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Assessment of Surface Water Runoff and Groundwater Recharge Using Mathematical Models

Khaled Omer Mohamed Haroon

United Arab Emirates University
Deanship of Graduate Studies

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By

Khaled Omar Mohamed Haroon

A Thesis Submitted to

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Thesis Title

Assessment of Surface Water Runoff and Groundwater Recharge Using Mathematical Models

Author's Name

Khaled Omar Mohamed Haroon

Supervisor

Prof. Mohsen Sherif
Professor of Water Resources
Civil and Environmental Engineering Department
College of Engineering
United Arab Emirates University

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Examination Committee

Prof. Mohsen Sherif,
Civil and Environmental Engineering Department
College of Engineering,
Al

Mohsen Sherif ....

Dr. Waleed Khalil Al-Zubari,
Desert and Arid Zones Sciences Program
College of Graduate Studies
Arabian Gulf University
Bahrain

....

Dr. Fares Howari,
Geology Department
College of Science,
Al

FARES Howari ....

United Arab Emirates University
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ABSTRACT

In arid and semi-arid regions, surface water resources are scarce and, in most cases, groundwater is the only natural resource of freshwater. Pumping of groundwater often exceeds natural recharge. Therefore, groundwater levels are declining and its quality is deteriorating. Sustainable management of groundwater is thus a key issue and requires implementation of appropriate technologies to augment groundwater resources. Artificial recharge augments the natural movement of surface water into the underground formations using some means of construction whereby surface water from streams or lakes is made to infiltrate into the ground.

The UAE is known by its arid conditions and limited renewable freshwater resources. Surface water in UAE is very limited and of a little significance in the water budget of the country. Despite the construction of many desalination plants, groundwater represents a vital natural resource. Although it may not be suitable, in most cases, for drinking, it represents the main source for irrigation. About 85% of the total water consumption in UAE is from groundwater. The sustainability of this precious resource is of prime concern in the UAE. Many dams have been constructed during the last two decades across the main wadis to harvest surface water runoff and recharge groundwater.

The importance of this study evolves from the need to assess surface water and groundwater resources in the main wadis which are of vital role in the sustainable development of UAE, specifically, the agricultural development. The study aims at the simulation and quantitative assessment of surface water runoff and the associated groundwater recharge in Wadi Ham, UAE. HEC-HMS and MODFLOW models were used. Due to data limitation, HEC-HMS was applied for the period 1979 to 1989 and MODFLOW was applied for the period form January, 1990 to December, 1993. The study provides a methodology that can be followed in other sites of similar hydrological and hydrogeological conditions. All the data and facilities were provided through a project entitled "Assessment of the effectiveness of Al-Bih, Al-Tawiyaen and Ham Dams in groundwater recharge using numerical models". The project was funded by the Ministry of Agriculture and Fisheries.

The catchment area and drainage network were delineated based on the available toposheets and remote sensing images using ArcView GIS and AutoCAD softwares. Different wadi tributaries and properties were identified. Comprehensive analyses were conducted to study the variations of rainfall, surface water flow and groundwater levels based on historical records. Several lithologic cross sections were developed to assess the hydrogeology of the area and identify the aquifer geometry. A rainfall/runoff model (HEC-HMS) was used to study the surface runoff process and quantify the total runoff yields. A three-dimensional groundwater flow model (MODFLOW) was used to quantify groundwater recharge and study flow directions and the water balance.

The total catchment area to the Wadi Ham Dam is approximately 195 km^2. This includes Wadi Ham itself and the catchment of Wadi Al-Farfar system. Rainfall distribution is intermittent and highly scattered. The mean annual rainfall, estimated for 23 years, is 154 mm. Surface water flow is also variable reflecting the intermittent nature of rainfall. Wadi Al-Farfar system has major contribution to the total runoff and accounts for about 40% of the total runoff yield accumulated at the dam site. The Wadi Ham Dam has an effective role in groundwater recharge and its effect is clearly reflected by rise in groundwater levels. The recharge from the dam ranged from 32% to 43% of the dam storage. The flow of seawater to the aquifer is reduced to very low levels during recharge events from the dam while water losses to the sea appear to be very minor.

The study suggests some recommendations including continuity of measurements and additional installations of flow gauges and observation wells. The groundwater model developed in this study can be enhanced and its capabilities can be expanded by conducting field inventory of pumping wells, new drillings and pumping tests of longer durations.

Keywords: Arid regions, UAE, Wadi Ham, artificial recharge, groundwater, surface runoff, Mathematical models.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

Chapter One

INTRODUCTION

CHAPTER ONE:

INTRODUCTION

1.1 Prologue

Doubtless, water is the most essential substance for life in the globe not only for humans but also for every living being. Water is the cornerstone for the socio-economic development and important for the functioning of a modern, developed society. In addition, water is essential for insuring the integrity of the Earth's ecosystems. With no exaggeration; water is the synonym of life.

Water is the most widely occurring substance on this planet but unfortunately most of the Earth's water is saline. The freshwater is very scarce. Of all the water in the world, 97% is salt water in the oceans (Bouwer, 1978). Of the remaining freshwater, two thirds is in the form of ice in arctic and mountainous regions. Of the remaining liquid freshwater, less than 2% is surface water in streams and lakes, and much of that is fed by groundwater. Thus, more than 98% of the world's liquid fresh water is groundwater (Bouwer, 2000a and 2002).

In recent years, the availability of and access to freshwater have been highlighted as among the most critical natural resource issues facing the world. The UN environmental report, GEO 2000, states that the global water shortage represents a full-scale emergency where the 'world water cycle seems unlikely to be able to adapt to the demands that will be made of it in the coming decades'. It has been estimated that today more than 2 billion people are affected by water shortage in over forty countries; 1.1 billion people do not have sufficient drinking water and 2.4 billion people have no provision for sanitation (WWAP, 2003).

The fact that the world faces a water crisis has become increasingly clear in recent years – most notably in arid and semi arid regions where drought conditions prevail. The tremendous population growth, higher living standards and industrialization cause ever increasing demand for freshwater, and thus shortage in freshwater is likely to restrain socio-economic development, especially in arid and semi-arid regions. In such areas, surface water resources are almost absent and, in most cases, groundwater is the only natural resource of freshwater, and often its pumping greatly exceeds natural recharge. Therefore, groundwater levels are declining and its quality is deteriorating.

Arid regions differ from humid regions in that their amount of precipitation is very little as compared to high evaporation rates. Rainfall events often occur as infrequent, short duration, high intensity storms which cause excess rainfall to propagate rapidly through watersheds resulting in flash floods which flow as ephemeral streams.

In order to achieve sustainable development in arid and semi arid regions, management of groundwater resources should be adopted through accurate assessments and wise use of water resources. Appropriate technologies to augment groundwater resources should also be implemented. To that end, artificial recharge of groundwater has become a common practice and is widely used as a conservative technique to store water at the time of abundance to be used during drought periods.

In contrast to natural recharge (which results from natural precipitation, flow of wadis and storm runoff), artificial recharge augments the natural movement of surface water into the underground formations by some means of construction or by artificially changing natural conditions. Artificial recharge may be defined as the planned activity of man whereby surface water from streams or lakes is made to infiltrate into the ground, commonly at rates and in quantities many times in excess of natural recharge, giving a corresponding increase in the magnitude of the safe yield. Artificial recharge may also be defined as a practice of increasing the amount of water reaching the subterranean reservoir by artificial means (Todd, 1980; Bouwer, 1978).

A variety of methods have been developed to recharge groundwater artificially, including water spreading, pits, recharge through wells, and pumping to induce recharge form surface water bodies. The most widely practiced method is water spreading generally classified as basin, stream channel, ditch and furrow flooding, and irrigation technique.

In most situations, artificial recharge projects not only serve as water-conservative mechanisms but also assist in overcoming problems associated with overdraft. Therefore, artificial recharge projects are conceptually designed to serve one or more of the following main purposes (Todd, 1980; Bouwer, 1978):

1. Maintain or augment the natural groundwater as an economic resource.
2. Combat adverse conditions such as a progressive lowering of groundwater levels, unfavorable salt balance, and saline water intrusion.
3. Coordinate operation of surface and groundwater reservoirs.
4. Provide subsurface storage for local or imported surface waters.
5. Reduce or stop significant land subsidence.
6. Provide a localized subsurface distribution system for established wells.
7. Provide treatment and storage for reclaimed wastewater for subsequent reuse.

1.2 Physical and Climatological Settings of UAE

The United Arab Emirates, with a mainland surface of about 83,600 km^2, lies in the southeastern part of the Arabian Peninsula between Latitudes 22^{o}40' and 26^{o}00' North and longitudes 51^{o}00' and 56^{o}00' East. It is bounded from the north by the Arabian Gulf, on the east by the Sultanate of Oman and the Gulf of Oman and on the south and the west by the Kingdom of Saudi Arabia (Fig. 1.1).

The United Arab Emirates may be divided into two distinct zones: the larger low-lying zone and the mountains zone. The first covers over 90% of the country's area, extending from the northwest to the eastern part of the country where it is truncated by the mountains zone (Al Hamady, 2003). The low-lying zone ranges in altitude from sea level up to 300 meters above mean sea level (amsl). Its major part is characterized by the presence of sand dunes which rise gradually from the coastal plain reaching their highest elevation of 250 m amsl. Along the coast of the Arabian Gulf, the low-lying land is punctuated by ancient raised beaches and isolated hills which may reach up to 40 m amsl in some locations (Baghdady, 1998).

The area adjacent to the Arabian Gulf Coast comprises a number of salt domes. These features often form islands in the sea and isolated hills on land. The highest of which is Jabal Dhana and it rises to 99 m amsl. Where the low-lying zone merges gradually with the mountains zone, several isolated anticlinal hills and mountains (trending generally, in N-S direction) occur. The highest and most extensive of these is Jabal Hafit with a maximum altitude of over 1000 m amsl (Al Shamsei, 1993).

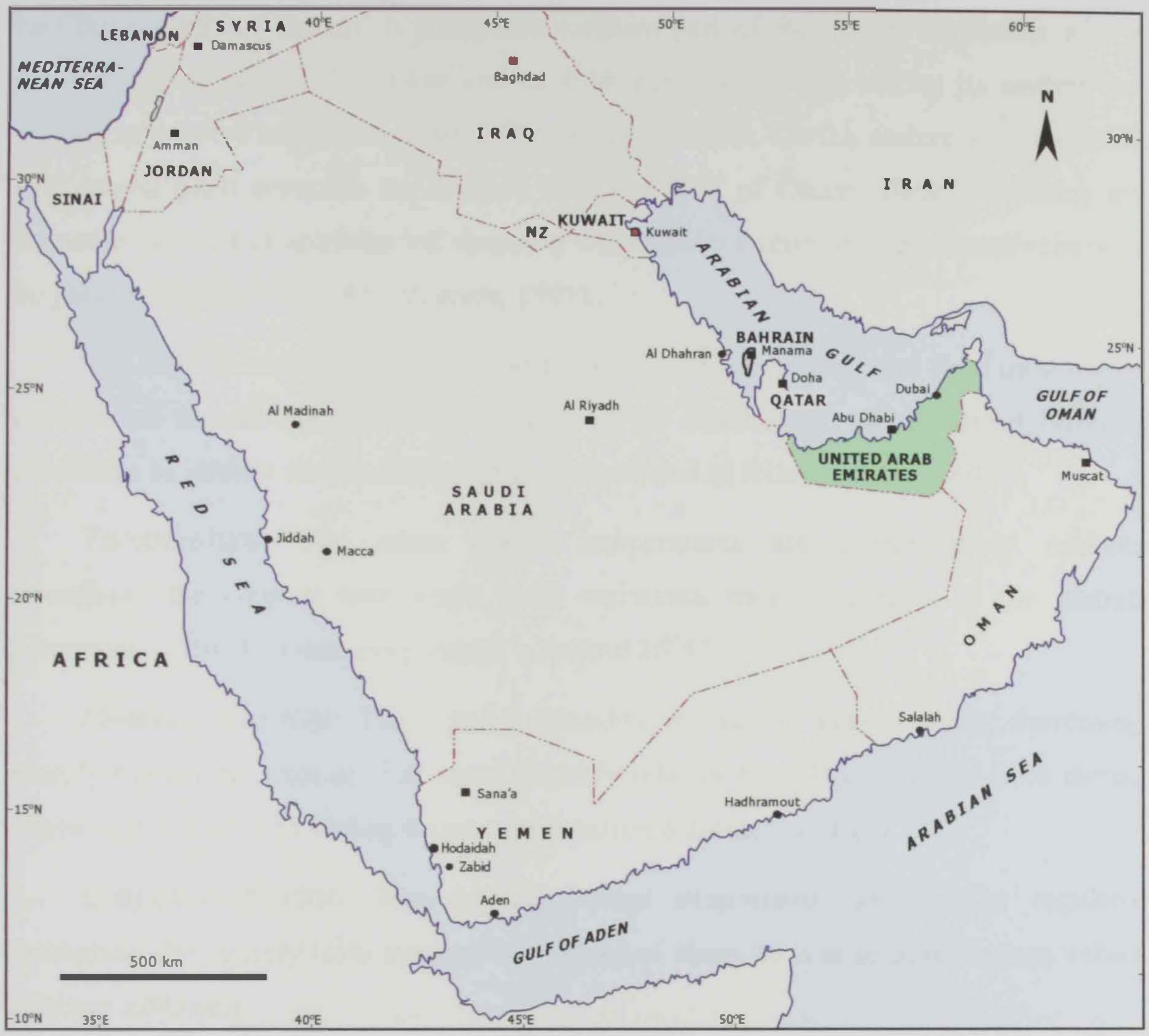

Figure 1.1. Arabian Peninsula and location of the United Arab Emirates (after Al-Nuaimi, 2003).

The mountains zone consists of a N-S trending ridges and is relatively parallel to the east coast (Gulf of Oman). It forms the northern part of the Oman Mountains with a maximum N-S extent of 150 km and an E-W extent of 50 km. Along its eastern and western edges, the mountains chain is fringed by Bahadas. On the eastern side, a narrow low coastal plain separates the Bahada from the Gulf of Oman. These mountains are formed as a result of uplifting and thrusting which leave a series of jagged peaks rising to heights over 1500 m amsl (Al Shamsei, 1993).

The UAE is known by its arid conditions. A long hot summer and short mild winter characterize the climate. The characteristics of the climatologic and hydrometeorologic conditions of interest for this study can be summarized as follow (MAF, 2001):

Temperature: The mean annual temperatures are approximately uniform throughout the country with slight local variations, most noticeable in the eastern mountains where the mean temperature is around 25° C.

Relative humidity: The relative humidity is high in coastal areas, decreasing sharply toward the interior. The mean monthly relative humidity is around 60% during winter and around 50% during summer with extreme diurnal variation.

Evapotranspiration: The annual average evapotranspiration varies regularly throughout the country from average low values of about 80 mm to peak average values of about 2200 mm.

Rainfall: The principal rain in UAE falls between November and March, with the maximum intensity during February and March. About 90% of precipitations fall during winter and spring. The average rainfall during winter is about 38 mm compared to 0.3 mm during summer.

The mean annual rainfall is about 110 mm with extreme variability in space and time. The mean annual rainfall in the eastern and northeastern mountains is about 160 mm, with extreme value as high as 350 mm in some parts. In contrast, during the drought period of 1984-1985 the mean rainfall was about 24 mm and as low as 6 mm in Abu Dhabi.

Winter rainfall is generally light and moderate and widespread in nature being frontal as the dry polar air masses meeting the warm moist air of the Arabian Gulf.

Summer rainfall occurs as heavy isolated events associated with the passage of retreating monsoon or as a result of convection giving rise to lifting mechanism. This rain is confined to the mountains and foothill areas. During the period extending from 1965 to 2000, the mean annual rainfall in UAE fluctuated between 7mm in 1999-2000 water-year to 382mm in 1995-96 water-year (MAF, 2001).

1.3 Water Resources Conditions in UAE

The UAE is known by its arid conditions and its limited renewable freshwater resources. Due to the increase in population and the rapid development in the country, water demands have increased abruptly in the last two decades. In order to meet the ever-increasing demand on water, the United Arab Emirates relies on non-conventional water resources including desalinated water and treated wastewater. A large part of the freshwater demand is being met by desalinated water mainly for drinking purposes. The treated wastewater is mainly used for forestation development. The conventional and non-conventional water resources are summarized in Table 1.1, (Al-Nuaimi, 2003; Rizk, 1999).

Seasonal floods, springs, falajes and groundwater represent the major conventional water resources in the country. However, surface water in UAE is very limited and of a little significance for direct utilization. Conversely, groundwater represents a vital natural resource and is regarded as the cornerstone for socio-economic development in UAE. Although it may not be suitable, in most cases, for drinking and other potable purposes, groundwater represents the main source for irrigation. About 85% of the total water consumption in UAE is groundwater (Rizk *et al*, 1997).

Table 1.1. Summary of conventional and non-conventional water resources in UAE (after Rizk, 1999)

Resource	Existing	Potential	In use	Source
Conventional Water Resources (MCM per year)				
Seasonal Floods	125	125	125	Al-Asam, 1996
Perennial Springs	3	6	3	Rizk and El-Etr, 1997
Seasonal Springs	22	40	---	MAF, 1998
Falajes	20	40	20	Rizk, 1998
Aquifer Recharge	120	120	120	Khalifa, 1995
Groundwater	---	---	880	MAF, 1998
Non-Conventional Water Resources (MCM per year)				
Desalinated Water	---	---	694	MEW, 1998
Reclaimed Water	150	---	150	Hamouda, 1995

In United Arab Emirates four major types of aquifers are recognized (Fig. 1.2). These include the Limestone Aquifer, the Ophiolite Aquifer, the Gravel Aquifer, and the Sand Dune Aquifer (Rizk, 1999). The largest reserve of fresh groundwater in UAE occurs in the gravel alluvial deposits extending along the western side of Oman mountain chain from Ras Al Khaymah to Al Ain. The Sand Dune Aquifer covers about 74% of the total area of UAE. It receives most of its recharge from the eastern mountains, whereas the Arabian Gulf and Gulf of Oman are the main discharge area. The Limestone Aquifers are seen in the northern region at Wadi Bih catchment, as well as Jabal Hafit catchment in Al-Ain region. Most of the natural recharge to the western and eastern aquifer systems is received at the heads of alluvial fans by infiltration form wadis' flows originated in the mountain zone.

Groundwater resources in the UAE have been over exploited to meet the increasing water demands, especially for agriculture purposes. The total withdrawal from the main western alluvial aquifer, mainly by agriculture sectors, has increased from 224 million cubic meters (MCM) in 1975 to 880 MCM in 1995 while the estimated mean annual recharge from rainfall is 120 MCM per year (Rizk, 1999). Due to the recharge-discharge imbalance, a remarkable depletion of the groundwater levels has occurred in many aquifers. The existing imbalance has originated as a consequence of lack of natural recharge and excessive discharge. Therefore, it is of great importance to replenish and recharge the depleted aquifers to ensure the sustainability of groundwater resources.

The Ministry of Agriculture and Fisheries (MAF) is undertaking major actions toward the implementation of appropriate rainwater harvesting technologies. Surface water from flash floods can be utilized to recharge the depleted aquifers and sustain the groundwater resources. Therefore, a large number of detention and retention dams have already been constructed during the last two decades across the main Wadis to harvest the surface water runoff and protect the cities at the downstream side of the Wadis (Sherif and Merabtene, 2002).

The importance of this study evolves from the need to assess surface runoff in the main wadis and quantify the associated groundwater recharge. Despite its limited availability, groundwater plays a vital role in the sustainable development of UAE, specifically, the agricultural development of UAE.

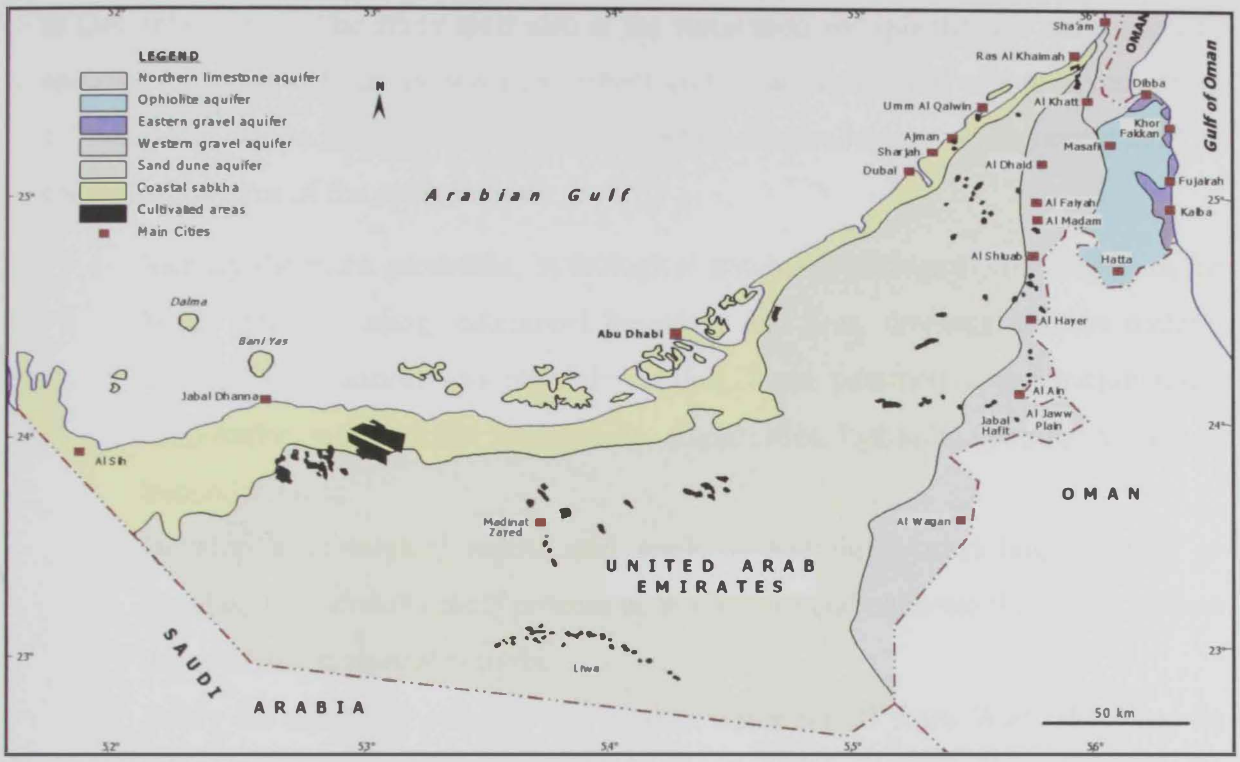

Figure 1.2. The main water bearing units (aquifers) in UAE, (after Rizk *et al*, 1997).

1.4 Objectives of Study

The overall objective of this study is to provide a quantitative assessment for the groundwater recharge at the dam site in Wadi Ham during the period from January, 1990 to December, 1993. The study aims also at the simulation and quantitative assessment of surface water runoff from the wadi catchment under various rainfall events in the period 1979 to 1989. These periods were selected based on the availability of required data. The specific objectives of the study include:

1- Identify the main geometric, hydrological and hydrogeological characteristics for Wadi Ham including, catchment boundary and area, drainage streams pattern, average areal annual and monthly rainfall, basin parameters, infiltration rates, evaporation rates, aquifer dimensions, storativities, hydraulic conductivities, and transmissivities.
2- Develop a conceptual model and apply a hydrologic modeling software to simulate the rainfall-runoff process at Wadi Ham and calibrate the model against the available historical records.
3- Apply the model to estimate the surface water runoff from Wadi Ham and its tributaries and the accumulated volume of surface water at the dam site for different rainfall events.
4- Develop a conceptual model and apply a numerical model to study the groundwater flow and recharge processes associated with water storage at Wadi Ham Dam site and calibrate the model against the available historical records.
5- Study the groundwater balance for the Wadi Ham aquifer system under both recharge and no-recharge conditions and provide a quantitative assessment for the groundwater recharge from the dam.
6- Study flow directions and assess the possibility of water losses to the Gulf of Oman or seawater flow to the aquifer from the Gulf based on the groundwater balance.

1.5 Methodology of Study

This thesis has been completed in the framework of a collaborative research project between the UAE University and the Ministry of Agriculture and Fisheries (MAF), entitled "Assessment of the Effectiveness of Al Bih, Al Tawiyean and Ham Dams in

Groundwater Recharge Using Numerical Models". The work presented in this thesis is focused on Wadi Ham.

