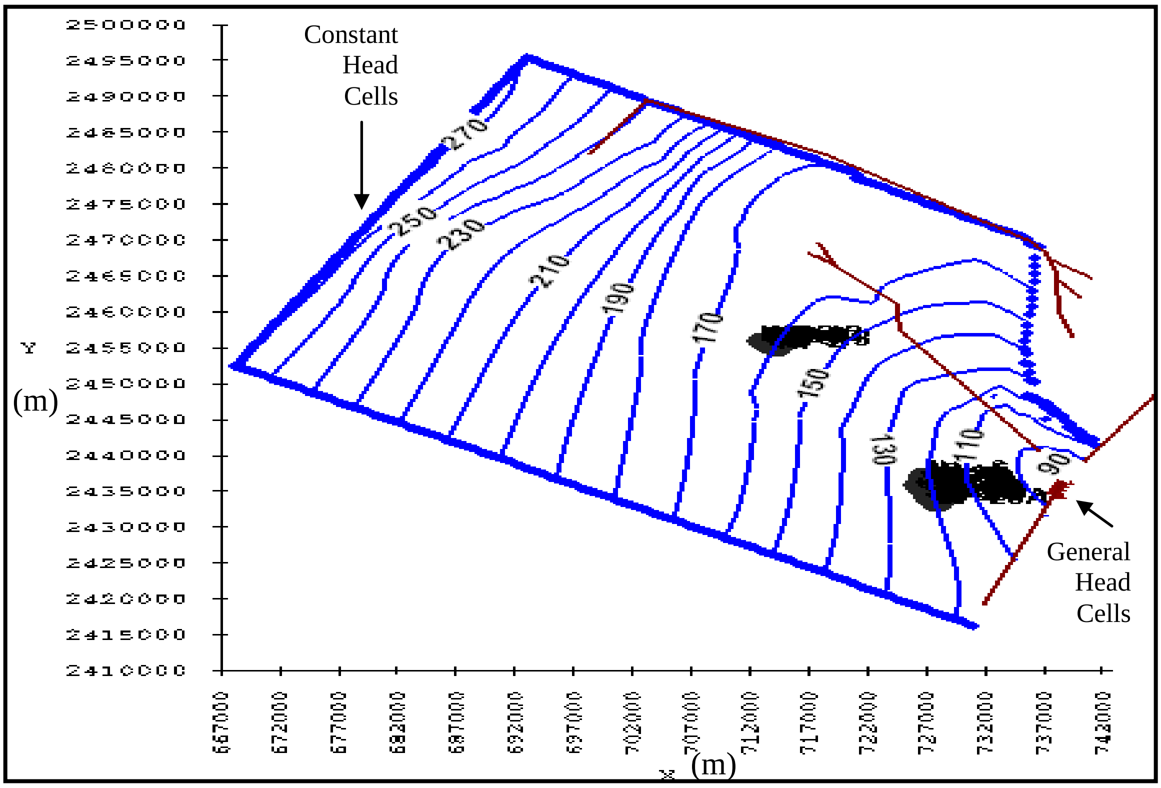


Figure 5.7: Model boundary conditions for layer 2 with water heads in 1997

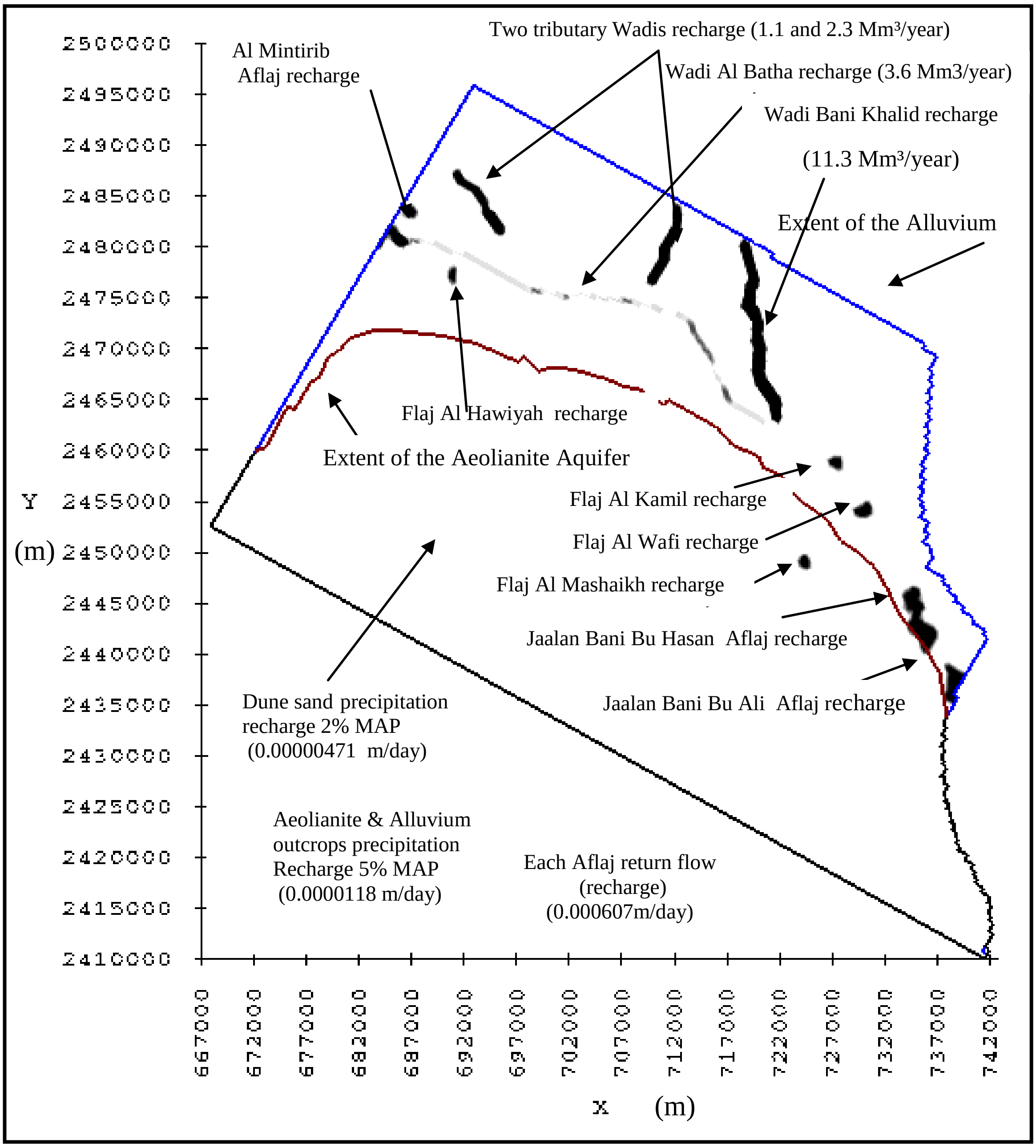
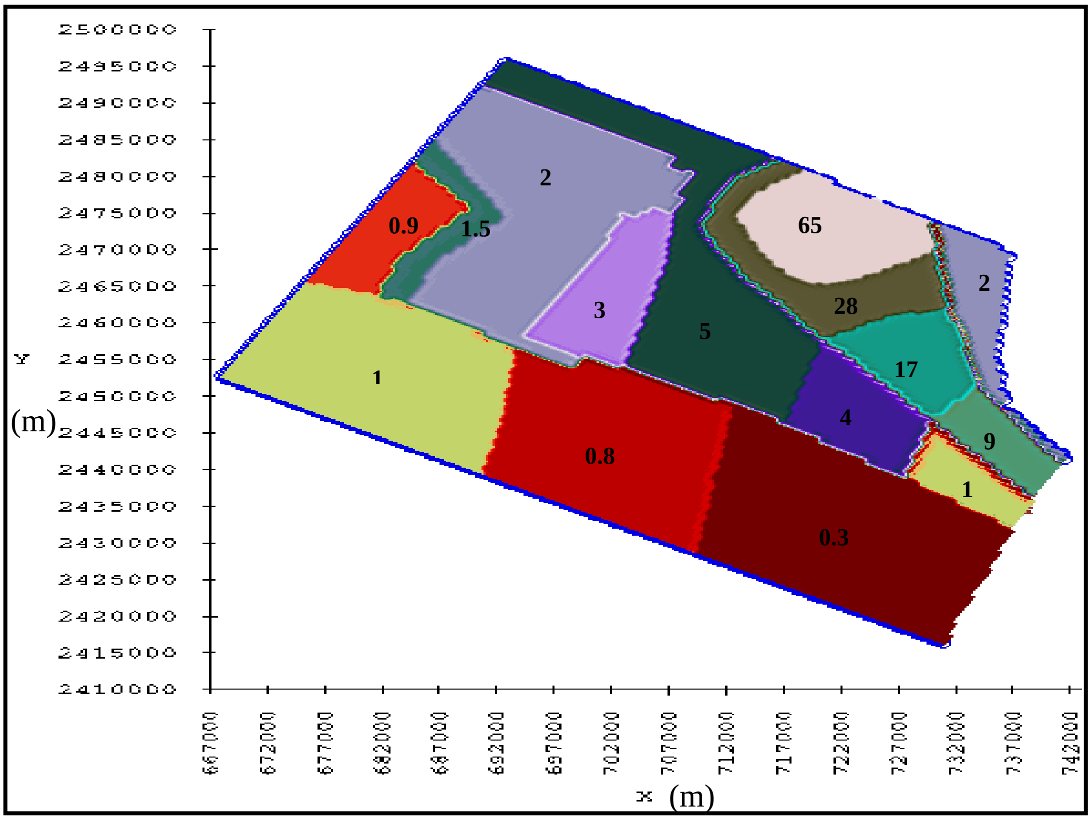


Figure 5.8: Distribution of recharge inputs for the two layers

a)



(b)

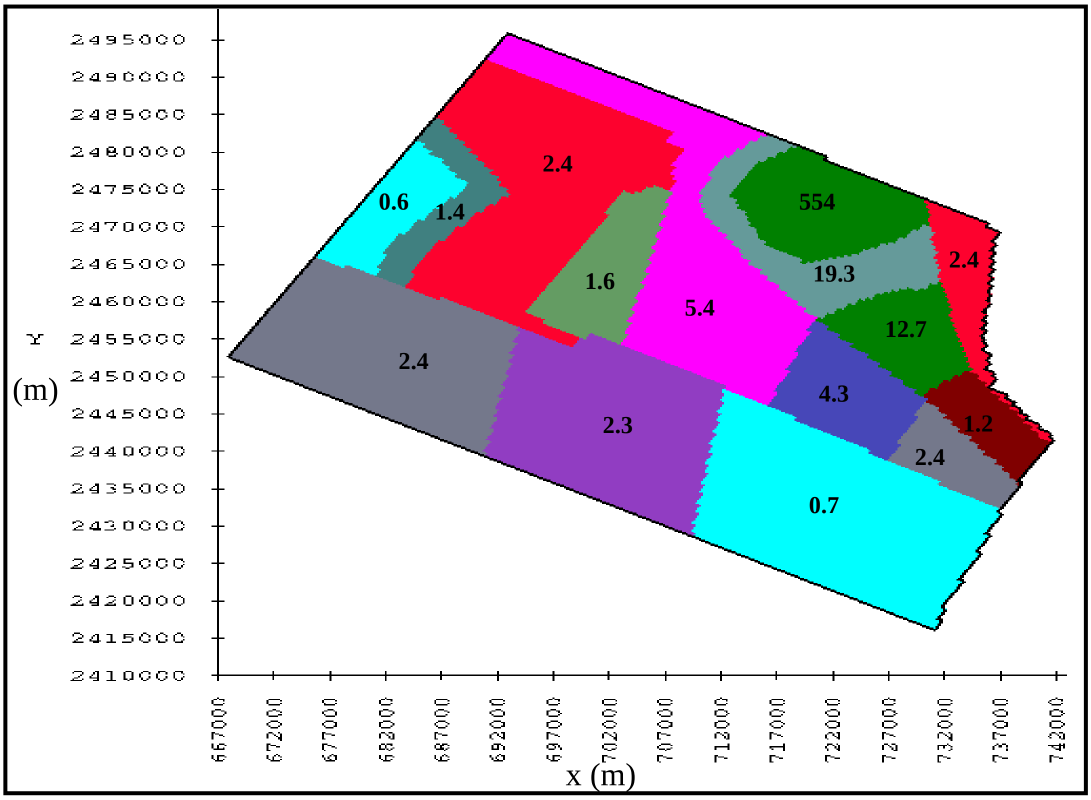


Figure 5.9: Hydraulic conductivity (m/day) distribution zones for layer-2 (a) initial calibrated values; (b) the automatic calibrated values

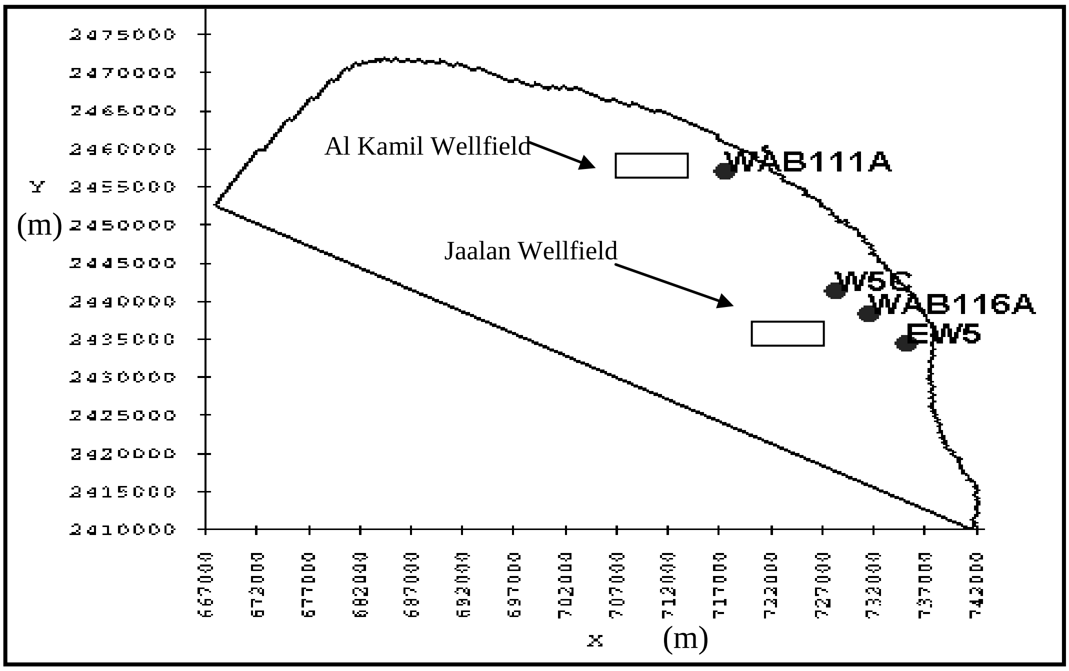


Figure 5.10: Observation wells used for transient model in layer 1

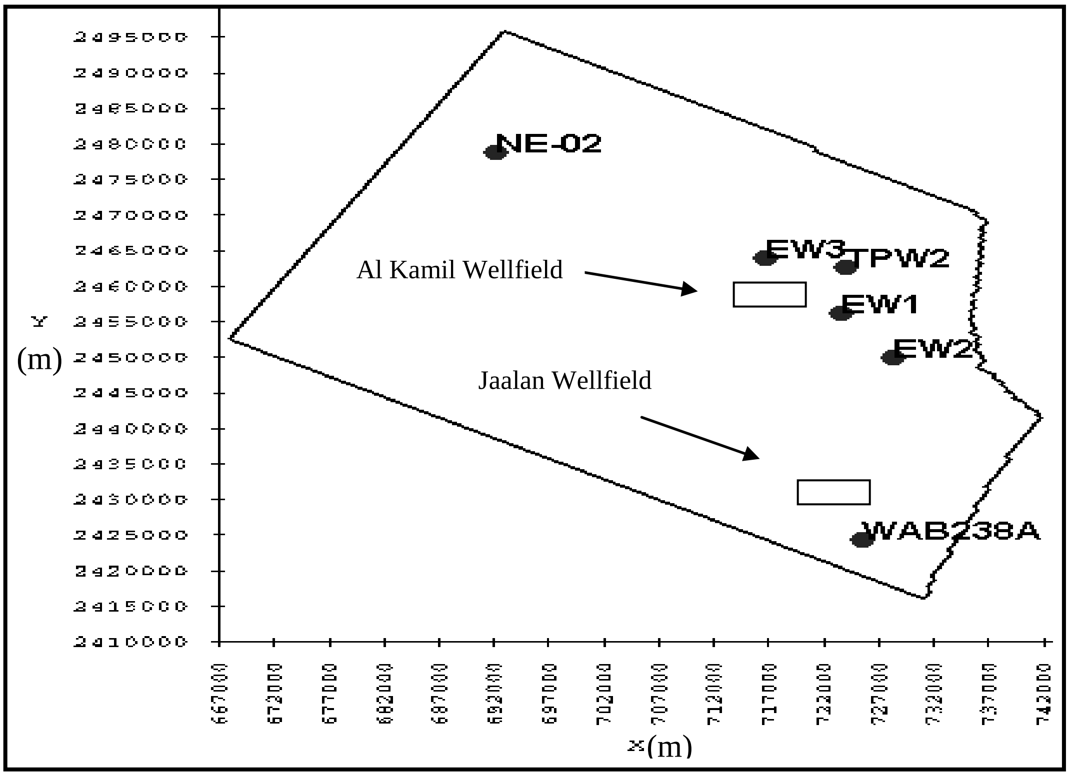


Figure 5.11: Observation wells used for transient model in layer 2

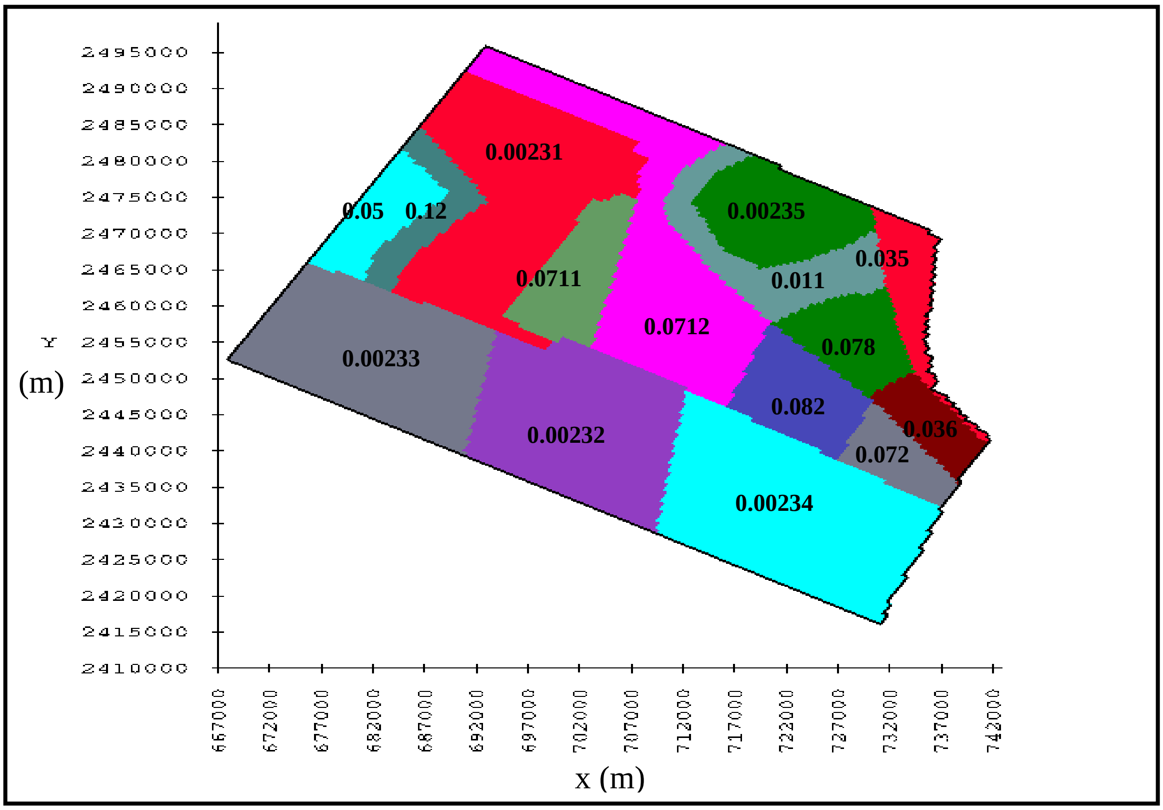


Figure 5.12: Calibrated specific yield (Sy) zones for layer 2

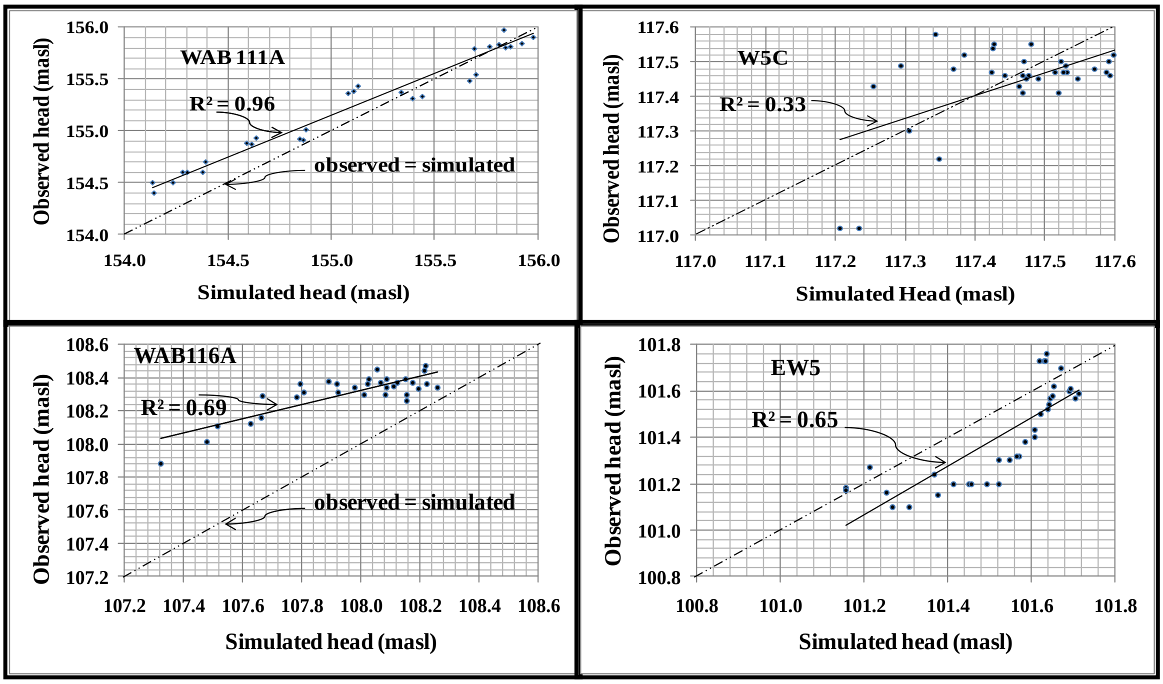


Figure 5.13: Statistical fit results for the observation wells used to validate the transient model in layer 1

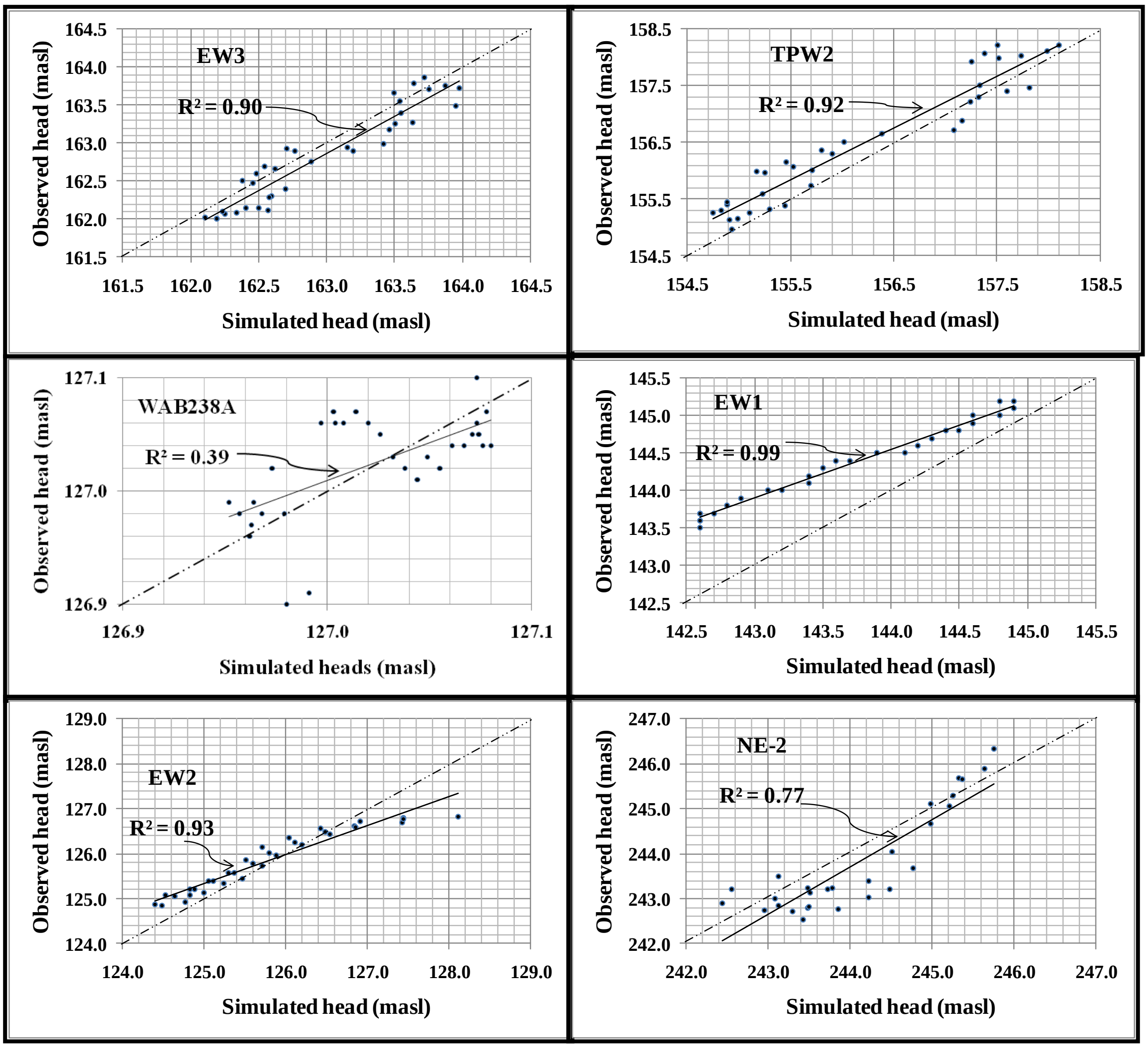


Figure 5.14: Statistical fit results for the observation wells used to validate the transient model in layer 2

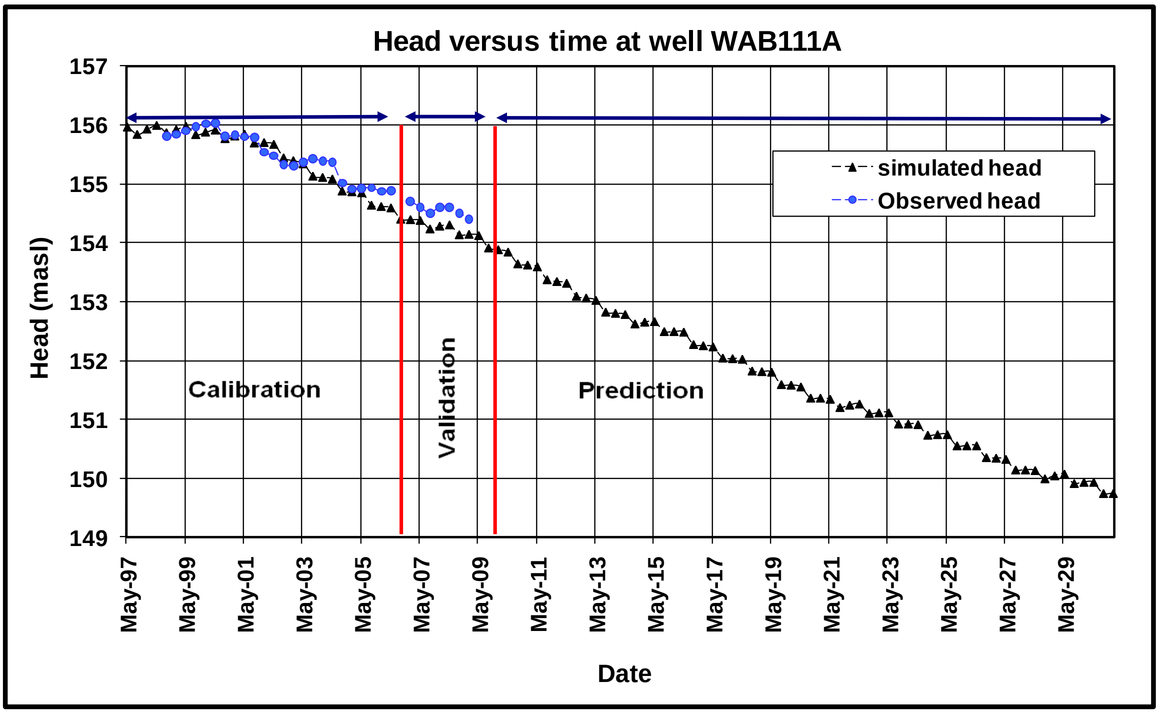


Figure 5.15: Comparison between observed and simulated heads in observation well WAB111A (layer 1)