All resources, collected data and field visits and measurements were made available through this project. The project was funded by the MAF. The following activities were completed to fulfill this study.

1- Literature Review

A comprehensive literature review of previous investigations related to Wadi Ham was completed. Reports of several previous studies carried out prior to, during the construction phase, and after the construction of Wadi Ham Dam have been reviewed. Reports about general climatic conditions, precipitation and evaporation rates were also reviewed. The available geological and lithological information for observation wells and boreholes were assessed. Several geological cross sections in Wadi Ham were available. Other related published research papers, unpublished reports and M.Sc. and Ph.D. theses were also reviewed (e.g., Bouwer, 2002; Rizk, 1999; Al-Nuaimi, 2003; and Baghdady, 1998).

2- Data Collection and Processing

The available geological, hydrological and hydrogeological data related to Wadi Ham were collected from MAF and previous studies. The data were then categorized, presented in graphical forms and prepared in different digital forms such that they could be used directly in other applications. The collected data have been critically examined and unreliable or erratic information were eliminated.

The collected data included available records for climatic conditions and rainfall events from the different rain gauges and meteorological stations in the vicinity of Wadi Ham, evaporation rates, records of surface water runoff and flood volumes, location of observation wells, records of groundwater levels, hydrogeological parameters based on the available information in the completion reports of observation wells and the geological sequence of the subsurface layers.

Topographic maps (toposheets) of different scales have been collected from the MAF and a remote sensing image has been purchased. All the necessary maps including catchment boundary, dam location map, drainage streams network map and model boundary map have been delineated and digitized based on these toposheets and the remote sensing image using ArcView GIS and AutoCAD softwares.

3- Field Work

Within the framework of the MAF/UAEU project, different field visits have been conducted to compensate for any lack in the needed information and to verify some of the collected data. During field visits, the important geological and hydrogeological features of Wadi Ham, including rock types; rock materials; and depths to water table, have been investigated. The coordinates of observation wells have been confirmed using GPS. Field activities also included conducting infiltration tests within the ponding area of the dam to identify the infiltration rates.

4- Surface Water Modeling

The study of surface water flow in Wadi Ham is considered a prerequisite for the study of groundwater system. Surface water modeling involved application of the Hydrologic Modeling System (HEC-HMS) to simulate the surface water runoff process from various rainfall events. The developed model helps to estimate flood volumes at the dam site to be used for the simulation of groundwater recharge.

Because rainfall is the major input for surface water modeling, historical rainfall data have been analyzed in both time and space. The historical records of surface water flow in Wadi Ham were also analyzed and several flood events were selected for the simulation.

5- Groundwater Modeling

The study of groundwater recharge at Wadi Ham Dam is the main objective of this thesis. The three dimensional groundwater modeling code (MODFLOW) has been applied to model the aquifer system. Prior to the modeling work, the hydrogeology of the study area was investigated. Several lithological cross sections have been developed.

To introduce the aquifer geometry to the model, contour maps for ground surface and aquifer bottom were prepared. Preliminary assessment of the aquifer response to recharge has been done by plotting water table level fluctuations with time at selected observation wells. After development, the model was calibrated against historical water table levels and the parameters were readjusted. The flow directions and the groundwater balance were examined to study flow conditions and to provide a quantitative assessment of groundwater recharge.

6- Thesis Preparation

Upon the completion of the previous activities, the output of the study was examined and conclusions were made. Different location maps, catchment maps and contour maps of model input, calibration and output were prepared in final forms. Illustrative figures and tables were also finalized and discussions were written in a scientific manner and presented in several chapters comprising this thesis.

1.6 Limitations of Study

The rainfall/runoff model presented in this study was restricted by the availability of data. Surface water flow records are available for Wadi Ham only during the period 1979 to 1990. The model was calibrated to match recorded total flow volumes for selected flood events during this period.

The groundwater model constructed in this study is of preliminary nature. It is intended to help providing a quantitative assessment for the recharge from the dam. The model had to be calibrated against recorded groundwater levels. Continuous records of groundwater levels were not available in all observation wells in the study area. The records were more available and better distributed during the period from January, 1990 to December, 1993. Therefore, the model is limited to calibration phase during this period.

1.7 Organization of Thesis

This thesis is composed of six main chapters and three appendices containing relevant data. Chapter one elaborates on the importance of groundwater resources in arid regions, in general, and UAE in particular. The importance of artificial recharge of groundwater is elaborated and its main purposes are reviewed. Physical and climatological settings and water resources conditions in UAE are discussed. The general and specific objectives of the study are presented and the methodology and limitations of study are explained.

Chapter two provides detailed discussions about hydrologic modeling and groundwater recharge. The concepts of hydrologic modeling and classification of simulation models are explained. The chapter goes on to discuss the different categories of both surface water and groundwater models. A detailed discussion about groundwater recharge is also presented. The different sources of groundwater recharge are explained with emphasis on artificial recharge. The concept of artificial recharge is elaborated and different artificial recharge systems are explained.

The location, geology and climatology of study area are presented in Chapter three. In this chapter, the general location of Wadi Ham and the Wadi Ham Dam are illustrated. The catchment area to the dam and the different tributaries of the wadi are identified. The characteristics of the dam and the geology of the study area are explained. The climatology of Wadi Ham is discussed with emphasis on rainfall. Historical rainfall data are analyzed on quantitative and probabilistic basis using statistical and frequency analyses.

Chapter four is devoted to the assessment and modeling of surface water runoff in Wadi Ham. The assessment is made based on historical records of surface water flows. Surface water flow data are analyzed and compared with rainfall data prior to modeling. The chapter goes on to provide an overview of the main features of the Hydrologic Modeling System (HEC-HMS). The application of this model to simulate surface runoff process in Wadi Ham is explained. Model conceptualization, construction, calibration and results are discussed in details.

Chapter five is devoted to the assessment and modeling of the groundwater system in Wadi Ham. The chapter provides detailed description of the hydrogeology of Wadi Ham plain area. Lithologic cross sections are presented and the different layers and aquifer geometry are identified. Preliminary assessment of aquifer response to recharge is made based on historical records of groundwater levels. This chapter is concluded with a comprehensive discussion on the modeling of Wadi Ham groundwater system. A brief description of theoretical base and capabilities of the modeling code (MODFLOW) is given and detailed explanation about the model conceptualization, construction, calibration and results is presented.

Chapter six includes the summary of all the work included in the thesis. The conclusions of the study are presented and several recommendations are proposed for future investigations and studies.

The thesis is supplemented by three appendices. Appendix A lists historical records of monthly rainfall depths at raingauges and the computed average aerial rainfall depths. Appendix B lists historical records of daily and monthly wadi flows measured by Bithnah Flow Gauge. Appendix C presents the rainfall patterns (hyetographs) for the storms simulated in Chapter four.

Chapter Two

HYDROLOGIC MODELING AND GROUNDWATER RECHARGE

CHAPTER TWO:

HYDROLOGIC MODELING AND GROUNDWATER RECHARGE

2.1 Prologue

Hydrologic engineers are often called upon to provide information such as rates and volumes of flow at any point of interest along a stream necessary for planning and designing new water projects, operating and/or evaluating existing water projects. Although many streams have been gauged to provide continuous records of streamflow, planners and engineers are sometimes faced with little or no available streamflow information. Moreover hydrologic information sometimes needs to be predicted. For example, a flood-damage reduction study may require an estimate of the increased volume of runoff for proposed changes to land use in a watershed. However, no record will be available to provide this information because the change has not yet taken place. The alternative is to use hydrologic modeling. Models are simplified systems that are used to simulate the real-life systems by relating something unknown (the output) to something known (the input) (USACE, 2000; Viessman and Lewis, 1995; and Diskin, 1970).

Simulation is defined as the mathematical description of the response of a hydrologic water resource system to a series of events during a selected time period. For example, a streamflow simulation model can be developed for calculating daily, monthly, or seasonal streamflow based on rainfall; or computing the discharge hydrograph resulting from a known or hypothetical storm; or simply filling in the missing values in a streamflow record. Similarly, a simulation model of a groundwater system might be developed to demonstrate the effects on groundwater storage of various pumping schemes (Viessman and Lewis, 1995).

2.2 Classification of Simulation Models

The varied nature of developed and applied simulation models has led to numerous classification attempts. However, models can be classified as follows.

2.2.1 Physical models

These are reduced-dimension representations of real world systems. A physical model of a watershed is a large surface with overhead sprinkling devices that simulate the precipitation input. The surface can be altered to simulate various land uses, soil types, surface slopes, and so on; and the rainfall rate can be controlled. The runoff can be measured, as the system is closed. A more common application of a physical model is simulation of open channel flow and many such models have been constructed and used to provide information for answering questions about flow in complex hydraulic systems (USACE, 2000). However, a physical model is often so small that proper scaling of the field situation can not be achieved. But this type of model does give an overall view of how the system behaves (Driscoll, 1986).

2.2.2 Analog models

These models represent the flow of water with the flow of electricity in electrical networks of resistors and capacitors. With these models, the input is controlled by adjusting the amperage, and the output is measured with a voltmeter. Historically, analog models have been used to calculate subsurface and groundwater flow (USACE, 2000). The concept of using electricity to model groundwater flow remains valid because Ohm's law for the flow of electricity is analogous to Darcy's law for the flow of groundwater. Individual electrical analog systems, however, were designed specifically for a single groundwater system and therefore could not be adapted to general use (Driscoll, 1986).

2.2.3 Mathematical models

These types are the most useful and the most commonly employed (Driscoll, 1986). A mathematical model relies on mathematical statements to represent the system. It includes a set of general laws or theoretical principles and a set of statements of empirical circumstances (Woolhiser and Brakensiek, 1982). The development of a mathematical model starts with a conceptual understanding of the system to be modeled. The conceptual model can then be translated into a mathematical framework (model) that produces governing equations which describe the physical processes in the system (Fig. 2.1).

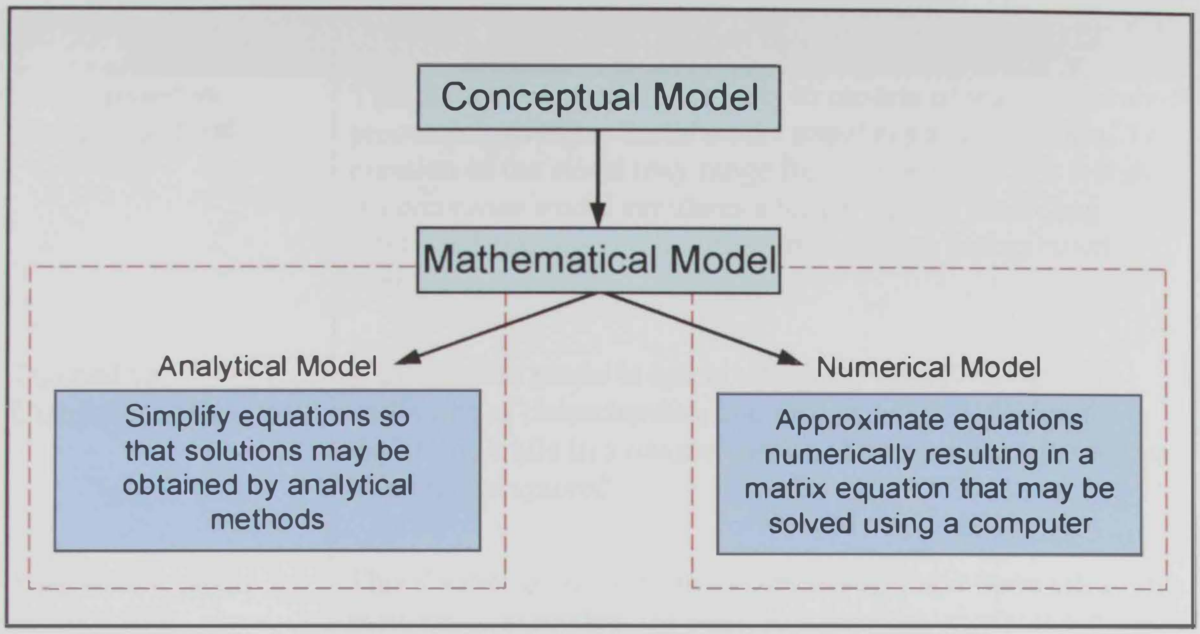

Figure 2.1. Logic diagram for developing a mathematical model (after Mercer and Faust, 1981).

The nature of a modeled system is often complicated and can rarely be described completely by mathematical expressions, so simplifying assumptions must usually be made to solve the governing equations for appropriate boundary and initial conditions.

If enough simplifying assumptions are made, the equations can be solved analytically but the accuracy of the model is reduced. However, more accurate equations are often so difficult to solve that numerical approximation techniques must be used. The use of computer for solving equations by numerical methods makes it possible to simulate extremely complicated systems (Driscoll, 1986).

Mathematical models can be classified using the criteria shown in Table 2.1. These focus on the mechanics of the model: how it deals with time, how it addresses randomness, and so on.

Table 2.1. Categorization of mathematical models. (after Ford and Hamilton, 1996).

Category	Description
Event-Based vs. Continuous Models	This distinction applies primarily to models of watershed-runoff processes. An *event-based model* simulates a single storm. The duration of the storm may range from a few hours to a few days. A *continuous model* simulates a longer period, predicting watershed response both during and between precipitation events.
Lumped vs. Distributed Models	A *distributed model* is one in which the spatial (geographic) variations of characteristics and processes are considered explicitly, while in a *lumped model*, these spatial variations are averaged or ignored.
Empirical (system theoretic) vs. Conceptual Models	This distinction focuses on the knowledge base upon which the mathematical models are built. A *conceptual model* is built upon a base of knowledge of the pertinent physical, chemical, and biological processes that act on the input to produce the output. An *empirical model*, on the other hand, is built upon observation of input and output, without seeking to represent explicitly the process of conversion.
Deterministic vs. Stochastic Models	If all input, parameters, and processes in a model are considered free of random variation and known with certainty, then the model is a *deterministic model.* If instead the model describes the random variation and incorporates the description in the predictions of output, the model is a *stochastic model*.
Measured-parameter vs. Fitted-parameter Models	This distinction is critical in selecting models for application when observations of input and output are unavailable. A *measured-parameter model* is one in which model parameters can be determined from system properties, either by direct measurement or by indirect methods that are based upon the measurements. A *fitted-parameter model*, on the other hand, includes parameters that can not be measured. Instead, the parameters must be found by fitting the model with observed values of the input and the output.

2.3 Surface Water Simulation Models

In recent decades the science of computer simulation of surface water resource systems has passed from scattered academic interests to a practical engineering procedure. Simulation of surface water system implies the use of computers to imitate historical events or predict the future response of the physical system to a specific plan or action. Surface water is simulated either for individual storm or continuous in time. A few of the numerous event-based, continuous and urban runoff computer models for simulating the hydrologic cycle are compared in Table 2.2.

Table 2.2. Digital hydrologic simulation models (after Viessman and Lewis, 1995).

Code Name	Model Name	Agency or Organization	Date of original development
Continuous Streamflow Simulation Models			
API	Antecedent Precipitation Index Mode	Private	1969
USDAHL	1970,1973.1974 Revised Watershed Hydrology	ARS	1970
SWM-IV	Stanford Watershed Model IV	Stanford University	1959
HSPF	Hydrocomp Simulation Program-FORTRAN	EPA	1967
NWSRFS	National Weather Service Runoff Forecast System		1972
SSARR	Streamflow Synthesis and Reservoir Regulation	US Army Corps of Engineers	1958
PRMS	Precipitation-Runoff Modeling System	USGS	1982
SWRRB	Simulator for Water Resources in Rural Basins	USDA	1990
Rainfall-runoff Event Simulation Models			
HEC-1	HEC-1 Flood Hydrograph Package	US Army Corps of Engineers	1973
HEC-HMS	HEC Hydrologic Modeling System	US Army Corps of Engineers	1998
TR-20	Computer Program for Project Hydrology	SCS	1965
USGS	USGS Rainfall-Runoff Model	USGS	1972
HYMO	Hydrologic Model Computer Language	ARS	1972
SWMM	Storm Water Management Model	EPA	1971
Urban Runoff Simulation Models			
UCUR	University of Cincinnati Urban Runoff Model	University of Cincinnati	1972
STORM	Quantity and Quality of Urban Runoff	US Army Corps of Engineers	1974
MITCAT	MIT Catchment Model	MIT	1970
SWMM	Storm Water Management Model	EPA	1971
ILLUDAS	Illinois Urban Drainage Area Simulator	Illinois State Survey	1972
DR3M	Distributed Routing Rainfall-Runoff Model	USGS	1978
PSURM	Pennsylvania State Urban Runoff Model	Pennsylvania State University	1979

Streamflow simulation models are either event-based or continuous models (Table 2.2). Urban runoff models also are primarily event simulation models. Both the event-based streamflow simulation (EBSS) and the continuous streamflow simulation (CSS) are employed to address a wide array of environmental and water resources problems.

Singh (1992) presented detailed discussion on these types of simulation. A brief description of the main features is presented here.

2.3.1 Event-based streamflow simulation (EBSS)

The EBSS models attempt to simulate rainfall-runoff processes resulting from a single storm thus their emphasis is on modeling the direct runoff hydrograph (DRH) or its peak characteristics. The period of simulation in EBSS models is usually as long as the DRH. For this reason some of hydrologic processes such as evaporation and transpiration, infiltration, interception, depression storage, subsurface flow, and baseflow are considered with considerable approximation, some are lumped and some are neglected.

The main elements required to build an EBSS include watershed representation, determination of effective rainfall amount, determination of effective rainfall hyetograph, computation of direct-runoff hydrograph, flow routing and parameter estimation. The data required for EBSS include watershed characteristics, rainfall characteristics, infiltration and other loss characteristics and streamflow characteristics. EBSS has diverse applications in hydrologic analysis and design such as design of hydraulic structures, design of urban and highway drainage, planning of flood control works, assessment of non point source pollution, evaluation of environmental impacts of land use and management practices, assessment of flood damage and evaluation of hydrologic consequences of climatic change (Singh, 1992).

2.3.2 Continuous streamflow simulation (CSS)

The CSS models allow simulation of streamflow for long periods of time (months or years) and maintain a continuous accounting of the water in storage in the watershed. In CSS, the emphasis is on simulation of the entire land phase of the hydrologic cycle and because of the long periods of time, such hydrologic processes as evaporation and transpiration, infiltration, interception, depression storage, subsurface flow, and baseflow are considered significantly.

Two phases are simulated: the land phase and the channel phase. The land phase is much more complicated in CSS than in EBSS, but the channel phase is about the same in both. The elements required to build a CSS model include watershed representation, mean areal rainfall, interception, depression storage, soil moisture storage, infiltration, evapotranspiration, interflow, baseflow, surface runoff, channel-flow routing and reservoir routing.

Data requirements for a CSS model must be as minimal as possible in order to be widely usable. Three types of data are usually available: (1) watershed characteristics: such as soil, land use, and topographic data, (2) climatic characteristics: which include rainfall and meteorological data such as temperature, radiation, humidity, pressure, etc., and (3) hydrological characteristics that may include not only observed hydrologic data such as streamflow, potential evapotranspiration, soil moisture, infiltration, etc., but also information on parameters of hydrologic models used in CSS.

Example problems for application of CSS include flow forecasting, watershed experimentation, evaluation of the effect of land-use practice on watershed response, design of urban drainage, highway culverts, reservoirs, etc., water-quality modeling, water-supply development and irrigation planning and management.

The CSS models are models of the hydrologic cycle, whereas EBSS models are models of the rainfall-runoff cycle. For that reason it is logical to say that CSS models are more general and encompass EBSS models as their special cases (Singh, 1992).

2.4 Groundwater Simulation Models

For many years, groundwater specialists have used various types of digital simulation models to study the storage and movement of water in a porous medium. Distributed rather than lumped parameter models are used to imitate observed events and to evaluate future trends in the development and management of groundwater systems.

Groundwater studies involve the adaptation of a particular code to the problem at hand. Several popular public domain computer codes for solving various types of groundwater flow problems are listed in Table 2.3. The codes become models when the system being studied is described to the code by inputting the system geometry and known internal operands (aquifer and flow field parameters, initial and boundary conditions, and water use and flow stresses applied in time to all or parts of the system).

Codes have emerged in four general categories: *groundwater flow* codes, *solute transport* codes, *particle tracking* codes, and *aquifer test data analysis* programs (Viessman and Lewis, 1995).

Table 2.3. Groundwater modeling codes (after Viessman and Lewis, 1995).

Code Name	Description	Source	Year
Groundwater-flow Models			
PLASM	Two-dimensional finite difference	Ill. SWS	1971
MODFLOW	Three-dimensional finite difference	USGS	1988
AQUIFEM-I	Two- and three-dimensional finite element	MIT	1979
GWFLOW	Package of 7 analytical solutions	IGWMC	1975
GWSIM-II	Storage and movement model	TDWR	1981
GWFL3D	Three-dimensional finite difference	TDWR	1991
MODRET	Seepage from retention ponds	USGS	1992
Solute-transport Models			
SUTRA	Dissolved substance transport model	USGS	1980
RANDOMWALK	Two-dimensional transport model	Ill. SWS	1981
MT3D	Three-dimensional solute transport	EPA	1990
ATi23D	Analytical solution package	DOE	1981
MOC	Two-dimensional solute transport	USGS	1978
HST3D	3-D heat and solute transport model	USGS	1992
Particle-tracking Models			
FLOWPATH	Two-dimensional steady state	SSG	1990
PATH3D	Three-dimensional transient solutions	Wisc GS	1989
MODPATH	Three-dimensional transient solutions	USGS	1991
WHPA	Analytical solution package	EPA	1990
Aquifer-test Analysis			
TECTYPE	Pump and slug test by curve matching	SSG	1988
PUMPTEST	Pumping and slug test	IGWMC	1980
THCVFIT	Pumping and slug test	IGWMC	1989
TGUESS	Specific capacity determination	IGWMC	1990

Groundwater flow codes provide the user with the distribution of heads in an aquifer that would result from a simulated set of distributed recharge-discharge stresses at cells or line segments. From Darcy's law, the flow passing any two points can be calculated from the head differential. The codes are used to model both confined and unconfined aquifers. Each can be structured to model regional flow, or flow in vicinity of a single well or well field. Steady-state and transient conditions can be evaluated. Boundaries can be barriers, full or partially penetrating streams and lakes, leaky zones, or constant head or constant gradient perimeters. By application of Darcy's law, the seepage velocities of groundwater can be determined after solving for the head differentials.

When groundwater seepage velocities are known, the advection, dispersion, and changes in concentration of solutes can be modeled. *Solute transport* models build on groundwater flow models by the addition of advection, dispersion, and/or chemical reaction equations. If the chemical, dispersion, or dilution concentration changes due to groundwater flow are not important, *particle tracking* codes model transport by advection and provide an easier method than solute transport models to track the path and travel times of solutes that move under the influence of head differentials. *Aquifer test data* programs provide users with computer solutions to many of the hand calculations needed to graph and interpret aquifer test data for determining aquifer and well parameters (Viessman and Lewis, 1995).

With few exceptions, the hydrodynamic equations for groundwater flow have no analytical solutions, and groundwater modeling relies on numerical techniques to provide approximate solutions to a wide variety of groundwater problems. The most widely used numerical techniques for solving groundwater flow problems are the *finite-difference* and *finite-element* methods. A detailed discussion on these methods was given by Viessman and Lewis (1995). However, such detailed discussion is beyond the scope of this thesis. A brief description is given in what follows. A graphical representation of these methods is shown in Figure 2.2.

2.4.1 The finite-difference method

Application of finite-difference techniques to groundwater flow problems requires that the region of concern be divided into many small sub-regions or elements (Fig. 2.2b). For each of these elements, characteristic values of all the variables in the governing differential equation are specified. These values are assigned to the centers of the elements, which are called nodes. The heads in adjacent nodes are related through a finite-difference equation, which is derived from the governing differential equation. These difference equations can be derived by an appropriate Taylor's series expansion or by mass balance considerations. The resulting algebraic equations can then be solved simultaneously to yield the heads at each node for each time step considered (Viessman and Lewis, 1995).

The success of any finite-difference scheme depends on the incremental values assigned to the element dimensions and the time steps. In general, the smaller the dimensions of elements and time increments, the closer the finite-difference approximation to the differential equation. However, as these partitions are made smaller, a price in computational costs and data needs must be paid. Furthermore, over-subdivision may even bring about computational intractability.

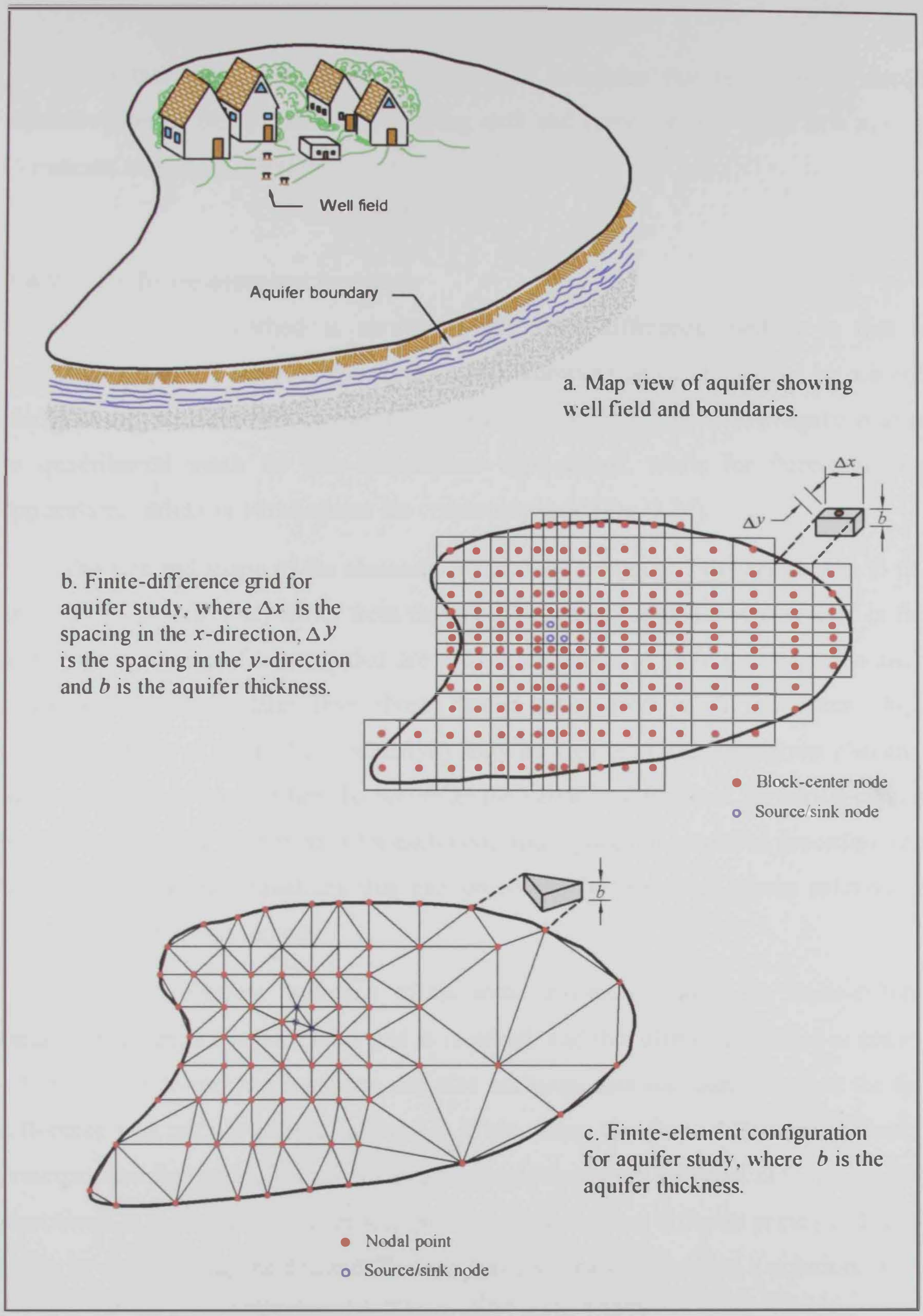

Figure 2.2. Two-dimensional grid systems for finite-difference and finite-element methods (after Driscoll, 1986).

Thus the object is to select the degree of definition that results in an adequate representation of the system while keeping data and computational costs at a minimum (Viessman and Lewis, 1995).

2.4.2 The finite-element method

The finite-element method is similar to the finite-difference method in that both approaches lead to a set of N equations in N unknowns that can be solved by relaxation. Nodes in the finite-element method are usually the corner points of an irregular triangular or quadrilateral mesh for two-dimensional applications, while for three-dimensional applications, bricks or tetrahedrons are commonly used (Fig. 2.2c).

The size and shape of the elements selected are arbitrary. They are chosen to fit the application at hand. They differ from the regular rectangular grid elements used in finite-difference modeling. Elements that are closest to points of flow concentration such as wells are usually smaller than those further away from such influences. Aquifer parameters such as hydraulic conductivity may be kept constant for a given element but may vary from one to another. To minimize the variational function, its partial derivative with respect to head is evaluated for each node and equated to zero. The procedure results in a set of algebraic equations that can be solved by iteration, matrix solution, or a combination of these methods.

The finite-element approach offers some advantages over the finite-difference technique. Often, a smaller nodal grid is required, and this offers economies in computer effort. The finite-element approach can also accommodate one condition that the finite-difference approach is unable to handle. When using the finite-difference method, the principal directions of anisotropy in an anisotropic formation are parallel to the coordinate directions. In cases where two anisotropic formations having different principal directions occur in a flow field, the finite-difference approach cannot produce a solution, whereas the finite-element approach can (Viessman and Lewis, 1995).

2.5 Groundwater Recharge

Groundwater use is of fundamental importance to meet the rapidly expanding urban, industrial, and agricultural water requirements, particularly in arid and semi-arid zones. To replenish and recharge groundwater resources is thus a prerequisite for sustainable development in these dry areas, where such resources are often the only water source, susceptible to contamination and are prone to depletion (De Vries and Simmers, 2002). Artificial recharge of groundwater is expected to become increasingly necessary in the future as growing populations require more water, and as more storage of water is needed to save water in times of water surplus for use in times of water shortage (Bouwer, 2002). Furthermore, groundwater recharge is a key component in any model of groundwater flow or contamination transport (Healy and Cook, 2002).

Since the mid-1980s, a relative explosion of recharge studies has been reported in the scientific literature (De Vries and Simmers, 2002). This section summarizes the various sources of groundwater recharge. Emphasis is accorded to artificial recharge which becomes increasingly adopted in many parts of the world. The discussions provided hereinafter are excerpted from Bouwer (2002).

2.5.1 Recharge sources

Groundwater recharge is defined as "the entry into the saturated zone of water made available at the water-table surface, together with the associated flow away from the water table within the saturated zone" (Freeze and Cherry 1979). Sources of groundwater recharge include *natural*, *enhanced*, *induced*, *incidental* and *artificial recharge.*

Natural recharge: is how natural (meteoric) groundwater is formed as the difference between water inputs into the soil (precipitation and infiltration from streams, lakes, or other natural water bodies) and outputs (evapotranspiration plus runoff). Natural recharge is typically about 30-50% of precipitation in temperate humid climates, 10-20% of precipitation in Mediterranean type climates, and about 0-2% of precipitation in dry climates (Bouwer, 1989, 2000b; and Tyler *et al*., 1996). Natural recharge rates are reflected by groundwater ages, which vary from a few hours or days in wet-weather springs or very shallow groundwater in high rainfall areas, to tens of thousands of years or more in dry climates with deep groundwater levels or in confined aquifers at considerable distances from their outcrops where they are recharged (Bouwer, 2002 and references therein).

Enhanced recharge: consists mainly of vegetation management to replace deep-rooted vegetation by shallow-rooted vegetation or bare soil, or by changing to plants that intercept less precipitation with their foliage, thus increasing the amount of water that reaches the soil. In wooded areas, this is achieved, for example, by replacing conifers with deciduous trees (Querner, 2000).

Induced recharge: is achieved by placing wells relatively close to streams or rivers, so that more river water is "pulled" into the aquifer as water tables near the streams are lowered by pumping the wells. The main objective of these "bank filtration" systems is often to get pretreatment of the river water as it moves through the river-bottom materials and the aquifers before it is pumped up for conventional drinking-water treatment and public water supply. Bank filtration is used particularly where river water is contaminated or where the public prefers groundwater over surface water (Kühn, 1999).

Incidental recharge: is caused by human activities that are not intended for recharge of groundwater as such. These activities include sewage disposal by septic-tank leach fields or cesspits, and drainage or deep percolation from irrigated fields (Bouwer, 2002). Another form of incidental recharge is obtained with urbanization, where most of the land is covered with streets, driveways, roofs, and other impermeable surfaces that produce more runoff and have much less evapotranspiration than the natural surfaces. This recharge could be significant in semi-arid areas, where rain typically falls in small amounts that do not penetrate the soil very deeply, so that most of the water evaporates. With urbanization, however, more runoff is produced, which can be collected for on-site storage and artificial recharge, or it flows naturally to ephemeral streams where it infiltrates into the soil and moves down to the groundwater (Lerner, 2002).

Artificial recharge: is achieved by ponding or flowing water on the soil surface with basins, furrows, ditches, etc. (Figs. 2.3 and 2.4); by placing it in infiltration trenches, shafts, or wells in the vadose zone (Fig. 2.5); or by placing it in wells for direct injection into the aquifer (Bouwer, 2002).

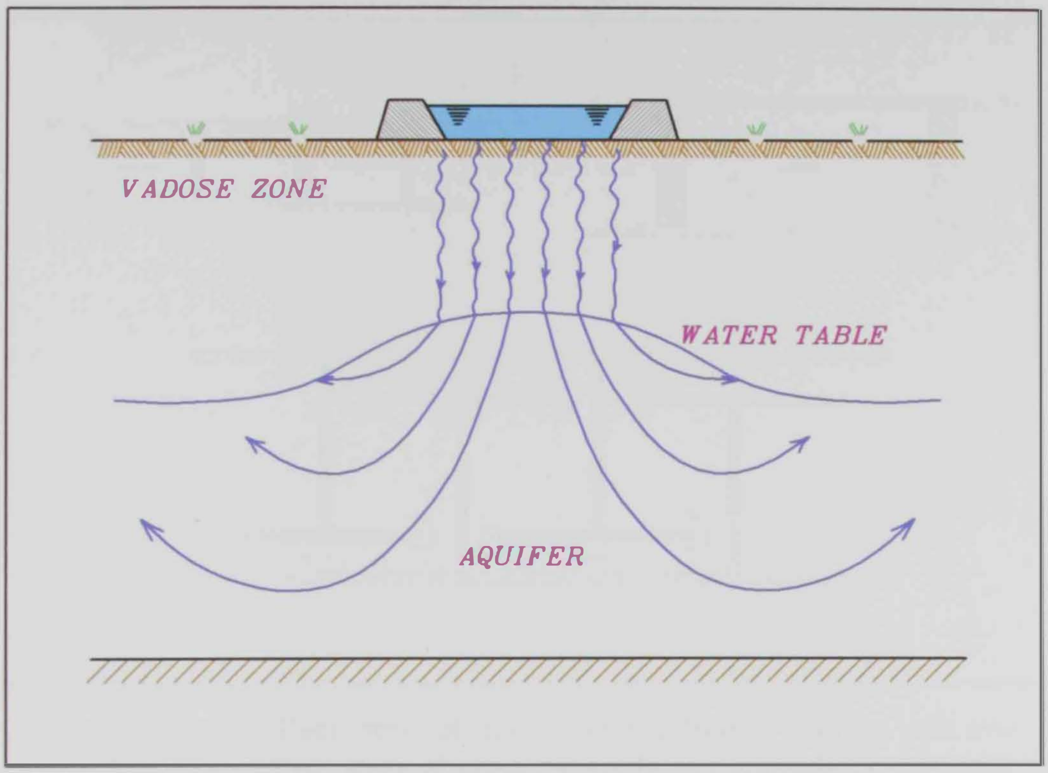

Figure 2.3. Section through a typical groundwater recharge system with infiltration basin and groundwater mound below the basin (after Bouwer, 2002).

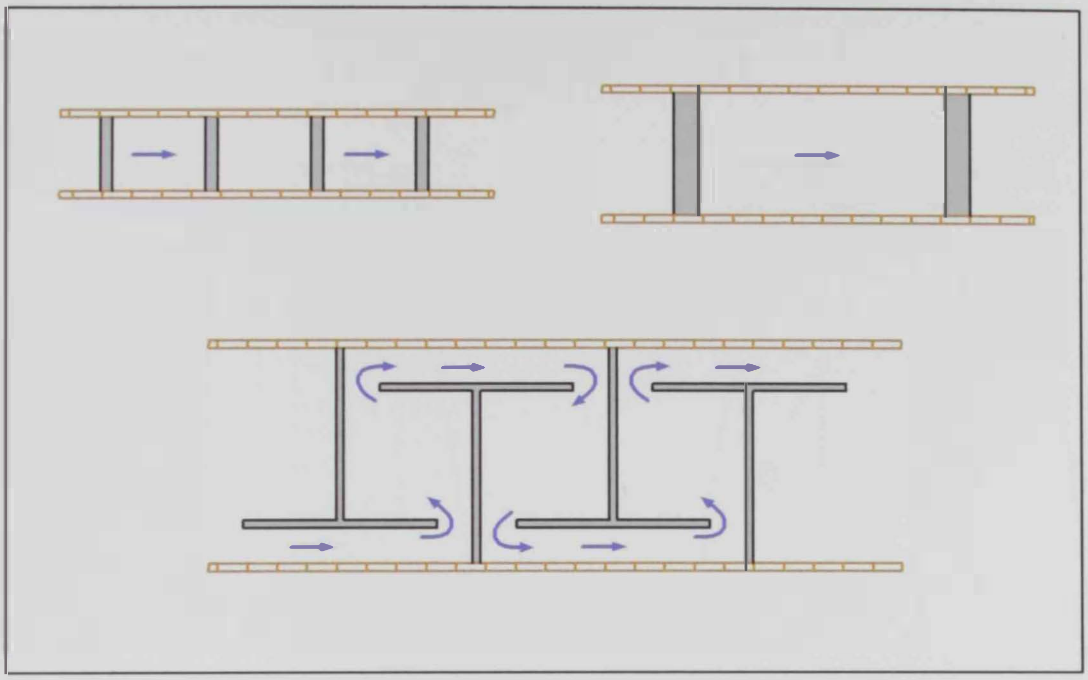

Figure 2.4. Plan views of in-channel infiltration systems with low weirs in narrow, steep channel (upper left); bigger dams in wider, more gently sloping channel (upper right); and T-levees in wide, flat channels (bottom) (after Bouwer, 2002).

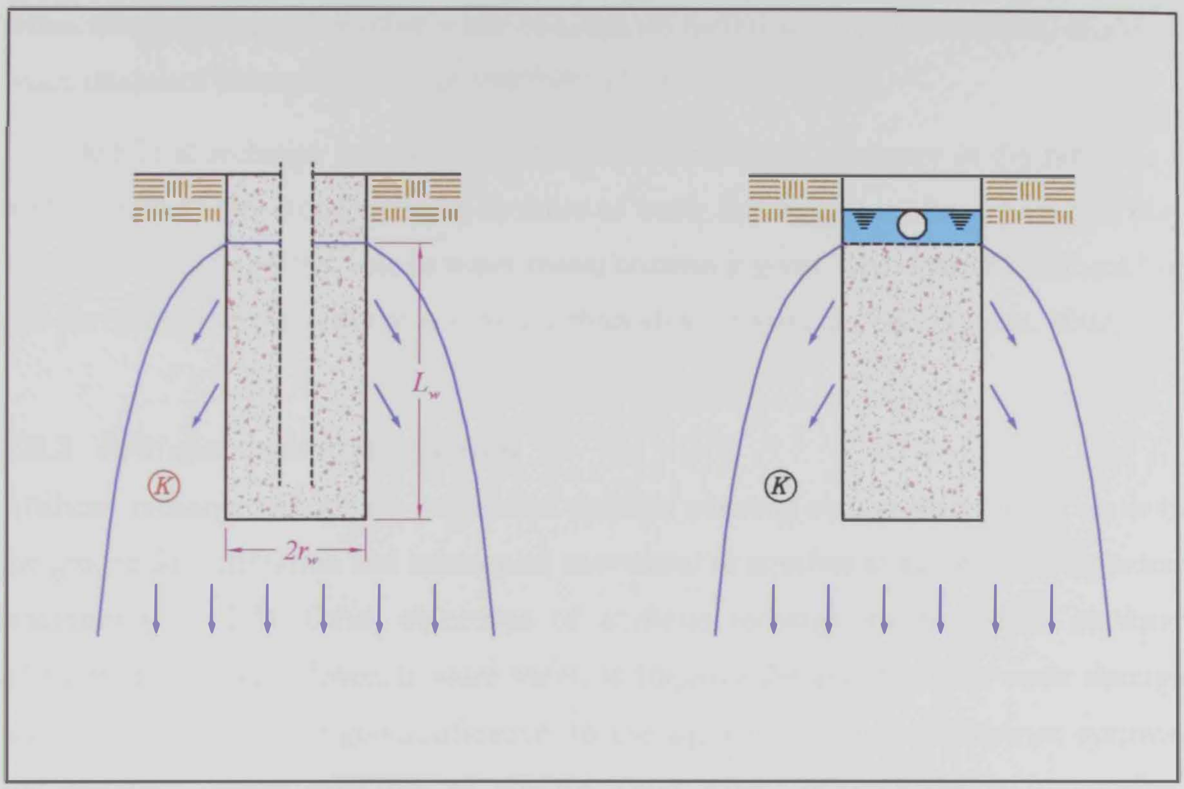

Figure 2.5. Sections showing vadose-zone recharge well (left) with sand or gravel fill and perforated supply pipe; and recharge trench (right) with sand or gravel fill, supply pipe on top of fill, and cover. Arrows represent downward flow in wetted zone with hydraulic conductivity K (after Bouwer, 2002).

Water sources for artificial recharge include water from perennial or intermittent streams that might or might not be regulated with dams, storm runoff (including from urban areas), aqueducts or other water-conveyance facilities, irrigation districts, drinking-water treatment plants, and sewage-treatment plants.

Artificial recharge is expected to become increasingly necessary in the future as a tool of groundwater storage for use in times of water shortage. It is also expected to play an increasingly important role in water reuse, because it gives "soil-aquifer treatment," or geo-purification of the effluent as it moves through soils and aquifers (Bouwer, 2002).

2.5.2 Artificial recharge systems

Artificial recharge systems are engineered systems whereby surface water is put on or in the ground for infiltration and subsequent movement to aquifers to augment groundwater resources (Fig. 2.3). Other objectives of artificial recharge are to reduce seawater intrusion or land subsidence, to store water, to improve the quality of the water through soil-aquifer treatment or geo-purification, to use aquifers as water conveyance systems, and to make groundwater out of surface water where groundwater is traditionally preferred over surface water for drinking (Bouwer, 2002).

Surface Infiltration

Surface infiltration systems for artificial recharge are divided into in-channel and off-channel systems. In-channel systems consist of dams placed across ephemeral or perennial streams to back the water up and spread it out, thus increasing the wetted area of the streambed or floodplain so that more water infiltrates into the ground and moves down to the groundwater (Fig. 2.4).

Some dams consist of low weirs spaced a small distance apart; others are larger dams spaced a greater distance apart (Fig. 2.4, top). The larger dams often need considerable spillway capacity to pass large flows. Sometimes they have a sacrificial section that washes out during high flows and is replaced when the flood danger is over. Steel weirs, earth dams, concrete dams, or inflatable rubber dams are used. The latter are injected with water or air; air is generally preferred. Air pressures are relatively low (about 10 psi, or 1.5 kPa).

When inflated, some water can spill over the dam, but for the big floods they are deflated to lie flat on their foundation. Where channels have small slopes and water depths, water is spread over the entire width of the channel or floodplain by placing T- or L-shaped earthen levees about 1 m high in the channel (Fig. 2.4, bottom) (Bouwer, 2002).

Off-channel surface recharge systems consist of specially constructed infiltration basins (Fig. 2.3), lagoons, old gravel pits, flood-irrigated fields, perforated pipes, or any other facility where water is put or spread on the ground for infiltration into the soil and movement to underlying groundwater.

Water sources for in- and off-channel recharge systems should be of adequate quality to prevent undue clogging of the infiltrating surface by deposition and accumulation of suspended solids (sediment, algae, and sludge); by formation of biofilms and biomass on and in the soil; by precipitation of calcium carbonate or other salts on and in the soil; and by formation of gases that stay entrapped in the soil, where they block pores and reduce the hydraulic conductivity. Gases sometimes also accumulate under the clogging layer, where they form a vapor barrier to downward flow. Pretreatment of the water to reduce suspended solids, nutrients, and organic carbon, and regular drying of the system to enable drying and cracking of the clogging layer and physical removal of the clogging layer might be necessary to minimize clogging effects. However, even when suspended solids, nutrients, and organic carbon are mostly removed from the water, clogging still is likely to occur because of microbiological growth on the infiltrating surface (Bouwer, 2002).

Surface infiltration systems normally require permeable surface soils to get high infiltration rates and to minimize land requirements. Where permeable soils occur deeper down and the less permeable overburden is not very thick, the overburden can be removed so that the basin bottom is in the more permeable material. Vadose zones should be free from layers of clay or other fine-textured materials that unduly restrict downward flow and form perched groundwater that waterlogs the recharge area and reduces infiltration rates. Perched water can also form on aquitards where aquifers are semi-confined.

Aquifers should be unconfined and sufficiently transmissive to accommodate lateral flow of the infiltrated water away from the recharge area without forming high groundwater mounds that interfere with the infiltration process. Also, soils, vadose zones, and aquifers should be free from undesirable contaminants that can be transported by the water and move to aquifers or other areas where they are not wanted (Bouwer, 2002).

Vadose-Zone Infiltration

Where sufficiently permeable soils and/or sufficient land areas for surface infiltration systems are not available, groundwater recharge can also be achieved with vertical infiltration systems, such as trenches or wells in the vadose zone. Recharge trenches are dug with a backhoe and are typically less than about 1 m wide and up to about 5 m deep (Fig. 2.5). They are backfilled with coarse sand or fine gravel. Water normally is applied through a perforated pipeline on the surface of the backfill, and the trench is covered to blend in with the surroundings. For example, a layer of topsoil for grass or other plantings is placed on top of the backfill to blend in with landscaping, or concrete slabs or other paving are added where the area is paved (Bouwer, 2002). Sand-filled ditches have been tested in agricultural areas in Jordan to intercept surface runoff for deeper infiltration into the vadose zone (Abu-Zreig *et al.*, 2000).

Vadose-zone wells (also called recharge shafts or dry wells) are normally installed with a bucket auger, and they are about 1 m in diameter and as much as 60 m deep (Fig. 2.5). The wells are also backfilled with coarse sand or fine gravel. Water is normally applied through a perforated or screened pipe in the center.

The main advantage of recharge trenches or wells in the vadose zone is that they are relatively inexpensive. The disadvantage is that eventually they clog up at their infiltrating surface because of accumulation of suspended soils and/or biomass. Because they are in the vadose zone, they cannot be pumped for "backwashing" the clogging layer, or redeveloped or cleaned to restore infiltration rates. To minimize clogging, the water should be pretreated to remove suspended solids. For recharge trenches, pretreatment is accomplished in the trench itself by placing a sand filter with possibly a geo-textile filter fabric on top of the backfill. Where this would reduce the flow into the backfill too much, the recharge trench could be widened at the top to create a T-trench with a larger filter area than the surface area of the trench itself (Bouwer, 2002).

Wells

Direct recharge or injection-wells are used where permeable soils and/or sufficient land area for surface infiltration are not available, vadose zones are not suitable for trenches or wells, and aquifers are deep and/or confined. Truly confined aquifers might still be rechargeable, because such aquifers accept and yield water by expansion and compression of the aquifer itself and, particularly, of inter-bedded clay layers and aquitards that are more compressible than the sands and gravels or consolidated materials of the aquifer. However, excessive compression of aquifer materials by over-pumping is mostly irreversible (Bouwer, 1978).

Recharge might also be possible through semi-confining layers. However, this situation creates quality deterioration in the lower aquifer if the groundwater above the aquitard is of low quality due to irrigation, septic-tank leach fields, and other incidental recharges. In the USA, the water used for well injection is usually treated to meet drinking-water quality standards for two reasons. One is to minimize clogging of the well-aquifer interface, and the other is to protect the quality of the water in the aquifer, especially where it is pumped by other wells in the aquifer for potable uses. Where groundwater is not used for drinking, water of lower quality can be injected into the aquifer (Bouwer, 2002).

Although clogged recharge wells can be redeveloped and rehabilitated with conventional techniques, a better approach is to prevent serious clogging by frequent pumping of the well, for example, about 15 min of pumping once, twice, or three times per day. This frequent "backwashing" of the clogging layer, which of course, requires a dedicated pump in the well, often prevents serious clogging. As a matter of fact, the frequent backwashing might eliminate the need for membrane filtration (Bouwer, 2002).