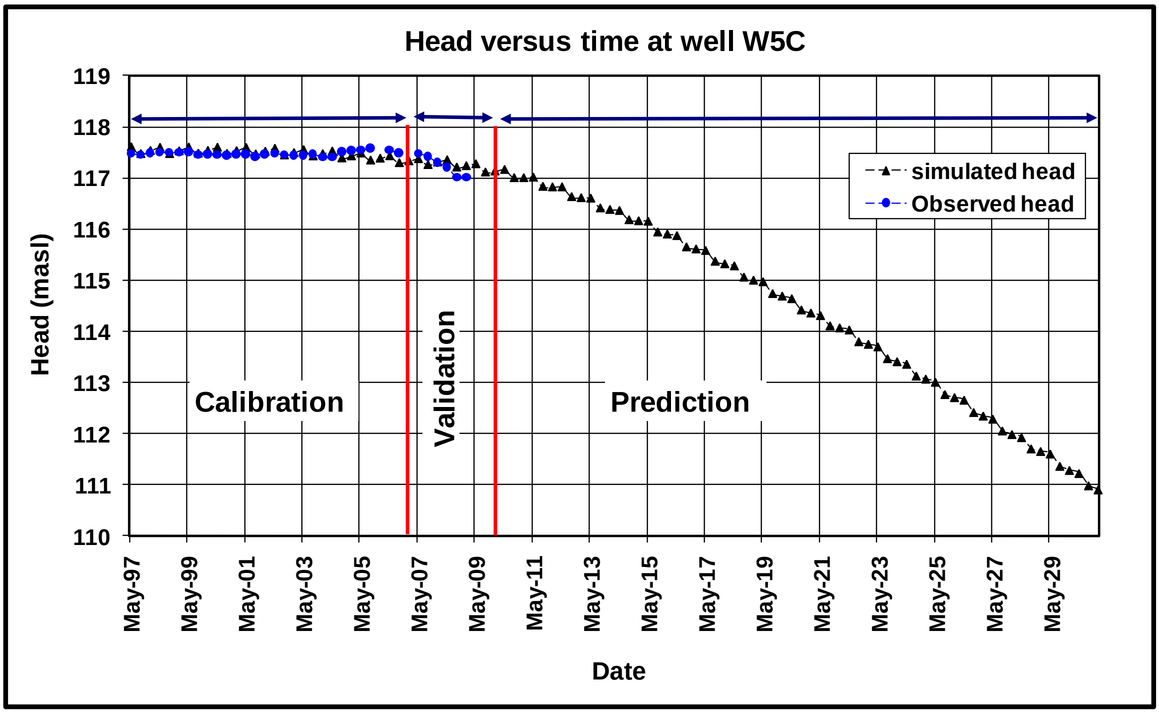


Figure 5.16: Comparison between observed and simulated heads in observation well W5C (layer 1)

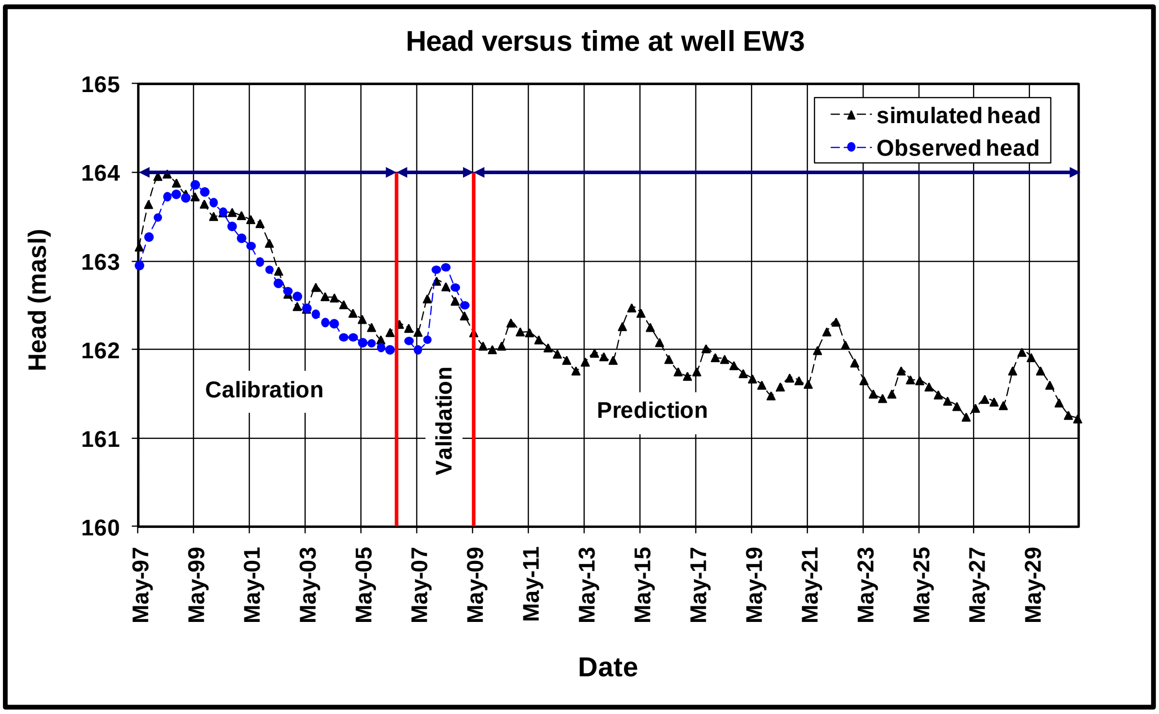


Figure 5.17: Comparison between observed and simulated heads in observation well EW3 (layer 2)

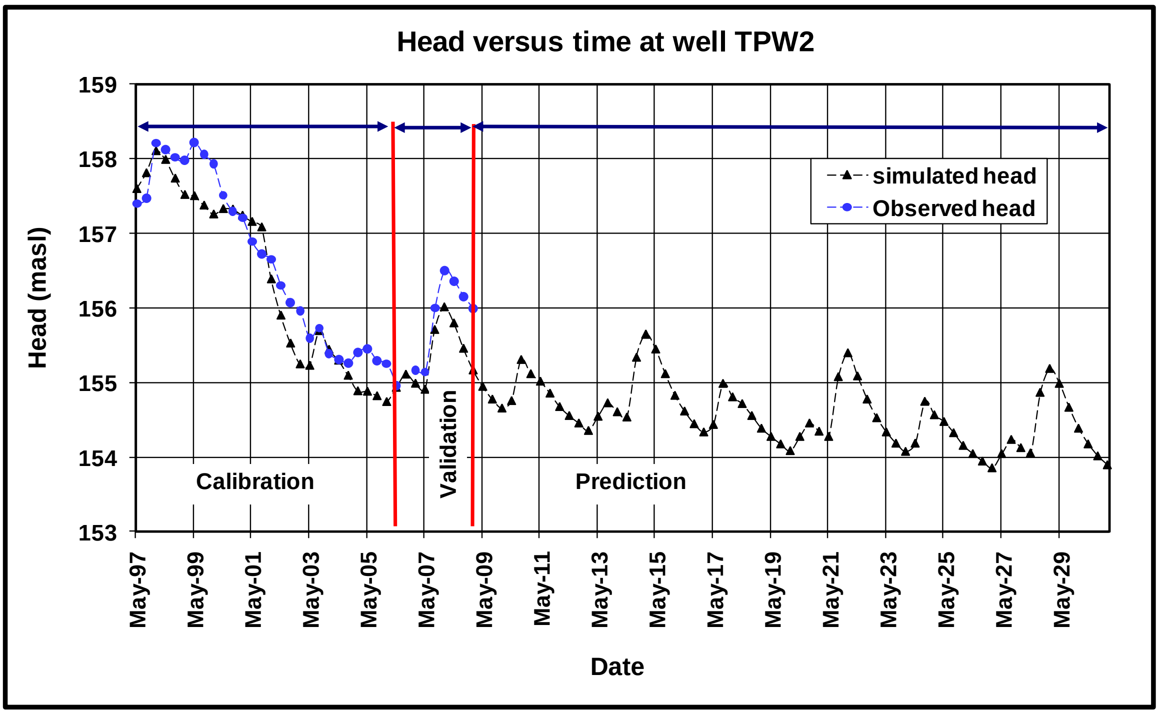


Figure 5.18: Comparison between observed and simulated heads in observation well TPW2 (layer 2)

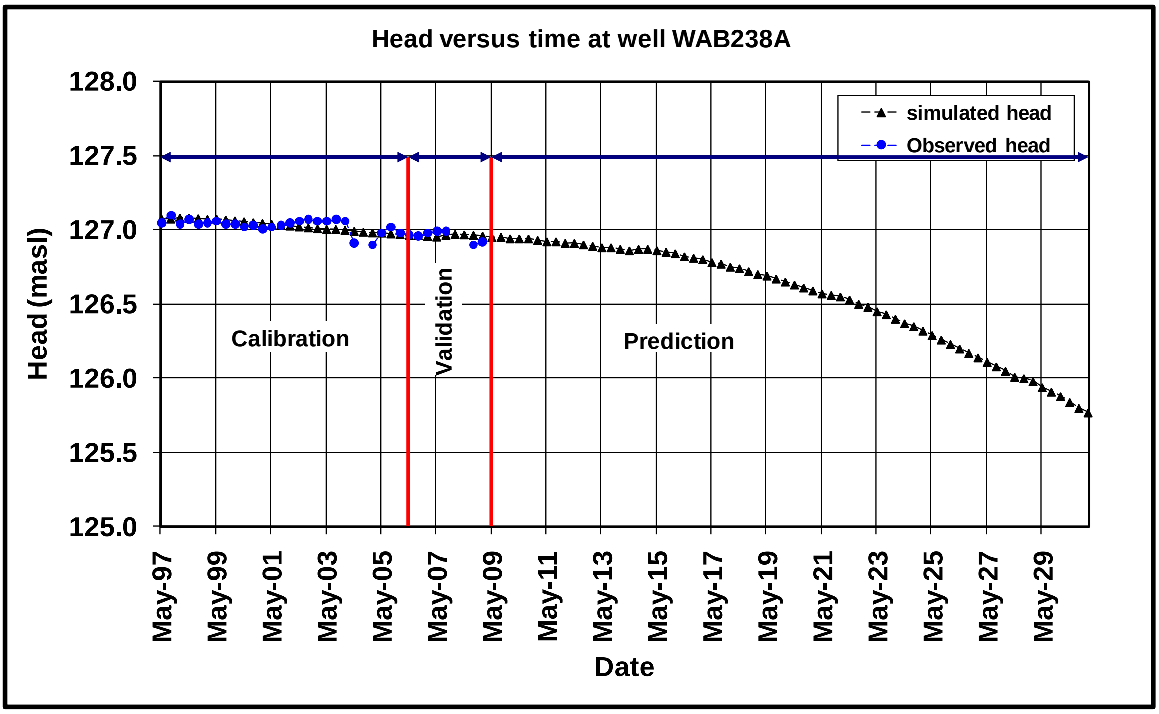


Figure 5.19: Comparison between observed and simulated heads in observation well WAB238A (layer 2)

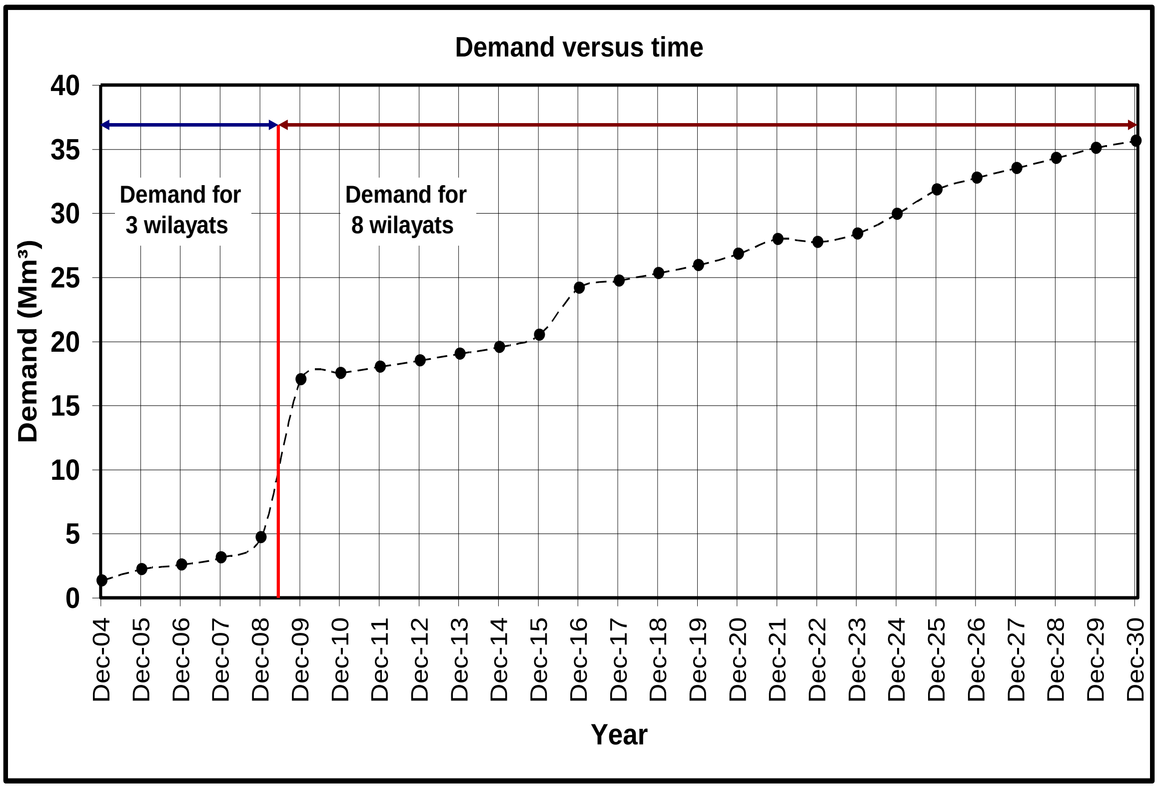


Figure 5.20: Projected domestic water demand required for the study area

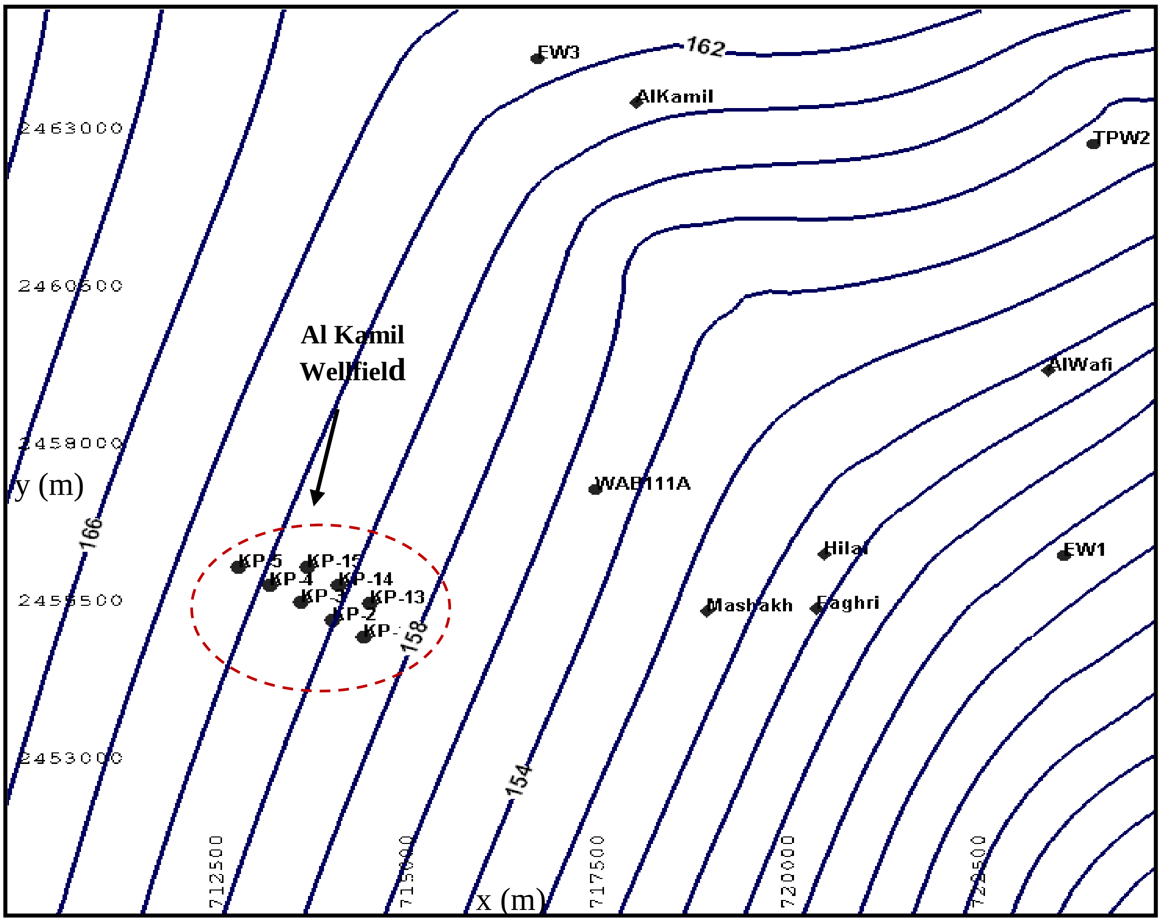


Figure 5.21: Simulated head at beginning operation of Al Kamil Wellfield in April 2004 showing close by Aflaj (♦) and some of observation wells (●)

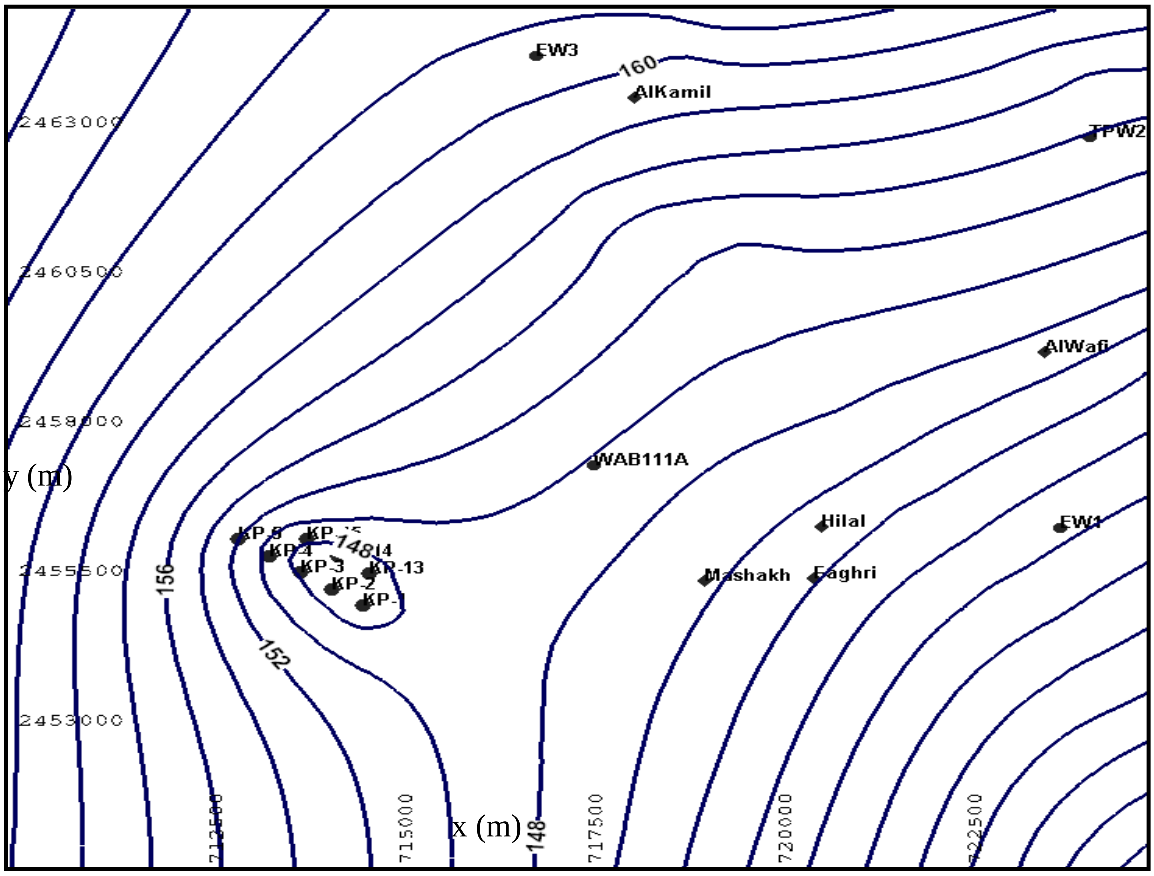


Figure 5.22: Simulated head at Al Kamil Wellfield at the end of 2030

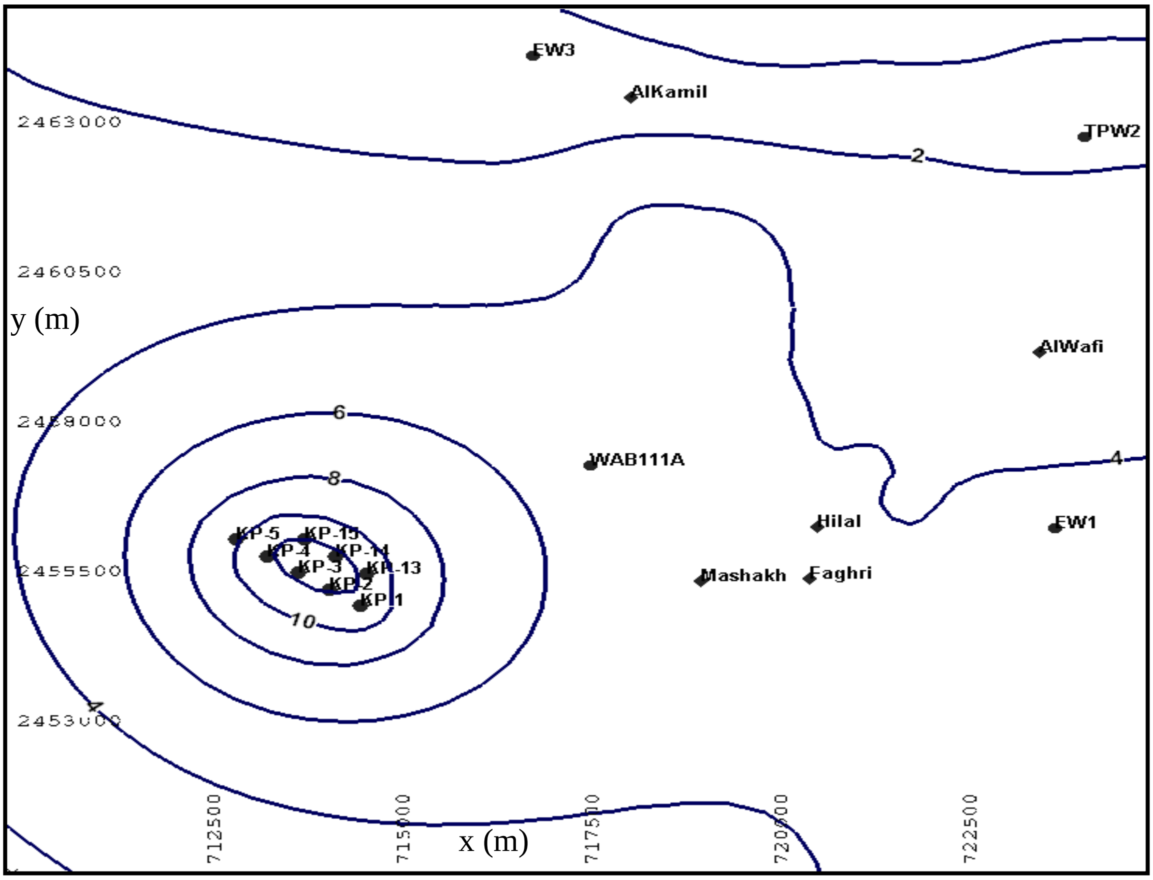


Figure 5.23: Simulated drawdown at Al Kamil Wellfield at the end of 2030

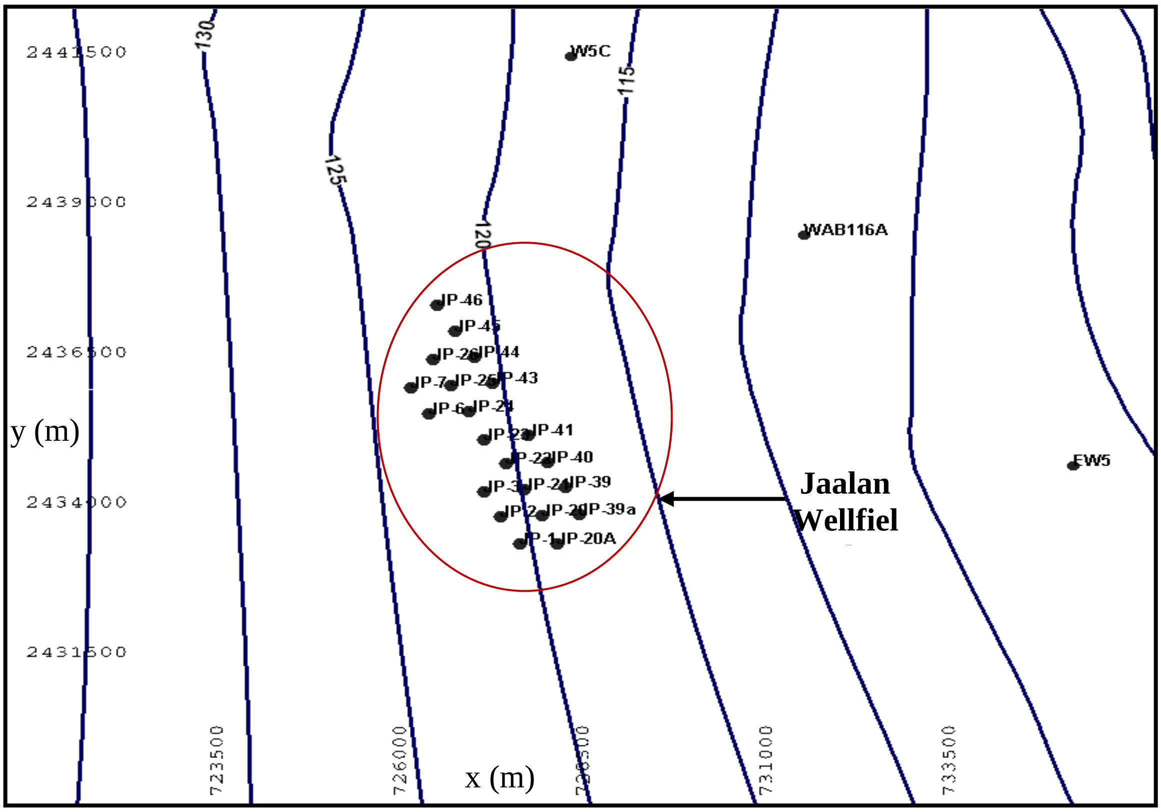


Figure 5.24: Simulated head at beginning operation of Jaalan Wellfield in April 2004 showing close by observation wells (●)

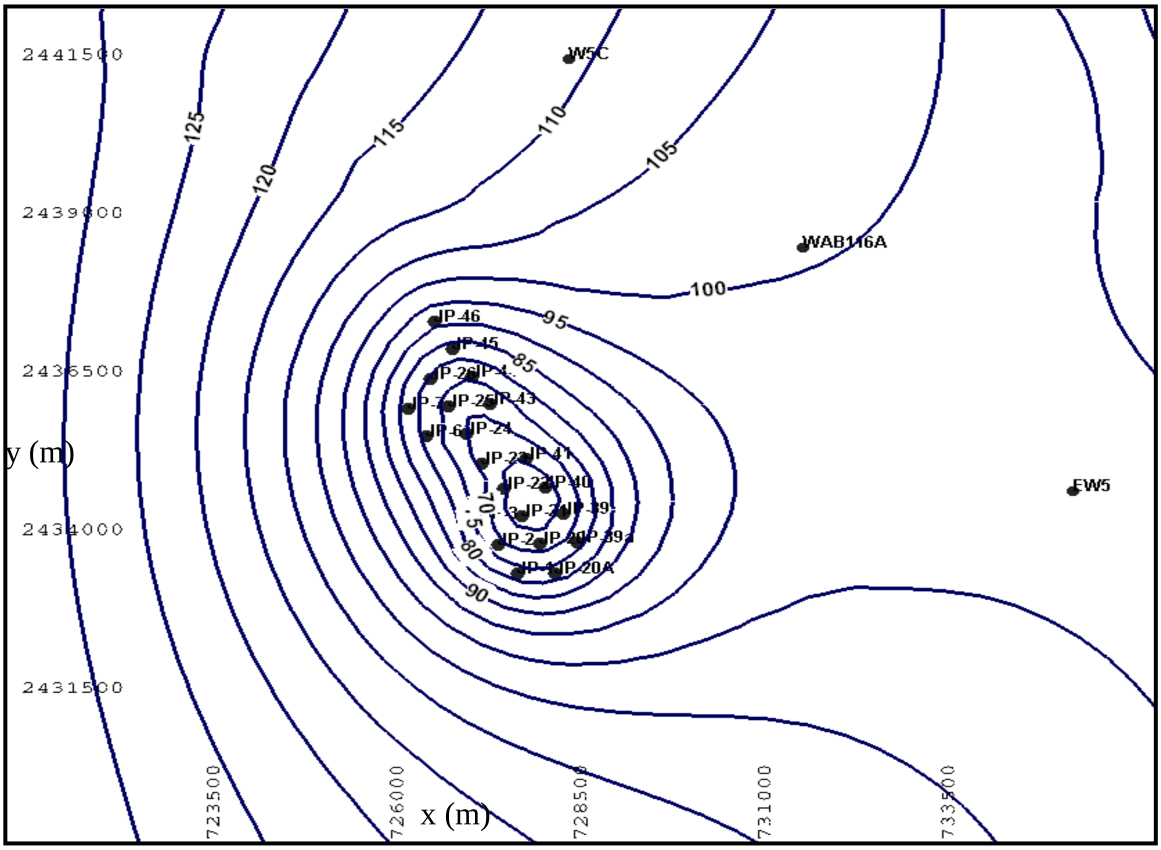


Figure 5.25: Simulated head at Jaalan Wellfield at the end of 2030

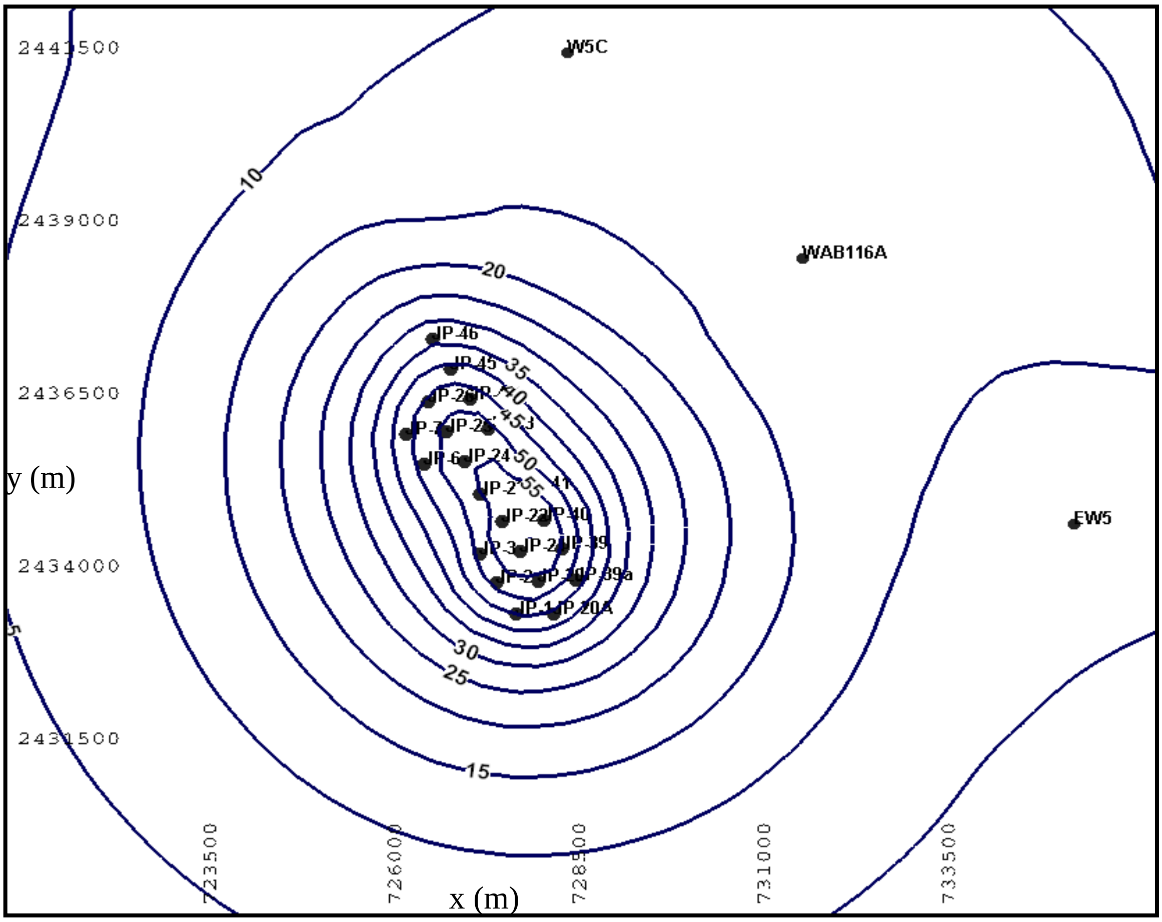


Figure 5.26: Simulated drawdown at Jaalan Wellfield at the end of 2030

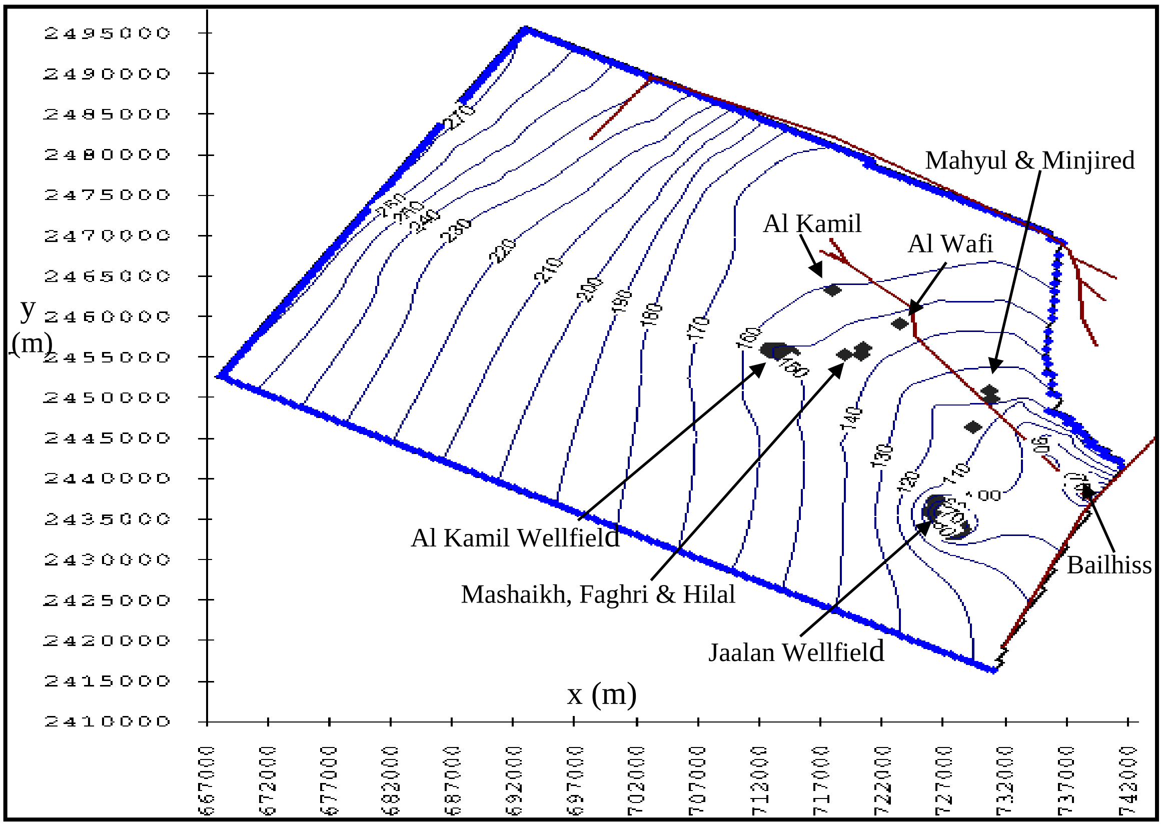


Figure 5.27: Aflaj locations (♦) and the transient simulated heads at the end of 2030

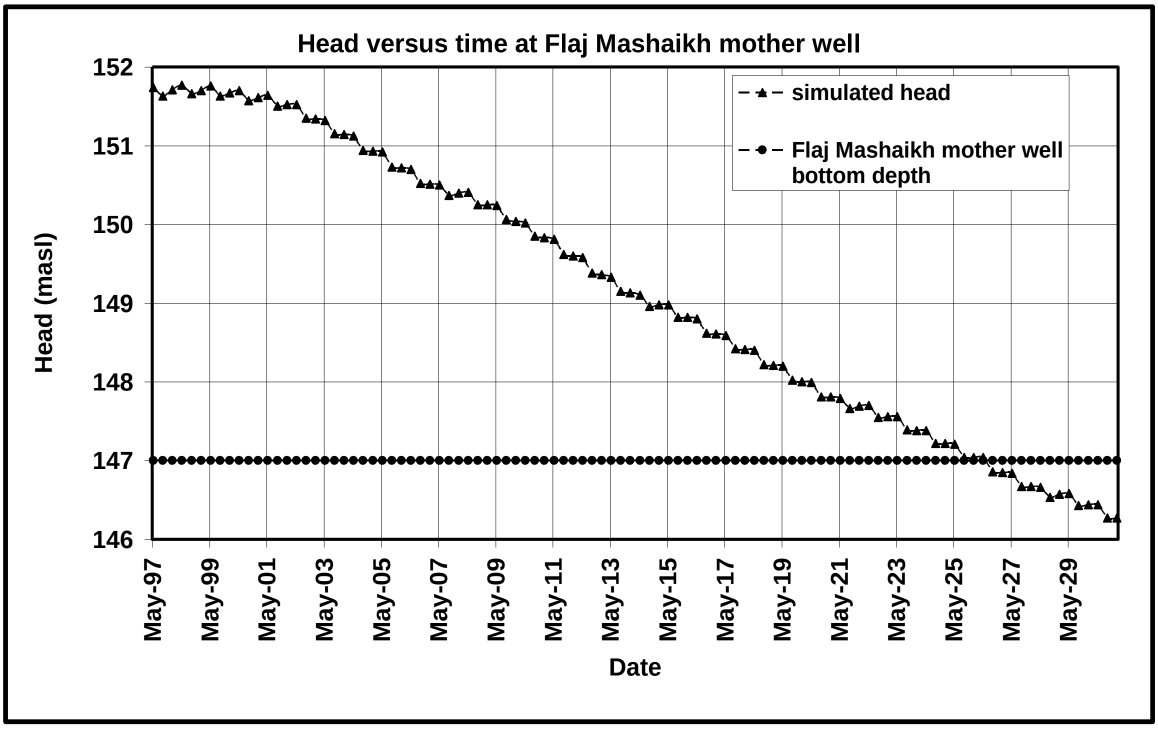


Figure 5.28: Head calculated up to the end of 2030 at Flaj Mashaikh mother well

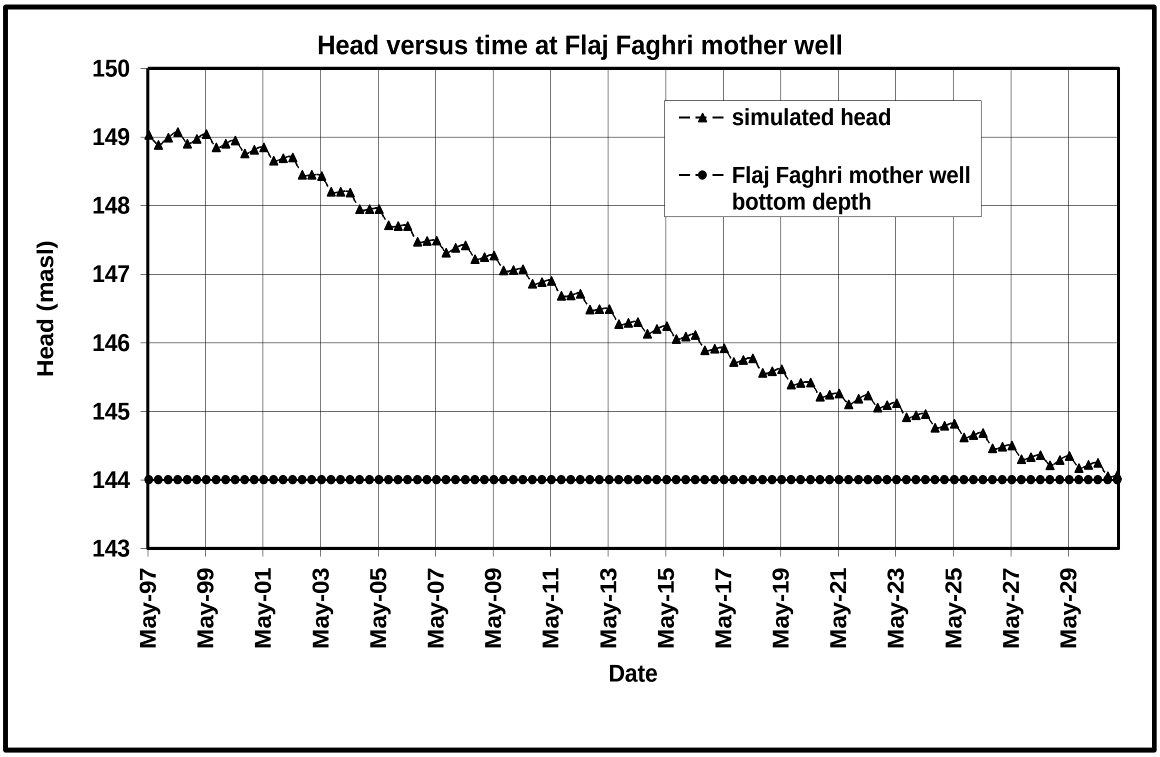


Figure 5.29: Head calculated up the end of 2030 at Flaj Faghri mother well

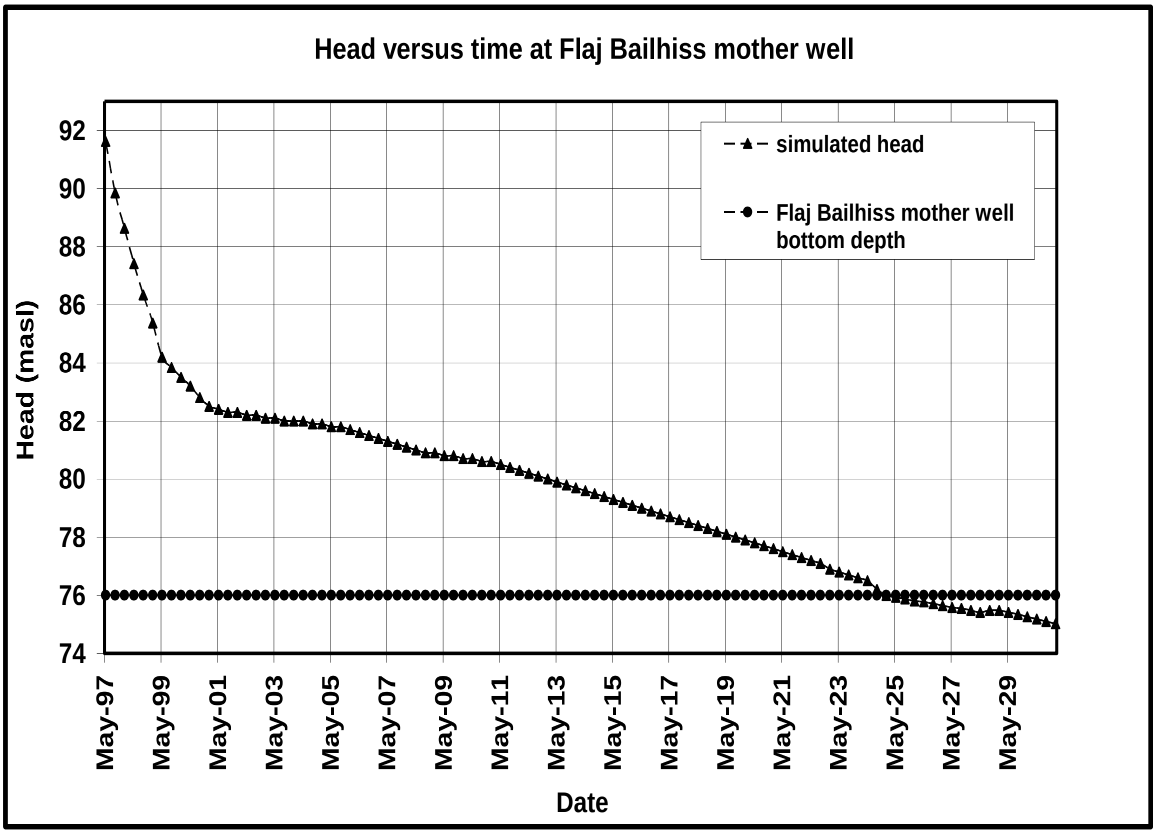


Figure 5.30: Head calculated up to the end of 2030 at Flaj Bailhiss mother well

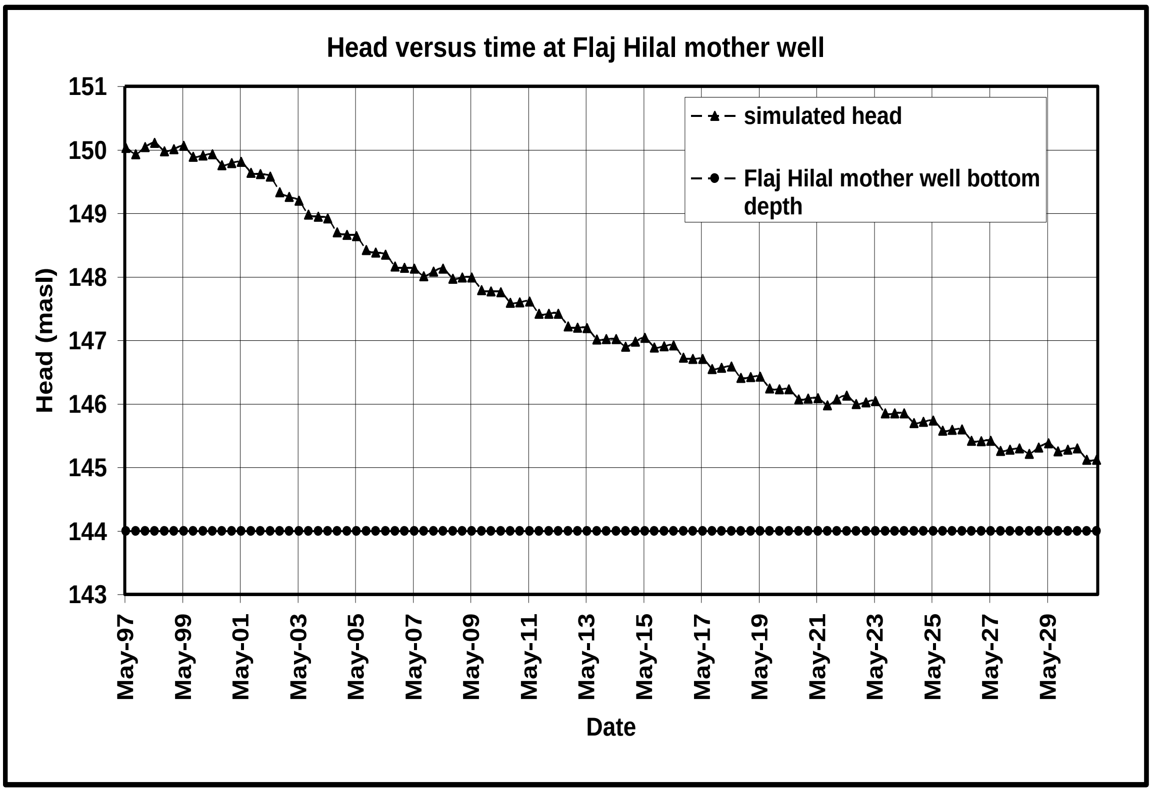


Figure 5.31: Head calculated up to the end of 2030 at Flaj Hilal mother well

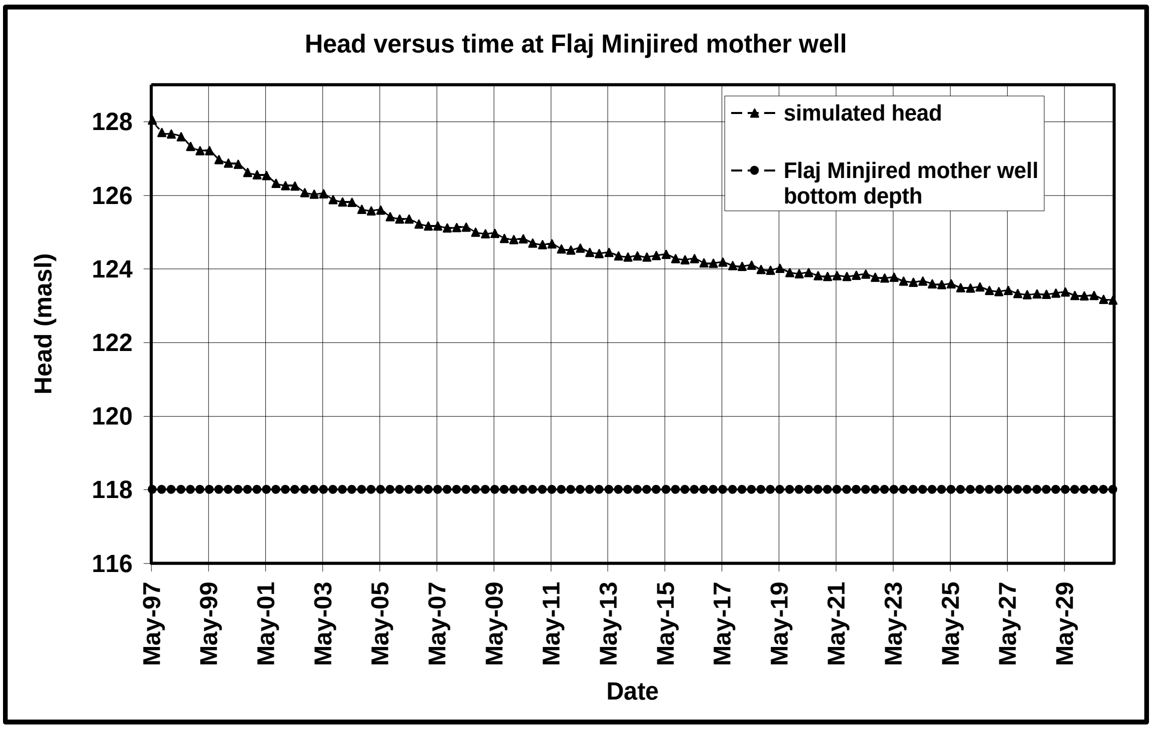


Figure 5.32: Head calculated up to the end of 2030 at Flaj Minjired mother well

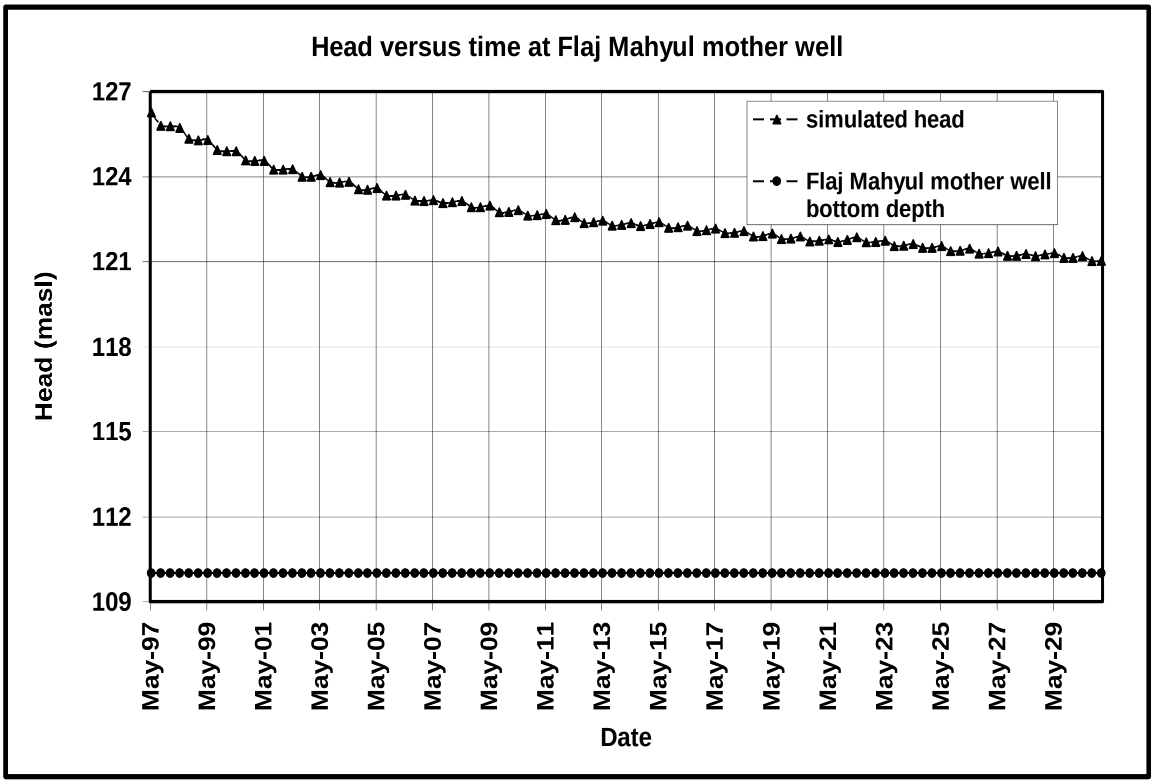


Figure 5.33: Head calculated up to the end of 2030 at Flaj Mahyul mother well

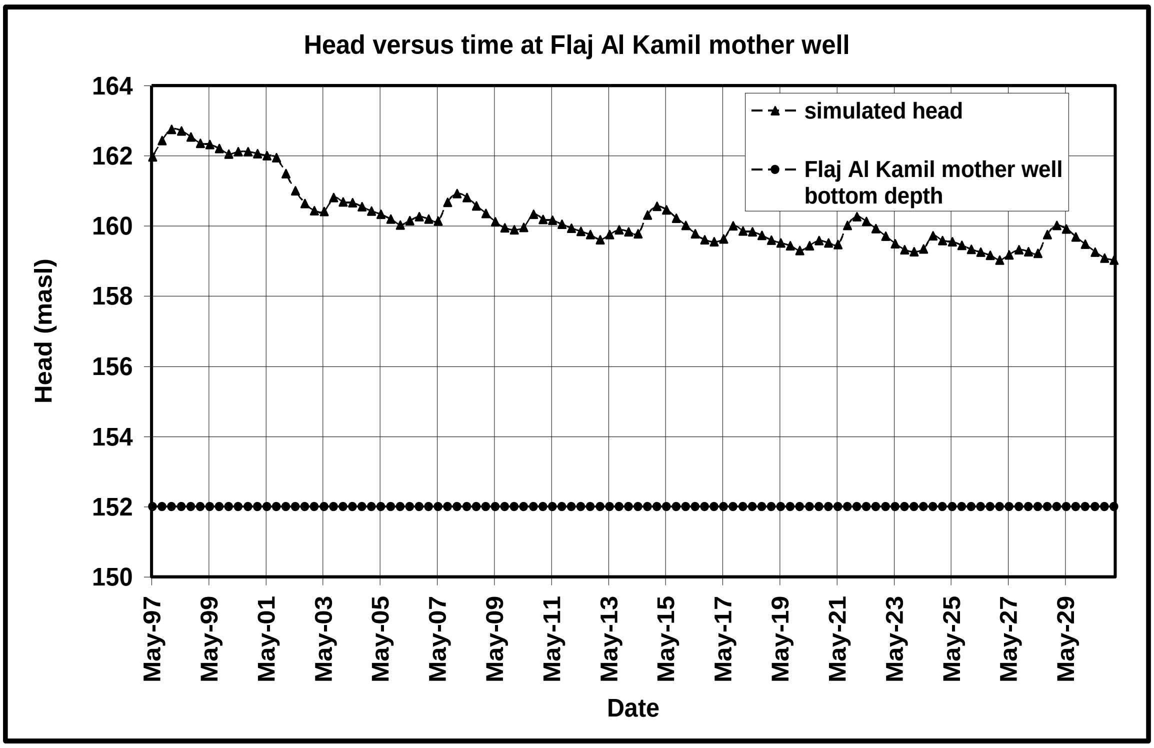


Figure 5.34: Head calculated up to the end of 2030 at Flaj Al Kamil mother well

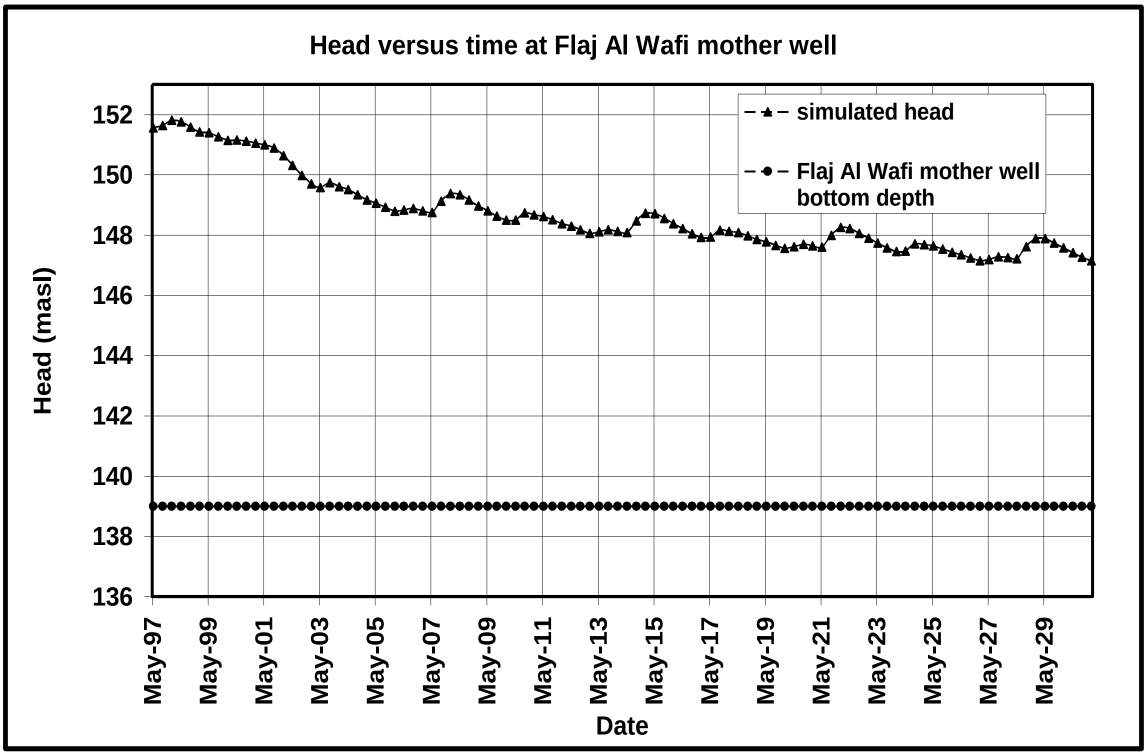


Figure 5.35: Head calculated up to the end of 2030 at Flaj Al Wafi mother well

CHAPTER 6

OPTIMIZATION MODEL

6.1 Introduction

In Chapter 5, the numerical simulation model of Ash Sharqiyah Sands Aquifer was used to assess the long-term impacts of supplying the eight Wilayats of Ash Sharqiyah Region with water from the 29 operational wells of the two groundwater wellfields by predicting the long-term behaviour until 2030 of the piezometric heads. The simulation results showed that the existing 29 operational wells of the two groundwater wellfields will be inadequate by the 1st of September 2025 to meet the domestic water supply needs for the eight Wilayats of Ash Sharqiyah Region without creating extensive drawdown and negative impacts on existing operational Aflaj and the environment. It is therefore clear that this is not a sustainable option for meeting the long-term water demands in the targeted Wilayats. Supplementing the abstraction from the well fields with desalinated water of the Sur Desalination Plant offers the prospect for combating the problem; however, given the relatively high cost of desalination in comparison with the cost of treating fresh groundwater, the blending strategy to be adopted must be such that the aggregated cost is minimal. This is thus a constrained optimization problem which will attempt to find the least cost combination of groundwater and desalinated water while satisfying environmental constraints imposed by the need to keep the Aflaj continuously flowing.