Combination Systems

Whenever possible, surface infiltration systems are preferred, because they offer the best opportunity for clogging control and the best soil-aquifer treatment if quality improvement of the water is of importance. If permeable soils occur at the ground surface or within excavatable depth, the water can directly move into the coarse soils. However, where deeper fine-textured layers significantly restrict the downward movement of the water to the aquifer, and perched groundwater rises too high, surface infiltration can still be used if vertical infiltration systems are installed through the restricting layer (Fig. 2.6). The upper parts of the systems then function as drainage systems of the perched groundwater, while the lower parts function as systems for infiltration and recharge of the aquifer. If the bottom of the restricting layer is not too deep (less than 3 m, for example), trenches can be used to drain the perched water and send it down to the aquifer (Fig. 2.6, left). For deeper restricting layers (up to about 40 m) vadose-zone wells can be used (Fig. 2.6, center), whereas conventional wells can be used where the restricting layers are beyond reach of bucket augers (Fig. 2.6, right). The wells would then be screened above and below the restricting or confining layer (Bouwer, 2002).

The advantage of the systems shown in Figure 2.6 is that the water has been pre-filtered through the soil and the perched groundwater zone, so that its clogging potential is significantly reduced. Even then, if the lower part of the system extends into the aquifer, as in Figure 2.6 (right), it would probably be good practice to regularly pump the well. Water-quality issues also must be considered, particularly where the water above the restricting layer is of lower quality than that in the aquifer itself.

The principle of draining perched groundwater for recharge of underlying aquifers with systems such as shown in Figure 2.6 has not been adequately tested in the field. Thus, pilot testing of these systems should always be done to see if they work satisfactorily and how they should best be managed before large projects are installed. Pilot testing is also desirable for the simpler systems of basins, trenches, vadose-zone wells, and aquifer wells, because how these systems perform and how they should be designed and managed depends very much on local conditions of soil, hydrogeology, climate, and water quality (Bouwer, 2002).

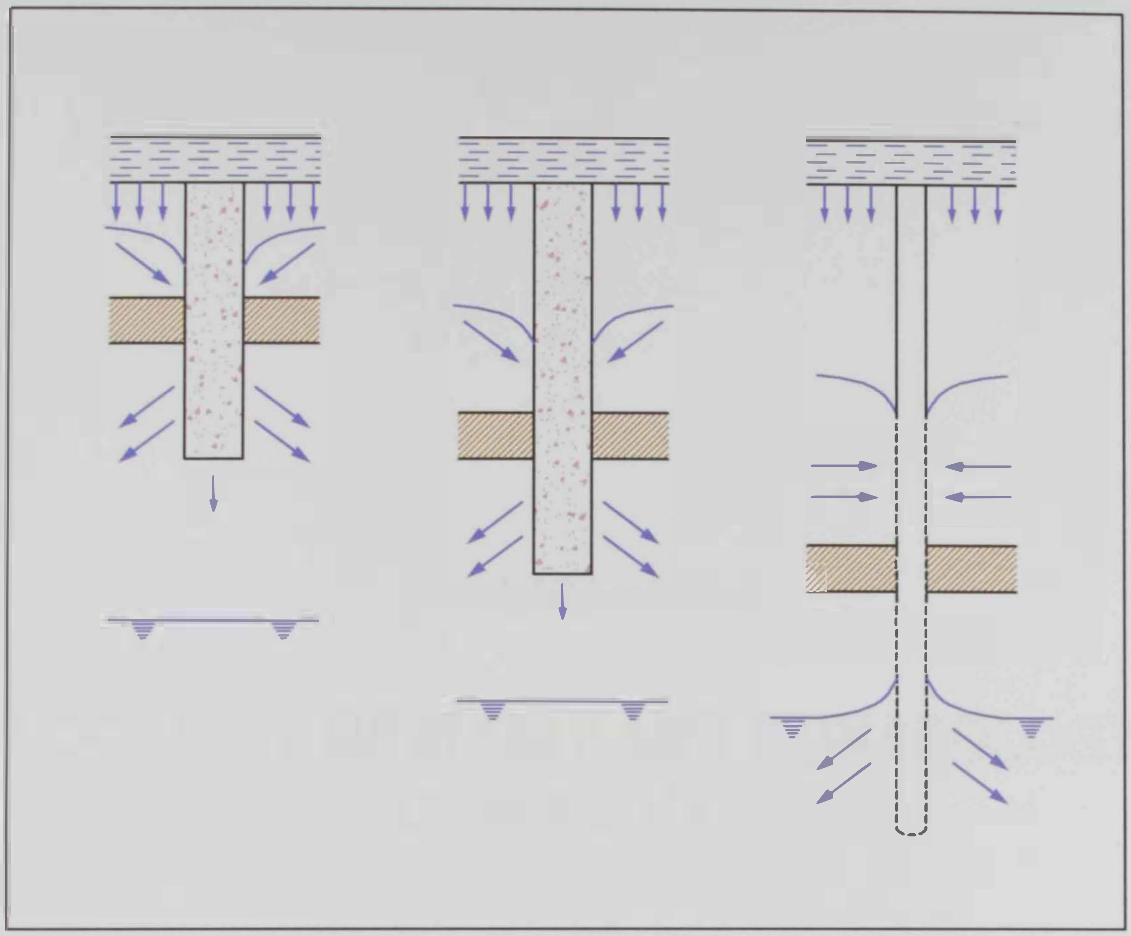

Figure 2.6. Sections showing surface infiltration systems with restricting layer (hatched) and perched groundwater draining to unconfined aquifer with trench (left), vadose-zone well (center), and aquifer well (right) (after Bouwer, 2002).

CHAPTER THREE:

LOCATION, GEOLOLOGY AND CLIMATOLOGY OF WADI HAM

3.1 Location and General Characteristics

Wadi Ham lies in the Eastern Region of the United Arab Emirates north west of Fujairah. It is a large south easterly flowing wadi complex comprising parts of the Masafi mountains in its north western upper part and alluvial plains to the east around Fujairah in its lower part. The upper part is characterized by narrow valleys and steep slopes while the lower part has a very low gradient and forms a broad flat plain of coarse alluvial gravels and boulders covering the area between the Ham Dam and the sea. The location of Wadi Ham is shown in Figure 3.1

3.1.1 The catchment

Three main tributaries can be recognized within the area of Wadi Ham, namely, Wadi Ham, Wadi Yabsah and Wadi Al-Farfar (Fig. 3.2). The main tributary, Wadi Ham, rises in the mountains immediately south and south east of Masafi draining south-eastward and eventually it drains into the Gulf of Oman between Fujairah and Kalba.

Wadi Yabsah drains the area south of Jabal Yabsah and Jabal Fariq, flows west of Jabal Mutarad and eventually confluences with Wadi Ham. However, the confluence of Wadi Yabsah with Wadi Ham is located in the downstream side of Wadi Ham Dam and consequently do not contribute to the storage accumulated behind the Ham Dam.

On the other hand, Wadi Al-Farfar constitutes a part of the catchment to Wadi Ham Dam. It includes a small sub-tributary, called Wadi Mimduk, which it confluences with at Al-Farfar. The confluence runoff of these two wadis flows north of Jabal Ruham and subsequently confluences with Wadi Ham about one kilometer upstream of the Ham Dam. The total catchment area formed by Wadi Ham and Wadi Al-Farfar is approximately 195 km^2 (Fig. 3.3). This represents the actual surface water catchment to the Ham Dam to be considered in the simulation of the surface runoff to the Ham Dam as described in Chapter four.

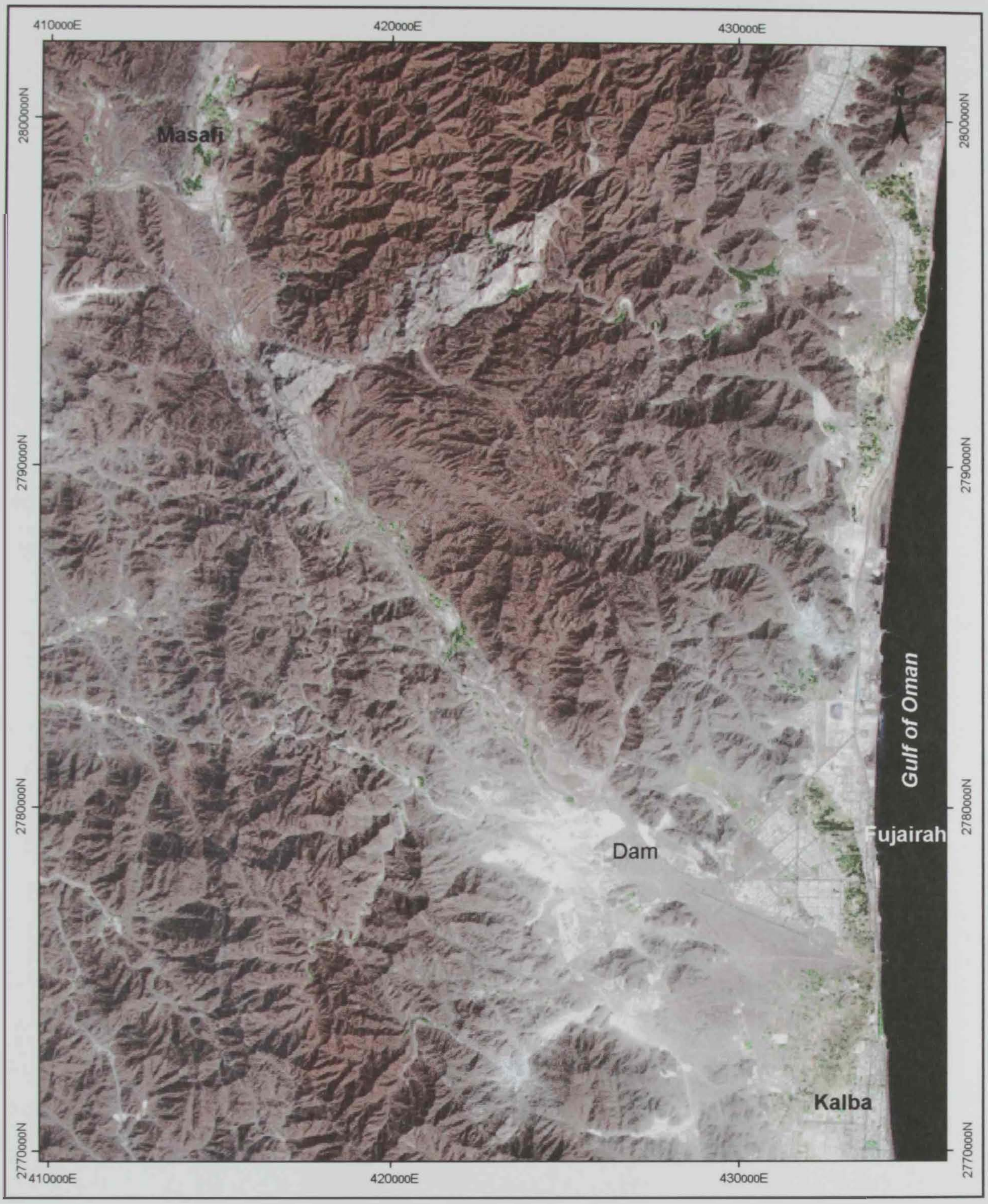

Figure 3.1. IRS true-color, 5 m resolution remote sensing image of Wadi Ham.

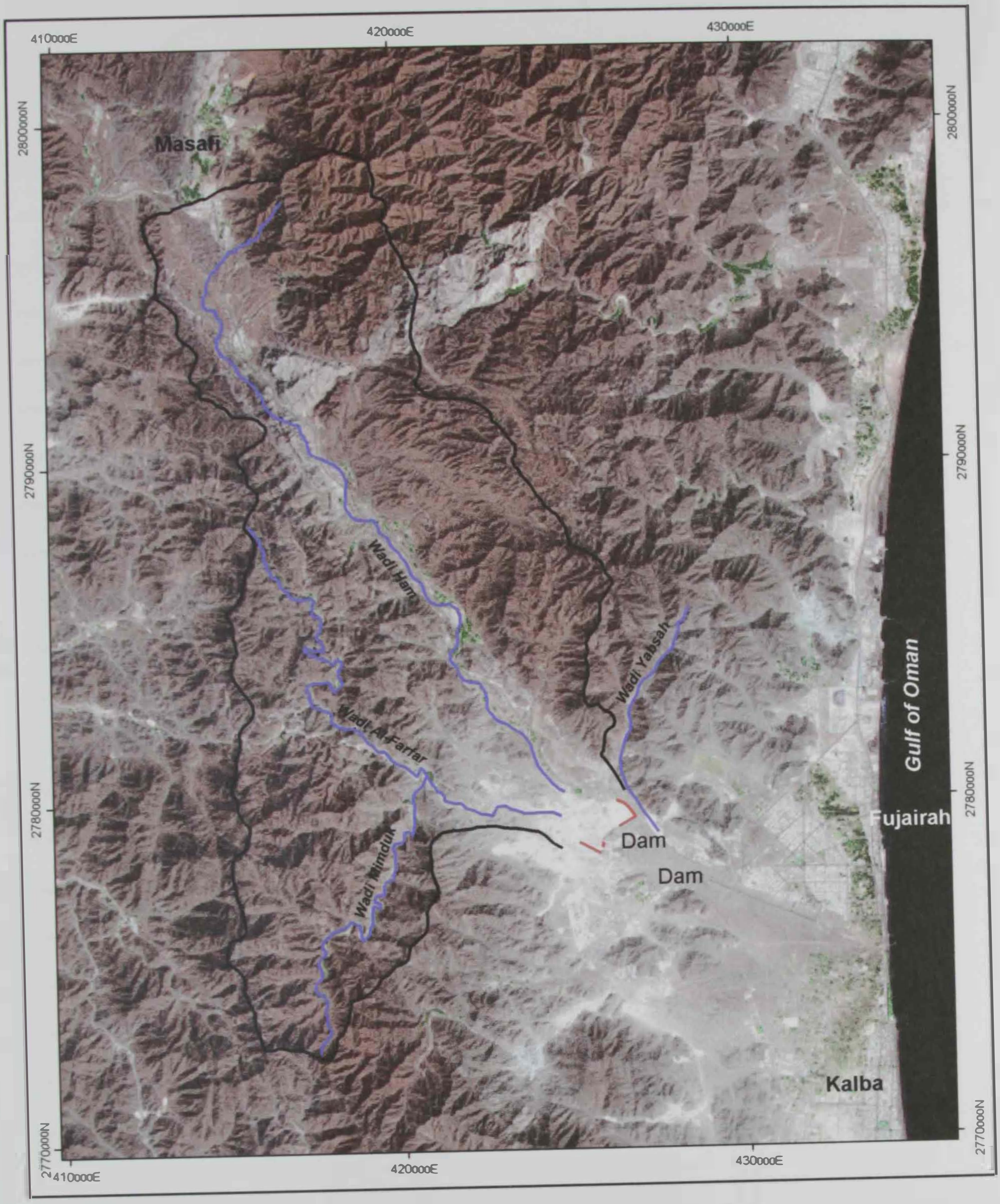

Figure 3.2. Wadi Ham tributaries.

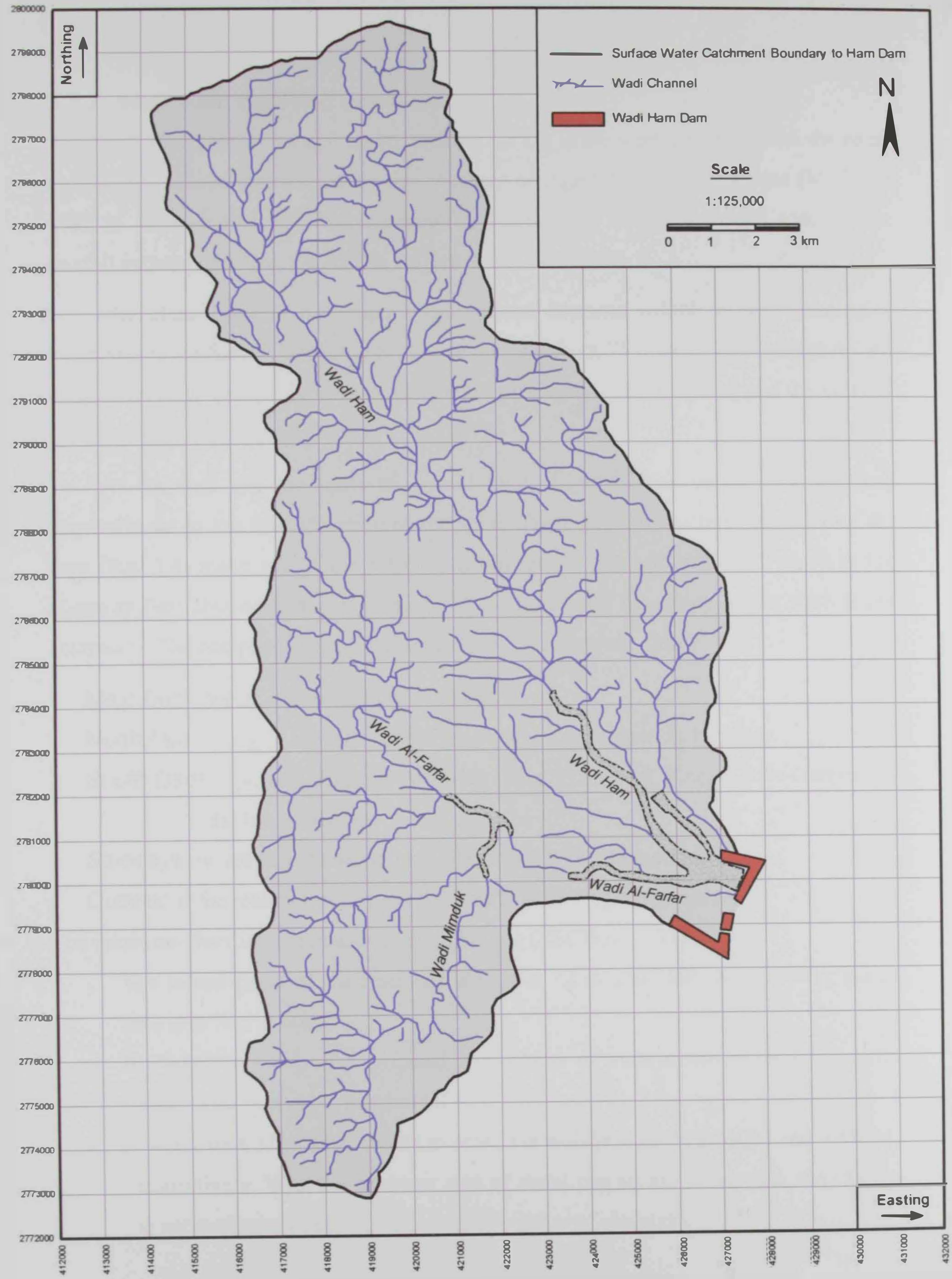

Figure 3.3. Wadi Ham catchment.

3.1.2 Wadi Ham dam and reservoir

The Wadi Ham Dam is located approximately 8.5 km to the west upstream from the coast at Fujairah. It was constructed by the Ministry of Agriculture and Fisheries (MAF) in 1982 to act as a recharge dam intercepting flash floods caused by rapid runoff from rainfall events within the Wadi Ham surface water catchment.

The Ham Dam is built upon wadi gravel deposits, which overlie lithologies belonging to the Samail Ophiolite Strata, on an altitude of 75m amsl. The average height of the dam is 16 m with its crest at an average elevation of 88.5 m amsl and the crest of spillways is at 84.5 m amsl (Electrowatt, 1981).

At the dam site the wadi is crossed by a ridge of rock outcrops which runs perpendicular to the flow direction of the wadi. The complex structures comprising the dam (Fig. 3.4) make use of the ridge by filling in the gap between a hill close to the Highway (left abutment) and a group of hills at the right hand side of the wadi (right abutment). The complex of structures consists of (Electrowatt, 1981):

Main Dam: baring the riverbed of Wadi Ham.

North Dam: forming the left wing of the structure to protect the highway.

South Dam: separated from the two other dams by a series of small hills comprising the two spillways.

Spillways: which have to work for major floods and of two outlets.

Outlets: of the reservoir which are the bottom of the right abutment.

The retention-elevation curve shows the following (ESCWA, 1997):

- The lowest point of the reservoir area is at 72 m amsl and the recording gauge zero is at 74.5 m amsl.
- At "lake" elevation of 82 m amsl, the volume of water stored in the reservoir is about 2 million cubic meters (MCM).
- At elevations 83 m amsl and 84 m amsl, the storage is about 3 MCM and 4 MCM, respectively. With the reservoir area of about one square kilometers, the storage is approximately one MCM in each one meter of elevation.

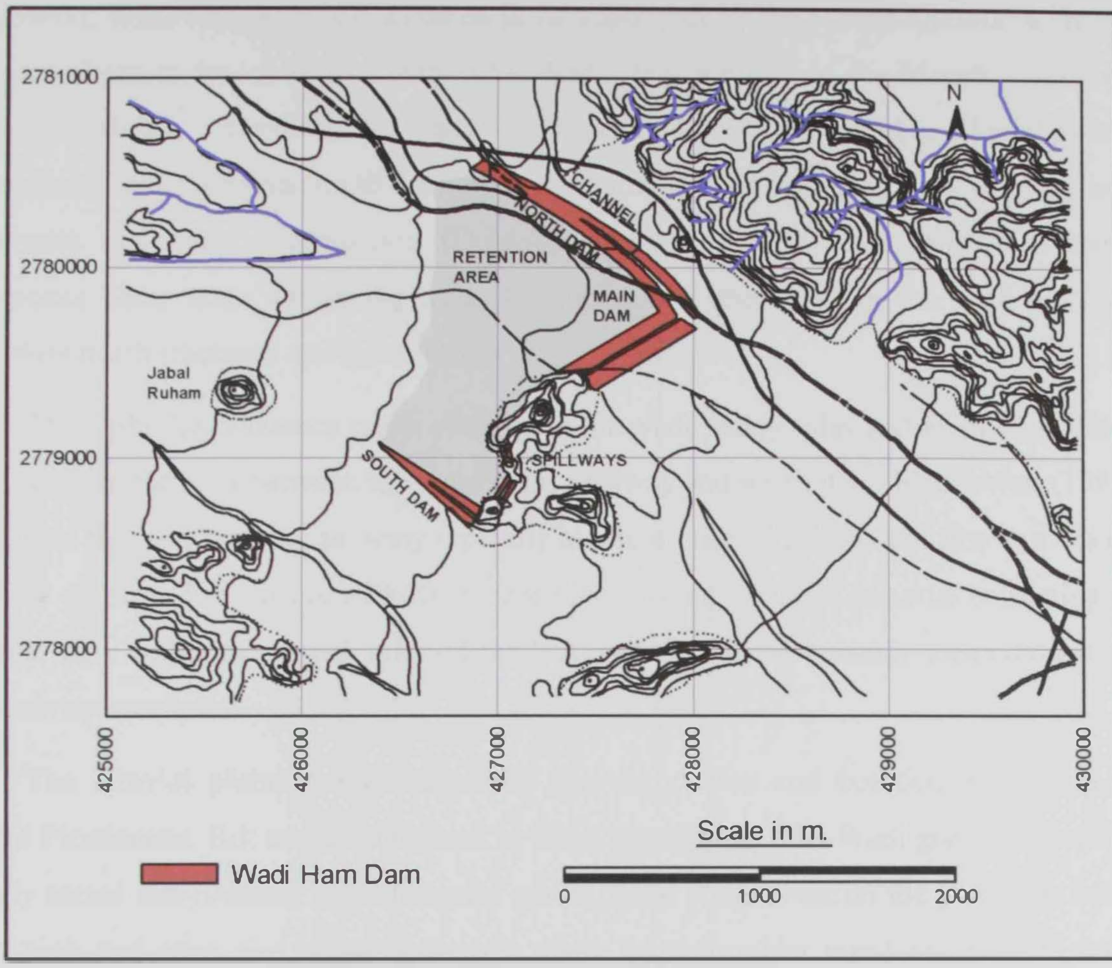

Figure 3.4. Wadi Ham Dam (after Electrowatt, 1981).