Chapter 2 presented a review of the use of optimization techniques in groundwater management. Most of the optimization formulations have been constrainted because, like the problem being addressed here, most or all of the decision variables can only take on prescribed values. It was also emphasized in Chapter 2 that where the objective function and decision variables are linear, solution can be readily obtained by linear programming; otherwise a non-linear optimization approach has to be used, unless they could be piecewise linearised.

A further feature of optimization in groundwater systems analysis is the coupling of a simulation model of the system with the optimization. This is necessary because although the optimization model will contain flow and heads as decision variables in the objective function and constraint equations, the relationship between these two is described by the simulation model. Thus, unless the simulation model is solved at the same time as the evaluation of the optimization, no solution of the optimization is possible. As noted in Chapter 2, this coupling can be achieved either by using the response matrix or the fully embedded method. The General Algebraic Modelling System (GAMS) software used for the coupling in this study can accommodate both options as described in the following section.

6.2 Formulating the management model of Ash Sharqiyah

One of the most essential stages in the development of management models is the formulation of the optimization problem and the selection of the most appropriate management goals through the determination of the mathematical expression of objective function and constraints. Therefore, a successful construction of the formulation of the problem requires both an understanding of the physical interpretation of the objective and the constraint, and the ability to anticipate the mathematical impact of the objective and the constraints on the form of the solution (Ahlfeld and Mulligan 2000). Figure 6.1 is a schematic diagram of different pumping locations of the water supply system for Ash Sharqiyah Region using desalinated water and groundwater.

6.2.1 Objective function

The objective function can be considered as to minimize the total cost of water production, which can be written as follows:

$$Min\ (\sum_{T=1}^{N_T} \Delta t((\ C_{Sd}\ \ Q_{Sd}\ (T) +\ C_{Sw}Q_{Sw}(T) + C_{SrKr}Q_{SrKr}(T) + C_{KrSr}Q_{KrSr}(T) + C_{Nw}Q_{Nw}(T) + C_{KrKfr}Q_{KrKfr}(T) + C_{KfrKr}Q_{Kfr\ Kr}(T) + C_{KrJfr}Q_{KrJfr}(T) + C_{JfrKr}Q_{JfrKr}(T) + C_{Kg}Q_{Kg}(T) + C_{Kw}Q_{Kw}(T) + C_{Jg}Q_{Jg}(T) + C_{Jw}Q_{Jw}(T))) \quad (6.1)$$

Where,

N_T : Number of time steps (here it is 63 no.).

Δt : Duration of time step (120 days).

C_{Sd} / $Q_{Sd}\ (T)$: Cost per unit pumping rate (RO/ m³) / pumping rate (m³/day) during period (T) from Sur Desalination Plant to Sur Reservoir respectively.

C_{Sw} / $Q_{Sw}(T)$: Cost per unit pumping rate (RO/ m³) / pumping rate (m³/day) during period (T) from Sur Reservoir to supply Sur Wilayat respectively.

C_{SrKr} / $Q_{SrKr}(T)$: Cost per unit pumping rate (RO/ m³) / pumping rate (m³/day) during period (T) from Sur Reservoir to Al Kamil Reservoir respectively.

C_{KrSr} / $Q_{KrSr}(T)$: Cost per unit pumping rate (RO/ m³) / pumping rate (m³/day) during period (T) from Al Kamil Reservoir to Sur Reservoir respectively.

C_{Nw} / $Q_{Nw}(T)$: Cost per unit pumping rate (RO/ m³) / pumping rate (m³/day) during period (T) from Al Kamil Reservoir to North Wilayats respectively.

C_{KrKwr} / Q_{KrKwr} : Cost per unit pumping rate (RO/ m³) / pumping rate (m³/day) during period (T) from Al Kamil Reservoir to Al Kamil Wellfield Reservoir.

C_{KwrKr} / $Q_{Kwr\,Kr}(T)$: Cost per unit pumping rate (RO/ m^3) / pumping rate (m^3/day) during period (T) from Al Kamil Wellfield Reservoir to Al Kamil Reservoir.

C_{KrJwr} / $Q_{KrJwr}(T)$: Cost per unit pumping rate (RO/ m^3) / pumping rate (m^3/day) during period (T) from Al Kamil Reservoir to Jaalan Wellfield Reservoir.

C_{JwrKr} / $Q_{JwrKr}(T)$: Cost per unit pumping rate (RO/ m^3) / pumping rate (m^3/day) during period (T) from Jaalan Wellfield Reservoir to Al Kamil Reservoir.

C_{Kg} / $Q_{Kg}(T)$: Cost per unit pumping rate (RO/ m^3) / pumping rate (m^3/day) during period (T) from Al Kamil Wellfield to Al Kamil Wellfield Reservoir.

C_{Kw} / $Q_{Kw}(T)$: Cost per unit pumping rate (RO/ m^3) / pumping rate (m^3/day) during period (T) from Al Kamil Wellfield Reservoir to supply Al Kamil Wilayat respectively.

C_{Jg} / $Q_{Jg}(T)$: Cost per unit pumping rate (RO/ m^3) / pumping rate (m^3/day) during period (T) from Jaalan Wellfield to Jaalan Wellfield Reservoir.

C_{Jw} / $Q_{Jw}(T)$: Cost per unit pumping rate (RO/ m^3) / pumping rate (m^3/day) during period (T) from Jaalan Wellfield Reservoir to supply Jaalan Wilayat.

Equation (6.1) is a constrained optimization problem and the constraints are presented in the following sections.

6.2.2 Constraint equations

6.2.2.1 Water balance at each of pumping location

Water balance is needed at each pumping location of the system and can be described by the following equations:

Water balance at Sur Reservoir

$$S_{Sr}(T+1) = S_{Sr}(T) + Q_{Sd}(T)\Delta t + Q_{KrSr}(T)\Delta t - Q_{Sw}(T)\Delta t - Q_{SrKr}(T)\Delta t \quad (6.2)$$

Where,

$$0 \leq S_{Sr}(T+1) \leq K_{Sr} \quad (6.3)$$

$S_{Sr}(T)$ is the storage (m^3)in Sur Reservoir at beginning of period (T)

$S_{Sr}(T+1)$ is the storage (m^3) in Sur Reservoir at end of period (T)

K_{Sr} is the capacity (m^3) of Sur Reservoir (See Table 6.1) and all the other variables are as defined under equation (6.1).

Water balance at Al Kamil Reservoir

$$S_{Kr}(T+1) = S_{Kr}(T) + Q_{SrKr}(T)\Delta t + Q_{KwrKr}(T)\Delta t + Q_{JwrKr}(T)\Delta t - Q_{KrSr}(T)\Delta t - Q_{KrKwr}(T)\Delta t - Q_{KrJwr}(T)\Delta t - Q_{Nw}(T)\Delta t \quad (6.4)$$

Where,

$$0 \leq S_{kr}(T+1) \leq K_{kr} \quad (6.5)$$

$S_{kr}(T)$ is the storage (m^3) in Al Kamil Reservoir at beginning of period (T)

$S_{kr}(T+1)$ is the storage (m^3) in Al Kamil Reservoir at end of period (T)

K_{kr} is the capacity (m^3) of Al Kamil Reservoir (See Table 6.1) and all the other variables are as defined under equation (6.1).

Water balance at Al Kamil Wellfield Reservoir

$$S_{Kwr}(T+1) = S_{Kwr}(T) + Q_{KrKwr}(T)\Delta t + Q_{Kg}(T)\Delta t - Q_{KwrKr}(T)\Delta t - Q_{Kw}(T)\Delta t \quad (6.6)$$

Where,

$$0 \leq S_{kwr}(T+1) \leq K_{kwr} \tag{6.7}$$

$S_{kwr}(T)$ is the storage (m^3) in Al Kamil Wellfield Reservoir at beginning of period (T)

$S_{kwr}(T+1)$ is the storage (m^3) in Al Kamil Wellfield Reservoir at end of period (T)

K_{kwr} is the capacity (m^3) of Al Kamil Wellfield Reservoir (See Table 6.1) and all the other variables are as defined under equation (6.1).

Water balance at Jaalan Wellfield Reservoir

$$S_{Jwr}(T+1) = S_{Jwr}(T) + Q_{KrJwr}(T)\Delta t + Q_{Jg}(T)\Delta t - Q_{JwrKr}(T)\Delta t - Q_{Jw}(T)\Delta t \tag{6.8}$$

Where,

$$0 \leq S_{jwr}(T+1) \leq K_{jwr} \tag{6.9}$$

$S_{jwr}(T)$ is the storage (m^3) in Jaalan Wellfield Reservoir at beginning of period (T)

$S_{jwr}(T+1)$ is the storage (m^3) in Jaalan Wellfield Reservoir at end of period (T)

K_{jwr} is the capacity (m^3) of Jaalan Wellfield Reservoir (See Table 6.1) and all the other variables are as defined under equation (6.1).

6.2.2.2 Pumping rates at the two wellfields

Pumping rates from Al Kamil Wellfield

$$Q_{Kg}(T) = \sum_{p=1}^{8} q_{Kg,p}(T) \tag{6.10}$$

Where,

$Q_{Kg}(T)$: Total pumping rates (m³/day) during stress period (T) from the eight Al Kamil production wells.

$q_{Kg,p}(T)$: Pumping rate (m³/day) during stress period (T) from production well p at the Al Kamil Wellfield.

Pumping rates from Jaalan Wellfield

$$Q_{Jg}(T) = \sum_{p=1}^{21} q_{Jg,p}(T) \tag{6.11}$$

Where,

$Q_{Jg}(T)$: Total pumping rates (m³/day) during stress period (T) from the twenty one Jaalan production wells.

$q_{Jg,p}(T)$: Pumping rate (m³/day) during stress period (T) from production well p at the Jaalan Wellfield.

6.2.2.3 Head constraints

Constraints on heads can be used to implement many conditions or preference such as:

1- Constant head cells that show to have equal lower and upper bounds;

2- Bounds that reflect the natural characteristics of the system, e.g. in a saturated flow system, the head should not drop below the base of the aquifer or in an unconfined layer the head is never higher than the top of that layer.

3- Bounds on head to limit the drawdowns to an acceptable limit, which in this case might be needed to keep the Aflaj constantly flowing.

Head constraints can be described by using the general form:

$$H^{l}{}_{i,j,NL}(T) \leq H_{i,j,NL}(T) \leq H^{u}{}_{i,j,NL}(T) \tag{6.12}$$

Where,

$H_{i,j,NL}(T)$: Computed head at cell (i,j) of layer number *NL* (1 or 2) during the stress period (T)

$H^{l}_{i,j,NL}(T)$: Lower bound on computed head at cell (i,j) of layer number *NL* (1 or 2) during the stress period (T).

$H^{u}_{i,j,NL}(T)$: Upper bound on computed head at cell (i,j) of layer number *NL* (1 or 2) during the stress period (T).

Head constraints on pumping from the two wellfields

This head constraint can be applied at a production well location during a stress period to avoid having the simulated head below a pump depth as follows:

$$H_{i,j,NL}(T) \geq Z_{i,j,NL,p} \tag{6.13}$$

Where,

$H_{i,j,NL}(T)$: Computed head at specified production well cell (i, j) of layer one or two during stress period (T).

$Z_{i,j,NL,p}$: Specified pump depth of a production well p at cell (i, j) of layer one or two.

The $Z_{i,j,NL,p}$ value for each production well is fixed at one meter above the pump depth in each well to avoid pump deterioration. This value remains unchanged during different stress periods.

Head constraints at Aflaj mother wells

$$H_{i,j,NL}(T) \geq Z_{i,j,NL,f} \tag{6.14}$$

Where,

$H_{i,j,NL}(T)$: Computed head at specified Flaj mother well cell (i, j) of layer one or two during stress period (T).

$Z_{i,j,NL,f}$: Specified depth of Flaj mother well f at cell (i, j) of layer one or two.

The $Z_{i,j,NL,f}$ value for each Flaj mother well is fixed as head constraint at half meter above the base of the Flaj mother well for each Flaj to avoid drying the Flaj. This value remains unchanged during different stress periods. An average constant flow for each different Flaj was used at each stress period although in reality it might decreases slightly with pumping time.

6.2.2.4 Pumping rates constraints

These constraints are applied to control the pumping rate at each production well to enforce natural conditions, legal rights or management goals. An upper bound is normally set to limit the maximum pumping such as the pump capacity, and prevent the model from computing impractical values. Lower bound is set to guarantee a minimum pumping and prevent the model from assigning no pumping to cells where pumping is already occurring. This constraint can be expressed by the following equations:

For Al Kamil Wellfield production wells

$$q_{Kg}{}^{l}{}_{i,j,NL,p}(T) \leq q_{Kg_{i,j,NL,p}}(T) \leq q_{Kg}{}^{u}{}_{i,j,NL,p}(T) \tag{6.15}$$

Where,

$q_{Kg_{i,j,NL,p}}(T)$: Computed pumping rate (m³/day) at production well p in cell (i, j) of layer one or two during stress period (T) at the Al Kamil Wellfield.

$Q_{Kg}{}^{u}{}_{i,j,NL,p}(T)$: The maximum pumping rate (m³/day) that can be achieved at production well p in cell (i, j) of layer one or two during stress period (T) at the Al Kamil Wellfield.

$Q_{Kg}{}^{l}{}_{i,j,NL,p}(T)$: The minimum pumping rate (m³/day) that can be achieved at production well p in cell (i, j) of layer one or two during stress period (T) at the Al Kamil Wellfield.

For Jaalan Wellfield production wells

$$q_{Jg}{}^{l}{}_{i,j,NL,p}(T) \le q_{Jg_{i,j,NL,p}}(T) \le q_{Jg}{}^{u}{}_{i,j,NL,p}(T) \tag{6.16}$$

Where,

$q_{Jg_{i,j,NL,p}}(T)$: Computed pumping rate (m³/day) at production well p in cell (i, j) of layer one or two during stress period (T) at the Jaalan Wellfield.

$q_{Jg}{}^{u}{}_{i,j,NL,p}(T)$: The maximum pumping rate (m³/day) that can be achieved at production well p in cell (i, j) of layer one or two during stress period (T) at the Jaalan Wellfield.

$q_{Jg}{}^{l}{}_{i,j,NL,p}(T)$: The minimum pumping rate (m³/day) that can be achieved at production well p in cell (i, j) of layer one or two during stress period (T) at the Jaalan Wellfield.

6.2.2.5 Pumping direction constraints

The existing system is designed to allow only pumping in one direction from one reservoir to another, as there is only one pipe line connecting between two reservoirs. Therefore, pumping direction constraint should be introduced in formulating the optimization problem. The following equations represent nonlinear constraints on pumping direction between different reservoirs:

Constraint on pumping direction between Sur Reservoir and Al Kamil Reservoir

$$Q_{SrKr}(T) \times Q_{KrSr}(T) = 0 \qquad (6.17)$$

Constraint on pumping direction between Al Kamil Reservoir and Al Kamil Wellfield Reservoir

$$Q_{KrKfr}(T) \times Q_{KfrKr}(T) = 0 \qquad (6.18)$$

Constraint on pumping direction between Al Kamil Reservoir and Jaalan Wellfield Reservoir

$$Q_{KrJfr}(T) \times Q_{JfrKr}(T) = 0 \qquad (6.19)$$

All of the above variables are as defined under equation (6.1).

6.2.2.6 Water demand constraints

The water demand to each Wilayat should be considered in formulating the optimization problem in order to allow enough water supplies to meet the water demand to each Wilayat. The following equations represent water demand constraints for different Wilayat as such:

Sur Wilayat water demand constraint

$$Q_{Sw}(T)\Delta t = D_S(T)\Delta t \tag{6.20}$$

Where,

$Q_{Sw}(T)$: Pumping rate (m^3/day) during stress period (T) from Sur Reservoir to supply Sur Wilayat.

$D_S(T)$: Water demand (m^3/day) for Sur Wilayat during stress period (T).

North Wilayats water demand constraint

$$Q_{Nw}(T)\Delta t = D_{Nw}(T)\Delta t \tag{6.21}$$

Where,

$Q_{Nw}(T)$: Pumping rate (m^3/day) during stress period (T) from Al Kamil Reservoir to supply north Wilayats.

$D_{Nw}(T)$: Water demand (m^3/day) for north wilayats during stress period (T).

Al Kamil Wilayat water demand constraint

$$Q_{Kw}(T)\Delta t = D_{Kw}(T)\Delta t \tag{6.22}$$

Where,

$Q_{Kw}(T)$: Pumping rate (m^3/day) during stress period (T)

from Al Kamil Wellfield Reservoir to supply Al Kamil Wilayat.

$D_{Kw}(T)$: Water demand (m^3/day) for Al Kamil Wilayat during stress period (T).

Jaalan Wilayat water demand constraint

$$Q_{Jw}(T)\Delta t = D_{Jw}(T)\Delta t \tag{6.23}$$

Where,

$Q_{Jw}(T)$: Pumping rate (m^3/day)during stress period (T)

from Jaalan Wellfield Reservoir to supply Jaalan Wilayat.

$D_{Jw}(T)$: Water demand (m^3/day) for Jaalan Wilayat during stress period (T).

6.2.2.7 Sur Desalination Plant capacity constraints

The existing Sur Desalination Plant was constructed to produce desalinated water at a maximum capacity of $80\mathrm{x}10^3\mathrm{m}^3$/day. Therefore, the plant capacity constraint should be introduced in formulating the optimization problem. That can be represented by the following equation:

$$Q_{Sd}(T) \leq 80{,}000 \tag{6.24}$$

Where,

$Q_{Sd}\ (T)$: Pumping rate (m^3/day) during stress period (T)

from Sur Desalination Plant to Sur Reservoir.

6.2.2.8 The implicit 3-D finite difference approximation of the flow equation

Unconfined aquifers which are similar to this study aquifer system are defined by the governing equation described in Chapter 2. Because these equations involve a product of the independent variable and its derivative, they are nonlinear differential equations. The numerical solution of the unconfined flow equations also produces a nonlinear system of equations (Ahlfeld and Mulligan 2000). In any groundwater management model using embedding method, the flow equations of the simulation model represent also the constraints of the optimization model (Theodossiou, 2004). The flow equation is derived using the block centred finite difference approach given by McDonald and Harbaugh (1988) and it was explained in detail in Section 2.4 of Chapter 2.

6.2.2.9 General head boundary constraint

This constraint is applied to cells where saturated flow is considered to be always present and it can be expressed as follows:

$$Q_{i,j,NL}(T) = CR_{i,j,NL}\ \left(Hb_{i,j,NL}(T) - H_{i,j,NL}(T)\right) \tag{6.25}$$

Where,

$Q_{i,j,NL}(T)$ **:** Computed flow (m^3/day) into the cell (i,j) of layer number *NL* (1 or 2) during the stress period (T).

$CR_{i,j,NL}$: Conductance between the external source and the cell (i,j) of layer number *NL* (1 or 2).

$Hb_{i,j,NL}(T)$: Head assigned to the external source at the cell (i,j) of layer number *NL* (1 or 2) during the stress period (T).

$H_{i,j,NL}(T)$: Head in the cell *(i,j)* of layer number *NL* (1 or 2) during the stress period(T).

6.3 Operational and maintenance costs

The costs of transporting water from the source to the consumer including the operational and maintenance are becoming significant components of the unit cost of water. The capital cost in this study will not be included in the optimization calculation as the systems already exist. Therefore, only the operational and maintenance costs will determine the cost per unit pumping rates.

In deciding what the appropriate annual operating budget for a facility is, there are several approaches that could be considered, including (Klammt, 2004):

1- Zero-based method: This method is used for a project which is in its first year of operation i.e. there is little history available and few benchmarks available for the specific operations incurred. It can be used for *Water Supply Scheme from Sur to Al Kamil* because the scheme only became operational in 2009. The predicted operation and maintenance costs per unit pumping rates determined by this method were calculated based on discount and inflation rates provided by the consultant designer of the scheme, Parsons Intern. & Co LLC (Parsons Intern. & Co LLC, 2005). As shown in Table 6.2, there are slight variations in the unit costs between years 2009 and 2030 that ideally should be considered in the optimisation. However, since the discount and inflation rates used in arriving at the future unit costs are mere forecasts with their inherent uncertainties, incorporating the year-to-year variation of the costs directly in the optimisation may not be advisable. Consequently, the inter-annual variations in the costs were ignored and a constant value given by the average cost (see the last column of Table 6.2) was used throughout.

2- Historical –based method can be considered for *Ash Sharqiyah Sands Scheme* as this project has been in operation since 2004 (MRMEWR, 2004-2009).

Records taken from *Ash Sharqiyah Sands Scheme* in each of three years - 2006, 2007 and 2008 - showed the real picture of operational and maintenance (O&M) costs versus water consumptions. A review of the data produced the following findings:

- The project first year of full operation was 2006.
- Electricity Consumption Cost (Rail Omani (R.O.) / m^3) was constant during the record life (i.e. for the three years).
- The (O&M) cost (R.O/m^3) started from 2006.
- The (O&M) cost during the third year of operation (2008) considerably increased by 30%.

Based on the above, the future operational and maintenance costs and the electricity consumption costs were predicted for the *Ash Sharqiyah Sands Scheme*. These are summarized in Table 6.3. However for simplicity, a constant value given by the average cost as presented in the last column of Table 6.3 was used for each cost category in the optimization calculations.

The cost of water from the Sur Central Desalination Plant to Sur Reservoir (C_{Sd}) is fixed at 0.16195 R.O/ m³ based on the water purchase agreement between Oman Government and Ash Sharqiyah Desalination Company SAOC which was signed on 17 January 2007 (Mot MacDonald, 2007). Table 6.4 summarizes the total cost per unit pumping (R.O/m³) used in the optimization problem. These costs of pumping unit from the three existing water supply sources to meet Ash Sharqiyah Region water demand were the summations of pumping costs of different pipe routes in Tables 6.2 and 6.3.

6.4 Results and discussions of the optimum water management scenarios

Two optimum water management scenarios were investigated in detail with regard to meeting the long-term demand situations up to 2030 in the eight Wilayats of Ash Sharqiyah Region:

1- Optimum conjunctive water supply of groundwater and desalinated water with the existing well pumping capacities at the two wellfields and

2- Optimum conjunctive water supply after increasing pumping rate from each of the wells by 50% of its current maximum operational capacity.

6.4.1 Existing Scenario: The optimum conjunctive water supply with the existing well pump capacities at the two wellfields

The optimization management model was run from 2010 up to 2030 to find the optimum solution from the three existing water sources. These sources are Al Kamil Wellfield, Jaalan Wellfield and Sur Desalination Plant to supply Al Kamil Wilayat, Jaalan Wilayat, Sur Wilayat and North Wilayats as illustrated in the schematic diagram in Figure 6.1.

On the basis of unit production costs alone (see Table 6.4), it is cheaper to supply Al Kamil Wilayat from Al Kamil Wellfield, Jaalan Wilayat from Jaalan Wellfield, and in general all other Wilayats from groundwater rather than from Sur Desalination Plant. However, optimization problem will need to balance this against satisfying the numerous constraints outlined in Section 6.3.2 and such a requirement may warrant releasing water from the Sur Desalination Plant to supplement the abstractions from the groundwater fields. Head constraint at Aflaj mother well, which was set at 0.5 m above the bottom depth of the Aflaj mother well, is invariably the main factor to control the limit on abstraction from the wellfields.

As noted in Chapter 5, when groundwater from the wellfields was relied upon as the sole source of meeting the future demands for the Wilayats, three of the Aflaj -Mashaikh, Faghri and Bailhiss– will run dry in the future. However, Figure 6.2 (see also Table 6.5) shows the trajectory for the optimised water levels in the Aflaj from where it is clear that for most of the Aflaj, there is not problem of drying out because, as noted in the simulation studies, these Aflaj are located upstream of the pumped wells and are hence least influenced by the pumping. The only exception was at Flaj Faghri which is located downstream of its associated pumping wells and where the water level in the Flaj just reached the constraint limit in year 2028.

Optimal water supply to meet Al Kamil Wilayat water demand

Al Kamil Wilayat required $1.39 \times 10^6 m^3$ of domestic water in 2010 and it is predicted to increase to $2.49 \times 10^6 m^3$ in 2030. The result of the management model is shown in Table 6.6 where it is clear that 90% of total required water to this Wilayat for all years from 2010 to 2030 can be provided from Al Kamil Wellfield with the remaining 10% being provided from Sur Desalination Plant. However, the significant drawdown in the wellfield caused by this abstraction mean that in 2026, groundwater contribution has reduced to 1% with desalinated water contributing the lion share. The share from desalination decreased in subsequent years as the wellfield recovers but desalinated water still dominated the total water supplied in 2029 in the Wilayat (see Table 6.6).

Al Kamil Wilayat will require approximately $41x10^6m^3$ of domestic water costing $2.84x10^6$ RO over the 20 years from 2010 and 2030 of this $36.53x10^6m^3$ costing $1.35x10^6$ RO will be provided by Al Kamil Wellfield and $4.23x10^6m^3$ costing $1.49x10^6$ RO will be supplied from Sur Desalination Plant (see Table 6.6). This reflects the huge cost of desalination relative to fresh groundwater system; indeed for the Al Kamil Wilayat supply, it is costing 9.5 times more to supply desalination water than the fresh water from Al Kamil Wellfield.