3.2 Geology of Wadi Ham

In general, Wadi Ham area is dominated in its upper part by the Masafi mountains and by alluvial plains in its lower part (Fig. 3.5). Wadi Ham initiates in the Masafi mountains where it is deeply incised forming narrow valleys which lead downward to alluvial plains. The Masafi mountains belong to the Samail formation which comprises rock of prevailing ultrabasic and basic composition (Ophiolite sequence). The wadi follows a major composite shear zone within the Samail complex and the rocks in the wadi area are therefore much fractured and tectonized.

The Ophiolite sequence rocks outcrop on the wadi valley sides and as small isolated hills, such as the ones between the main dam, spillway and south dam. Electrowatt (1981) described the rock material as being typically medium grained gabbro and fine to medium grained diorites. The surface of hills is generally covered by tallus material consisting of angular rock components with silt and sand. At outcrops these volcanic rocks have been extensively weathered.

The alluvial plains contain basically gravels, cobles and boulders belonging to recent Pleistocene. Silt and sand content is often remarkable. The Wadi gravels comprise poorly sorted sub-rounded to sub-angular gravel. Sand content within the gravels is often very high and clast size ranges from silt grade up to boulder sized material. Erosion presumably had already started in Paleocene times and progressively drowned the pre-Paleocene relief and resulted in narrow valleys with flat gravel floors.

The gravels are typically composed of basic igneous clasts with other clasts of very well cemented sandstone and conglomerates. Electrowatt (1981) subdivided them into *recent gravels*, being slightly silty sand gravel with some cobbles; *young gravels*, which are silty sandy gravels with many cobbles and boulders, and finally *old gravels*, which are silty sandy gravels with many cobbles and boulders which are weathered and cemented.

3.3 Climatology of Wadi Ham

The catchment to the Ham Dam and the general area lies within the hot arid desert climatic zone. Evaporation rates are very high in the region of Wadi Ham due to the prevailing arid climatic conditions that dominates the Eastern Region of the UAE.

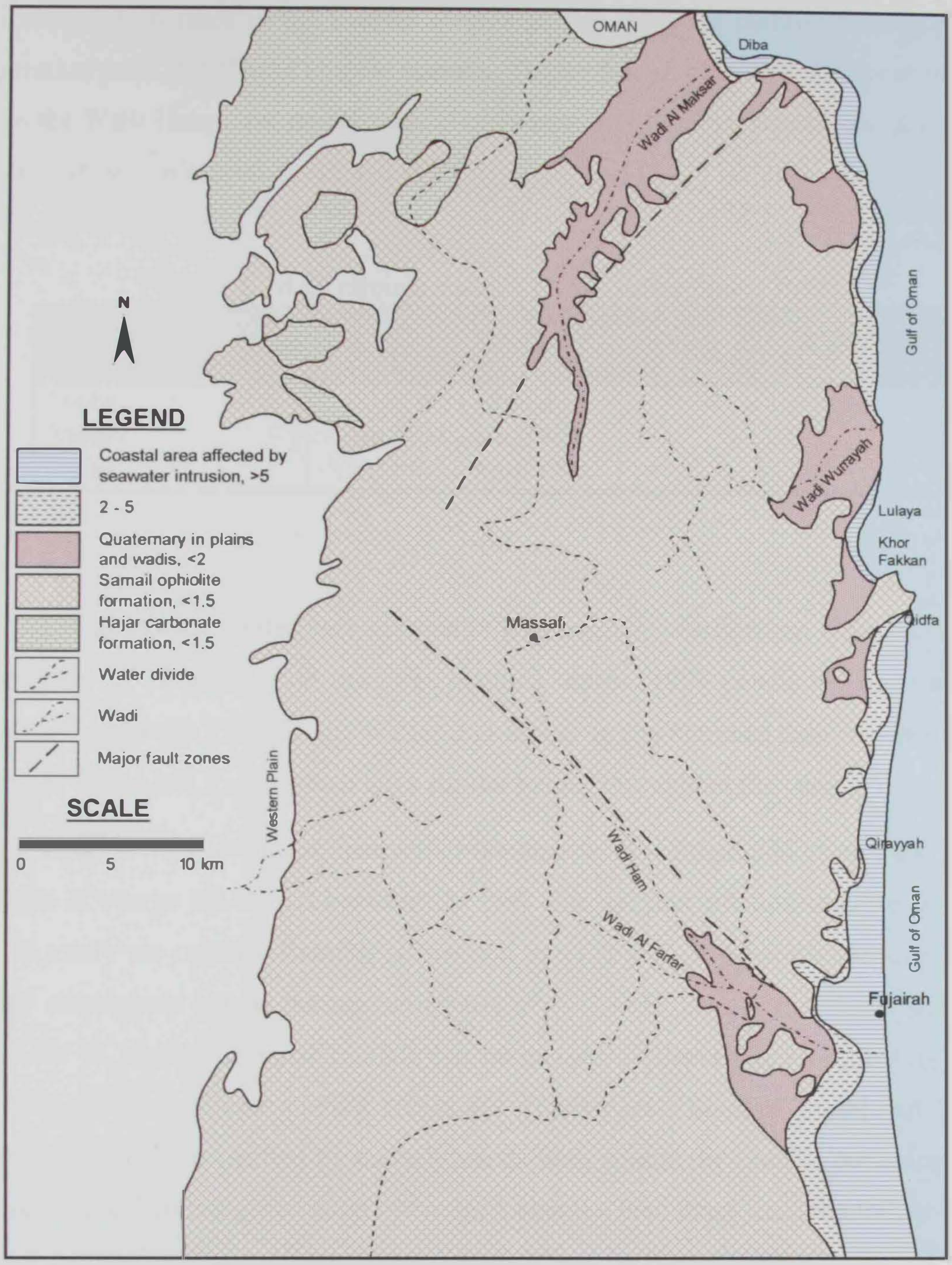

Figure 3.5. Geological map of the eastern region of UAE (revised after ESCWA, 1997).

Evaporation rates in the Eastern Region of the UAE are measured using class-A evaporation pans at different climate stations. Three of these climate stations are installed within the Wadi Ham area, namely, the climate stations at Dibba, Masafi and Kalba. The details of these stations are presented in Table 3.1.

Table 3.1. MAF climate stations details (after Entec, 1996)

Climate Station	Length of Evaporation Data Record	Elevation, m (AMSL)
Dibba	Oct. 1973 - Present	10
Masafi	Jan.1974 – Jan. 1985	450
Kalba	Oct. 1967 – April 1985	15.0

Mean evaporation rates vary seasonally with average yearly mean evaporation rates in the order of 10 mm/day at the three stations. Entec, 1996 concluded that values at Masafi are generally lower in the winter and higher in the summer than the two coastal stations at Dibba and Kalba which have similar values throughout the year.

Rainfall is very low in comparison to the high levels of potential evapotranspiration and there is a large net deficit between the two. The net precipitation input to the Wadi Ham is solely via rainfall. Rainfall is low and variable and is generally associated with 'frontal' movements across the northern parts of the UAE. During the winter period of November to March, irregular rainfall can be caused by depressions originating in the eastern Mediterranean and moving across the Arabian area. Summer months of June to September tend to be typified by easterly winds with spasmodic rainfall occurring along the inter-tropical convergence zone where it crosses the land mass from the Indian Ocean. Rainfall during the winter months is usually much heavier (Entec, 1996).

Rainfall is the primary input vector of the hydrologic cycle and is a major input in any hydrologic modeling work. Rainfall varies greatly in both time and space. The variability can be visualized by analyzing rainfall records of different gauging stations. The variability in space is addressed by studying the rainfall areal distribution and the variability in time is addressed using statistical analysis of historical records. Comprehensive analyses of rainfall data of Wadi Ham are presented in what follows.

3.4 Rainfall Analysis

Rainfall is the primary input to the hydrologic budget and thus it is essential to study and critically analyze the available historical records of rainfall in Wadi Ham. Rainfall is measured at a number of MAF Rainfall Recording and Climate Stations in and around the catchment area of the Wadi Ham. The stations are located at Bithna, Farah, Fujairah, Jebel Sharmah, Masafi and Sifuni. Rainfall data exists for these stations for variable periods dating back to 1967. The locations of these stations in relation to the surface water catchment of Wadi Ham are shown in Figure 3.6. The station details are listed in Table 3.2.

Table 3.2. MAF rainfall recording stations details (after Entec, 1996)

Rainfall Recording Station	Installation Date	Elevation, m (AMSL)
Bithna	April1971	190.0
Farah	May 1980	220.0
Fujairah	October 1967	10.0
Jebel Sharmah	January 1981	410.0
Masafi Climate St.	October 1965	450.0
Sifuni	August 1976	335.0

For most hydrologic analyses, it is important to know the areal distribution of precipitation. Usually, weighted average depths for representative portions of the watershed are determined and used for this purpose. Methods of calculating areal rainfall averages include the arithmetic average of gauged quantities, the isohyetal map method and the Thiessen polygon method. The arithmetic average may be incorrect if the precipitation is nonuniform and the stations are unevenly distributed within the area.

The isohyetal method is generally considered to be the most accurate method for computing the average areal rainfall over a drainage basin (Singh, 1992). This method is accomplished by drawing contours of equal rainfall (isohyetal map) and the area enclosed between each two successive contours (isohyets) is considered representative of these two isohyets. However, the number of raingauges available in Wadi Ham is not sufficient to develop such an isohyetal map.

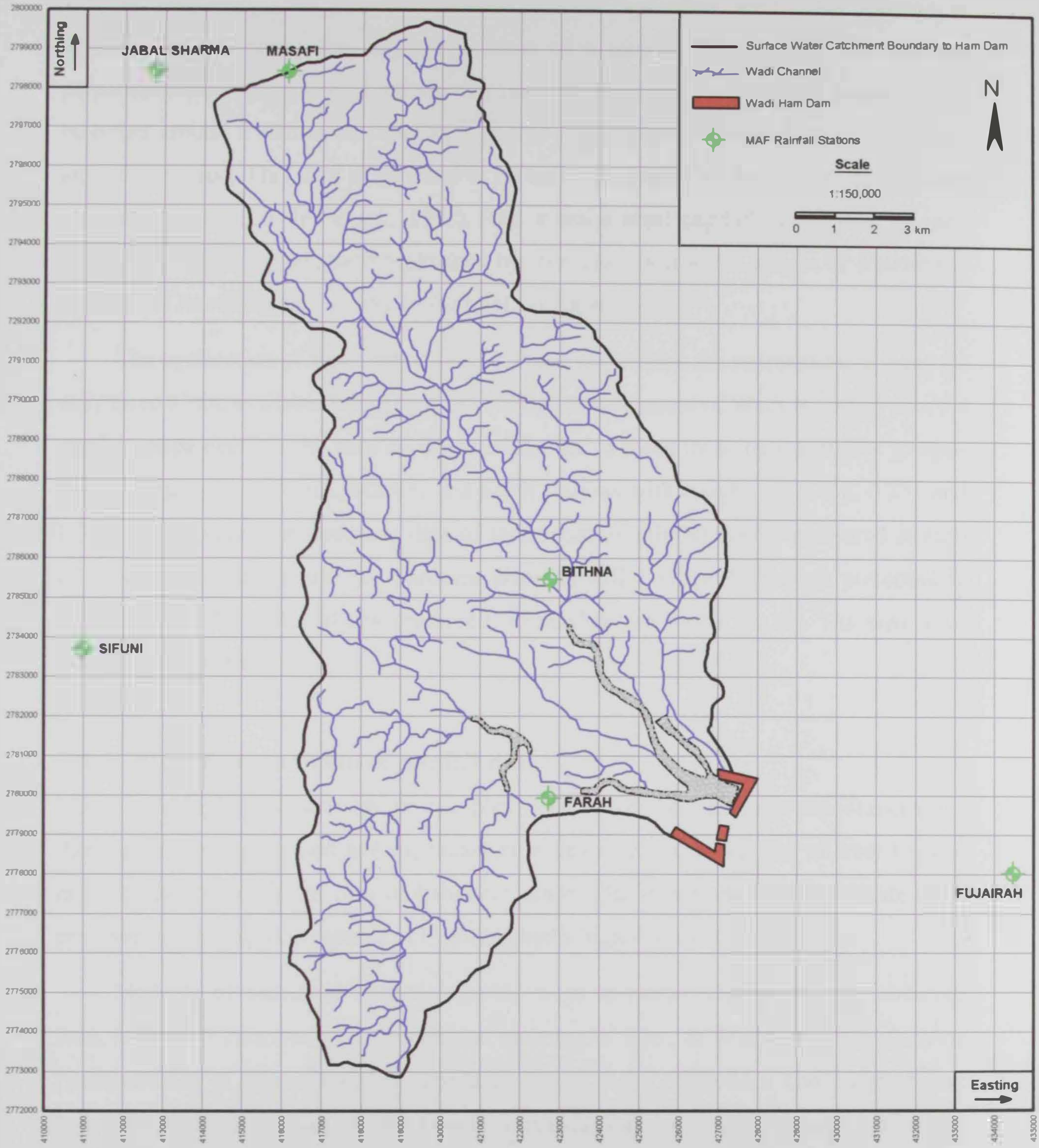

Figure 3.6. Location of rain gauges.

In this study, the Thiessen polygon method has been used. This method is accomplished by connecting adjacent stations on a map by straight lines and erecting perpendicular bisectors to each connecting line. The polygon formed by the perpendicular bisectors around a station encloses an area that is everywhere closer to that station than any other station. This area is assumed to be best represented by the rainfall depth at the enclosed station (Linsley *et al.*, 1992). The average areal rainfall is the sum of each individual station depths, each multiplied by its weight where the weight of a station is the ratio of its representative area to the total catchment area.

The application of this method (Fig. 3.7) has shown that the representative areas for only three of the available raingauges fall within the catchment of Wadi Ham and thus the areal average depth is the sum of weighted depths of these three representative gauges. These are gauges at Bithna, Masafi, and Farah stations with weights of 0.438, 0.231 and 0.331 respectively. The complete data of total monthly rainfall depths measured at each of these three stations and the Thiessen areal rainfall average depths are presented in Appendix A. All the analyses presented hereinafter are based on the Thiessen areal rainfall average depths.

3.4.1 Statistical analysis of rainfall data

Many hydrologic processes are so complex because of their inherent randomness that they can not be interpreted and explained on a deterministic basis. The alternative is to analyze and explain them in a probabilistic sense. The information to investigate these processes is contained in records of historical hydrologic observations.

Methods of statistical analysis provide ways to reduce and summarize observed data, to present information in precise and meaningful form, to determine the underlying characteristics of the observed phenomena, and to make predictions concerning future behavior. Statistical analysis deals with methods for drawing inferences about the population based on examination of sample values from the population. The inferences include information about the central tendency, range, distribution within the range, variability around the central tendency, degree of uncertainty, and frequency of occurrence of values (Viessman and Lewis, 1995).

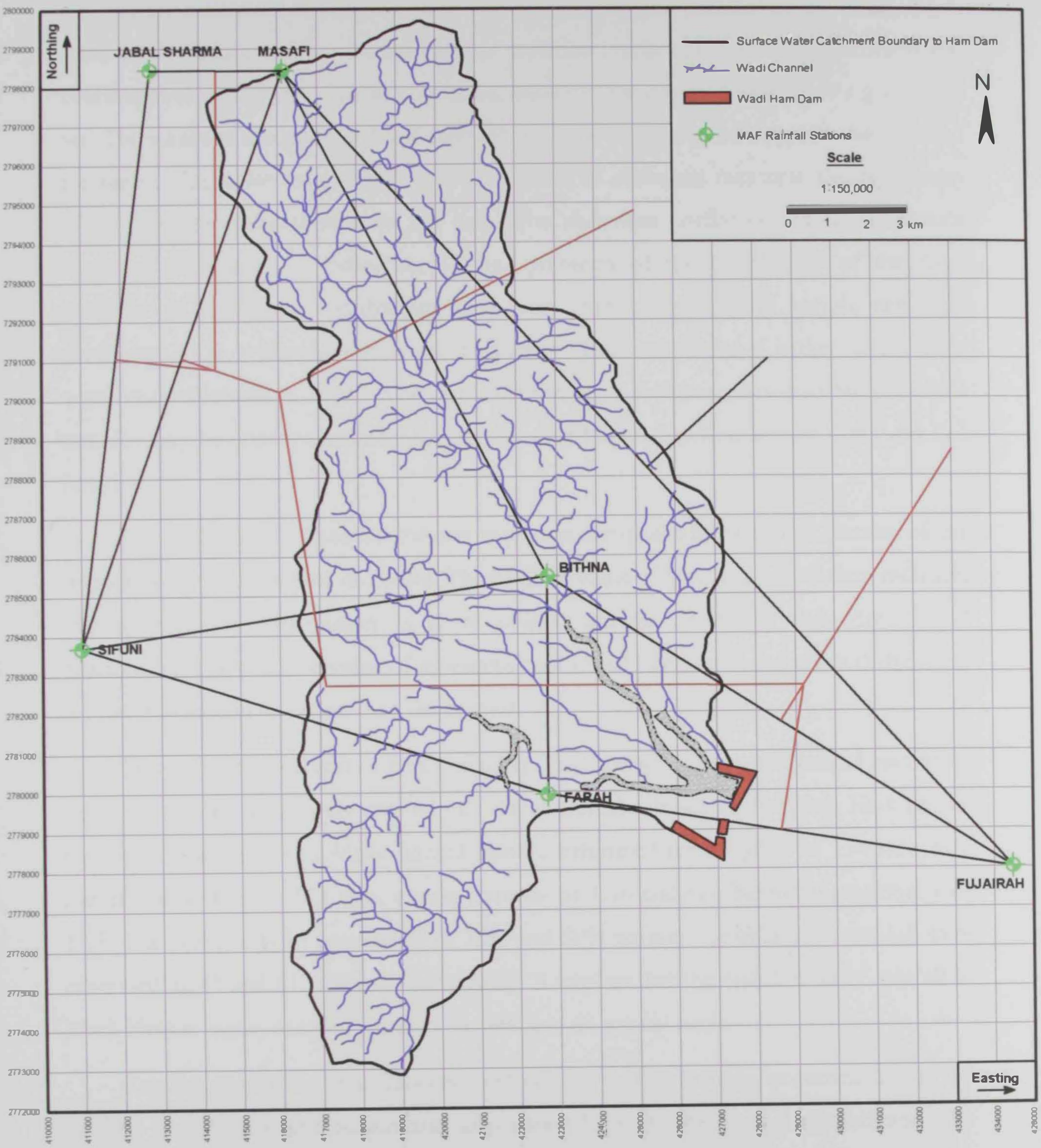

Figure 3.7. Application of Thiessen Polygon method.

These inferences are expressed in terms of distribution parameters including mean, standard deviation, range and coefficient of variation (variance), skewness coefficient and coefficient of asymmetry. The sample mean measures the central tendency of a given data set. The standard deviation and variance measure the dispersion of sample values around the mean. The skewness coefficient or coefficient of skewness measures the asymmetry of the frequency distribution of the data. The skewness coefficient has an important meaning since it gives indication of the symmetry of the distribution of the data. Symmetrical frequency distributions have very small or negligible sample skewness coefficient (C_s) while asymmetrical frequency distributions have either positive or negative coefficients. A small value of C_s often indicate that frequency distribution, of the sample, may be approximated by the normal distribution function since $C_s = 0$ for this function.

The coefficient of excess kurtosis measures the peakedness or the flatness of the frequency distribution near its centre. The positive value of an excess coefficient indicates that a frequency distribution is more peaked around its centre than the normal distribution. The negative value of an excess coefficient indicates that a given distribution is more flat around its centre than the normal.

The statistical analyses of mean monthly, total annual and one day annual maximum rainfall for Wadi Ham were carried out. Total annual rainfall in the Wadi Ham ranges from 8 mm to 499 mm. Mean annual rainfall estimated for 23 years is 154 mm with standard deviation of 123 mm, excess kurtosis of 1.36 and coefficient of asymmetry of 1.18. The probability of occurrence of 75% and 50% normal (mean annual) rainfall were estimated as 47 and 61 percent, respectively. It implies that the distribution of rainfall in Wadi Ham is highly scattered and not dependable on annual basis.

Rainfall distributed from January to December with maximum occurring during the months of February and March and also about 50% of total annual rainfall normally occurs during these two months. Monthly rainfall values range from 0 to 198 mm, mean monthly varies from 1 mm to 42 mm with variation of standard deviation from 2 mm to 62 mm. The monthly standard deviation exceeds the monthly average precipitation and reveals that year-to-year monthly variation in precipitation is quite extreme in the area.

One day annual maximum rainfall values range from 12-137 mm with mean of 50 mm, standard deviation of 34 mm, excess kurtosis of 0.69 and coefficient of asymmetry of 1.06. The probability of occurrence of 75% and 50% of mean rainfall were 58 and 79 percent, respectively. The statistical results are listed in the Table 3.3 and Table 3.4 and presented graphically in Figure 3.8 and Figure 3.9.

Table 3.3. Statistical values of mean monthly rainfall distribution in Wadi Ham

Month	Rainfall Range (mm)	Mean (mm)	% of Normal Rainfall	S.D (mm)
October	0-74	6	4	16
November	0-33	7	4	10
December	0-178	17	11	38
January	0-111	20	13	26
February	0-198	42	27	62
March	0-129	37	24	47
April	0-54	11	7	17
May	0-28	3	2	8
June	0-8	1	1	2
July	0-72	5	3	15
August	0-23	2	2	5
September	0-7	3	2	5

Table 3.4. Statistical values of annual rainfall distributions in Wadi Ham

Rainfall range (mm)	Mean (mm)	S.D (mm)	Kurtosis	Coefficient. of Asymmetry.	Probability of Occurrence of 75% normal	Probability of Occurrence of 50% normal
Total Annual Rainfall Distribution						
8-499	154	123	1.36	1.18	47	61
One Day Annual Maximum Rainfall Distribution						
12-137	50	34	0.69	1.06	58	79

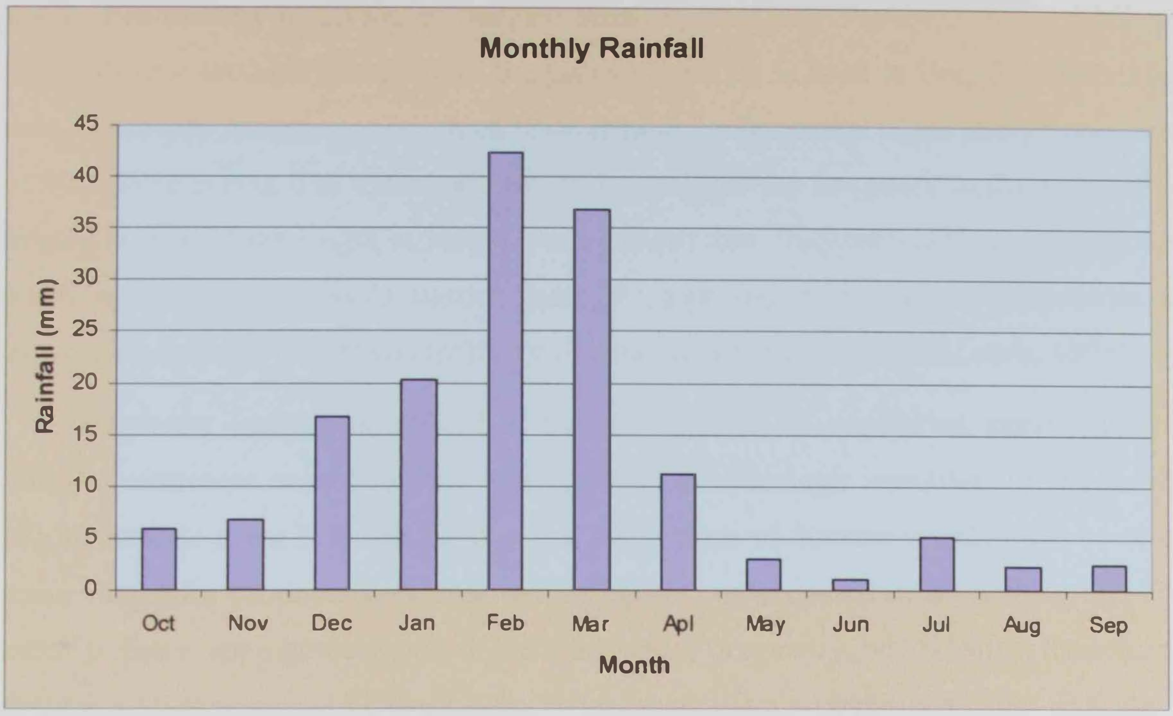

Figure 3.8. Distribution of mean monthly rainfall for Wadi Ham.

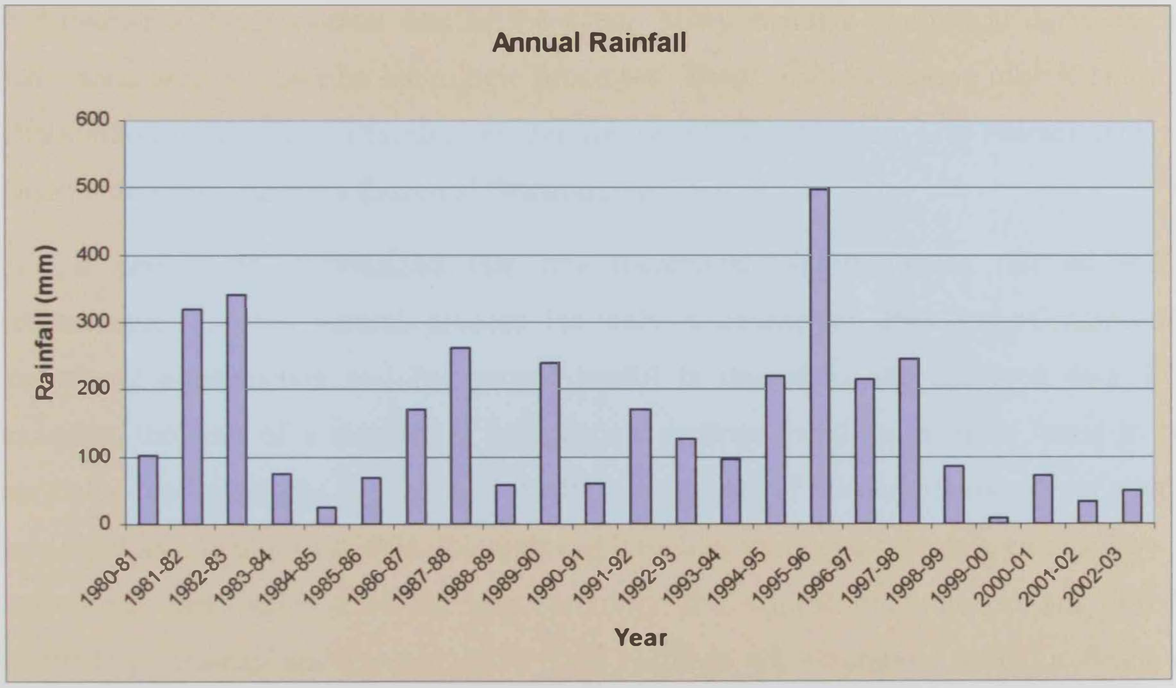

Figure 3.9. Distribution of total annual rainfall for Wadi Ham.

3.4.2 Frequency analysis of rainfall data

The statistical analysis presented in the previous section is used to describe the rainfall data. Frequency analysis is performed to determine the frequency of the likely occurrence of hydrologic events. The hydrologic data to be analyzed for frequency analysis should be treated in light of the length of record. As a general rule, frequency analysis is cautioned when working with records shorter than 10 years and in estimating frequencies of expected hydrologic events greater than twice the record (Viessman and Lewis, 1995).

Frequency analysis is defined as the investigation of population sample data to estimate recurrence or probabilities of magnitudes of hydrologic variables. The *frequency* of a hydrologic event is the probability that some value of discrete variable will occur or some value of a continuous variable will be equaled or exceeded in any given year. The latter is more appropriately called the *exceedance frequency* but is often termed the *frequency* (Viessman and Lewis, 1995). The frequency is a probability and has no units of measure. The reciprocal of the exceedance frequency is the *return period* in years. Frequency analysis of a hydrologic event is performed by fitting a theoretical frequency distribution to the historical data of the event. Many standard theoretical distributions have been used to describe hydrologic processes. These include, among others, Normal Distribution, Log-normal Distribution, Pearson type III Distribution, Log-Pearson type III Distribution and Gumbel's Extremal Distribution.

It should be emphasized that any theoretical distribution is not an exact representation of the natural process but only a description that approximates the underlying phenomenon and has proved useful in describing the observed data. For example, the sum of a number of independent random variables is often found to be normally distributed like annual rainfall which is the sum of the daily rains each of which is viewed as a random variable. Examples of variables that have been known to follow a log-normal distribution include daily, monthly and annual precipitation and runoff volumes (Viessman and Lewis, 1995). Two methods are commonly used for fitting a theoretical distribution to the observed data, namely, the graphical method and the frequency factor method. The graphical method has the advantage of simplicity and visual presentation and has been adopted for the analysis of rainfall data of Wadi Ham.

This method involves fitting of an assumed probability distribution to observed data. The sample data are arranged in descending order of magnitude and each data point is assigned a rank starting with 1 for the highest value , 2 for the second highest value, and so on, and the lowest value will have a rank of n. This arrangement gives an estimate of the exceedance probability (frequency), that is, the probability of a value being equal to or greater than the ranked value. The probabilities of data points are determined from a plotting-position formula. Many plotting-position formulas are available in the literature. The most commonly used plotting-position formula in hydrology is the Weibull formula (Singh, 1992). The Weibull formula takes the form:

$$P_m = \frac{m}{N+1} \tag{3.1}$$

where P_m is the exceedance probability (frequency) of the mth data point (observed value) in the sample of N observations arranged in descending order.

The return period of the mth data point, T_m, is then:

$$T_m = \frac{1}{P_m} = \frac{N+1}{m} \tag{3.2}$$

The observed values and their exceedance probability (frequency) are then plotted on a probability paper corresponding to the assumed probability distribution. The objective of using the probability paper is to linearize the distribution so that plotted data can be represented by a straight line. Several theoretical distributions plot as straight lines on special graph paper developed for use with equations (3.1) and (3.2). For example, a probability paper with an arithmetic scale ordinate and a logarithmic scale probability abscissa is used to evaluate whether a Normal Distribution is approximated by the data. A straight line plot would identify a Normal Distribution. The same paper but with a logarithmic scale as the ordinate tests the apparent fit to a Log-normal Distribution (Viessman and Lewis, 1995).

Frequency analysis of one day annual maximum rainfall and total annual rainfall data of Wadi Ham has been performed using Weibull formula for calculating frequencies and return periods. Different trials have been made to fit the data using probability papers corresponding to different theoretical distributions. The Normal Distribution was found perfectly fitting ($R^2 = 0.99$) with the observed data (Fig. 3.10) and (Fig. 3.11).

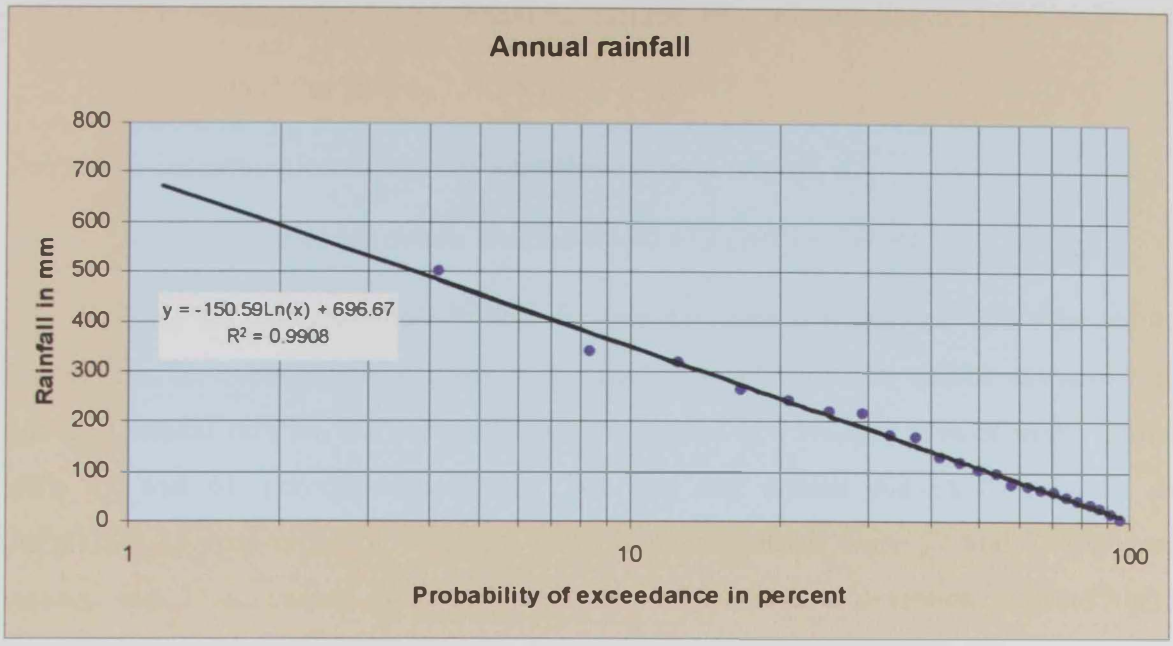

Figure 3.10. Normal distribution fit of total annual rainfall versus frequency.

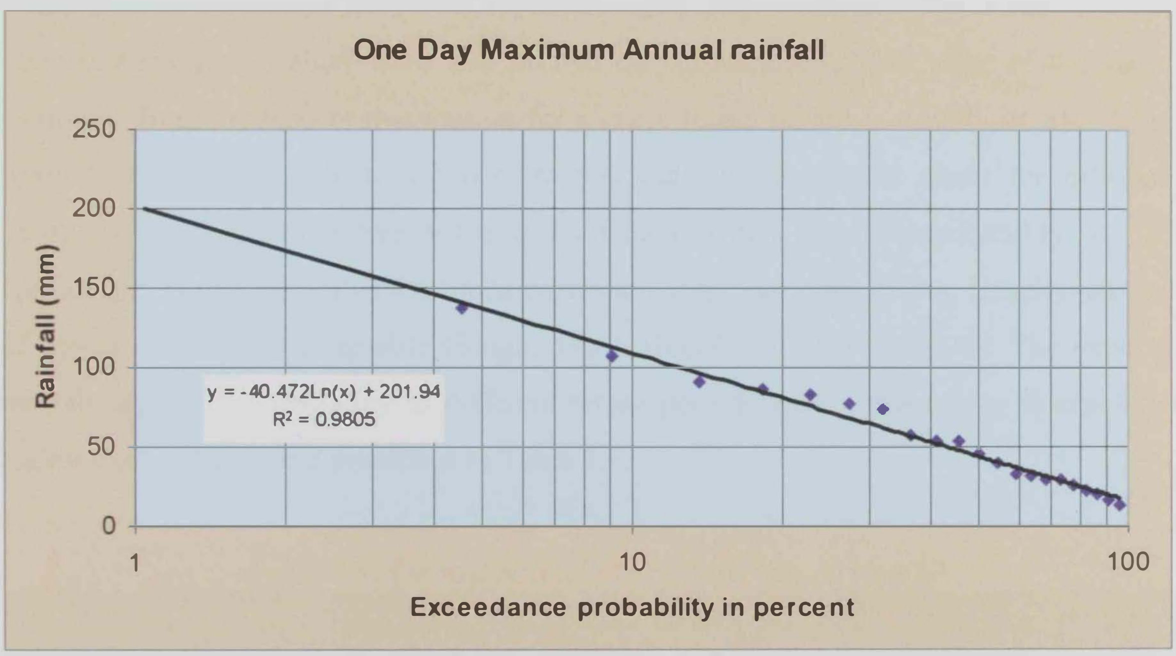

Figure 3.11. Normal distribution fit of one day maximum annual rainfall versus frequency.

The model for estimation of total annual rainfall for the corresponding frequency is:

$$Total\ Annual\ Rainfall = -150.59\ Ln(P) + 696.67 \quad (3.3)$$

The model for estimation of one day annual maximum rainfall is:

$$One\ Day\ Annual\ Maximum\ Rainfall = -40.472\ Ln(P) + 201.94 \quad (3.4)$$

Rainfall for different probabilities for one day annual maximum and total annual rainfall can be extracted from probability graphs by interpolation and/or extrapolation. For total annual rainfall, the probability of occurrence of 75% and 50% of mean rainfall were 47 and 61 percent respectively. For one day annual maximum rainfall, the probability of occurrence of 75% and 50% of mean rainfall were 57 and 79 per cent, respectively. These results along with relatively high standard deviation indicate highly scattered and inconsistent nature of rainfall.

Total annual rainfall and one day annual maximum rainfall data were plotted against return periods and found fit to Normal Distribution (Fig. 3.12) and (Fig. 3.13). The 95% confidence interval values were also plotted on the same graph. A value of the variate estimated from probability distribution for a given return period is usually in error due to limited sample size. The confidence interval indicates the limits about the estimated values within which the true value is contained with a specific probability. A 95% confidence interval indicates the limits corresponding to an error of 5%. Usually, an error of 5% is considered acceptable (Singh, 1992; Helsel and Hirsch, 2000). The expected rainfall depths corresponding to different return periods were extracted by interpolation and/or extrapolation and presented in Table 3.5.

Table 3.5. Estimated return periods of annual rainfall

Return Period (Years)	Estimated Rainfall (mm)	
	Total Annual	One Day Annual maximum
2	118	44
5	253	81
10	355	109
25	490	146
50	592	174

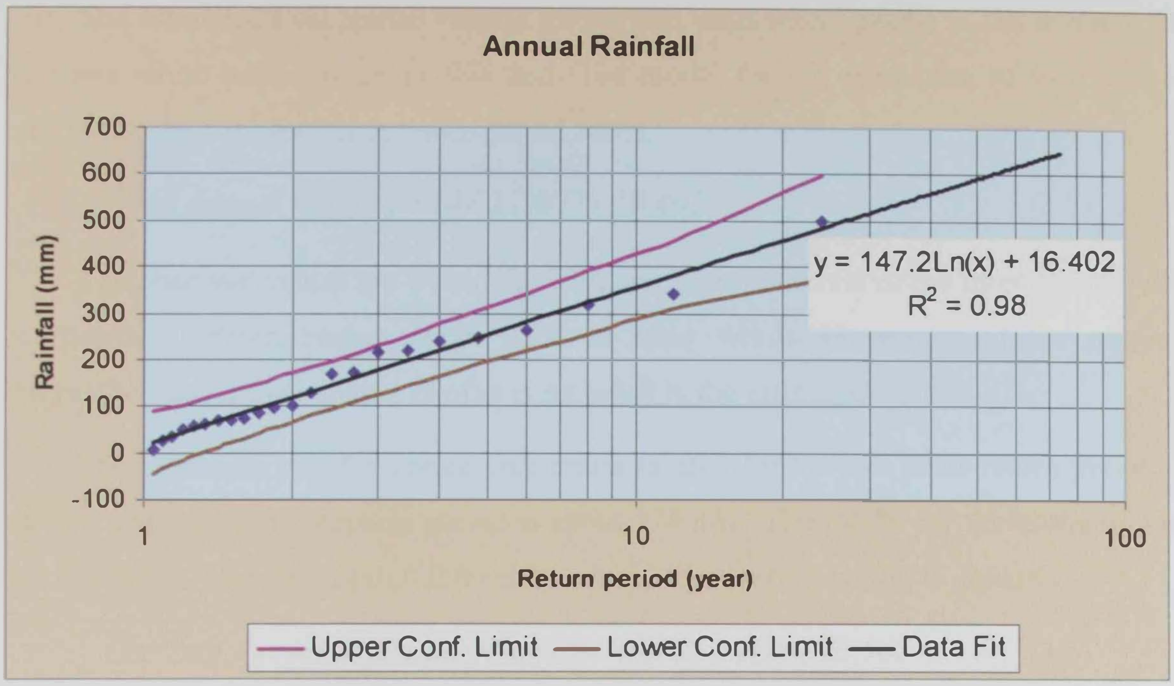

Figure 3.12. Normal distribution fit of total annual rainfall versus return period with confidence limits.

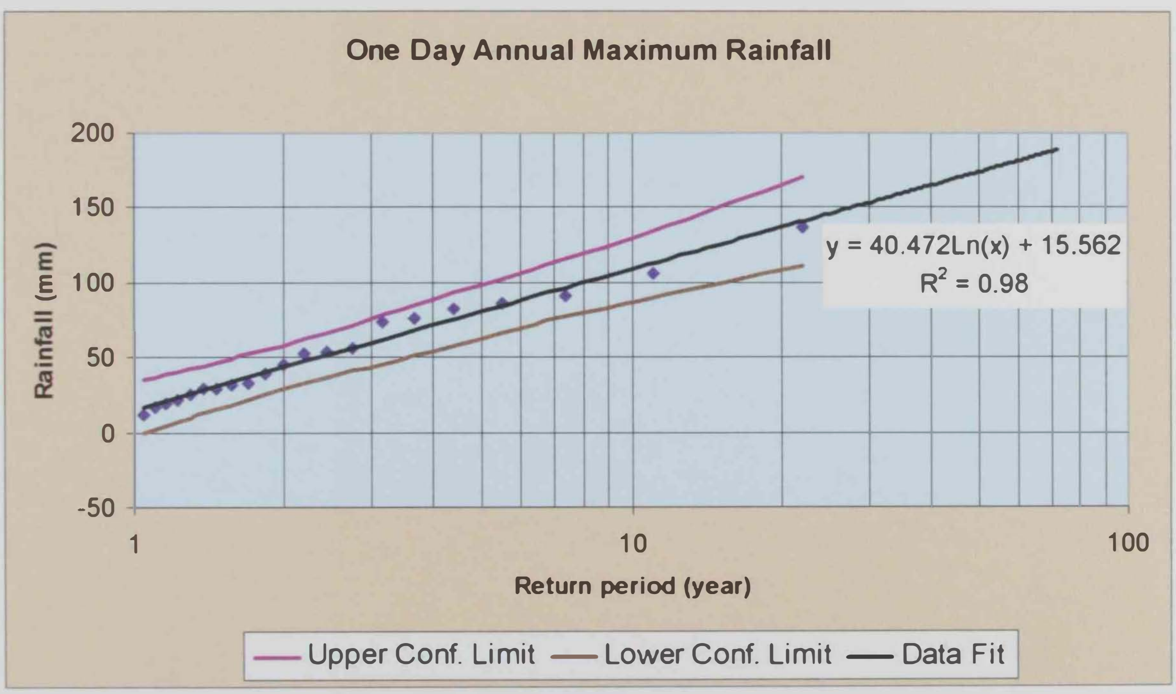

Figure 3.13. Normal distribution fit of one day maximum annual rainfall versus return period with confidence limits.

The estimated total annual rainfall for the two years return period is 118 mm and for 50 years return period is about 592 mm. The model for the estimation of total annual rainfall for the corresponding return period is:

$$Total\ Annual\ Rainfall = 147.2\ Ln(T) + 16.402 \qquad (3.5)$$

The observed values are within the 95% confidence interval of the fitted values with coefficient of determination about 0.98. In other words 98 percent of the original information about total annual rainfall is included in the estimated rainfall.

The estimated one day annual maximum rainfall for the two years return period is 44 mm and for 50 years return period is about 174 mm. The model for the estimation of one day annual maximum rainfall for the corresponding return period is given by:

$$One\ Day\ Annual\ Maximum\ Rainfall = 40.472\ Ln\,(T) + 15.562 \qquad (3.6)$$

The observed values are within the 95% confidence interval of the fitted values with coefficient of determination about 0.98.

Chapter Four

SURFACE WATER ASSESSMENT AND MODELING

CHAPTER FOUR:

SURFACE WATER ASSESSMENT AND MODELING

4.1 Prologue

Wadi Ham lies within the hot arid desert climatic zone. The catchment to Wadi Ham Dam is dominated by the Masafi mountains belonging to the Samail formation (Ophiolite sequence). The drainage stream network of Wadi Ham is a dendritic system (Fig. 4.1). Over the years, surface water flow affected the surface geometry through the process of erosion resulting in numerous channels with steep slopes. The small channels join into higher-order streams; eventually form the main wadi course. The main wadi course starts in the mountains immediately south and south east of Masafi. It is deeply incised in its upper portion due to high flow velocities. It opens out to a broad wadi floor upstream of the dam consisting of alluvial gravels and boulders of various sizes deposited as a result of the reduced flow velocities. The catchment to Wadi Ham Dam includes the catchment of Wadi Al-Farfar tributary which confluences with Wadi Ham about one kilometer upstream of the Ham Dam. The total catchment area formed by Wadi Ham and Wadi Al-Farfar is approximately 195 km^2 (Fig. 4.1).

4.2 Analysis of Surface Water Runoff

The net precipitation input to Wadi Ham is solely via rainfall. Rainfall events often occur as infrequent, short duration and high intensity storms. Due to low porosity and permeability of the prevailing igneous and metamorphic rocks, the surface layer of the mountainous region is unable to infiltrate the incident rainfall, resulting in precipitation excess and surface water runoff that propagates rapidly through the watershed. Therefore, large amount of rainwater occurs as surface runoff or flash flood events. The surface water flow starts usually at the mountainous region near Masafi, moves south easterly through Wadi Ham course, passes by Bithna and eventually ponds behind Wadi Ham Dam. The surface water collected by Wadi Al-Farfar system flows separately through Wadi Al-Farfar course and its sub-tributary, Wadi Mimduk, and joins Wadi Ham upstream of the Ham Dam.

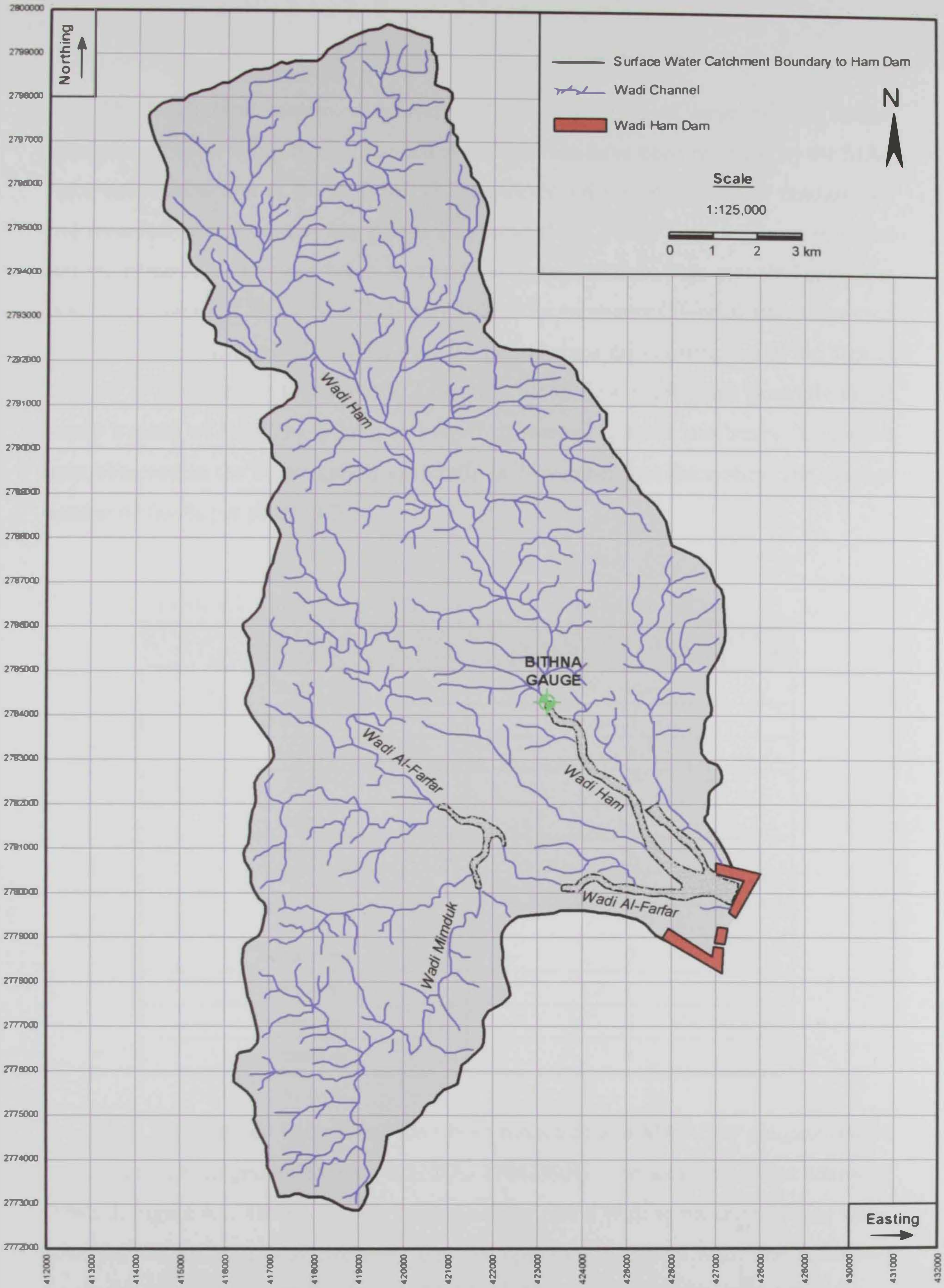

Figure 4.1. Wadi Ham catchment and location of Bithna flow gauge.