Optimal water supply to meet Jaalan Wilayat water demand

Jaalan Wilayat required $5.35x10^6m^3$ of domestic water in 2010 and it is predicted to increase to $9.47x10^6m^3$ in 2030. The result of the management model is shown in Table 6.7 where it is clear that 96% of total required water to this Wilayat for all years from 2010 to 2030 can be provided from groundwater, mostly (94%) from Jaalan Wellfield with the remaining 4% being provided from Sur Desalination Plant. However, the significant drawdown in the wellfields caused by this abstraction mean that by 2026 with exception of 2028 as the wellfields recovers , groundwater contribution has reduced to approximately 80% with desalinated water contributed the remaining share (see Table 6.7).

Jaalan Wilayat will require approximately $155x10^6m^3$ of domestic water costing $6.71x10^6$ RO over the 20 years from 2010 and 2030 of this $146.52x10^6m^3$ costing

4.25×10^6 RO will be provided by Jaalan Wellfield, $3.02 \times 10^6 m^3$ costing 0.44×10^6 RO will be provided by Al Kamil Wellfield and $5.79 \times 10^6 m^3$ costing 2.02×10^6 RO will be supplied from Sur Desalination Plant (see Table 6.7). This reflects the huge cost of desalination relative to fresh groundwater system; indeed for the Jaalan Wilayat supply, it is costing 11.1 times more to supply desalination water than the fresh water from the two wellfields.

Optimal water supply to meet Sur Wilayat water demand

The result of the management model shows 100% water supply to Sur Wilayat will be from the costly Sur Desalination Plant from 2010 to 2030 due to head constraint at Flaj Faghri mother well. The Wilayat required $4.60 \times 10^6 m^3$ of domestic water in 2010 and it is predicted to increase to $8.68 \times 10^6 m^3$ in 2030. The total water demand over the 20 years from 2010 and 2030 will be approximately $136 \times 10^6 m^3$ costing 27.55×10^6 RO. As the Sur Desalination Plant is located in the same Wilayat, the cost of water supply to Sur Wilayat from Sur Desalinated Plant is costing almost the same cost (1.1 more) of water supply from the 60km away wellfields.

Optimal water supply to meet North Wilayat water demand

North Wilayats required $6.18 \times 10^6 m^3$ of domestic water in 2010 and it is predicted to increase to $15.01 \times 10^6 m^3$ in 2030. The result of the management model is shown in Table 6.8 where it is clear that only 21% of total required water to these Wilayats for all years from 2010 to 2030 can be provided from groundwater due to head constraint at Flaj Faghri mother well, with the remaining 79% being provided from Sur Desalination Plant. However, the water supply will begin with 79% from the two wellfields and only 21% from Sur Desalination Plant in 2010 but due to the significant drawdown in the wellfields caused by this abstraction, this percentage will decrease to zero by 2026 and the whole supply in subsequent years will be provided by the Sur Desalination Plant (see Table 6.8).

North Wilayats will require approximately 224.69x10^{6}m³ of domestic water costing 61.69x10^6 RO over the 20 years from 2010 and 2030 of this 32.50x10^{6}m³ costing 3.74x10^6 RO will be provided by Al Kamil Wellfield, 15.33x10^{6}m³ costing 1.90x10^6 RO will be provided by Jaalan Wellfield and 176.86x10^{6}m³ costing 56.05x10^6 RO will be supplied from Sur Desalination Plant (see Table 6.8). This reflects the high cost of desalination relative to fresh groundwater system; indeed for the North Wilayats supply, it is costing 2.7 times more to supply desalination water than the fresh water from the two wellfields.

Total optimal water supply from three existing water sources to meet Ash Sharqiyah Region water demand

The eight Wilayats of Ash Sharqiyah Region required a total of 17.52x10^{6}m³ of domestic water in 2010 and it is predicted to increase to 35.65x10^{6}m³ in 2030. The result of the management model is shown in Table 6.9 where it clear that 42% of total required water to Ash Sharqiyah Region for all years from 2010 to 2030 can be provided by groundwater, 13% by Al Kamil Wellfield and 29% by Jaalan Wellfield, with the remaining 58% being from Sure Desalination Plant. However, the water supply will begin with 66% from the two wellfields and only 34% from Sur Desalination Plant in 2010 but due to the significant drawdown in the wellfields caused by this abstraction and head constraint at Flaj Faghri mother well, this percentage will decrease to 29% by 2030 and most of the supply (71%) by that year will be provided by Sur Desalination Plant (see Table 6.9).

Ash Sharqiyah Region will require approximately $557x10^6m^3$ of domestic water costing approximately $99x10^6$ RO over the 20 years from 2010 to 2030 of this $234\ x10^6m^3$ costing only $12\ x10^6$ RO will be provided by the two wellfields and $323x10^6m^3$ costing $87\ x10^6$ RO will be supplied from Sur Desalination Plant (see Table 6.9). This reflects the huge cost of desalination relative to fresh groundwater system; indeed for the Ash Sharqiyah Region supply (see Figure 6.3), it is costing 5.3 times more to supply desalination water than the fresh water from the two wellfields.

6.4.2 Alternative Scenario 2: The optimum conjunctive water supply after increasing pumping rate in each of the well in Jaalan Wellfield by 50% of its current maximum operational capacity

The previous results of the optimization management model output showed that all of the 21 wells of Jaalan Wellfield were able to reach their maximum existing pumping rate capacities during all of stress periods from 2010 to 2030 because most of the targeted protected eight Aflaj are located upstream of this wellfield, unlike Al Kamil Wellfield where these Aflaj are located downstream. Given this, it will be worthwhile to investigate the effect of increased pumping from wells at the Jaalan well field, which if possible without violating the head constraints at the Aflaj mother wells, should relieve pressure on the Aflaj associated with the Al Kamil Wellfield. To get an idea of the possible increased pumping rate at the Jaalan, the optimization management model was run several times to investigate the optimum higher pump rate capacities for each well in Jaalan Wellfield to pump as much water as possible from Ash Sharqiyah Sands Aquifer as it is a cheaper source of water supply without changing any of other constraints. It was found, by re-running the management model several times and observing the 0.5m limit between the head and the base of Aflaj mother wells, that the pumping rate in each of the well in Jaalan Wellfield can be increased by 50% of its current operational capacity shown in Table 6.10 but keeping all other constraints unchanged. On the other hand, the well pumping rate capacities of Al Kamil Wellfield were kept unchanged because most of the eight wells at Al Kamil Wellfield will not be able to reach their maximum pumping capacities starting from 2026. That is because the head constraint at Flaj Faghri mother well which was set at 0.5m will reach its maximum limit starting from that year up to 2030 as explained in Section 6.5.1. There is no extra capital cost needed to implement this scenario as the systems already exist and capable to operate with the required demands until the year 2030 according the consultant designer of the scheme, Parsons Intern. & Co LLC (Parsons Intern. & Co LLC, 2005). The results of the management model for this scenario are described in the following Sections.

Optimal water supply to meet Al Kamil Wilayat water demand

The result of management model to meet Al Kamil Wilayat water demand is shown in Table 6.11 where it is clear that 87% instead of 90% as in the previous scenario of total required water to this Wilayat for the years from 2010 to 2030 can be provided from Al Kamil Wellfield because Al Kamil Wellfield will not produce any water in 2026 and 2027 as the head constraint at Flaj Faghri mother well will reach its maximum limit due to influence of increase pumping from Jaalan Wellfield (see Table 6.11). Positively, the other 13% will be provided from Jaalan Wellfield instead of being from the costly Sur Desalination Plant as in the previous scenario. Therefore, the optimal water supply to the Wilayat will be 100% from groundwater over the 20 years from 2010 to 2030. The cost is reduced to 2.17×10^6 RO instead of being 2.84×10^6 RO as in the previous option (see Table 6.11).

Optimal water supply to meet Jaalan Wilayat water demand

The result of management model to meet Jaalan Wilayat water demand shows 100% water supply to Jaalan Wilayat will be provided from Jaalan Wellfield over the 20 years from 2010 to 2030, instead of being 94% from Jaalan Wellfield, 2% from Al Kamil Wellfield and 4% from Sur Desalination Plant as in the previous scenario. The cost is reduced to $4.51x10^6$ RO instead of being $6.71x10^6$ RO as in the previous option.

Optimal water supply to meet Sur Wilayat water demand

The result of management model to meet Sur Wilayat water demand is shown in Table 6.12 where it is clear that 7% instead of zero as in the previous scenario of the required water to Sur Wilayat will be provided by Jaalan Wellfield from 2010 to 2030. The percentage contribution from Sur Desalinated Plant will be reduced from 100% to 93% resulting in slightly reduction in cost from being $27.6x10^6$ RO to $27.3x10^6$ RO (see Table 6.12).

Optimal water supply to meet North Wilayats water demand

The result of management model to meet North Wilayats water demand is shown in Table 6.13 where it is clear that 49% instead of 21% as in the previous scenario of required water to these Wilayats from 2010 to 2030 will be provided by groundwater, 16% by Al Kamil Wellfield and 33% by Jaalan Wellfield. Thus Sur Desalination Plant supply contribution will be reduced from 79% to 51% resulting in total cost reduction from being 61.69x106 RO to $49.79x10^6$ RO (see Table 6.13).

Total optimal water supply from three existing water sources to meet Ash Sharqiyah Region water demand

The result of the management model to meet Ash Sharqiyah Region water demand is shown in Table 6.14 where it is clear that Ash Sharqiyah Sands Aquifer supply contribution, to the eight Wilayats of Ash Sharqiyah Region water demand over the 20 years from 2010 to 2030, will increase from being $234x10^6m^3$ (equivalent to 42%) to 309 $x10^6m^3$ or 56%. Thus Sur Desalinated Plant supply contribution will reduce from being $323x10^6m^3$ (equivalent to 58%) to 248 $x10^6m^3$ (equivalent to 44%) (see Figure 6.4). Subsequently, the total optimum cost will be reduced from 99 $x10^6$ RO to 84 $x10^6$ RO (see Figure 6.5). Therefore, the best optimum water management scenario, to meet Ash Sharqiyah Region water demand using conjunctive groundwater and desalinated water over the 20 years from 2010 to 2030, would be to use the conjunctive water supply after increasing pumping rate in each of the well in Jaalan Wellfield by 50% of its current operational capacity.

6.5 Results and discussions of the sensitivity analysis on some of the economic factors

As the unit pumping production cost, desalination production cost and water projected demand for potable water was considered to have a high degree of uncertainty, economic sensitivity analysis on them was performed. In all sensitivity analysis runs, only the parameter of interest was changed, others were kept constant. Comparison of results from each optimum water management run and the corresponding economic results will indicate how sensitive the optimization model is to the tested parameter. The proportional percentages decrease / increase of the above three parameters were chosen to vary as

-20%, -10%, +10% and +20%, as they were not expected to vary more than that, as there were based on sufficient measured, or estimated data.

As stated in Section 5.2.2, the modelled area was discretized into square grids of 500 m spacing, which were refined to a square grid of 250 m spacing at the stress areas (wellfields). The numerical finite difference solution adopted assumes that the hydraulic head is uniform within a given grid square. Whilst this is not a major problem in grid cells where there are no external stresses (i.e. well abstractions), it may not accurately describe the rapid drawdown caused by turbulence and well losses in the proximity of the pumped wells. To better model such effects, a much finer mesh, typically with spacing of the order of the diameter of the pumped well, would be required. However, this will cause the computation time to increase astronomically and may run the risk of causing instability of the numerical solution scheme. It is precisely to avoid such problems that a relatively coarse time interval of 4 months was adopted for the discretisation in the time domain for the unsteady state simulations. For the broad objective of developing an optimal, conjunctive groundwater-seawater desalination use strategy as implemented in the current study, such a “lumped” approach involving relatively coarse spatial and temporal discretisation scales should suffice. Nonetheless, a recommendation to investigate this assumption will be included in the suggestions for further work at the end of the thesis.

The results of sensitivity analysis are presented in the following sections for both management scenarios.

6.5.1 Sensitivity analysis results for scenario 1

The results of the sensitivity of variations in the unit pumping production cost, desalination production cost and water projected demand for scenario 1 are presented in

Tables 6.15, 6.16 and 6.17 respectively. The results of the analysis are discussed as follows:

Variation in unit pumping costs

Variation in unit pumping costs was tested mainly to assess the effects that possible changes in energy and maintenance costs can have on the global costs of the optimal solution. Therefore, the unit pumping costs presented in Table 6.4 from the three existing water supply sources were varied by -20%, -10%, +10% and +20%. However, the cost of water production from the Sur Central Desalination Plant was kept fixed at 0.16195 R.O/ m³ based on the water purchase agreement between Oman Government and Ash Sharqiyah Desalination Company SAOC (Mott MacDonald, 2007). The sensitivity analysis results (see Table 6.15) showed that the amount of water provided from each of the three water sources to meet Ash Sharqiyah Regional demand over the 20 years from 2010 to 2030 would remain the same. This is to be expected given that the variations in pumping costs were applied uniformly across all the sources. However, had different variations been applied to different sources, then a different outcome would have resulted with the cheaper source being made to contribute more water to the total water supplied.

However, the cost of water from each water source would increase or decrease due to increasing or decreasing of the unit pumping costs respectively. The results (see Table 6.15) show that the optimal total cost of 98.8 $x10^6$ RO would be reduced to 94.1 $x10^6$ RO equivalent to 4.7% reduction in cost and to 89.5 $x10^6$ RO equivalent to 9.4% reduction in cost due to decrease in unit pumping costs by 10% and 20% respectively. Also, the optimal total cost of 98.8 $x10^6$ RO would increased to 103.5 $x10^6$ RO equivalent to 4.7% extra in cost and to 108.1 $x10^6$ RO equivalent to 9.4% extra in cost due to increase in unit pumping costs by 10% and 20% respectively.

A common feature of all of the above cases is that the reduction in total cost is not linearly related to the reduction in the unit cost; in fact, in proportional terms, changes in total cost were less that the corresponding changes in the unit pumping costs. One possible reason for this is the inherently non-linear of the objective function as presented in equation 6.1. Another factor that may have caused the disparity between the proportional change in the unit pumping costs and the total costs is because a significant part of the total cost, especially in the later years, is made up of the Sur desalinated water production costs. As noted earlier, the unit cost of the desalination plant was not changed at all for this sensitivity analysis.

Variation in desalination production cost

As stated above, the unit cost of water production from the Sur Central Desalination Plant is fixed at 0.16195 R.O/ m^3 until the year of 2027 based on the water purchase agreement between Oman Government and Ash Sharqiyah Desalination Company SAOC (Mott MacDonald, 2007). However, if the cost of desalinated water will become cheaper due to desalination technology improvement, the government may decide to change the agreement. Therefore, sensitivity analysis was carried out on the variation in desalination production costs. It is unlikely to see an increase in desalination cost, but sensitivity analysis was also tested for cost increasing for comparison purposes. The cost therefore was varied as -20%, -10%, +10% and +20%. The results (see Table 6.16) showed that the amount of water provided from each of the three water sources to meet Ash Sharqiyah Region over the 20 years from 2010 to 2030 would remain the same. The cost of water from groundwater source would also remain the same as assumed there were no changes in unit pumping cost. The results (see Table 6.16) show that the optimal total cost of 98.8 $x10^6$ RO would be reduced to 93.6 $x10^6$ RO equivalent to 5.3% reduction in cost and to 88.4 $x10^6$ RO equivalent to 10.5% reduction in cost due to decrease in desalination production costs by 10% and 20% respectively. As was the case with the previous case in which only the pumping unit costs were varied, the response of the total is not linearly related to the change in the pumping costs. However, because changes in the costs of production at the Sur desalination plant are being considered, the simulated proportional change in the total costs exceeds that for the pumping costs change alone. For, the reduction in cost is better by 0.6% when changes in both the unit pumping and Sur desalination costs are reduced by 10% than when only the unit pumping costs were reduced by the same percentage change. Also, the optimal total cost of 98.8 $x10^6$ RO is expected to increase to 104.1 $x10^6$ RO equivalent to 5.4% extra in cost and to 109.4 $x10^6$ RO equivalent to 10.7% extra in cost due to increase in desalination production costs by 10% and 20% respectively.

Variation in water projected demand

A variation in water projected demand was preformed to test its effect on the optimal total cost due to decrease or increase in projected demand and the sources of water. The projected demand was varied as -20%, -10%, +10% and +20%. The sensitivity analysis results (see Table 6.17) shows that the amount of water provided from each of the two wellfields to meet Ash Sharqiyah Region over the 20 years from 2010 to 2030 will remain the same. However, the amount of desalinated water will decrease or increase as the projected demand decreases or increases respectively, resulting in similar relation for the cost of the required desalinated water. The cost of water from groundwater would increase as the projected water decrease because the groundwater could be pumped further to serve further Wilayats as far as Sur Wilayat. Oppositely, the cost of water from groundwater would decrease as the projected water increase because the groundwater would be required to serve closer Wilayat like Al Kamil and Jaalan which require less cost to pump the water to serve them. However, the results (see Table 6.17) show that the overall optimal total cost of 98.8 $x10^6$ RO would be reduced to 83.7 $x10^6$ RO equivalent to

15.3% reduction in cost and to 69.6 x10^6 RO equivalent to 29.6% reduction in cost due to decrease in projected water demand by 10% and 20% respectively. The proportional reduction in total demand is obviously larger than the corresponding reduction in unit and this is caused by the huge effect of the desalination cost on total production. Reducing or decreasing the contribution of desalination to the total water supplied will so dominate the effect on the total cost that the ultimate effect will be a significant disparity in the proportional change in total production cost as observed here. In monetary terms, the optimal total cost of 98.8 x10^6 RO would increase to 114.3 x10^6 RO equivalent to 15.7% extra in cost and to 130.1 x10^6 RO equivalent to 31.7% extra in cost due to increase in projected water demand by 10% and 20% respectively.

6.5.2 Sensitivity analysis results for scenario 2

Same sensitivity analysis varied as -20%, -10%, +10% and +20%, was carried out for the same three parameters for scenario 2. The results are shown in Tables 6.18, 6.19 and 6.20 for the unit pumping production cost, desalination production cost and water projected demand respectively. The results of the analysis are discussed beneath.

Variation in unit pumping costs

The sensitivity analysis results (see Table 6.18) showed that the amount of water provided from each of the three water sources to meet Ash Sharqiyah Region over the 20 years from 2010 to 2030 would remain the same. However, the cost of water from each water source would increase or decrease due to increasing or decreasing in the unit pumping costs respectively. The results (see Table 6.18) show that the optimal total cost of 83.8 x10^6 RO would be reduced to 80.7 x10^6 RO equivalent to 3.7% reduction in cost and to 75.9 x10^6 RO equivalent to 9.4% reduction in cost due to decrease in unit pumping costs by

10% and 20% respectively. Also, the optimal total cost of 83.8 x10^6 RO would increase to 89.3 x10^6 RO equivalent to 6.6% extra in cost and to 93.8 x10^6 RO equivalent to 11.9% extra in cost due to increase in unit pumping costs by 10% and 20% respectively.

Variation in desalination production cost

The results (see Table 6.19) show that the amount of water provided from each of the three water sources to meet Ash Sharqiyah Region over the 20 years from 2010 to 2030 would remain the same except with a -20% reduction in the desalinated production cost. In this case, the amount of water will be more from Sur Desalination Plant and less from Jaalan Wellfield than other cases because the whole Sur Wilayat will be supplied with desalinated water as it will be cheaper than from the Jaalan Wellfield (see Table 6.19). The results also show that the optimal total cost of 83.8 x10^6RO would be reduced to 81.7 x10^6 RO equivalent to 2.5% reduction in cost and to 80.9 x10^6 RO equivalent to 3.5% reduction in cost due to decrease in desalination production costs by 10% and 20% respectively. Also, the optimal total cost of 83.8 x10^6 RO is expected to increase to 85.8 x10^6 RO equivalent to 2.4% extra in cost and to 93.8 x10^6 RO equivalent to 11.9% extra in cost due to increase in desalination production costs by 10% and 20% respectively.

Variation in water projected demand

The projected demand was also varied as -20%, -10%, +10% and +20%. The sensitivity analysis results (see Table 6.20) show that the amount of water provided from each of the two wellfields to meet Ash Sharqiyah Region over the 20 years from 2010 to 2030 would remain the same except when water projected demand reduced by -20% demands. In this case, in the years - 2010, 2011 and 2012 - all Wilayats will get their water demands from groundwater and the pumps in Jaalan Wellfield will be not required to reach their total maximum capacities to provide all Wilayats with their required demands. However, the amount of desalinated water would decrease or increase as the projected demand decrease or increase respectively resulting in similar relation for the cost of the required desalinated water. The cost of water from groundwater would increase as the projected water decrease because the groundwater could be pumped further to serve further Wilayats as far as Sur Wilayat. Oppositely, the cost of water from groundwater would decrease as the projected water increase because the groundwater would be required to serve closer Wilayat like Al Kamil and Jaalan which require less cost to pump the water to serve them. However, the results (see Table 6.20) show that the overall optimal total cost of 83.8 $x10^6$ RO would be reduced to 71.9 $x10^6$ RO equivalent to 14.2% reduction in cost and to 59.3 $x10^6$ RO equivalent to 29.3% reduction in cost due to decrease in projected water demand by 10% and 20% respectively. Also, the optimal total cost of 83.8 $x10^6$ RO would increase to 98.2 $x10^6$ RO equivalent to 17.2% extra in cost and to 112.4 $x10^6$ RO equivalent to 34.1% extra in cost due to increase in projected water demand by 10% and 20% respectively.

In summary (see Table 6.21), the variation in water projected demand showed to be a more sensitive parameter than the unit pumping production cost or desalination production cost. The overall optimal cost of water to meet Ash Sharqiyah Regional demand over the 20 years from 2010 to 2030 can vary from up to -30% or + 34% due to decrease or increase in projected water demand by - 20% or +20% respectively. However, increasing or decreasing the pumping or desalination cost by 20% produced 10% change (increase/decrease) in the total projected cost.

6.6 Summary

This chapter described the optimization of the water supply demand arrangement for the Ash Sharqiyah Region. The main driver of the optimization was the need to reduce the current abstractions from the existing wellfields and thus eliminate some of the associated negative environmental impacts, notably the drying up of the Aflaj that derive their flows from the groundwater. A decision was made to supplement the wellfields supplies with water from the newly installed Sur Desalination Plant but given the huge cost of desalination, the problem was set up as an optimization problem to determine the least cost blend of the two sources. The constrained optimization problem has as its objective function the minimization of total cost of meeting the water demand up to the year 2030. The constraints ranged from maintaining a minimum level of water in the wells that ensure that the Aflaj flows continuously to meeting the total water demand for domestic, agriculture and industrial purposes.

Two water management scenarios were investigated in detail; (i) conjunctive water supply using the existing well pump capacities as there are in each of the two wellfields; and (ii) conjunctive water supply after increasing pumping rate from each of the well in Jaalan Wellfield by 50% of its current maximum operational capacity. Option (ii) was prosecuted because option (i) revealed that wells in the Jaalan field could be made to produce more water without seriously affecting the flows in the Aflaj most of which are located upstream of the wellfield. The result of the management model shows 309 $x10^6 m^3$ (equivalent to 56%) by using the second scenario instead of $234x10^6 m^3$ (equivalent to 42%) as in the existing scenario of total optimum required water to Ash Sharqiyah Region for the next 20 years will be provided from Ash Sharqiyah Sands Aquifer. The contribution of Sur Desalinated Plant will reduce from $323x10^6 m^3$ (equivalent to 58%) to 248 $x10^6 m^3$ (equivalent to 44%). As a consequence, the total optimum cost will be reduced from being $99x10^6$ RO to $84x10^6$ RO. Therefore, it is recommended to use the second management scenario to supply Ash Sharqiyah Region with water over the next 20 years up to the year of 2030.

The variation in water projected demand proved to be more significant effect on the results than the unit pumping production cost or desalination production cost. The overall optimal cost of water to meet Ash Sharqiyah Regional demand over the 20 years from 2010 to 2030 can vary from up to -30% or + 34% due to decrease or increase in projected water demand by 20%, reflecting the huge disparity between desalination production costs and groundwater production costs. Because of this, desalination costs will dominate the total costs and so any changes in the desalinated water quantity are bound to dominate the resulting effects on the total production costs, which is clearly the case in the sensitivity studies reported here. However, increasing or decreasing the pumping or desalination cost by 20% produced only a 10% change (increase/decrease) in the total projected cost. This latter situation could be attributed to the inherently non-linear nature of the objective function, resulting in the difference (in proportional terms) between changes in the input costs and the resulting changes in the total production costs.

Table 6.1: Reservoirs capacity at different pumping stations of the water supply system for Ash Sharqiyah region using desalinated water and groundwater (Parsons Intern. & Co LLC, 2005)

No.	Reservoir's name	Capacity (m^3)
1	Sur Reservoir (K_{sr})	160,000
2	Al Kamil Reservoir (K_{kr})	6,000
3	Al Kamil Wellfield Reservoir (K_{kwr})	6,000
4	Jaalan Wellfield Reservoir (K_{Jwr})	12,300

Table 6.2: Summary costs of unit pumping calculated based on discount and inflation rates from different main ground reservoirs of the water supply system for Ash Sharqiyah Region (after Parsons Intern. & Co LLC, 2005)

No.	Pipe Route	Cost per unit pumping (R.O/ m^3)			
		2009	2015	2030	Average
1	Sur Reservoir ***To*** Sur Wilayat(C_{Sw})	0.030	0.041	0.049	0.04
2	Sur Reservoir ***To*** Al Kamil Reservoir (C_{SrKr})	0.078	0.088	0.112	0.093
3	Al Kamil Reservoir ***To*** Al Kamil Wellfield Reservoir (C_{KrKwr})	0.057	0.080	0.090	0.076
4	Al Kamil Reservoir ***To*** Jaalan Wellfield Reservoir (C_{KrJwr})	0.063	0.080	0.090	0.078
5	Al Kamil Reservoir ***To*** North Wilayats (Bidiyah, Qabil, Ibra& Mudhaibi) (C_{Nw})	0.051	0.060	0.076	0.062
6	Al Kamil Wellfield Reservoir ***To*** Al Kamil Reservoir (C_{KwrKr})	0.028	0.038	0.043	0.036
7	Jaalan Wellfield Reservoir ***To*** Al Kamil Reservoir (C_{JwrKr})	0.039	0.045	0.061	0.048
8	Al Kamil Reservoir ***To*** Sur Reservoir (C_{KrSr})	0.049	0.069	0.116	0.078

Table 6.3: Summary costs per unit pumping for Ash Sharqiyah Sands Scheme (after Ash Sharqiyah Sands Main Water Supply System Annual Water Production Reports, MRMEWR (2004-2009))

No.	Pipe route	Cost per unit pumping (R.O/ m³)					
		2006	2007	2008	2015	2030	Average
1	Al Kamil Wellfield ***To*** Al Kamil Wellfield Reservoir (C_{kg})	0.012	0.011	0.013	0.017	0.023	0.017
2	Al Kamil Wellfield Reservoir ***To*** Al Kamil Wilayat (C_{Kw})	0.012	0.011	0.012	0.020	0.029	0.020
3	Jaalan Wellfield ***To*** Jaalan Wellfield Reservoir (C_{jg})	0.010	0.011	0.013	0.014	0.017	0.014
4	Jaalan Wellfield Reservoir ***To*** Jaalan Wilayat (C_{jw})	0.009	0.009	0.010	0.014	0.020	0.015

Table 6.4: Summary of the total cost per unit pumping (R.O/m³) from the three existing water supply sources to meet Ash Sharqiyah Region water demand

	Source of the water supply		
Place to be supplied by water	**Sur Desalination Plant**	**Al Kamil Wellfield**	**Jaalan Wellfield**
Sur Wilayat	0.20195	0.171	0.18
North Ash Sharqiyah Wilayats	0.31695	0.115	0.124
Al Kamil Wilayat	0.35095	0.037	0.158
Jaalan Wilayat	0.34795	0.146	0.029

Table 6.5: Management model outputs of the optimal differences between the protected Flaj mother well depths and the predicted groundwater heads (m)

Year	Al Kamil	Mahyul	Al Wafi	Minjired	Bailhiss	Hilal	Mash aikh	Faghri
2010	13.33	12.72	9.45	6.23	4.98	3.54	3.17	2.35
2011	13.22	12.57	9.20	6.05	4.81	3.36	2.94	2.17
2012	13.08	12.50	8.90	5.98	4.70	3.16	2.72	1.99
2013	13.09	12.43	8.90	5.90	4.59	2.99	2.51	1.82
2014	13.33	12.47	9.36	5.93	4.64	2.97	2.39	1.77
2015	13.18	12.38	9.08	5.85	4.51	2.86	2.24	1.65
2016	13.00	12.28	8.73	5.76	4.37	2.69	2.06	1.49
2017	13.06	12.22	8.84	5.70	4.28	2.58	1.92	1.37
2018	12.97	12.14	8.66	5.61	4.14	2.45	1.78	1.25
2019	12.84	12.07	8.41	5.54	4.04	2.31	1.64	1.11
2020	12.88	12.00	8.45	5.47	3.95	2.20	1.51	1.00
2021	13.12	12.03	8.95	5.50	3.99	2.23	1.46	1.00
2022	12.99	11.95	8.71	5.43	3.86	2.17	1.37	0.93
2023	12.81	11.85	8.38	5.34	3.71	2.05	1.26	0.82
2024	12.89	11.82	8.51	5.30	3.61	1.98	1.17	0.74
2025	12.80	11.75	8.35	5.24	3.46	1.89	1.07	0.65
2026	12.69	11.69	8.12	5.18	3.35	1.79	1.02	0.55
2027	12.73	11.63	8.20	5.12	3.25	1.75	1.00	0.50
2028	12.99	11.69	8.73	5.18	3.28	1.85	1.02	0.57
2029	12.87	11.63	8.53	5.13	3.14	1.85	1.03	0.56
2030	12.70	11.55	8.22	5.05	2.99	1.78	0.99	0.50