The Wadi Ham system is subject to flash flooding from large rainfall events. Estimates of flood volumes accumulated at the dam site have been recorded by the MAF since the construction of the dam in 1982. However, critical evaluation of rainfall data and recorded flood volumes has shown that some flood events have not been recorded and sometimes the estimated volumes are not accurate. The maximum recorded storage at Wadi Ham Dam is 7.50 MCM in February 1988. The numbers of flood storages observed and recorded at the Wadi Ham Dam over the period since the construction of the dam in 1982 up to 2003 are presented in the Table 4.1. Flood flow takes place generally in the winter months of February, March and April. However, isolated and heavy flows have been observed in the other months especially in November and December. The average number of floods per year is 1.3.

Table 4.1. Occurrence of flood events over the years 1982 to 2003

Month	Number of Flood Events
January	0
February	6
March	9
April	5
May	0
June	0
July	0
August	0
September	0
October	1
November	3
December	4

Wadi Ham surface water flows have been measured at a MAF flow gauging station at the approximate grid reference (423208E, 2784299N). The location of this station is shown in Figure 4.1. The gauge was located below Bithna Weir to measure surface water flows for the portion of the catchment that lies upstream of Bithna weir. The catchment area to the flow gauge is approximately 91.5 km^2. The observed surface water flows were measured by using a rating curve procedure (stage-discharge curve).

Surface water flow data for this station exists from 1979 to February 1990 when the gauging station was destroyed in a flood and has not been reconstructed. The complete discharge records on a daily basis as well as the total monthly flow are listed in Appendix B. The available data are analyzed hereinafter in order to detect variations in surface water flow on yearly and monthly basis. The total annual flow volumes are listed in Table 4.2. Annual and monthly flows are also presented graphically in Figures 4.2 and 4.3.

Table 4.2. Total annual flows for Bithna flow gauge (MAF 1980-1989).

Meteorological Year	Flow (MCM/year)
1979 – 1980	0.3234
1980 – 1981	0.05921
1981 – 1982	13.0638
1982 - 1983	3.6428
1983 – 1984	0.080
1984 – 1985	0.0
1985 – 1986	0.0
1986 – 1987	0.63417
1987 – 1988	7.1280
1988 – 1989	0.530
Average	**2.54613**

Critical evaluation of surface water flow data and rainfall data has shown that flash flooding normally results from large rainfall events with a very large proportion of runoff passing through the system very rapidly. Most flood events are over within a relatively short period of time, with many being over in less than a day. The flow data shows the volume of gauged flow to be extremely variable reflecting the intermittent nature of the rainfall events discussed in Chapter 3. Gauged flows vary considerably from year to year and range from a maximum recorded daily flow of 5.3557 MCM/day on 17/02/1988 to complete calendar years with no flow at all such as 1985 and 1986. Average yearly total discharge from the available data for Bithna flow gauge is 2.54613 MCM/yr.

It should be emphasized, however, that Wadi Ham Bithna Flow Gauge measures flow from only part of the surface water catchment to the Ham Dam. The catchment to the flow gauge is approximately 91.5 km^2 compared to 195 km^2 to the Ham Dam. This does not include neither any of the flow being fed down by Wadi Ham itself nor the whole catchment of Wadi Al-Farfar system (Fig. 4.1).

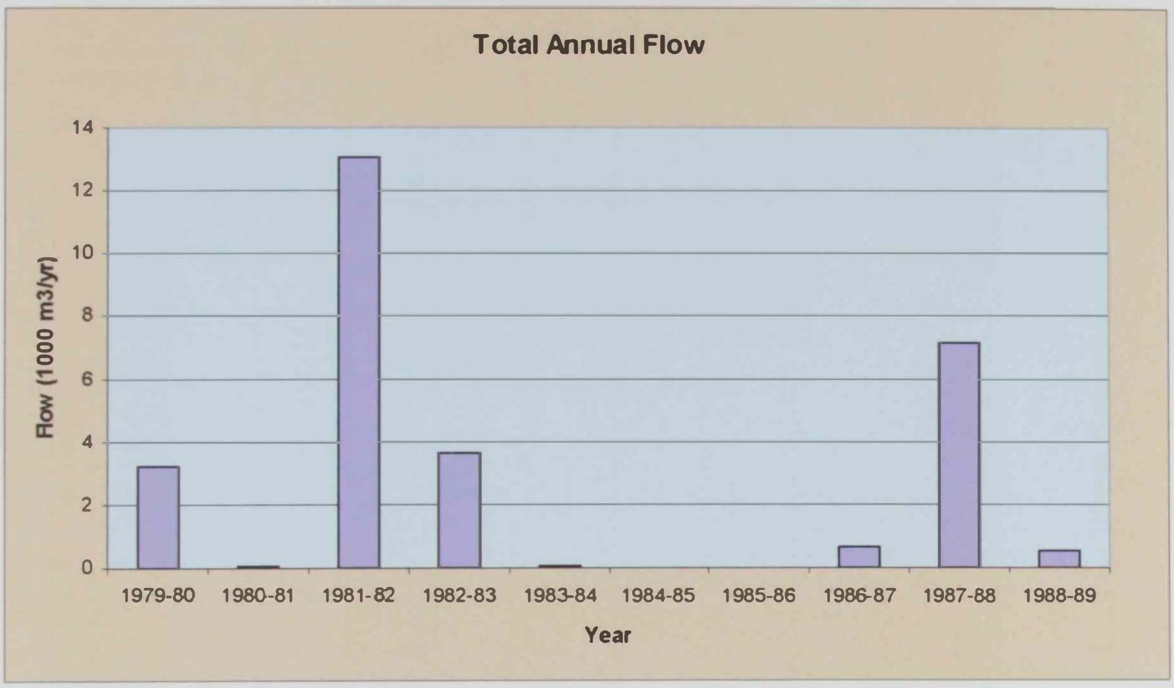

Figure 4.2. Total annual flows for Bithna flow gauge.

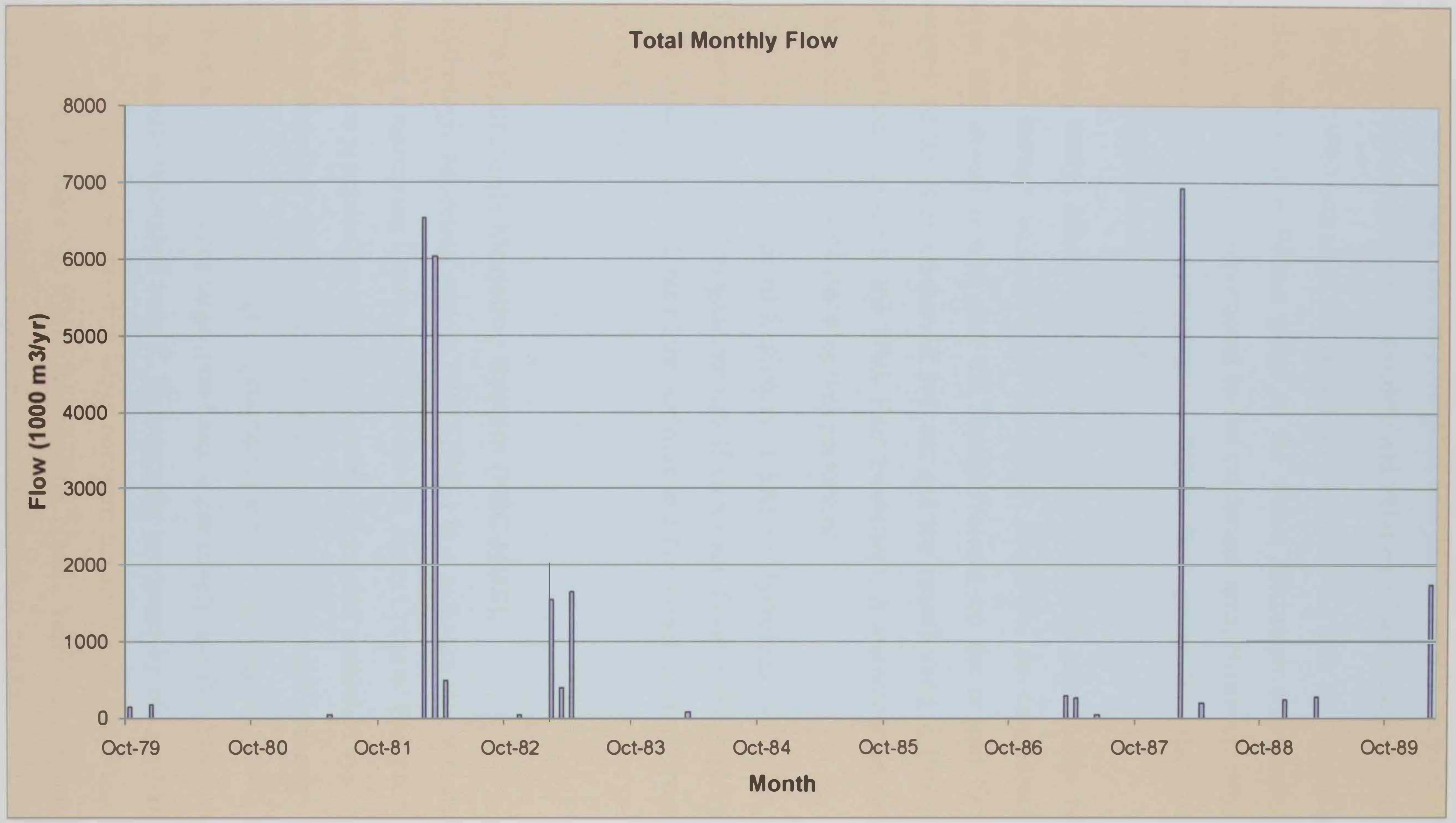

Figure 4.3. Total monthly flows for Bithna flow gauge

No measurements are available for the surface water flow drained by the whole catchment of Wadi Ham. The only recorded data are the total accumulated volume at the dam site and even these are not complete and volumes of some flood events are missed.

Entec (1996) estimated the volume of storage at the dam site by scaling up the flow volumes measured at Bithna gauge to the whole catchment area assuming the runoff volume to be directly proportional to the catchment area. However, major flood events were encountered after destruction of Bithna flow gauge like the ones occurred in February 1990 and December 1995.

In this study, efforts have been made to establish relationship between annual rainfall and storage volume at the dam site. However, no consistency was noticed between the annual rainfall and the storage volume on the annual basis. For proper assessment of the Rainfall-Runoff process and the runoff yield, a Precipitation-Runoff model has been applied to the Wadi Ham catchment. A systematic analysis of intensity and duration of storm with flooding was performed.

The US Army Corps of Engineers' (USACE) Hydrologic Modeling system (HEC-HMS) package was used to generate runoff yields and flood hydrographs at dam site. An overview of the main features of the software and the modeling work are presented in the following sections.

4.3 The Hydrologic Modeling System (HEC-HMS).

The Hydrologic Modeling system (HEC-HMS) is a software that was developed at the Hydrologic Engineering Center (HEC) of the US Army Corps of Engineers. It is designed to simulate the precipitation-runoff processes of dendritic watershed systems. HEC-HMS provides a variety of options for simulating precipitation-runoff processes. It is designed to be applicable in a wide range of geographic areas for solving the widest possible range of problems. This includes large river basin water supply and flood hydrology, and small urban or natural watershed runoff. Hydrographs produced by the program can be used directly or with conjunction with other software for studies of water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and etc (USACE, 2001).

4.3.1 HEC-HMS components

HEC-HMS employs an easy-to-use Graphical User Interface (GUI) that allows manipulation of hydrologic elements, hydrologic data entry and organization of the components, which make up each hydrologic modeling task (project). In HEC-HMS, a project consists of three separate components: the Basin Model, the Meteorologic Model, and the Control Specifications.

The Basin Model

The physical representation (conceptual model) of watersheds or basins and rivers is configured in the basin model. It contains the basin and routing parameters of the model as well as connectivity data for the basin. Hydrologic elements are connected in a dendritic network to simulate runoff processes. HEC-HMS provides a variety of models to represent each component of the runoff process and these are listed in Table 4.3. Available hydrologic elements are: ***subbasin***, ***reach***, ***junction***, ***reservoir***, ***diversion***, ***source***, and ***sink***. Computation proceeds from upstream elements in a downstream direction. ***Subbasins*** represent the physical areas within the basin and produce a discharge hydrograph at the outlet of their respective areas. The hydrograph produced is calculated from precipitation data minus the losses.

Loss rates are simulated by one of loss models (Table 4.3). The resulting precipitation excess is transformed using a direct-runoff model (Unit Hydrograph methodology) to compute runoff at the outlet (Table 4.3). The computed runoff at the outlet is then added to **baseflow** which is simulated using one of the available baseflow models. No baseflow is also an option, as in simple hydrologic models over short time periods or highly urbanized basins with channels, baseflow can usually be neglected. The resulting runoff is routed through ***reaches*** using one of the routing models available. A ***junction*** represents a point where two reaches are joined.

The effect of adding a detention pond to a basin can be modeled by using a ***reservoir***. A reservoir stores the inflow from upstream elements and produces an outflow hydrograph based on a monotonically increasing storage-outflow relationship. A reservoir can be entered with one of three possible types of relationships: storage vs. outflow; elevation vs. storage vs. outflow; or elevation vs. area vs. outflow. The inflow entering the reservoir must be contained with the minimum and maximum value of the data entered. The user must also select an initial condition of storage, elevation, outflow, or select inflow equal to outflow.

Table 4.3. Models included in HEC-HMS (USACE, 2000)

Model	Categorization
Runoff-Volume Models (Losses Models)	
Initial and constant-rate	event, lumped, empirical, fitted parameter
SCS curve number (CN)	event, lumped, empirical, fitted parameter
Gridded SCS CN	event, distributed, empirical, fitted parameter
Green and Ampt	event, distributed, empirical, fitted parameter
Deficit and constant rate	continuous, lumped, empirical, fitted parameter
Soil moisture accounting (SMA)	continuous, lumped, empirical, fitted parameter
Gridded SMA	continuous, distributed, empirical, fitted parameter
Direct-runoff models(Runoff Transform Models)	
User-specified unit hydrograph (UH)	event, lumped, empirical, fitted parameter
Clark's UH	event, lumped, empirical, fitted parameter
Snyder's UH	event, lumped, empirical, fitted parameter
SCS UH	event, lumped, empirical, fitted parameter
ModClark	event, distributed, empirical, fitted parameter
Kinematic wave	event, lumped, conceptual, measured parameter
Baseflow models	
Constant monthly	event, lumped, empirical, fitted parameter
Exponential recession	event, lumped, empirical, fitted parameter
Linear reservoir	event, lumped, empirical, fitted parameter
Routing models	
Kinematic wave	event, lumped, conceptual, measured parameter
Lag	event, lumped, empirical, fitted parameter
Modified Puls	event, lumped, empirical, fitted parameter
Muskingum	event, lumped, empirical, fitted parameter
Muskingum-Cunge Standard Section	event, lumped, quasi-conceptual, measured parameter
Muskingum-Cunge 8-point Section	event, lumped, quasi-conceptual, measured parameter
Confluence	continuous, conceptual, measured parameter
Bifurcation	continuous, conceptual, measured parameter

Sources are elements that represent a discharge into the basin as an observed hydrograph or a hydrograph generated by a previous simulation. ***Sinks*** are elements that have an inflow and no outflow. The only inputs are the name and description of the sink. It may represent the lowest point of the drainage area or the outlet. ***Diversions*** represent abstraction of flow from the stream. Diversions have two "downstream" connections, one being the routed path and the other the diverted path. The user specifies the diverted flow only, and whatever flow is not diverted will travel the main path.

In HEC-HMS, the basin model is merely a representation of the actual watershed (conceptual model), and the visual location and sizes of each element do not matter as long as the numerical data and connectivity are correct. A background map containing subbasin boundaries and streams can be entered from a GIS map file as a visual reference, but it is not used for any calculations. Figure 4.4 shows standard HEC-HMS Basin Model window showing a basin with map file background.

The Meteorologic Model

The Meteorologic Model contains the precipitation data, either historical or hypothetical, for the HEC-HMS model and can even account for evapotranspiration. The options in historical precipitation inputs include hyetographs, gauge weighting, and inverse-gauge weighting and capable of handling unlimited number of recording and non-recording gauges. Hypothetical precipitation data can be derived from frequency storm and standard project storm (SPS) models. In addition HEC-HMS has the capability to model gridded rainfall, such as NEXRAD-estimated radar rainfall.

The Control Specifications

The Control Specifications contains all the timing information for the model, including the start time and date, stop time and date, and computational time step of the simulation. A series of runs can be easily organized using this option with several different scenarios.

4.3.2 HEC-HMS running and calibration

The user may specify different data sets for each component within a project and then run the hydrologic simulation using different scenarios. For example, several Meteorologic Models representing different storms can be modeled using the same basin model and control specifications to compare the resulting flows. With several basin models saved in the same project, the effects of adding diversions and reservoirs to a basin can also be modeled.

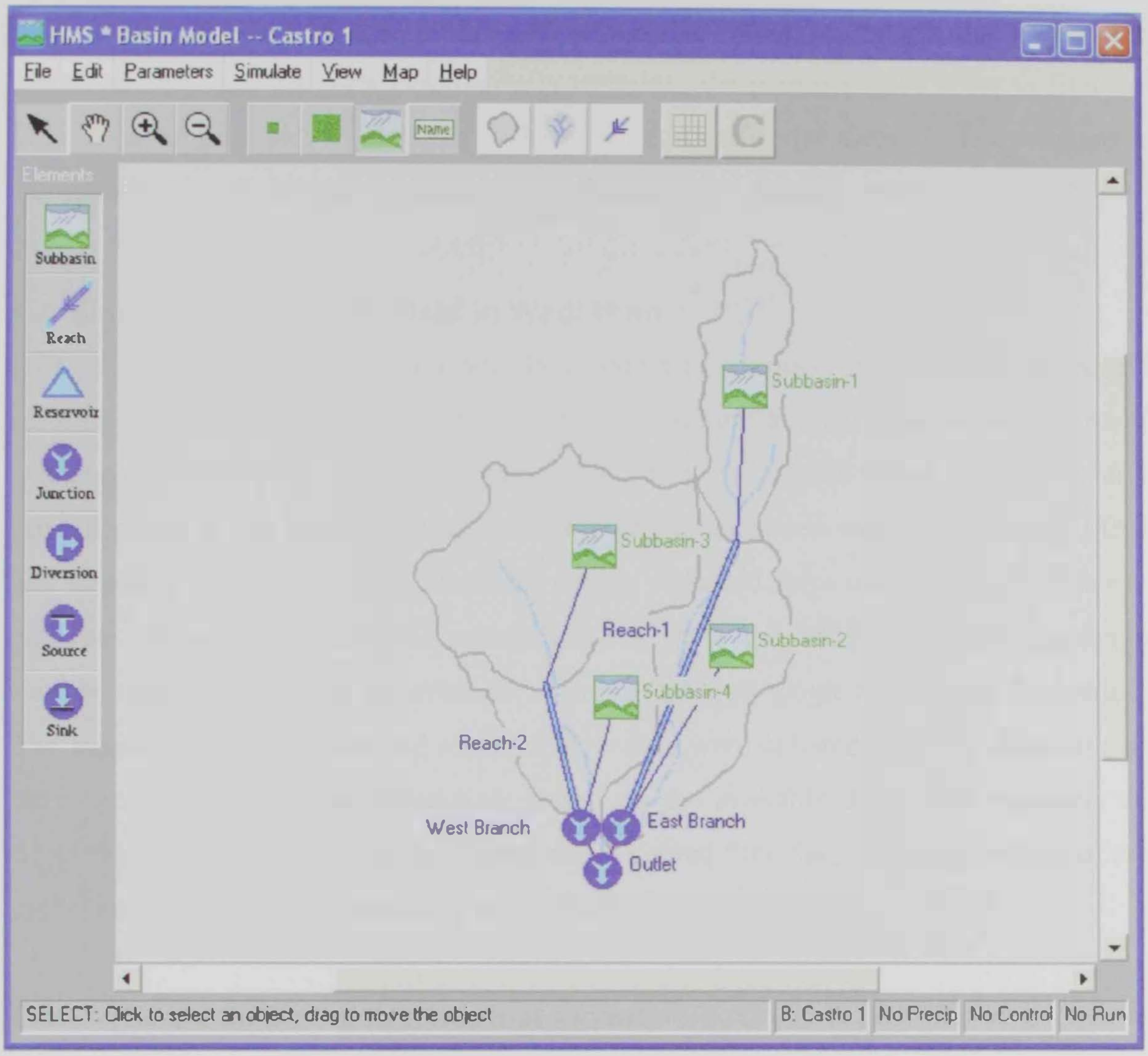

Figure 4.4. Standard HEC-HMS Basin Model window with map file background.

The parameter estimation and optimization function is used to compare resulting hydrographs to observed hydrographs, so at least one element in the basin must have observed data. The program automatically estimates the parameters in order to find the best fit of the generated hydrograph to the observed one for one element. The program is run iteratively until the user is satisfied with the objective function value.

4.4 Simulation of Runoff Yield in Wadi Ham

HEC-HMS has been applied to Wadi Ham system to simulate the surface water runoff resulted from various historical rainfall events. To perform a rather accurate simulation of a historical flood event, it is a prerequisite to know the rainfall event pattern. Several rainfall charts of the Bithna rainfall recording gauge have been collected from the MAF and rainfall patterns of several events have been extracted form these charts. In order to calibrate the model, some flood events between 1979 and 1989 have been simulated. Surface water flow records are available at the Bithna flow gauge only during this period. The available rainfall charts and recorded flow data were collated and **four** flood events have been selected for the simulation. Based on the available data, these represent all flood events that have both rainfall charts and observed flow data. The construction of the model and the simulation procedure are explained in what follows.

4.4.1 The Basin Model (conceptual model)

The catchment of Wadi Ham was divided into five subbasins which are formed naturally within the catchment (Fig. 4.5). The model of Wadi Ham was conceptualized using only 4 of 7 hydrologic elements available in HEC-HMS basin model. Basin elements like reservoir, sources and diversions are omitted from the basin model as there are no such elements present in Wadi Ham. The model of Wadi Ham contained 11 hydrologic elements, made up of 5 subbasins, 3 reaches, 2 junctions and 1 sink. Junction-2 was set at the Bithna flow gauge location and the sink was set at Wadi Ham Dam (Fig. 4.5).The catchment boundary, subbasin boundaries and stream network were delineated and digitized based on available toposheets and remote sensing images using ArcView-GIS and AutoCAD softwares. Geometric data such as stream lengths, areas and centroid location were then measured and introduced to the model.