Table 6.6: Optimal water supply solution for Al Kamil Wilayat (existing scenario)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	10^6m^3	10^6m^3	%	10^6m^3	%	10^6m^3	%	10^6m^3	%
2010	1.39	1.39	100	0	0	1.39	100	0	0
2011	1.43	1.43	100	0	0	1.43	100	0	0
2012	1.47	1.47	100	0	0	1.47	100	0	0
2013	1.51	1.51	100	0	0	1.51	100	0	0
2014	1.54	1.54	100	0	0	1.54	100	0	0
2015	1.66	1.66	100	0	0	1.66	100	0	0
2016	1.74	1.74	100	0	0	1.74	100	0	0
2017	1.79	1.79	100	0	0	1.79	100	0	0
2018	1.84	1.84	100	0	0	1.84	100	0	0
2019	1.88	1.88	100	0	0	1.88	100	0	0
2020	1.94	1.94	100	0	0	1.94	100	0	0
2021	2.02	2.02	100	0	0	2.02	100	0	0
2022	2.01	2.01	100	0	0	2.01	100	0	0
2023	2.06	2.06	100	0	0	2.06	100	0	0
2024	2.17	2.17	100	0	0	2.17	100	0	0
2025	2.23	2.23	100	0	0	2.23	100	0	0
2026	2.31	0.02	1	0	0	0.02	1	2.29	99
2027	2.37	1.70	72	0	0	1.70	72	0.67	28
2028	2.44	2.44	100	0	0	2.44	100	0	0
2029	2.49	1.21	49	0	0	1.21	49	1.28	51
2030	2.49	2.49	100	0	0	2.49	100	0	0
Total demand (10^6m^3)	**40.76**	**36.53**	**90% from total**	**0**	**0% from total**	**36.53**	**90% from total**	**4.23**	**10% from total**
Total cost (10^6 RO.)	**2.84**	**1.35**	**48% from total**	**0**	**0% from total**	**1.35**	**48% from total**	**1.49**	**52% from total**

Table 6.7: Optimal water supply solution for Jaalan Wilayat (existing scenario)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	$10^6 m^3$	$10^6 m^3$	%	$10^6 m^3$	%	$10^6 m^3$	%	$10^6 m^3$	%
2010	5.35	0	0	5.35	100	5.35	100	0	0
2011	5.52	0	0	5.52	100	5.52	100	0	0
2012	5.66	0	0	5.66	100	5.66	100	0	0
2013	5.81	0	0	5.81	100	5.81	100	0	0
2014	5.94	0	0	5.94	100	5.94	100	0	0
2015	6.31	0	0	6.31	100	6.31	100	0	0
2016	6.61	0	0	6.61	100	6.61	100	0	0
2017	6.81	0	0	6.81	100	6.81	100	0	0
2018	6.99	0	0	6.99	100	6.99	100	0	0
2019	7.14	0	0	7.14	100	7.14	100	0	0
2020	7.38	0	0	7.38	100	7.38	100	0	0
2021	7.66	0	0	7.66	100	7.66	100	0	0
2022	7.64	0	0	7.64	100	7.64	100	0	0
2023	7.83	0.10	1	7.73	99	7.83	100	0	0
2024	8.24	0.53	6	7.71	94	8.24	100	0	0
2025	8.48	0.74	9	7.73	91	8.48	100	0	0
2026	8.77	0	0	7.67	87	7.67	87	1.10	13
2027	9.02	0	0	7.71	85	7.71	85	1.31	15
2028	9.26	1.49	16	7.73	84	9.22	100	0.04	0
2029	9.45	0	0	7.71	82	7.71	82	1.74	18
2030	9.47	0.16	2	7.71	81	7.87	83	1.60	17
Total demand ($10^6 m^3$)	**155.34**	**3.0**	**2% from total**	**146.5**	**94% from total**	**149.5**	**96% from total**	**5.79**	**4% from total**
Total cost (10^6 RO.)	**6.71**	**0.44**	**7% from total**	**4.25**	**63% from total**	**4.69**	**70% from total**	**2.02**	**30% from total**

Table 6.8: Optimal water supply solution for North Wilayats (existing scenario)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	**$10^6 m^3$**	**$10^6 m^3$**	**%**	**$10^6 m^3$**	**%**	**$10^6 m^3$**	**%**	**$10^6 m^3$**	**%**
2010	6.18	2.52	41	2.36	38	4.88	79	1.30	21
2011	6.37	2.49	39	2.21	35	4.71	74	1.66	26
2012	6.53	2.44	37	2.05	31	4.50	69	2.03	31
2013	6.71	2.40	36	1.90	28	4.30	64	2.41	36
2014	6.86	2.35	34	1.73	25	4.07	59	2.78	41
2015	7.33	2.25	31	1.40	19	3.65	50	3.67	50
2016	9.94	2.15	22	1.05	11	3.20	32	6.73	68
2017	10.23	2.12	21	0.90	9	3.02	30	7.21	70
2018	10.50	2.08	20	0.74	7	2.82	27	7.68	73
2019	10.73	2.03	19	0.57	5	2.60	24	8.12	76
2020	11.09	1.97	18	0.33	3	2.30	21	8.78	79
2021	11.51	1.87	16	0.01	0	1.88	16	9.62	84
2022	11.48	1.90	17	0.07	1	1.97	17	9.50	83
2023	11.77	1.76	15	0	0	1.76	15	10.01	85
2024	12.38	1.21	10	0	0	1.21	10	11.17	90
2025	13.44	0.95	7	0	0	0.95	7	12.49	93
2026	13.62	0	0	0	0	0	0	13.62	100
2027	14.01	0	0	0	0	0	0	14.01	100
2028	14.37	0	0	0	0	0	0	14.37	100
2029	14.66	0	0	0	0	0	0	14.66	100
2030	15.01	0	0	0	0	0	0	15.01	100
Total demand ($10^6 m^3$)	**224.69**	**32.50**	**14% from total**	**15.3**	**7% from total**	**47.83**	**21% from total**	**176.9**	**79% from total**
Total cost (10^6 RO.)	**61.69**	**3.74**	**6 % from total**	**1.90**	**3% from total**	**5.64**	**9% from total**	**56.05**	**91% from total**

Table 6.9: Optimal water supply solution for Ash Sharqiyah regional demand (existing scenario)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	**10^6m^3**	**10^6m^3**	**%**	**10^6m^3**	**%**	**10^6m^3**	**%**	**10^6m^3**	**%**
2010	17.52	3.91	22.3	7.71	44.0	11.62	66.3	5.90	33.7
2011	18.05	3.92	21.7	7.73	42.8	11.65	64.5	6.40	35.5
2012	18.51	3.91	21.1	7.71	41.7	11.62	62.8	6.89	37.2
2013	19.02	3.91	20.6	7.71	40.5	11.62	61.1	7.40	38.9
2014	19.45	3.89	20.0	7.67	39.4	11.56	59.4	7.89	40.6
2015	20.52	3.91	19.1	7.71	37.6	11.62	56.6	8.90	43.4
2016	24.04	3.89	16.2	7.67	31.9	11.56	48.1	12.48	51.9
2017	24.75	3.91	15.8	7.71	31.2	11.62	47.0	13.13	53.0
2018	25.41	3.92	15.4	7.73	30.4	11.65	45.9	13.76	54.1
2019	25.94	3.91	15.1	7.71	29.7	11.62	44.8	14.32	55.2
2020	26.82	3.91	14.6	7.71	28.8	11.62	43.3	15.20	56.7
2021	27.84	3.89	14.0	7.67	27.5	11.56	41.5	16.28	58.5
2022	27.76	3.91	14.1	7.71	27.8	11.62	41.9	16.14	58.1
2023	28.47	3.92	13.8	7.73	27.2	11.65	40.9	16.82	59.1
2024	29.95	3.91	13.1	7.71	25.7	11.62	38.8	18.33	61.2
2025	31.92	3.92	12.3	7.73	24.2	11.65	36.5	20.27	63.5
2026	32.59	0.02	0.1	7.67	23.5	7.69	23.6	24.90	76.4
2027	33.52	1.70	5.1	7.71	23.0	9.41	28.1	24.11	71.9
2028	34.38	3.92	11.4	7.73	22.5	11.65	33.9	22.73	66.1
2029	35.09	1.21	3.4	7.71	22.0	8.92	25.4	26.17	74.6
2030	35.65	2.65	7.4	7.71	21.6	10.36	29.1	25.29	70.9
Total demand (10^6m^3)	**557.2**	**72.1**	**13% from total**	**161.9**	**29% from total**	**233.9**	**42% from total**	**323.3**	**58% from total**
Total cost (10^6 RO.)	**98.78**	**5.53**	**6% from total**	**6.15**	**6% from total**	**11.68**	**12% from total**	**87.10**	**88% from total**

Table 6.10: Recommended pumping rates at Jaalan Production Wellfield for alternative management strategy (scenario 2)

Well No. *	Easting	Northing	Well depth (masl)	Pump depth (masl)	pumping capacity (m^3/day)	
					Existing	Recommended
JP-1	727646.8	2433314	47.20	88.20	864	1296
JP-2	727392.2	2433756	37.36	84.36	1555	2333
JP-3	727147.6	2434181	37.30	73.30	1210	1815
JP-6	726398.9	2435480	66.58	86.58	346	519
JP-7	726149.3	2435914	43.20	88.20	432	648
JP-20	7277955	2433780	28.30	88.30	1555	2333
JP-20A	728157.9	2433320	33.00	76.00	691	1037
JP-21	727705.7	2434214	47.47	91.47	1296	1944
JP-22	727456.1	2434647	50.98	88.98	1296	1944
JP-23	727159.5	2435053	39.69	84.69	1296	1944
JP-24	726957.0	2435513	39.23	72.23	1037	1556
JP-25	726707.4	2435947	44.44	71.44	691	1037
JP-26	726457.8	2436380	50.68	85.68	778	1167
JP-39	728263.7	2434247	34.88	75.88	691	1037
JP-39A	728458.8	2433799	35.07	90.07	432	648
JP-40	728016.6	2434676	40.94	88.94	518	777
JP-41	727764.6	2435113	43.04	88.04	1080	1620
JP-43	727265.4	2435980	0	91.73	518	777
JP-44	727015.8	2436413	0	98.16	1728	2592
JP-45	726766.3	2436846	0	81.00	1555	2333
JP-46	726516.7	2437279	0	86.51	1555	2333

* All of these wells are producing from layer 1

Table 6.11: Optimal water supply solution for Al Kamil Wilayat (scenario-2)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	**$10^6 m^3$**	**$10^6 m^3$**	**%**	**$10^6 m^3$**	**%**	**$10^6 m^3$**	**%**	**$10^6 m^3$**	**%**
2010	1.39	1.39	100	0	0	1.39	100	0	0
2011	1.43	1.43	100	0	0	1.43	100	0	0
2012	1.47	1.47	100	0	0	1.47	100	0	0
2013	1.51	1.51	100	0	0	1.51	100	0	0
2014	1.54	1.54	100	0	0	1.54	100	0	0
2015	1.66	1.66	100	0	0	1.66	100	0	0
2016	1.74	1.74	100	0	0	1.74	100	0	0
2017	1.79	1.79	100	0	0	1.79	100	0	0
2018	1.84	1.84	100	0	0	1.84	100	0	0
2019	1.88	1.88	100	0	0	1.88	100	0	0
2020	1.94	1.94	100	0	0	1.94	100	0	0
2021	2.02	2.02	100	0	0	2.02	100	0	0
2022	2.01	2.01	100	0	0	2.01	100	0	0
2023	2.06	2.06	100	0	0	2.06	100	0	0
2024	2.17	2.17	100	0	0	2.17	100	0	0
2025	2.23	2.23	100	0	0	2.23	100	0	0
2026	2.31	0.00	0	2.31	100	2.31	100	0	0
2027	2.37	0.00	0	2.37	100	2.37	100	0	0
2028	2.44	2.44	100	0	0	2.44	100	0	0
2029	2.49	1.74	70	0.75	30	2.49	100	0	0
2030	2.49	2.49	100	0	0	2.49	100	0	0
Total demand ($10^6 m^3$)	**40.76**	**35.33**	**87% from total**	**5.43**	**13% from total**	**40.76**	**100% from total**	**0**	**0% from total**
Total cost (10^6 RO.)	**2.17**	**1.31**	**60% from total**	**0.86**	**40% from total**	**2.17**	**100% from total**	**0**	**0% from total**

Table 6.12: Optimal water supply solution for Sur Wilayat (scenario-2)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	**10^6m^3**	**10^6m^3**	**%**	**10^6m^3**	**%**	**10^6m^3**	**%**	**10^6m^3**	**%**
2010	4.60	0	0	2.56	55.7	2.56	55.7	2.04	44.3
2011	4.74	0	0	2.21	46.5	2.21	46.5	2.53	53.5
2012	4.86	0	0	1.83	37.6	1.83	37.6	3.03	62.4
2013	5.00	0	0	1.45	29.0	1.45	29.0	3.55	71.0
2014	5.11	0	0	1.05	20.6	1.05	20.6	4.06	79.4
2015	5.22	0	0	0.18	3.5	0.18	3.5	5.04	96.5
2016	5.75	0	0	0	0	0	0	5.75	100
2017	5.92	0	0	0	0	0	0	5.92	100
2018	6.07	0	0	0	0	0	0	6.07	100
2019	6.20	0	0	0	0	0	0	6.20	100
2020	6.41	0	0	0	0	0	0	6.41	100
2021	6.65	0	0	0	0	0	0	6.65	100
2022	6.64	0	0	0	0	0	0	6.64	100
2023	6.81	0	0	0	0	0	0	6.81	100
2024	7.16	0	0	0	0	0	0	7.16	100
2025	7.77	0	0	0	0	0	0	7.77	100
2026	7.89	0	0	0	100	0	100	7.89	100
2027	8.11	0	0	0	100	0	100	8.11	100
2028	8.32	0	0	0	0	0	0	8.32	100
2029	8.49	0	0	0	30	0	30	8.49	100
2030	8.68	0	0	0	0	0	0	8.68	100
Total demand (10^6m^3)	**136.4**	**0**	**0% from total**	**9.3**	**7% from total**	**9.3**	**7% from total**	**127.1**	**93% from total**
Total cost (10^6 RO.)	**27.34**	**0**	**0% from total**	**1.67**	**6% from total**	**1.67**	**6% from total**	**25.67**	**94% from total**

Table 6.13: Optimal water supply solution for North Wilayats (scenario-2)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	**10^6m^3**	**10^6m^3**	**%**	**10^6m^3**	**%**	**10^6m^3**	**%**	**10^6m^3**	**%**
2010	6.18	2.52	41	3.65	59	6.18	100	0	0
2011	6.37	2.49	39	3.87	61	6.37	100	0	0
2012	6.53	2.44	37	4.08	63	6.53	100	0	0
2013	6.71	2.40	36	4.30	64	6.71	100	0	0
2014	6.86	2.35	34	4.51	66	6.86	100	0	0
2015	7.33	2.25	31	5.08	69	7.33	100	0	0
2016	9.94	2.15	22	4.89	49	7.04	71	2.90	29
2017	10.23	2.12	21	4.76	47	6.88	67	3.35	33
2018	10.50	2.08	20	4.61	44	6.69	64	3.82	36
2019	10.73	2.03	19	4.43	41	6.46	60	4.27	40
2020	11.09	1.97	18	4.19	38	6.16	56	4.93	44
2021	11.51	1.87	16	3.84	33	5.72	50	5.79	50
2022	11.48	1.90	17	3.93	34	5.83	51	5.65	49
2023	11.77	1.86	16	3.76	32	5.62	48	6.14	52
2024	12.38	1.74	14	3.26	26	5.00	40	7.38	60
2025	13.44	1.69	13	2.92	22	4.61	34	8.83	66
2026	13.62	0.00	0	0.09	1	0.09	1	13.53	99
2027	14.01	0.00	0	2.52	18	2.52	18	11.49	82
2028	14.37	1.49	10	2.00	14	3.48	24	10.89	76
2029	14.66	0.00	0	1.65	11	1.65	11	13.01	89
2030	15.01	0.16	1	1.49	10	1.65	11	13.36	89
Total demand (10^6m^3)	**224.7**	**35.5**	**16% from total**	**73.8**	**33% from total**	**109.4**	**49% from total**	**115.3**	**51% from total**
Total cost (10^6 RO.)	**49.79**	**4.08**	**8% from total**	**9.16**	**19% from total**	**13.24**	**27% from total**	**36.55**	**73% from total**

Table 6.14: Optimal water supply solution for Ash Sharqiyah regional demand (Scenario 2)

	Total water demand	Source of the water supply							
		Groundwater						Sur Desalination Plant	
		Al Kamil Wellfield		Jaalan Wellfield		Total			
Year	**10^6m³**	**10^6m³**	**%**	**10^6m³**	**%**	**10^6m³**	**%**	**10^6m³**	**%**
2010	17.52	3.91	22	11.57	66	15.48	88	2.04	12
2011	18.05	3.92	22	11.60	64	15.52	86	2.53	14
2012	18.51	3.91	21	11.57	62	15.48	84	3.03	16
2013	19.02	3.91	21	11.57	61	15.48	81	3.55	19
2014	19.45	3.89	20	11.50	59	15.39	79	4.06	21
2015	20.52	3.91	19	11.57	56	15.48	75	5.04	25
2016	24.04	3.89	16	11.50	48	15.39	64	8.65	36
2017	24.75	3.91	16	11.57	47	15.48	63	9.27	37
2018	25.41	3.92	15	11.60	46	15.52	61	9.89	39
2019	25.94	3.91	15	11.57	45	15.48	60	10.47	40
2020	26.82	3.91	15	11.57	43	15.48	58	11.34	42
2021	27.84	3.89	14	11.50	41	15.39	55	12.44	45
2022	27.76	3.91	14	11.57	42	15.48	56	12.28	44
2023	28.47	3.92	14	11.60	41	15.52	55	12.95	45
2024	29.95	3.91	13	11.50	38	15.41	51	14.54	49
2025	31.92	3.92	12	11.40	36	15.32	48	16.60	52
2026	32.59	0.00	0	8.87	27	8.87	27	23.72	73
2027	33.52	0.00	0	11.54	34	11.54	34	21.98	66
2028	34.38	3.92	11	11.25	33	15.18	44	19.21	56
2029	35.09	1.74	5	11.10	32	12.84	37	22.25	63
2030	35.65	2.65	7	10.96	31	13.61	38	22.04	62
Total demand (10^6m³)	**557.2**	**70.8**	**13% from total**	**238.5**	**43% from total**	**309.3**	**56% from total**	**247.9**	**44% from total**
Total cost (10^6 RO.)	**83.81**	**5.39**	**6% from total**	**16.19**	**19% from total**	**21.58**	**26% from total**	**62.23**	**74% from total**

Table 6.15: Sensitivity of unit pumping production costs (R.O/ m³) for Scenario 1

Unit pumping cost variation	Total		Source of the water supply							
			Groundwater						Sur Desalination Plant	
	demand	cost	Al Kamil Wellfield		Jaalan Wellfield		Total			
	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO
Existing	557.2	98.8	72.1	5.5	161.9	6.2	233.9	11.7	323.3	87.1
-10%	557.2	94.1	72.1	4.9	161.9	5.5	233.9	10.4	323.3	83.7
-20%	557.2	89.5	72.1	4.4	161.9	4.9	233.9	9.2	323.3	80.2
+10%	557.2	103.5	72.1	6.0	161.9	6.8	233.9	12.8	323.3	90.7
+20%	557.2	108.1	72.1	6.5	161.9	7.4	233.9	13.9	323.3	94.2

Table 6.16: Sensitivity of desalination production costs (R.O/ m³) for Scenario 1

Unit pumping cost variation	Total		Source of the water supply							
			Groundwater						Sur Desalination Plant	
	demand	cost	Al Kamil Wellfield		Jaalan Wellfield		Total			
	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO
Existing	557.2	98.8	72.1	5.5	161.9	6.2	233.9	11.7	323.3	87.1
-10%	557.2	93.7	72.1	5.5	161.9	6.2	233.9	11.7	323.3	82.0
-20%	557.2	88.4	72.1	5.5	161.9	6.2	233.9	11.7	323.3	76.7
+10%	557.2	104.1	72.1	5.5	161.9	6.2	233.9	11.7	323.3	92.5
+20%	557.2	109.4	72.1	5.5	161.9	6.2	233.9	11.7	323.3	97.7

Table 6.17: Sensitivity of projected water demand for Scenario 1

Water demand variation	Total		Source of the water supply							
			Groundwater						Sur Desalination Plant	
	demand	cost	Al Kamil Wellfield		Jaalan Wellfield		Total			
	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO
Existing	557.2	98.8	72.1	5.5	161.9	6.2	233.9	11.7	323.3	87.1
-10%	501.5	83.7	72.1	5.6	161.9	7.1	233.9	12.7	267.6	71.0
-20%	445.8	69.6	72.1	6.0	161.9	8.5	233.9	14.5	211.9	55.1
+10%	612.9	114.3	72.1	5.2	161.9	5.5	233.9	10.7	379.0	103.6
+20%	668.6	130.1	72.1	4.9	161.9	5.1	233.9	10.0	434.7	120.1

Table 6.18: Sensitivity of unit pumping production costs (R.O/ m³) for Scenario 2

Unit pumping cost variation	Total		Source of the water supply							
			Groundwater						Sur Desalination Plant	
	demand	cost	Al Kamil Wellfield		Jaalan Wellfield		Total			
	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO
Existing	557.2	83.8	70.8	5.4	238.5	16.2	309.3	21.6	247.9	62.2
-10%	557.2	80.7	70.8	4.9	238.5	14.1	309.3	19.0	247.9	61.7
-20%	557.2	75.9	70.8	4.3	238.5	12.3	309.3	16.6	247.9	59.3
+10%	557.2	89.3	70.8	5.9	238.5	16.9	309.3	22.8	247.9	66.5
+20%	557.2	93.8	70.8	6.5	238.5	18.4	309.3	24.9	247.9	68.9

Table 6.19: Sensitivity of desalination production costs (R.O/ m³) for Scenario 2

Unit pumping cost variation	Total		Source of the water supply							
			Groundwater						Sur Desalination Plant	
	demand	cost	Al Kamil Wellfield		Jaalan Wellfield		Total			
	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO
Existing	557.2	83.8	70.8	5.4	238.5	16.2	309.3	21.6	247.9	62.2
-10%	557.2	81.7	70.8	5.4	238.5	16.2	309.3	21.6	247.9	60.1
-20%	557.2	76.8	70.8	5.4	229.2	13.7	300.0	19.1	257.2	57.7
+10%	557.2	85.8	70.8	5.4	238.5	16.2	309.3	21.6	247.9	65.2
+20%	557.2	93.8	70.8	5.4	238.5	16.2	309.3	21.6	247.9	72.2

Table 6.20: Sensitivity of projected water demand for Scenario 2

Water demand variation	Total		Source of the water supply							
			Groundwater						Sur Desalination Plant	
	demand	cost	Al Kamil Wellfield		Jaalan Wellfield		Total			
	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO	10^6 m³	10^6 RO
Existing	557.2	83.8	70.8	5.4	238.5	16.2	309.3	21.6	247.9	62.2
-10%	501.5	71.9	70.8	5.7	238.5	17.2	309.3	22.9	192.2	45.0
-20%	445.8	59.3	70.8	5.9	235.1	18.7	305.9	24.6	139.9	34.7
+10%	612.9	98.2	70.8	5.1	238.5	13.6	309.3	18.7	303.6	79.5
+20%	668.6	112.4	70.8	4.9	238.5	12.1	309.3	17.0	359.3	95.4

Table 6.21: Summary of the sensitivity analysis for the total Ash Sharqiyah water demand cost

% variation in the parameter	% variation in the total water demand cost due to variation in					
	Unit Pumping Cost for Scenario		Desalination Production Cost for Scenario		Projected Water Demand for Scenario	
	1	2	1	2	1	2
-10%	- 4.7	-3.7	-5.3	-2.5	-15.3	-14.2
-20%	-9.4	-9.4	-10.5	-8.4	-29.6	-29.3
+10%	+4.7	+6.6	+5.4	+2.4	+15.7	+17.2
+20%	+9.4	+11.9	+10.7	+11.9	+31.7	+34.1

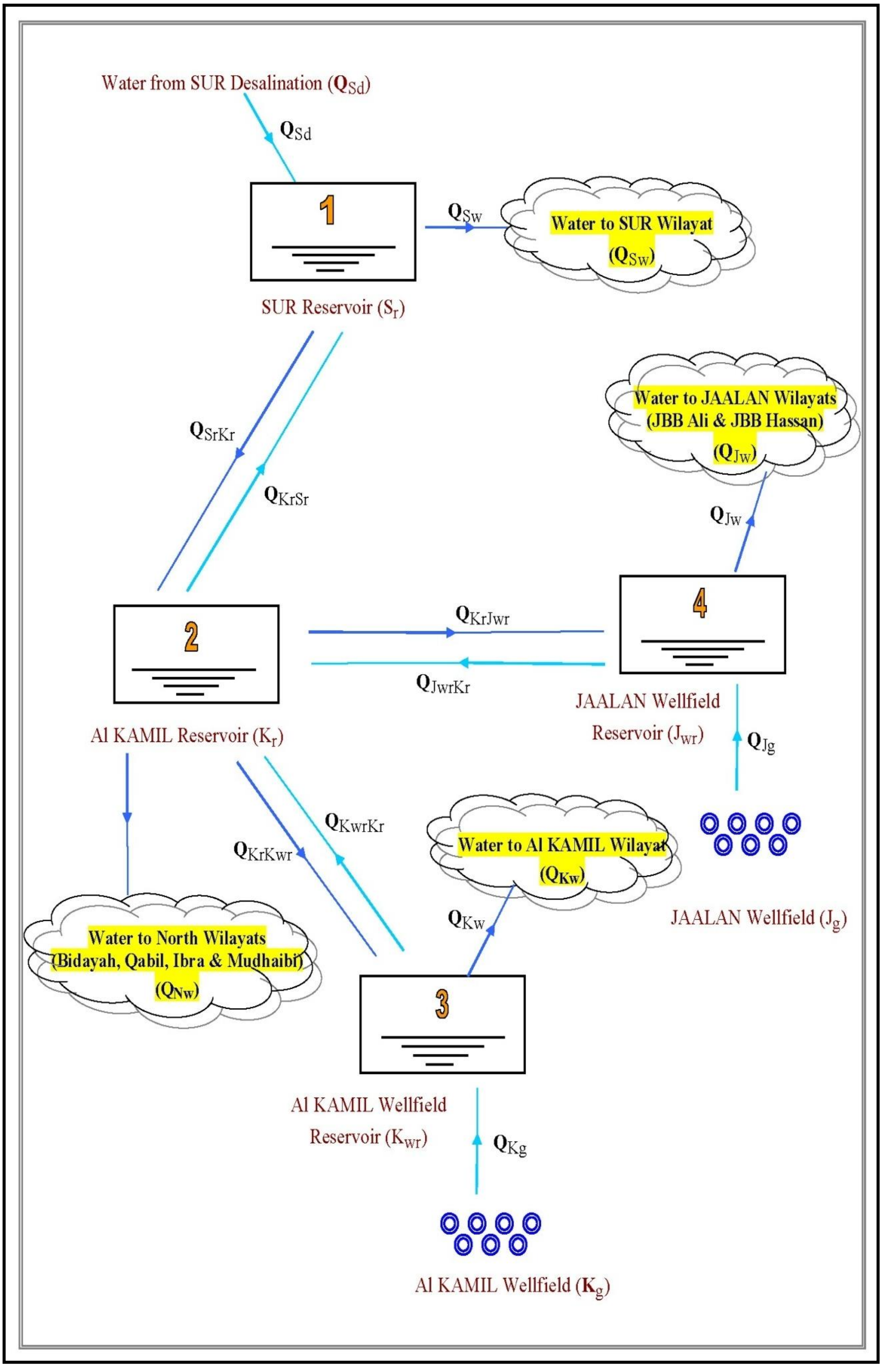


Figure 6.1: A schematic diagram of different pumping locations of the water supply system for Ash Sharqiyah region using desalinated water and groundwater

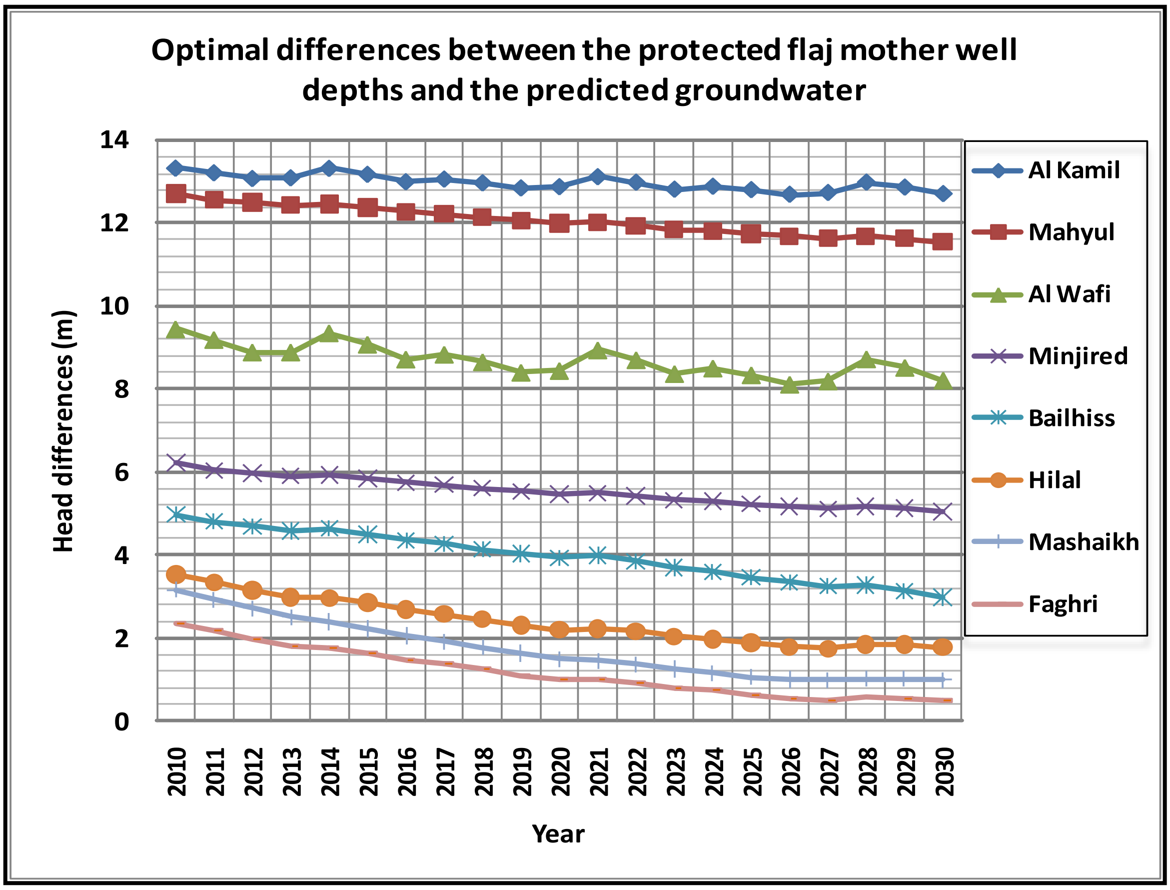


Figure 6.2: Optimal differences between the eight protected Flaj mother well depths and the predicted groundwater calculated up to 2030

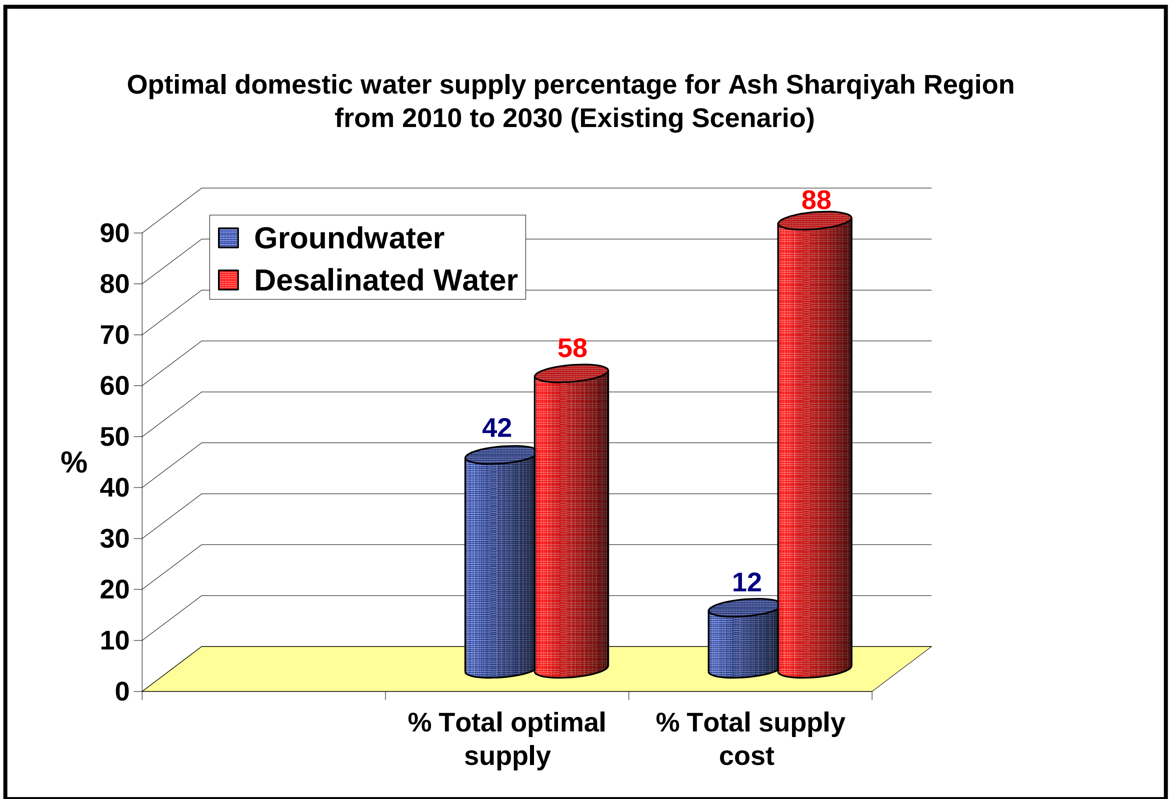


Figure 6.3: Optimal water supply solution for Ash Sharqiyah regional demand from groundwater and desalinated water for 20 years (2010 - 2030), (Existing Scenario)

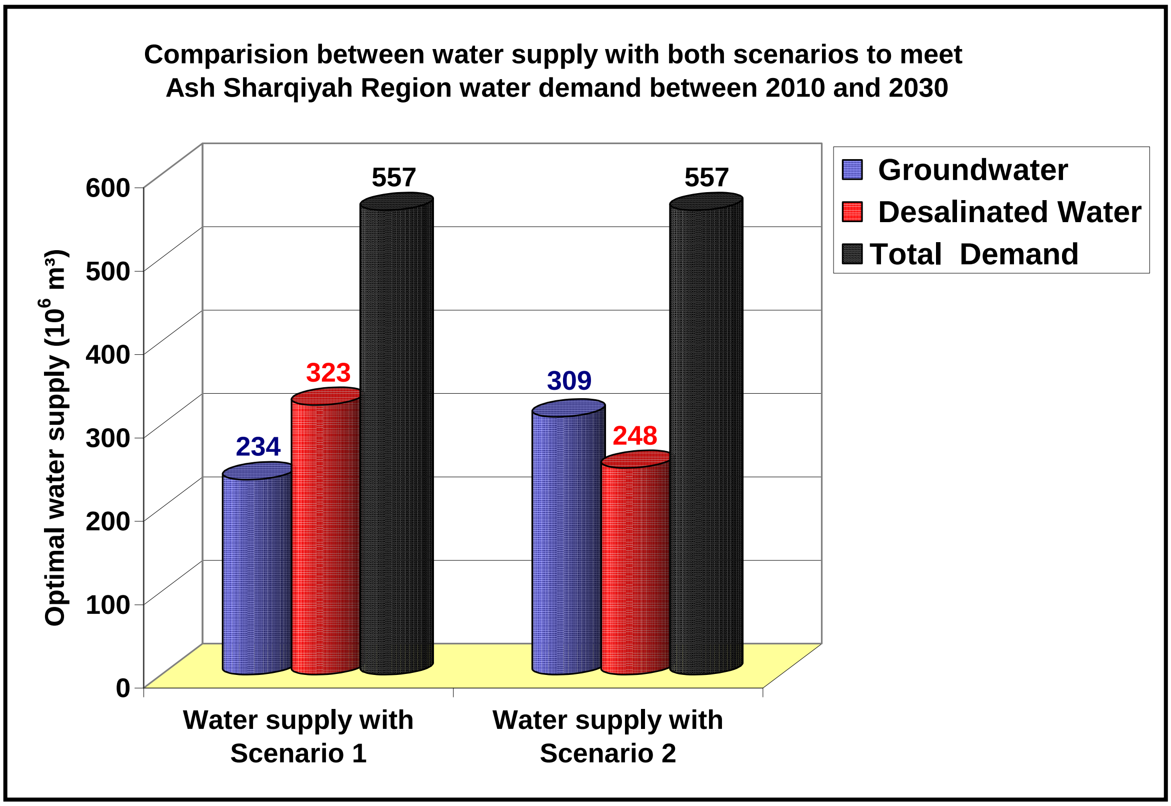


Figure 6.4: Comparison between water supply with the existing scenario 1 and with the recommended scenario 2 for Ash Sharqiyah regional demand for 20 years (2010 - 2030)

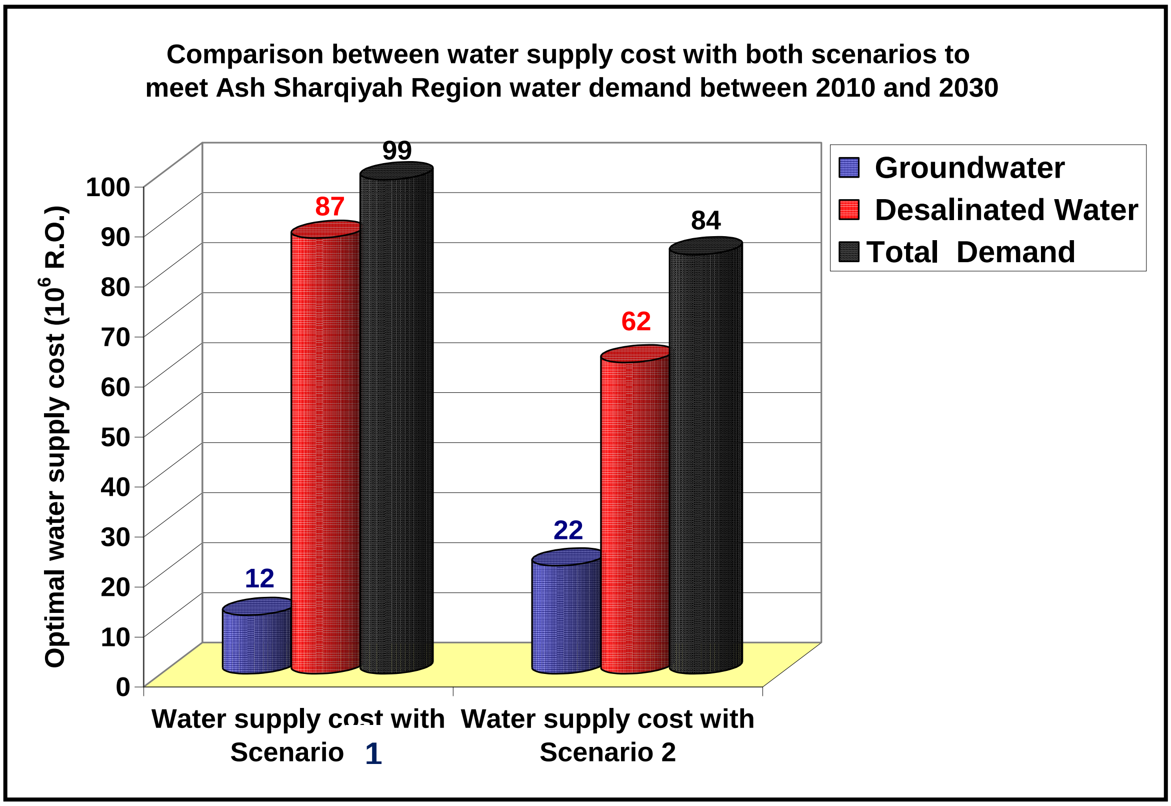


Figure 6.5: Comparison between water supply costs with the existing scenario 1 and with the recommended scenario 2 for Ash Sharqiyah regional demand for 20 years (2010 - 2030)

CHAPTER 7

DISCUSSIONS, CONCLUSIONS AND RECOMMENDATIONS

7.1 Discussions

Fresh groundwater resources are limited particularly in arid to semi-arid regions and at a premium in the Sultanate of Oman while their use is subjected to the competing needs of various activities. The government of Oman is very concerned about this situation because there is a continuous shortage of domestic water supply, which is threatening the development of the country. Consequently, in order to secure a sustainable, reliable potable water supply for the Sultanate, the government decided to use desalinated sea water for the sole purpose of supplementing the water available from groundwater sources. This conjunctive use of desalinated water and groundwater, it is hoped, will secure the long-term future water supply situation for most of the country. However, there is no clear management strategy was established yet. Therefore, this study was conducted taken Ash Sharqiyah Region as a case study to determine the optimum (minimum cost) water management strategy for the conjunctive use of both groundwater and desalinated water for domestic water supply up to the year of 2030.

Before discussing the sources of uncertainty and presenting the main conclusions of this work, it will be important first to review the original objectives and establish to what degree they have been achieved. The objectives as outlined in Chapter 1 were as follows:

1- Develop a groundwater simulation model to describe the existing conditions of the Ash Sharqiyah Sands Aquifer, and then apply the model for assessing the long-term impacts of current groundwater management strategies at the two existing wellfields.

2- Formulate a constrained optimization model for the conjunctive use of groundwater and desalinated water in the region that will have as its objective the minimization of the total production cost while meeting a number of environmental and physical constraints.

3- Develop a practical and reliable management model to couple the Ash Sharqiyah Sands Aquifer simulation model with the constrained optimization model in order to find optimal, acceptable, sustained water resources management solution to the water supply situation in the region.

4- Investigate through extensive sensitivity studies the impact of variations in various assumptions made on the developed optimal water management strategy for the region.

5- Make recommendations to policy makers and other stakeholders on the best conjunctive use strategy for water recourses development to meet the future Ash Sharqiyah Region domestic water demand.

Following an extensive review of the literature and other useful background information about the Ash Sharqiyah Sands Aquifer and Wadi al Batha Basin, the groundwater simulation development model was presented in Chapter 5. In order to design the simulation model, many geological and hydrological data from the study area were collected and analyzed as presented in Chapter 4. The management model to couple the Ash Sharqiyah Sands Aquifer simulation model with the optimization model was accomplished by using the fully embedded approach via the General Algebraic Modelling System software (GAMS) as presented in Chapter 6.

7.1.1 Sources of uncertainty

Despite the success recorded in this study, there is no disputing the fact that the management model's accuracy will depend on the accuracy of the data- hydraulic characteristics, economic data, water demand and abstraction data, etc. - that went into the model as well as on how detailed the conceptualisation and characterisation of the geo-hydrological processes had been. The deterministic optimisation carried out in this study has assumed that these factors were known relatively accurately. However, this may not be the case and as the subsequent sensitivity analyses carried out showed, any inaccuracies in some of these factors may result in large variations in the optimised water management plan. It is therefore important that these sources of uncertainties are recognised and their possible effects documented as outlined below.

1. Remote sensing was applied to assist to estimate water consumptions within the study area as discussed in Chapter 4. It has been used to evaluate the extent, density and water consumption of the natural woodlands (prosopis cineraria) which is a significant consumer of water in the model. It has also been used to estimate the area covered by Sabka (salty water) and the outflow to this Sabka. It has furthermore been used to evaluate regional vegetation analysis including differentiation of Aflaj- and non-Aflaj (wells) -fed agriculture which lead to estimate. All of these water abstraction estimations were considered as significant inputs for the simulation model. However, the accuracy of the remote sensing technology depends on the pixel size and its resolution. It would be more accurate if Aflaj- and non-Aflaj (wells) -fed agriculture were metered but that was not possible as no regulations has been implemented yet to do so and it would be costly practice. Furthermore, other abstraction rates from private wells and Aflaj were roughly estimated as discussed in Chapter 4 from the National

Well Inventory Project in 1995 and the Aflaj Inventory Project during 1997 and 1998.

2. Projected water demand to the year of 2030 was based on the Ministry of National Economy (MONE) population growth forecasts and the results of the 2003 Census and design criteria done by Parsons International & Co LLC (the designer consultant) as discussed in Chapter 4. Any uncertainty in the project demand is a big problem because as clearly revealed by the sensitivity analysis, variations in the abstraction rates produced the largest sensitivity in the optimised management plan. Given the current concerns about climate change and its effect on water demand, both domestic and agricultural, this is an issue that warrants more detailed independent studies.

3. The long-term rainfall record for Oman since 1895 has indicated that the rainfall pattern approximately repeats itself every seven years as discussed in Chapter 3. Therefore, the same recharge data events every seven years were adopted as recharge inputted for the simulation and optimization models starting from the year 2009 up to 2030. This assumption may not be valid due to climate changes and other effects, impacting on infiltration and recharge. However, percentage changes in recharge were tested in the sensitivity analysis and the results were were found to be relatively insensitive to changes in the minimal recharge that occurs in the region because of the low rainfall.discussed in simulation model in Chapter 5. It was concluded that the simulation model is relatively not sensitive to the range varied in recharge rate and incertitude within -20% to 20% will not affect the results of the model.

4. The predicted energy cost or operation and maintenance costs per unit pumping rates used for the optimization model were calculated based on discount and inflation rates provided by the consultant designer of the scheme, Parsons Intern. & Co LLC as discussed in Chapter 6. These showed slight variations in the unit costs between years 2009 and 2030 that ideally should be considered in the optimisation. However, since the discount and inflation rates used in arriving at the future unit costs are mere forecasts with their inherent uncertainties, incorporating the year-to-year variation of the costs directly in the optimisation may not be advisable. Consequently, the inter-annual variations in the costs were ignored and a constant value given by the average cost was used throughout. However, if rapid increase or decrease in energy cost happened in the future, there will be impact on pumping cost and thus on the overall conclusions of this study.

5. Geophysical survey provided only an approximate indication of the base of layer 2 as no drilled wells encountered the base of this layer yet. The aquifer thickness of this layer is consequently still largely unknown. However, based on the borehole geophysical logging as discussed in Chapter 5, it is thought that it could be up to 200m. Therefore, layer 2 was assigned a uniform thickness of 150m throughout the model domain for modelling purposes. This results in an increase in saturated thickness away from the highland front where the water table is quite deep (60m to 80m below ground surface). Therefore, a further future detailed study to update the simulation model based on deep drilling wells project in different parts of the aquifer could be conducted to penetrate the base of the complicated heterogeneous alluvium layer in order to map the actual thickness of the layer.

6. Heterogeneity in the hydraulic characteristic of the aeolianite (layer 1) aquifer was also ignored. While this had to be the case because of lack of data, it is unlikely to be a valid assumption. Furthermore, alluvium (layer 2) has a much more diverse litho-logical and hydraulical characteristic. Thus, unlike the horizontal extent of this layer which was reasonably well defined, the vertical extent is not well defined and must be approximated for model purposes as discussed earlier in point 2.

7. As discussed in Chapter 5, the modelled area was discretized into square grids of 500 m spacing, which were refined to a square grid of 250 m spacing at the stress areas (wellfields). The numerical finite difference solution adopted assumes that the hydraulic head is uniform within a given grid square. Whilst this is not a major problem in grid cells where there are no external stresses (i.e. well abstractions), it may not accurately describe the rapid drawdown caused by turbulence and well losses in the proximity of the pumped wells. To better model such effects, a much finer mesh, typically with spacing of the order of the diameter of the pumped well, would be required. However, this will cause the computation time to increase astronomically and may run the risk of causing instability of the numerical solution scheme. It is precisely to avoid such problems that a relatively coarse time interval of four months was adopted for the discretisation in the time domain for the unsteady state simulations. For the broad objective of developing an optimal, conjunctive groundwater-seawater desalination use strategy as implemented in the current study, such a “lumped” approach involving relatively coarse spatial and temporal discretisation scales should suffice. Nonetheless, a recommendation to investigate this assumption will be included in the suggestions for further work at the end of the thesis.

7.1.2 Significances and implications of the research findings to the wider field of knowledge for water management in arid regions

As stated earlier, continuously increasing water demand in various sectors is intensifying the water scarcity problem particularly in arid and semi-arid regions like Oman. Some countries, e.g. Libya, with similar climatic conditions despite their limited groundwater resources are still reliant solely on groundwater. Libya started in 1984 abstracting fossil water at a rate of 5.7 million m^3 per day from aquifers in the south to meet water demands in the north of the country where most of the population lives by developing the 4,000km-long Great Man-Made River Project (Shaki and Adeloye, 2007). These non-renewable aquifers are located in the arid desert where natural recharge is almost non-existent, thereby presenting similar challenges as in the Omani situation if abstractions continue at the current rate. Other arid to semi-arid countries such as the Gulf States depend on the sea desalinated water as strategic domestic water supply without fully exploring the potential of their groundwater resources. Desalination is expensive and energy intensive; hence it cannot realistically be the sole source of drinking water in the arid region. Rather, an optimal conjunctive use of groundwater and desalination as demonstrated by this research could be the best way in arid and semi-arid regions to meet water demands while ensuring the sustainability of the groundwater resources. This study therefore has tested successfully this promising water supply management approach by applying it as case study for Ash Sharqiyah Region in Oman combining the use of optimization techniques, hydrogeology, groundwater modelling, and system cost analysis. The following are seen as greatest significances and implications of the research findings to the wider field of knowledge for water management in arid regions:

1. Traditionally, proper management of groundwater-surface water interaction has been relied upon to ensure that available water resources continue to meet

demands. However, for most arid and semi-arid areas, fresh surface water resources are scarce or non-existent, which means groundwater is over-exploited causing pollution such as salt intrusion and other environmental problems. This study has successfully demonstrated a feasible option for achieving the protection of groundwater resources in regions having no freshwater resources through the use of desalinated water. This will have ramifications for future research on groundwater management in arid regions of the world.

2. The study has also shown how optimization techniques, hydrogeology, groundwater modelling, and system cost analysis and environmental considerations can be integrated to produce optimal strategy for groundwater management. It is a truism that previous studies have been limited to surface water-groundwater considerations and have almost always paid short-shrift to environmental considerations.

3. This study has successfully demonstrated a feasible option for achieving the protection of groundwater resources in regions having no freshwater resources through the use of desalinated water. The hybrid simulation-optimisation model is flexible enough to accommodate most of the practical possible scenarios related to the use of groundwater and desalinated water for domestic uses and could be adapted in other arid to semi-arid regions as a viable strategy for managing groundwater resource depletion

4. The research provides water resources managers with a valuable management tool with many constraints to determine the “optimal” long-term strategy for developing their limited groundwater resources by blending it with desalinated sea water in such a way that the aggregated cost is minimal.

5. This management model could be adapted in other arid to semi-arid regions to avoid creating extensive drawdown of aquifers and its consequent negative environmental impact.

6. Although recently the cost of domestic water supply from sea desalinated water is more expensive than groundwater, almost three times in Oman, it is quite likely with the development of new desalination technology in future the cost of desalination may be reduced or become cheaper than abstracting groundwater. The developed water supply management model of this research is flexible enough to accommodate these changes in cost to produce different management scenarios as demonstrated by the outcome of the model in the scenario 2 as discussed in Chapter 6.

7. Although including the demand management as part of this management model is beyond the scope of this work, it must be stressed that managing the domestic water supply can not be implemented alone without considering the demand management in order to save every drop of water, build an awareness of, and continual concern about, water conservation into every aspect of life by increasing efficiency in water supply and water usage and promoting water.

7.2 Conclusions

From the above, it is clear that all the objectives set out in Chapter 1 for the study have been achieved. From the entire study, the following specific conclusions were obtained:

1. The study area can be considered as two layered aquifer systems based on geophysical and other data collected. These are referred to as layer 1 (or the

aeolianite) and layer 2 (or the alluvium). Layer 1 is litho-logically homogenous and unsaturated along part of its northern boundary, but reaches a maximum saturated thickness of approximately 100m in the south east. On the other hand, layer 2 is more complex. It comprises a diverse group of sediments which underlie layer 1. Geophysical survey provided only an approximate indication of the base of layer 2 as no drilled wells encountered the base of this layer yet but based on the borehole geophysical logging; it is thought that it could be up to 200m. Therefore, layer 2 was assigned a uniform thickness of 150m throughout the model domain for modelling purposes.

2. It has been possible to effectively model the Ash Sharqiyah Sands Aquifer as unconfined aquifer. Calibration and validation of the model at both steady and transient states revealed that the MODFLOW model was capable of reproducing the hydraulic heads in the two layered aquifer systems accurately.

3. Hydraulic conductivity and specific yield distribution zones had to be used to capture the heterogeneity of layer 2. However, due to lack of sufficient information, layer 1 was considered homogeneous with respect to both the hydraulic conductivity and specific yield

4. The numerical simulation model of Ash Sharqiyah Sands Aquifer was used to assess the long-term impacts of supplying the eight Wilayats of Ash Sharqiyah Region with water from the 29 operational wells of the two groundwater wellfields by predicting the long-term behaviour until 2030 of the piezometric heads. The results show that the drawdown will reach a maximum of approximately 12 m at the Al Kamil Wellfield by the end of 2030 to deliver the required domestic water demand at the eight operational wells of Al Kamil

Wellfield. This drawdown, however, will not effect the production from all of the eight operational wells as it will be above the pump installation depth assuming rainfall and other hydrological conditions within the basin remain as assumed. On the other hand, the drawdown will reach a maximum of approximately 55 m at the Jaalan Wellfield at the end of 2030. Unlike Al Kamil Wellfield, however, the drawdown at the Jaalan Wellfield will effect the production from 16 operational wells out of 21 wells as it will be below the pump depths. As there are more production wells in the Jaalan Wellfield, the results show that the drawdown is more and very distinguished in the area of Jaalan Wellfield compared to the one of Al Kamil Wellfield.

5. Evaluation of the long-term water demand projection for the Wilayats confirmed the insufficiency of the two wellfields to meet the projected long-term demands. The need for conjunctive use with desalinated water was clear from the simulation model because the two wellfields could not be considered alone as a sustainable option for meeting the long-term water needs without affecting the sustainability of the Aflaj deriving their flows from the aquifers. Thus, supplementing the abstraction from the well fields with desalinated water of the Sur Desalination Plant offered the prospect for combating the future water demands after the 1st of September 2025 to meet the domestic water supply needs for the eight Wilayats of Ash Sharqiyah Region without creating extensive drawdown and avoiding negative impact on existing operational Aflaj and environment.

6. The constrained optimization problem formulated with its objective function being the minimization of total cost of meeting the water demand up to the year 2030 was successfully solved to provide optimal blend of groundwater and

desalinated water for the Region. The constraints ranged from maintaining a minimum level of water in the wells that ensure that the Aflaj flows continuously, to meeting the total water demand for domestic, agriculture and industrial purposes.

7. The results of the optimization revealed increasing contribution of desalination water in later years as groundwater becomes depleted and the risk of drying out Aflaj becomes greater. The water supply by the existing scenario will begin with 66% from the two wellfields and only 34% from Sur Desalination Plant in 2010. However, due to the significant drawdown in the wellfields caused by this abstraction and head constraint at Flaj Faghri mother well, the percentage supplied by wellfields will decrease to 29% by 2030 with most of the supply (71%) by that year being provided by the Sur Desalination Plant. Ash Sharqiyah Region will require approximately $557x10^6 m^3$ of domestic water costing approximately $99x10^6$ RO over the 20 years from 2010 to 2030 of this $234 \, x10^6 m^3$ costing only $12 \, x10^6$ RO will be provided by the two wellfields and $323x10^6 m^3$ costing $87 \, x10^6$ RO will be supplied from Sur Desalination Plant. This reflects the huge cost of desalination relative to fresh groundwater system; indeed for the Ash Sharqiyah Region supply, it is costing 5.3 times more to supply desalination water than the fresh water from the two wellfields.

8. Sensitivity analysis of the parameters and assumptions were carried out to establish how robust the results of both the simulations and optimization are to these factors. It was found that the simulation model was relatively not sensitive within -20% to 20% to the recharge rate, hydraulic conductivity and specific yield indicating that the values used in the simulation model were determined very accurately. The model was also found to be relatively not sensitive to the

boundary condition especially for layer 2 when its boundary condition was changed from being constant heads to be general head for both layers. However, the simulation model is sensitive (-/+6.4%) to the +/-20% variation in abstraction. It was also found that the inflow and the outflow from model boundaries as influenced by the abstractions have a greater influence on the model behaviour than changes to the recharge, boundary conditions, hydraulic conductivity or specific yield. The variation in water projected demand in the optimization model showed to be a more sensitive parameter than the unit pumping production cost or desalination production cost. The overall optimal cost of water to meet Ash Sharqiyah Regional demand over the 20 years from 2010 to 2030 can vary from up to -30% or + 34% due to decrease or increase in projected water demand by - 20% or +20% respectively. The other two parameters almost showed similar sensitivity measured approximately – 10% or +10% by decreasing or increasing in the unit pumping production cost or desalination production cost by - 20% or +20% respectively.

9. While some of the Aflaj were drying out, it was also clear that some were barely affected at all by the groundwater pumping principally because the concerned wells were located downstream of major Aflaj. This suggests potential for increasing the pumping abstractions from some of the wells, which was investigated by the optimization. The results show that pumping capacities at the wells of the Jaalan Wellfield can be increased by 50% of their current size (Scenario 2) leading to a significant reduction in the water demand from the Sur Desalination Plant and hence the overall scheme cost. The groundwater supply contribution by the recommended scenario 2 to meet Ash Sharqiyah regional demand over the 20 years from 2010 to 2030, will increase from being

$234 \times 10^6 m^3$ (equivalent to 42%) to $309 \times 10^6 m^3$ (equivalent to 56%). Thus, Sur Desalinated Plant supply contribution will reduce from being $323 \times 10^6 m^3$ (equivalent to 58%) to $248 \times 10^6 m^3$ (equivalent to 44%). Subsequently, the total optimum cost will be reduced from 99×10^6 RO to 84×10^6 RO.

7.3 Recommendations for further researches

Despite the success recorded in this study, there are certain aspects which have been identified that would benefit from further investigations. Therefore, the following are suggested as areas for further work:

1. The alluvium layer-2 was assigned a uniform thickness of 150m throughout the model domain for modelling purposes because the base of this layer could not be encountered. Therefore, a further future detailed study to update the simulation model based on deep drilling wells project in different parts of the aquifer could be conducted to penetrate the base of the complicated heterogeneous alluvium layer in order to map the actual thickness of the layer.

2. A source of uncertainty was ignoring the heterogeneity in the hydraulic characteristic of the layer 1 aquifer because of lack of data. More detailed monitoring; including further pumping test of layer 1 should be carried out to redress this problem.

3. The modelled area was discretized into coarse square grids of 500 m spacing, which were refined to a square grid of 250 m spacing at the stress areas (wellfields), it is recommended to test the assumption of finer spacing discretisation.

4. An average constant flow for each different Flaj was used at each stress period although in reality it might decrease slightly with pumping time. Therefore, a further future detailed study could be conducted to know this Aflaj flow and in practically water table dynamic at each different stress period near mother well and near Flaj tunnel area.

5. Some of the hydro-metrological conditions assumed for the simulation and optimization works, e.g. rainfall and evapotranspiration, have ignored the possible effects of climate change on them. While the assumption of the impacts of climate change is beyond the scope of this work, it is quite likely based on scientific evidence published by the IPCC that the future rainfall and evapotranspiration will be impacted by climate change (IPCC, 2007). If this happened, there will be impact on infiltration, water demands in both domestic and agricultural needs and thus on the overall conclusions of this study. A follow-on study to investigate these impacts would be useful.

6. The management model could be used for other similar regions in Oman where conjunctive uses are applicable for domestic water supply with minor changes with respect to their hydro-geological inputs and water demands. Furthermore, other Gulf States which have similar hydro-geological characteristics and groundwater scarcity or these countries using surface water and groundwater as water supply could adapt this model technique without major changes.

7. Finally, partly as a way of addressing (4) above, the optimal water management model for Ash Sharqiyah Region domestic water supply using both desalinated and groundwater should be updated at least every ten years based on the actual water demands for the region.

REFERENCES

Abdalla, O. A. E. (2008) Groundwater discharge mechanism in semi-arid regions and the role of evapotranspiration. *Hydrological Processes*, 22: 2993–3009.

Adeloye, A.J. and Montaseri, M. (2002) Preliminary streamflow data analyses prior to water resources planning study. *Hydrological sciences-Journal –des sciences hydrologyiques*, 47(5), 679-692.

Aguado, E. and Remson, I. (1976) Groundwater hydraulics in aquifer management. *Journal of Hydraulics Division*, ASCE 100(HY1), 103-119.

Ahlfeld, D. P and Mulligan, A. E. (2000) Optimal management of flow in groundwater systems. Academic press. 185pp.

Ahlfeld, D. P and Riefler, R. G (1999) Documentation for MODOFC: A Program for Solving Optimal Flow Control Problems Based on MODFLOW Simulation. Version 2, 11.

Aldar Consultancy (2006) Water metering technical and financial feasibility study final report, 165pp, MRMEWR, Muscat , Sultanate of Oman.

Al Shaqsi, S. (1996) Aflaj management in the Sultanate of Oman: case study of Flaj Al Hamra and flaj Kasfah. M.Sc. Dissertation, University of Wales, Bangor, unpublished.

Anderson, M.P, and Woessner, W.W (1992) Applied groundwater modelling simulation of flow and advective transport. San Diego: Academic Press, Inc.

Bahremand, A. and De Smedt, F. (2008) Distributed hydrological modeling and sensitivity analysis in Torysa Watershed, Slovakia. *Water Resource Manage*, 22, 393-408.

Barlow, J.R.B., and Clark, B.R. (2011) Simulation of water-use conservation scenarios for the Mississippi Delta using an existing regional groundwater flow model: U.S. Geological Survey Scientific Investigations Report 2011–5019, 14 p.

Binnie and Partners overseas Ltd (2000) National water resources master plan. Ministry of Water Resources, Muscat, Sultanate of Oman.

Boronina, A., Renard, P., Balderer, W. and Christodoulides, A. (2003) Groundwater resources in the Kouris catchment (Cyprus): data analysis and numerical modeling, *Journal of hydrology*, 271, 130-149.

Bredehoeft, J.D. and Young, R.A. (1970) Temporal allocation of groundwater: a simulation approach. *Water Resources Research*, 6(1), 3-29.

Brooke, A., Kendrick, D., Meeraus, A. (1988) GAMS: a user's guide. The Scientific Press, California, 289pp.

Brown, K. (1988) Ecophysiology of prosopis cineraria in the Wahiba Sands with reference to its re- afforestation potential in Oman. *Journal of Oman Studies*, 3, 257-270.

Century Architects (2007) Groundwater modeling of the Nejd aquifers final report, Muscat, Sultanate of Oman.

Chebaane, M. (1994) Springs in the Sultanate of Oman. Ministry of Water Resources (MWR), Muscat, Sultanate of Oman, unpublished, 150pp.

Chebaane, M. (1996) Mean annual rainfall isohyets and mean annual rainfall yield in the Sultanate of Oman. Ministry of Regional Municipalities Environment and Water Resources (MRMEWR), Muscat, Sultanate of Oman, unpublished.

Clarke, M.H. (1990) Oman's geological heritage, Petroleum Development of Oman, Muscat, Sultanate of Oman.

Cooper, H.Jr. (1966) The equation of groundwater flow in fixed and deforming coordinates. Geophysics Resources 100(HY5), 667-687.

Das, A., and Datta, B. (2001) Application of optimization techniques in groundwater quantity and quality. *Sadhana*, 26(4), 293-316.

Deninger, R.A. (1970) Systems analysis of water supply systems. Water Resources Bulletin 6(4), 573-579.

Don N.C., Araki H., Yamanishi H. and Koga K. (2005) Simulation of groundwater flow and environmental effects resulting from pumping. *Environmental Geology*, 47, 361-374.

Dr. Ahmed Abdel Warith & Partners LLC (1999) A consultancy services agreement to carry out the engineering design and supervise the implementation for Ash Sharqiyah Sands Aquifer Project, Muscat, Sultanate of Oman.

El Harrouni, K., Ouazar, D., Walters, G. A. and Cheng, A. H. -D. (1996) Groundwater optimization and parameter estimation by genetic algorithm and dual reciprocity boundary element method. Engineering Analysis with Boundary Elements 18(4), 287-296.

EMRL (2004) Groundwater modeling system GMS 5.0 tutorials. Environmental Modeling Research Laboratory. Brigham Young University.

Faust, C.R., and Mercer, J.W. (1980) Groundwater modeling: numerical models. Groundwater, 18(4), 395-409.

Freeze and Cherry (1979) Groundwater. Prentice Hall Inc. Englewood Cliffs, NJ, USA, 604pp.

Freeze, R.A. (1971) Three dimensional, transient, saturated-unsaturated flow in a groundwater basin. *Water Resources Research*, 7, 347-366.

Frind, E.O. and Verge, M.J. (1978) Three-dimensional modeling of groundwater flow system. *Water Resources Research*, 14(5), 844-856.

Gardner, R.A.M. (1988) Aeolianites and marine deposits of the Wahiba Sands. *Journal of Oman Studies*, 3, 55-60.

Geo-Resources (2001) Final report for inventory of cisterns (Birkat) in the Musandam Governorate, Muscat, Sultanate of Oman.

Geo-Resources (2005) Drilling & Aquifer Testing Project in the Western Al Wusta Desert, final report, Muscat, Sultanate of Oman.

Gharbi, A (1991) Optimal groundwater quantity and quality management with application to the Salt Lake Valley. Utah State University, Utah, USA.

Glennie, K.W. (1970) Desert sedimentary environment. Developments in sedimentology 14. Elsevier, Amsterdam, Holland.

Glennie, K.W. (1988) The geological setting of the Wahiba Sands of Oman. *Journal of Oman Studies*, 3, 65-70.

Gordu, F., Yurtal, R. and Motz, H (2001) Optimization of groundwater use in the Goksu Delta at Silifke in Turkey. First international conference on saltwater intrusion and coastal aquifers-monitoring modeling and management. Essaouira, Morocco.

Gorelick, S.M. (1982) A model for managing sources of groundwater pollution. *Water Resources Research*, 18, 773-781.

Gorelick, S.M. (1983) A review of distributed parameter groundwater management modeling methods. *Water Resource Research*, 19(2), 305-319.

GULFAR / SADE consortium (2001) Ash Sharqiyah Sands main water supply system, contract no. 00-4757, Muscat, Sultanate of Oman.

Hallaji, K. and Yazicigil, H (1996) Optimal management of a coastal aquifer in southern Turkey, *Journal of Water Resources Planning and Management*, ASLE, 122, 233-244.

Harbaugh, A.W., Banta, E.R., Hill, M.C, McDonald, M. G. (2000) Modflow-200 modular groundwater model – user guide to modularization concepts and the groundwater flow process. US Geological Survey Open-File Report 00-92, 121 pp.

Harbaugh, A.W., McDonald, M.G. (1996) Programmer's documentation for MODFLOW-96: an update to the US Geological Survey modular finite-difference groundwater flow model. US Geological Survey Open-File Report 96-486, 220 pp.

Hill, M.C. (1990) Preconditioned conjugate-gradient 2 (PCG2), a computer program for solving ground-water flow equations: U.S. Geological Survey Water-Resources Investigations Report 90-4048, 43 pp.

Hornberger, G. M., Raffensperger, J. P., and P. L. Wilberg (1998) Elements of physical hydrology.The Johns Hopkins Press, Baltimore, 302 p.

IPCC, 2007. Climate changes 2007: Synthesis Report. Contribution of Working Groups I, II and III to the Fourth Assessment Report of the Intergovernmental Panel on Climate Changes [Core Writing Team, Pachauri, R.K and Reisinger, A.(eds.)]. IPCC, Geneva, Switzerland, 104pp.

Jain, A., Srinivasalu, S., Bhattacharjya, R.K. (2005) Determination of an optimal unit pulse response function using real-coded genetic algorithm. *Journal of Hydrology*, 303,199-214.

Katsifarakis, K. L., Karpouzos, D. K. Theodossiou, N. (1999) Combined use of BEM and genetic algorithms in groundwater flow and mass transport problems. *Engineering analysis with boundary elements*, 23, 555-565.

Klammt, F. (2004) Financial management for facility managers. Facility design and management handbook. McGraw-Hill, Inc. Hightstown, N.J 08520, USA.

Liu, S., Konstantopoulou, F., Gikas, P. and Papageorgiou, L.G. (2011) A mixed integer optimization approach for integrated water resources management. *Computers & chemical engineering*, 35(5), 858-875.

Maizels, J.K. and Anderson, E. (1988) Surface water in the Sharqiyah flash floods February-March 1986. Journal of Oman Studies, special report No.3. Scientific results of the Royal geographical society's Oman Wahiba Sands project 1985-1987, 217-230.

Mandle, R.J. (2002) Groundwater modeling guidance. Michigan department of environmental quality, Draft 1.0, 54pp.

Mays, L.W., and Tung, Y.K (1992) Hydrosystems engineering & management. McGraw-Hill, Inc. Hightstown, N.J 08520, USA.

McDonald, M.G. and Harbaugh, A. W. (1988) A modular three-dimensional finite-difference groundwater flow model. Techniques of Water Resources Investigations, Book 6. U.S. Geological Survey, Virginia, USA.

MCI (1978) Oman standard No8-drinking water. Ministry of Commerce and Industry, Muscat, Sultanate of Oman, 4pp.

McPhee, J. and Yeh, W. W-G. (2004) Multi-objective optimization for sustainable groundwater management in semiarid regions. *Journal of Water Resources Planning and Management*, ASCE, 130 (6), 490-497.

MHEW (2004) Water supply to Ibra, Al Qabil and Bidyah wilayats: tender No. 31/2004. Ministry of Housing, Electricity and Water, Muscat, Sultanate of Oman.

MONE (2003) General census of population, housing and establishments-2003 for the Sultanate of Oman (in Arabic). Ministry of National Economy, Muscat, Sultanate of Oman.

Montgomery, D. C. and Runger, G, C. (2003). Applied statistics and probability for engineers., 3rd Edition, John Wiley & Sons Inc., New York, NY, USA.

Morel, E.H. and Birtles, A.B. (1983) Optimization of groundwater abstraction from a coastal aquifer. *Hyrological sciences journal*, 28:1, 169-182.

Motz, L. H., Gordu, F. and Yurtal, R. (2001) Simulation and optimization of groundwater use in coastal areas. American Geophysical Union, Fall meeting 2001, H12D-0325

Mott MacDonald International (1991) National water resources master plan. Volume3, water resources modeling, MWR, Muscat, Sultanate of Oman.

Mott MacDonald (1997) Ash Sharqiyah sands aquifer development scheme: pre-feasibility study, MWR, Muscat, Sultanate of Oman.

Mott MacDonald (2007) Sur independent water project. Volume1, Sharqiyah Desalination Company SAOC, Muscat, Sultanate of Oman.

MRMEWR (2001) Aflaj inventory project summary report, Ministry of Regional Municipalities Environment and Water Resources (MRMEWR), Muscat, Sultanate of Oman, 49pp.

MRMEWR (2004-2009) Ash Sharqiyah Sands Main Water Supply System Annual Water Production Reports (6), MRMEWR, Muscat, Sultanate of Oman.

MRMEWR (2005) Water Resources in Oman, MRMEWR, Muscat, Sultanate of Oman, 137pp.

MRMEWR (2006a) Dams in the Sultanate of Oman, MRMEWR, Muscat, Sultanate of Oman, 83pp.

MRMEWR (2006b) Wadi Dayqah dam and water supply scheme to Muscat and Quriyah, 43pp, MRMEWR, Muscat , Sultanate of Oman.

MRMEWR (2006c) Ash Sharqiyah water distribution system Asilah and al Gwabi development scheme: contract No. (7/2006). MRMEWR , Muscat, Sultanate of Oman.

MRMWR (2008) Aflaj Oman in the world heritage list, MRMEWR, Muscat, Sultanate of Oman, 51 pp.

MRMWR (2010) Water resources management in Oman (unpublished report), Ministry of Regional Municipalities and Water Resources (MRMWR), Muscat, Sultanate of Oman, 51pp.

Murtagh, B.A. and Saunders, M.A. (1982) A projected langrangian and its implementation for sparse nonlinear constraints, mathematical programming study 16, algorithms for constrained minimization of smooth nonlinear functions, pp 84-117.

MWR (1996a) National well inventory, catchment technical report (catchments 59 and 60). Final internal report, MWR, Muscat, Sultanate of Oman.

MWR (1996b) Hydrochemistry of Wadi Al Batha, Sharqiyah Region, (2) volumes, March 1996, MWR, Muscat, Sultanate of Oman.

MWR (1997a) Sharqiyah Sands Aquifer Assessment, Vol. 1 main report, MWR, Muscat, Sultanate of Oman.

MWR (1997b) Sharqiyah Sands Aquifer Assessment, Vol. 3 main report, appendix 3: drilling investigation, MWR, Muscat, Sultanate of Oman.

MWR (1997c) Sharqiyah Sands Aquifer Assessment, Vol. 2 main report, appendix 1: surface geophysics, MWR, Muscat, Sultanate of Oman.

MWR (1997d) Sharqiyah Sands Aquifer Assessment, Vol. 2 main report, appendix 2: borehole geophysics, MWR, Muscat, Sultanate of Oman.

MWR (1997e) Sharqiyah Sands Aquifer Assessment, Vol. 4 main report, appendix 4: aquifer testing, MWR, Muscat, Sultanate of Oman.

MWR (1997f) Sharqiyah Sands Aquifer Assessment, Vol. 6 main report, appendix 6: groundwater sampling, analysis, and portability assessment, MWR, Muscat, Sultanate of Oman.

MWR (1997g) Sharqiyah Sands Aquifer Assessment, Vol. 5 main report, appendix 5: hydrological analysis of the Wadi al Batha basin, MWR, Muscat, Sultanate of Oman.

MWR (1997h) Sharqiyah Sands Aquifer Assessment, Vol. 6 main report, appendix 7: remote sensing, MWR, Muscat, Sultanate of Oman.

MWR (1997i) Sharqiyah Sands Aquifer Assessment, preliminary numerical modelling, MWR, Muscat, Sultanate of Oman, 32pp.

MWR (1998) Dams in the Sultanate of Oman, MWR, Muscat, Sultanate of Oman.

MWR (2000) Statistics and tables of aflaj in Sultanate of Oman: national aflaj inventory report, in Arabic, MWR, Muscat, Sultanate of Oman.

Nabi, A., Gallarda, A.H. and Ahmed S. (2011) Optimization of a groundwater monitoring network for a sustainable development of the Maheshwaram Catchment in India. *Sustainability,* 3, 396-409.

Olsthoorn, T. N. (1999) A comparative review of analytic and finite difference models used at the Amsterdam Water Supply. *Journal of Hydrology*, 226, 139-143.

Parsons International & Co LLC (2005) Final design report, water supply schemes to Al Sharqiyah region. Volume1-report, 71pp, Ministry of Housing, Electricity and Water, Muscat, Sultanate of Oman.

Peaceman, D.W. and Rachford.Jr, H.H. (1955) The numerical solution of parabolic and elliptic differential equations. Society Indian Applied. Math 3(1), 28-41.

Peralta, R.C. and Datta, B. (1990) Reconnaissance-level alternative optimal groundwater use strategies. *Journal of Water resources planning and management Division*., ASCE, 116(5), 676-692.

Pezeshk, S., Helweg, O. J and Oliver, K. E. (1994) Optimal operation of groundwater supply distribution systems. *Journal of water resources planning and management*, 120 (5), 573-586.

Pool, D.R., Blasch, K.W., Callegary, J.B., Leake, S.A., and Graser, L.F. (2011) Regional groundwater-flow model of the Redwall-Muav, Coconino, and alluvial basin aquifer systems of northern and central Arizona. U.S. Geological Survey Scientific Investigations Report 5180, 101 pp.

Prasad, T.D., Park, N.J., (2004) Multi-objective genetic algorithm for design of water distribution networks. *Journal of Water Resources Planning and Management division*, ASCE, 130 (1), 73–82.

Psilovikos, A. A. (1999) Optimization models in groundwater management, based on linear and mixed integer programming. An application to a Greek hydrogeological basin. Phys. Chem. Earth (B) 24, 139 –144.

Pye, K. (1984) Models of transgressive coastal dune building episodes and relationship to Quaternary sea level changes, a discussion with reference to eastern Australia. In Clark M (ed). Coastal Research, UK. Perspective, 81-104. Norwich: GeoBooks.

Rosenthal, R. E. (2008) GAMS – a user's guide. GAMS Development Corporation, Washington, DC, USA.

Safavi R.H., Darzi F. and Mariño M.A. (2009) Simulation-Optimization modelling of conjunctive use of surface water and groundwater. *Water Resources Management*, 24, 1965-1988.

Seneviratne, A.A. (2007) Development of steady state groundwater flow model in lower Walawa Basin in Seri Lanka. Integrating GIS, remote sensing and numerical groundwater modelling. Unpublished Msc Thesis submitted to the international institute for geology information science and earth observation enschede. The Netherlands 70pp.

SEPA (2006) Modelling methods for groundwater abstractions, version 2, 28pp, Scottish Environment Protection Agency, Edinburgh, Scotland, UK.

Shaki. A.A. and Adeloye. A.J. (2007) Mathematical modeling of effects of Irawan irrigation project water abstraction on the Murzuq aquifer system in Libya, J. Arid environment, Vol. 71(5), 133-156.

Shaw, E.M. (2004) Hydrology in practice. Third edition, Routledge, Abingdon, Oxon, UK, 563pp.

Space Designers International (2006) Consultancy services for the pilot study for improving domestic water demand efficiency using water saving technology report, 62pp, MRMEWR, Muscat , Sultanate of Oman.

Szekely, F., Senay, Y., Al-Rashed, M., Al-Sumait, A., Al-Awadi, E. (2000) Computer simulation of the hydraulic impact of water well fields in Kuwait. *Journal of Hydrology*, 235, 205 – 220.

Shu Takahashi , S. and Peralta, R.C. (1995) Optimal perennial yield planning for complex nonlinear aquifers: methods and examples. Advances in water resources, 18(1), 49-62

Theodossiou, N.P. (2004) Application of non-linear simulation and optimization models in groundwater aquifer management. *Water Resources Management*, 18, 125-141.

Todd, D. K. (1980) Groundwater hydrology. Second edition, *John Wiley & Sons*, New York, USA, 535pp.

Todd, D. and Mays, L (2005) Groundwater hydrology. Third edition. *John Wiley & Sons*, New York, USA, 636pp.

Tung, Y.K. and Koltermann, C.E. (1985) Some computational experience using embedding technique for groundwater management. *Journal of groundwater*, 23, 455-464.

Vieira, J., Cunha, M.C., Nunes, L., Monteiro, J.P., Ribeiro, L., Stigter, T., Nascimento, J., and Lucas, H. (2011) Optimization of the operation of large-scale multisource water-supply systems. *Journal of Water Resources Planning Management*, 137(2), 12 pp.

Vigerske, S (2009) The solver Manuals for GAMS; COIN-OR solver manual. GAMS development corporation, Washington, DC, USA, 608pp.

Wachter, A. and Biegler L. T. (2006) On the implementation of a primal-dual interior point filter line search algorithm for large-scale nonlinear programming. *Mathematical Programming*, 106(1), 25-57.

Wagner B. and Gannett (2010) The use of a coupled groundwater simulation and optimization model to guide groundwater management in the upper Klamath Basin, Oregon and California, USA. Second joint federal interagency conference, Las Vegas, Nevada, USA.

Wang, F. and Anderson, P. (1981) Introduction to groundwater modeling finite difference and finite element methods. W.H. Freeman and company, New York, USA, 223pp.

Warren, A. (1988a) The dunes of the Wahiba Sands. Journal of Oman Studies, special report No.3. Scientific results of the Royal geographical society's Oman Wahiba Sands project 1985-1987, 131-160.

Warren, A. (1988b) A note on sand movement and vegetation in the Wahiba Sands. Journal of Oman Studies, special report No.3. Scientific results of the Royal geographical society's Oman Wahiba Sands project 1985-1987, 251-255.

Willis, R., & Finney, B.A. (1988) Planning model for optimal control of saltwater intrusion. *Journal of Water Resources Planning and Management*, 114(2), 163-178.

Willis, R. and Yeh, W. W-G. (1987) Groundwater systems planning and management. Prentice-Hall, Englewood Cliffs, New York, USA.

Winston, R. B. (1999) MODFLOW-related freeware and shareware resources on the internet. Computers & Geosciences, 25, 377-382.

Yager, R.M., and Ratcliffe, N.M. (2010) Hydrogeology and simulation of groundwater flow in fractured rock in the Newark basin, Rockland County, New York. U.S.Geological Survey Scientific Investigations, Report 2010–5250, 139 p.

Yeh, W. W-G. (1992) Systems analysis in groundwater planning and management. *Journal of Water Resources Planning and Management*, ASCE, 118 (3), 224-237.

Yuan, L.-R., Xin, P., Kong, J., Li, L. and Lockington, D. (2011) A coupled model for simulating surface water and groundwater interactions in coastal wetlands. Hydrological Processes, 25

Zhou, Y. and Li, W. (2011) A review of the historical development of regional groundwater flow modelling. Geocience Frontiers 2(2) 204-214.

The GAMS/CoinIpopt web site http: //projects.coin-or.org/Ipo

RESUME

Said K. Al-Khamisi completed his basic education and passed High School in May, 1983 from Oman. After graduating, he was given a Government scholarship to study Geological Engineering at the University of Arizona, USA, between September, 1983 and December, 1988. He was also sponsored to do his Masters Degree between September 1998 and December 1999. He completed his Masters Degree in "Environmental Remote Sensing and Geographic Information System (GIS)" from Boston University in December 1999 with distinction (GPA 4.0).

Immediately after his B. Sc. graduation, he commenced a post as well site drilling and petroleum engineer in the Petroleum Development Oman Company (PDO), where he supervised exploration drilling and the development of production oil wells in many different concession areas in the Sultanate. After this, he was promoted to the position of production geologist in the Department of Petroleum Engineering. Here his duties included writing oil field development proposals, siting of exploration wells and the final design of oil production wells. During his five years of service with the company, he attended short courses in geology and oil production in the Netherlands, and also courses in computer programming. He participated also in writing a technical paper entitled "Horizontal Drilling in the Nimer Oil Field", and presented the paper at an oil conference held by OPEAC in Paris in 1992. In addition, he represented PDO at the 1992 Muscat International Exhibition and at the Oman Cultural week in Qatar in February, 1993.

In August 1994, he decided to resign from PDO and take up a position of geologist in the Water Resources Protection Department of the Ministry of Water Resources (MWR) in the Sultanate of Oman; recently re-named the Ministry of Regional Municipalities and Water Resources (MRMWR). Between 1994 and 2008, he has subsequently held the positions of Director of Water Resources Conservation Department, Director of Aflaj and Supporting Wells Department, Director of Research Department, Director of Surface and Groundwater and his last position in this ministry, Director of Ash Sharqiyah Sands Aquifer Project Department (ca. $ 100 million water supply project).

In April 2008, he transferred to work in the Public Authority for Electricity and Water (PAEW) as Director of Water Projects Implementations, looking after the execution of water supply transmission and distribution network projects. He monitored the construction phase of approximately 35 water supply projects during 2008 and 2009 costing a total of more than $1000 millions, where he liaised with other governmental authorities, consultants and the contractors to resolve issues related to the construction of projects to ensure timely execution of the projects and monitor project budgets. He looked after designing various water projects including reviewing draft design and contract documents, evaluation of tenders and selection of contractors based on their technical and financial offers. He participated in the studies done by M/s KPMG for the privatization of Ash Sharqiyah Sands Project and drafting of the contract, defining the terms of reference and the conditions of the privatization contract for Ash Sharqiyah Sands Aquifer Project.

His 22 years of work with PDO, MWR, MRMWR and now with PAEW has included participation in the studies and field works to make the master plan for the water wellfields protection zones and monitoring pollution to the water resources and conservation of water in the Sultanate of Oman, participation in the preparation of pre-feasibility & feasibility studies for development of wellfields for water supply schemes such As Sharqiyah Sands Water Supply Project. He has also prepared and introduced viable water resources conservation programmes to conserve water resources and public participation programmes to generate public awareness, participating in public, media campaigns, conducted seminars on various forums such as regional officials, citizens, schools and public media (Radio, news papers & magazines) all to conserve water usages. He participated also in the plan and implementation of Aflaj management strategies with the owners of the Aflaj to promote and encourage public participation for effective routine maintenance of the Aflaj at their own cost as a long term strategy. He was an active member in the coordination and implementation of new research projects to boost the conservation of water resources of the country. Supervised different cooperative joint research projects with Sultan Qaboos University, PDO and outside Universities (France). Examples of these researches are potential water resource coming from Aphiolite rock formation, monitoring studies for protection of Wadi Ronab Aquifer in Al-Wusta Region of Oman and pollution problems originating from uses of pesticides and fertilizers and their effect on the ground water resources. He participated in the initiation and implementation of water supply projects in Oman using renewable energy sources such as wind power and solar power.

He has also during his 23 years of work participated at several seminars and international conferences related to oil and water resources management, conservation programs, environment protection, research and water assessment. In addition, he has attended courses on administration, development of supervision skills and other management related issues. He has also been an active member on several government committees, including the committee for establishing national guidelines on discharge of water from hospital waste and also a committee formulated to investigate the problems of oil well pollution in Oman. He has given several presentations on the activities and responsibilities of the respective bodies.