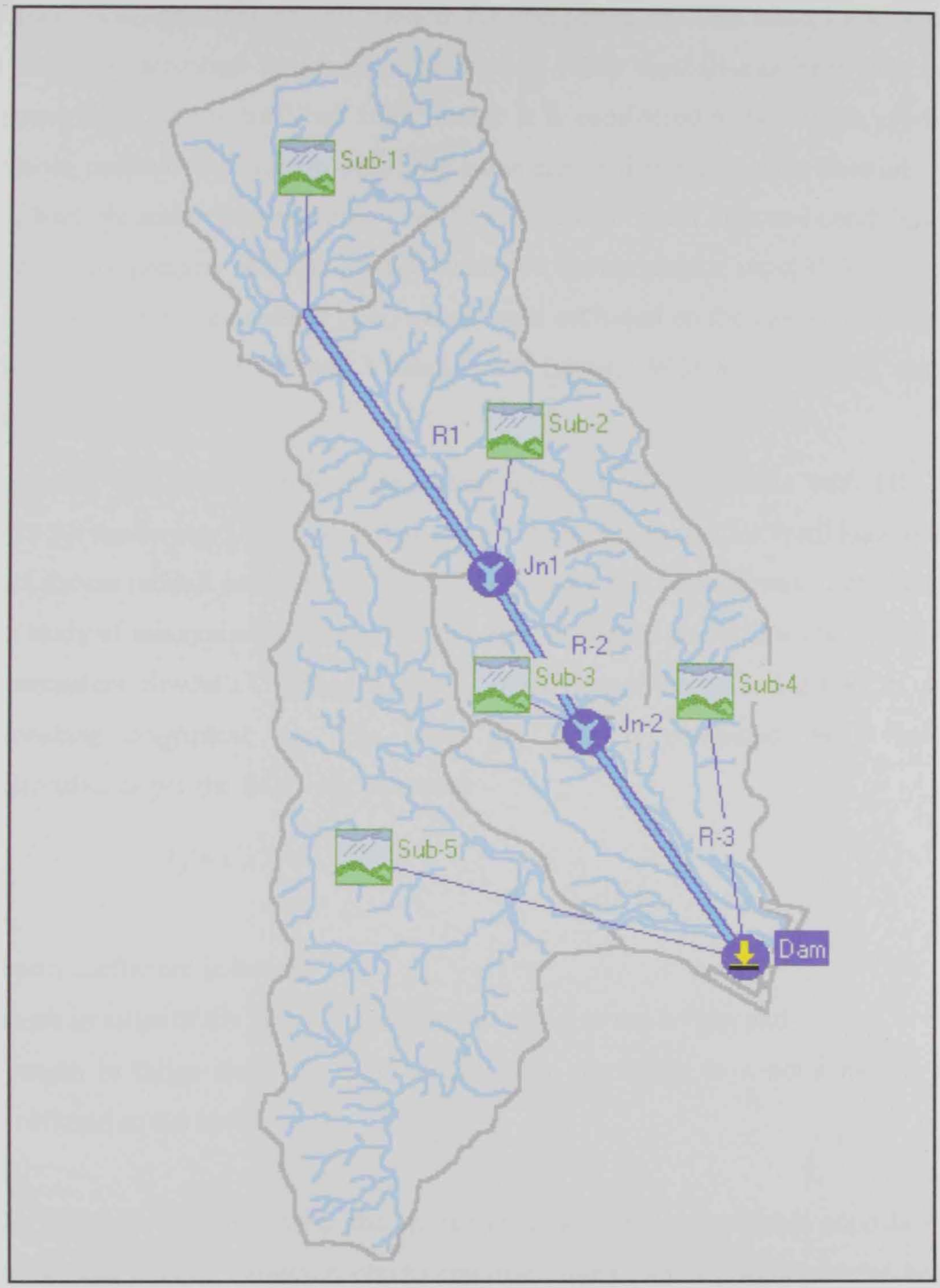

Figure 4.5. Wadi Ham Basin Model.

HEC-HMS provides several models for computing the loss rate (Table 4.3). The SCS (Soil Conservation Service) curve number (CN) method has been selected for computing the loss rate for Wadi Ham model. It is considered to be simple, predictable and stable method. It relies on only one parameter which varies as a function of soil group, land use and treatment, surface condition and antecedent moisture condition which can be readily grasped and also well-documented environmental input (USACE, 2000). Initial guesses of curve numbers (CN) values were set based on the values documented in the literature (USACE, 2001 and Viessman and Lewis, 1995) for conditions similar to that of Wadi Ham.

Among the direct runoff models (transform models) available with HEC-HMS (Table 4.3), the Snyder Unit Hydrograph model has been selected for Wadi Ham model to convert excess rainfall into surface runoff. The Snyder UH method was developed based on the study of mountainous watersheds and uses watershed characteristics for estimating UH parameters. Snyder's UH model requires specifying of the *basin lag time*, t_p, and the *UH peaking coefficient*, C_p. The basin lag time is computed using watershed characteristics as per the following equation:

$$t_p = C_t(L \times L_c)^{0.3} \tag{4.1}$$

where,

C_t = basin coefficient in hours,

L = length in miles of the main stream from the outlet to the divide, and

L_c = length in miles along the main stream from the outlet to a point nearest to the centroid of the basin.

L and L_c were delineated and measured based on the available toposheets and remote sensing images. However, other parameters like C_t and C_p were selected based on the values reported in the literature that C_t typically ranges from 1.8 to 2.2 and it has been found to vary in mountainous areas from 0.4 to 8.0 where steeper slopes tend to generate lower values. It is also reported that C_p ranges from 0.4 to 0.8, where larger values of C_p are associated with smaller values of C_t (Viessman and Lewis, 1995).

HEC-HMS offers various routing models (Table 4.3) to route through reaches the runoff resulted from different subbasins. Each of these models computes the downstream hydrograph given the upstream hydrograph by solving the continuity and momentum equations. The lag model is the simplest of these models and requires only the lag time as the input. With the lag model, the outflow hydrograph is simply the inflow hydrograph, but with all ordinates translated (lagged in time) by specified duration. The flows are not attenuated, so the shape is not changed. Other models use numerical (finite difference) technique to approximate the continuity and momentum equations and solve for flows (USACE, 2000). These models simulate flows attenuation but require many input information including, among others, description of the channel (width, bed slope and cross section shape), energy loss parameters and boundary conditions (upstream, lateral and tributary inflow hydrographs).

The selection of the routing model for Wadi Ham was made in light of the objective of the simulation. Because the simulation aims at computing volumes of surface water flow rather than the flow pattern, the lag model was selected.

4.4.2 The Meteorologic Model

The meteorologic model of Wadi Ham contained historical rainfall data from three rainfall gauges, namely, Masafi, Bithna and Farah rain gauges. The total storm rainfall depths for the four rainfall events selected for the simulation are listed in Table 4.4.

Table 4.4. Total storm rainfall depths, input to Wadi Ham model.

SN	Storm date	Depth (mm)		
		Masafi	**Bithna**	**Farah**
1	13.02.1982	120	148	158
2	11.02.1983	45	105.3	130
3	30.03.1983	62.1	98.8	107
4	16.02.1988	148.1	161.4	211

The pattern (temporal distribution) of rainfall for each rainfall event was assumed to follow the hyetograph pattern of Bithna recording rain gauge for that particular event. The hyetograph ordinates were extracted from rainfall charts recorded at Bithna gauge. The hyetograph ordinates for the simulated events are listed in Appendix C.

The user gauge weighting method was selected to input total storm depths with recording (Bithna) and non-recording (Masafi and Farah) rain gauges. The Thiessen Polygon method was applied to identify representative gauges for each subbasin and gauge weights were computed. The number of rain gauges and their Thiessen weights considered for the subbasins are presented in Table 4.5.

Table 4.5. User gauge weighting, input to Wadi Ham model.

Sub-basin	Area (Km^2)	User Gauge Weighting	
		Rain gauge	**Thiessen Weight**
1	31.48	Masafi	1.00
		Bithna	-
		Farah	-
2	47.67	Masafi	0.40
		Bithna	0.60
		Farah	-
3	18.75	Masafi	1.00
		Bithna	-
		Farah	-
4	30.08	Masafi	0.70
		Bithna	-
		Farah	0.30
5	66.52	Masafi	-
		Bithna	0.35
		Farah	0.65

4.4.3 Control specifications

Time information for the simulated flood events were specified based on the historical flood events data. The control specifications for the model are listed in Table 4.6.

Table 4.6. Control specifications, input to Wadi Ham model.

SN	Starting date	Starting Time	Ending Date	Ending Time	Time Interval
1	13.02.1982	06:00	16.02.1982	20:00	1 Hour
2	11.02.1983	04:00	12.02.1983	24:00	1 Hour
3	30.03.1983	14:00	06.04.1983	08:00	1 Hour
4	16.02.1988	16:00	21.02.1988	20:00	1 Hour

4.4.4 Model calibration and results

The model for Wadi Ham was calibrated against the historical surface water flow data recorded at the Bithna flow gauge. The model was initially run using initial guesses of basin parameters quoted from literature (USACE, 2001 and Viessman and Lewis, 1995) as discussed previously. Several runs then have been performed and basin parameters were adjusted by trial and error method to achieve best possible results comparable with the observed flow values. The final calibrated values used to produce the model results are listed in Table 4.7.

Table 4.7. Basin parameters, input to Wadi Ham model.

Subbasin	Area (Km^2)*	CN^+	C_t^+	C_p^+	L (km)*	L_c (km)*	t_p (hr)$^{++}$
1	31.48	68	0.5	0.7	13.9	4.5	1.21
2	47.67	68	0.5	0.7	12.7	6	1.35
3	18.75	65	0.5	0.7	8.2	4.5	1.19
4	30.08	65	0.5	0.7	10.2	5	1.21
5	66.52	65	0.6	0.7	19.7	8	2.00
Reach (R1) = 7.8 km, Lag = 100 min.		**Reach (R2) = 6.0 km, Lag = 60 min.**			**Reach (R3) = 4.4 km, Lag = 60 min.**		

* Measured on Map

\+ Calibrated Values

++ Calculated Using Equ. 4.1.

It should be noted, however, that the calibration of the model has been performed to match the flow values observed at Bithna flow gauge which measures flow from only part of the surface water catchment to the Ham Dam. Junction-2 in the model was set at the location of Bithna flow gauge for this purpose. The flow values produced by the model at Junction-2 were compared with observed values and the model parameters were adjusted until best possible agreement was obtained. Given the calibrated parameters, the model was then assumed to accurately produce total runoff yields at the dam site. The simulated versus observed flow volumes at Bithna and the simulated flow volumes at dam site and from Wadi Al-Farfar system are listed in Table 4.8.

Table 4.8. Simulated flow volumes, output of Wadi Ham model.

Storm date	Bithna Gauge			Dam Site	Wadi Al-Farfar	
	Simulated flow (MCM)	Observed flow (MCM)	% Error	Simulated flow (MCM)	Simulated flow (MCM)	% of Total Yield
13.02.1982	5.0224	5.388123	6.79	10.863	4.0747	37.5
11.02.1983	1.5127	1.591329	4.94	5.0368	2.5432	50.5
30.03.1983	1.7124	1.747948	2.03	4.3408	1.8464	42.5
16.02.1988	6.5449	6.935626	5.63	14.949	6.0694	40.6

The model has shown good agreement with observed flow values. The simulated flow volumes were below the actual measured flow volumes for all simulated events. The percentage of error between simulated results and the actual measurements is ranging from 2.03 to 6.79 percent. The model has shown that Wadi Al-Farfar system has great contribution to the total runoff yield accumulated at the dam site. The percentage of the runoff yielded by Wadi Al-Farfar to the total runoff yield ranged from 37.5 to 50.5 percent. Runoff hydrographs produced by the model are shown in Figures 4.6 through 4.9. The hydrographs shows several peaks corresponding to the successive rainfall storms provided to the model through rainfall patterns. The total flow times as simulated by the model agree with the recorded data with only one to two hours difference.

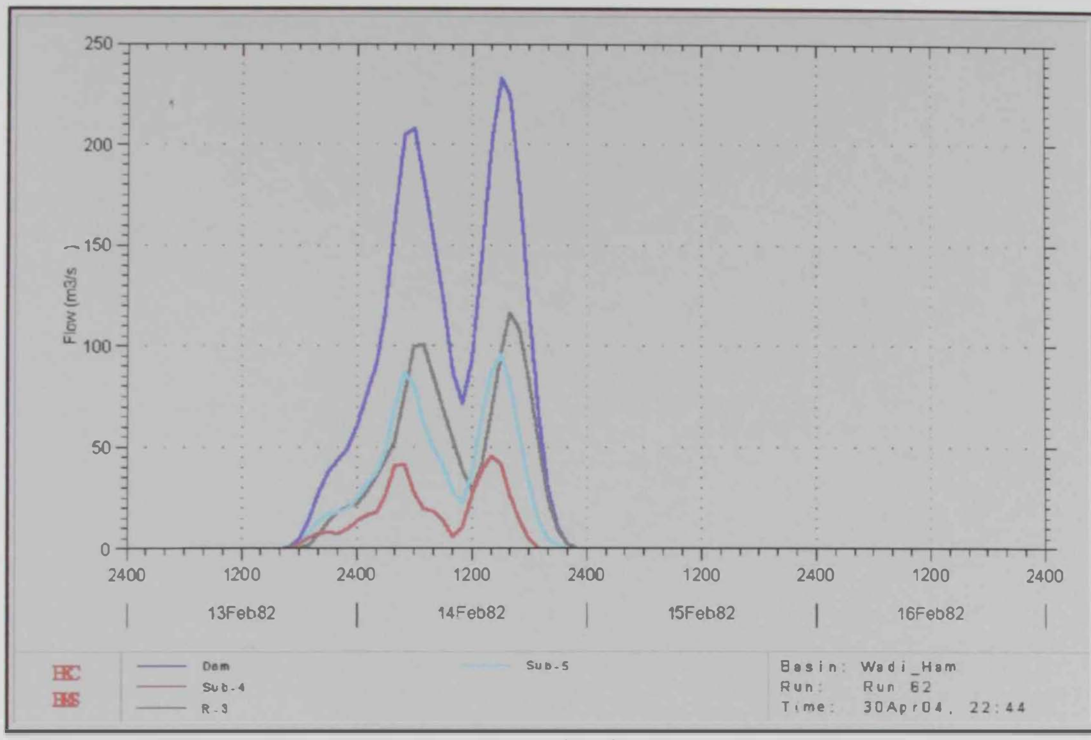

Figure 4.6. Runoff hydrograph at dam site for the storm on February 13, 1982.

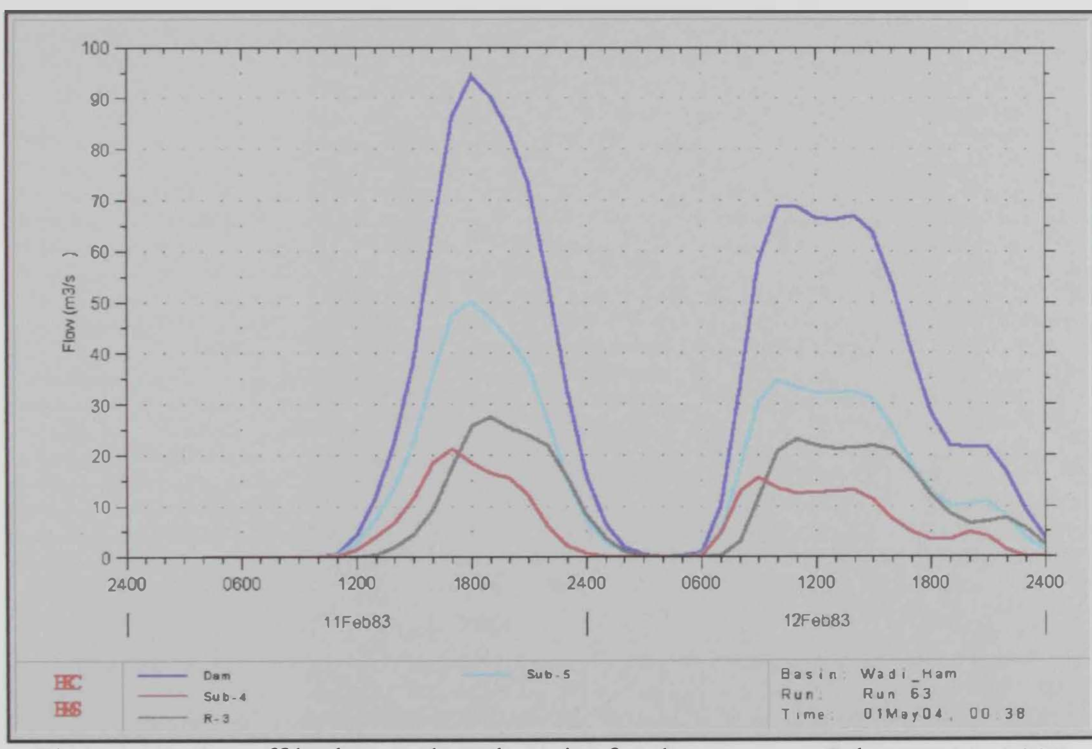

Figure 4.7. Runoff hydrograph at dam site for the storm on February 11, 1983.

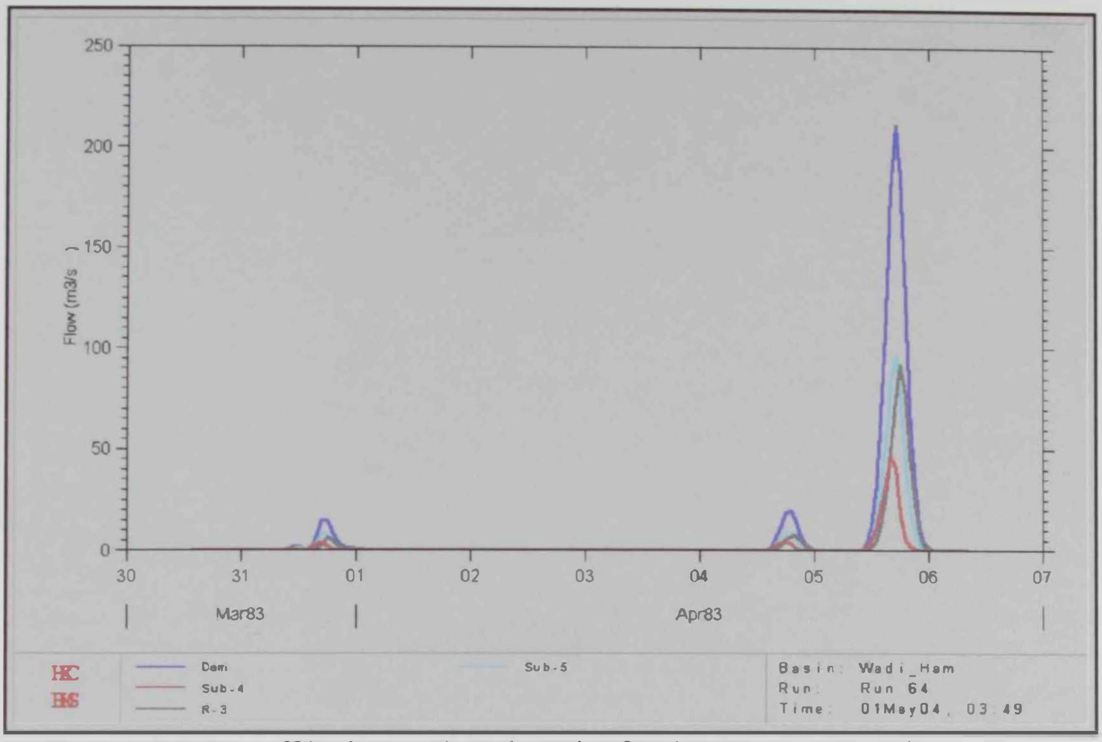

Figure 4.8. Runoff hydrograph at dam site for the storm on March 30, 1983.

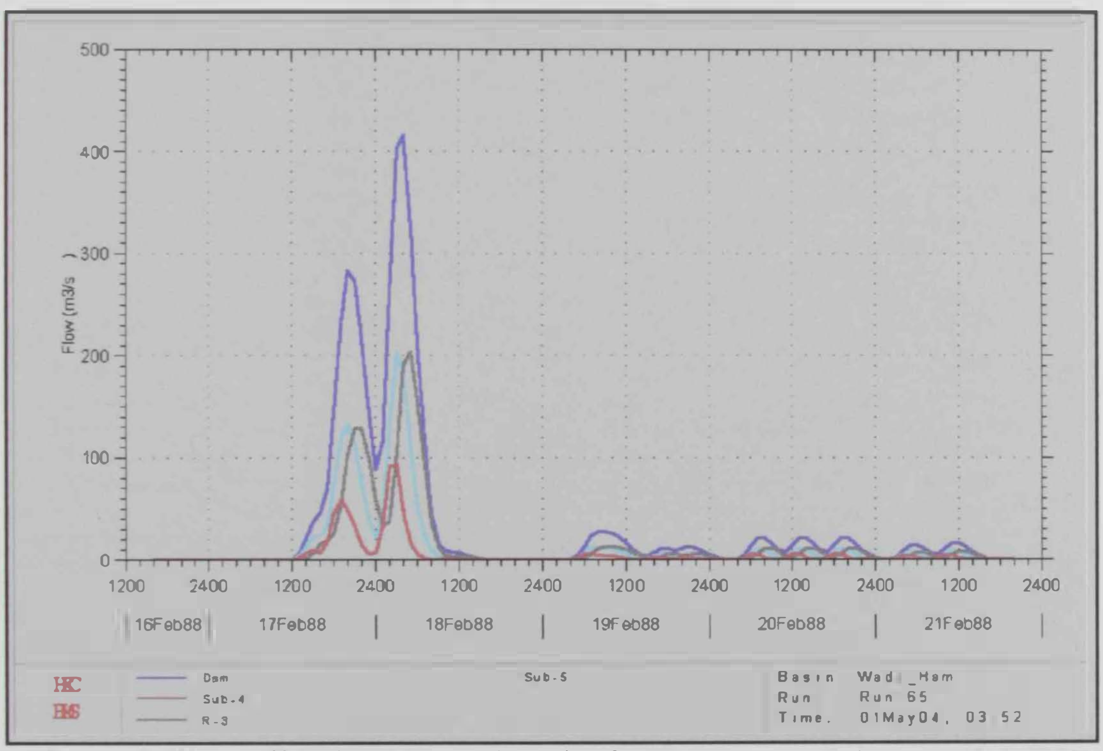

Figure 4.9. Runoff hydrograph at dam site for the storm on February 16, 1988.

Chapter Five

GROUNDWATER ASSESSMENT AND MODELING

CHAPTER FIVE:

GROUNDWATER ASSESSMENT AND MODELING

5.1 Prologue

Wadi Ham opens out to a broad wadi floor upstream of the dam consisting of alluvial gravels and boulders of various sizes deposited as a result of the reduced flow velocities. This alluvial deposits form a broad flat plain which covers the area between the Ham Dam and the Gulf of Oman. The plain starts about two kilometers upstream of the Ham Dam where it covers the retention area behind the dam and extends downstream where it becomes more broader comprising the coastal plain area from north of Fujairah to south of Kalba (Fig. 5.1).

The alluvial deposits in this plain provide an important exploitable aquifer with moderate well yields. Natural groundwater recharge is, however, very limited in the arid climatic conditions that dominate Wadi Ham area and the UAE in general. On the other hand, intensive groundwater extraction from the aquifer, mainly for irrigation, has resulted in remarkable depletion of groundwater storage and caused problems of seawater intrusion and thus deterioration of groundwater quality.

One of the main technical options for conservation of groundwater resources is to enhance aquifer recharge by artificial means. To that end, Wadi Ham Dam was constructed by the Ministry of Agriculture and Fisheries (MAF) in 1982 to act as a retention dam intercepting flash floods caused by rapid runoff from rainfall events within the Wadi Ham surface water catchment.

The dam helps to augment groundwater storage by artificial recharge caused by surface infiltration through the retention area behind the dam. The increase in groundwater storage, in turn, enhances groundwater quality and the rise of groundwater table levels fights back the seawater intrusion. This chapter is devoted to assess the hydrogeology of Wadi Ham and groundwater conditions and provides quantitative assessment of the recharge from the dam using numerical modeling techniques.

Figure 5.1. Alluvial plain of Wadi Ham.

5.2 Hydrogeology of Wadi Ham

Wadi Ham area is dominated by alluvial plains in its lower part. The lower plain of wadi Ham is composed of recent Pleistocene wadi gravels which comprise the major aquifer system. This gravels plain is underlain by the rocks of the Samail formation (Ophiolite sequence). The degree of consolidation varies from recent uncemented sandy gravel with older better consolidated gravels to oldest well cemented gravel horizons.

The hydrogeology has been investigated by a number of exploration boreholes by the Ministry of Agriculture and Fisheries (MAF). The locations of these observation boreholes are shown in Figure 5.2. A number of very short pumping tests were carried out on borehole numbers BHF-1B to BHF5 and BHF10 to BHF14.

Details of the individual boreholes and transmissivity values derived from these by Geoconsult and Bin Ham Ltd (1985) and IWACO (1986) in a series of well and summary reports are listed in Table 5.1 (Entec, 1996). The lithology information of other wells which are not listed in Table 5.1 was taken from the study by ESCWA (1997).

The hydrogeology of the Wadi Ham plain area from dam site and Wadi Ham downstream of the dam to the Gulf of Oman can be classified into two units, namely, wadi gravels; and gabbro and diorite of the Samail Ophiolite.

5.2.1 Wadi gravels

The wadi gravels form the major aquifer system in Wadi Ham plain. The gravels maintain a thick layer beneath the plain extending to Fujairah with its thickness varies considerably. Beneath the main dam it averages from 15 m to 25 m in depth, while further down the Wadi it reaches a maximum of 99 m in depth. The minimum thickness of wadi gravels (15 m) is found at well number BHF-19 which is in the area upstream of Wadi Ham Dam close to mountain series. The maximum thickness of wadi gravels (99 m) is observed at well number BHF-14 which lies downstream of Wadi Ham Dam in the area near the coast of the Gulf of Oman.

Figure 5.2. Locations of boreholes and observation wells (after Sherif *et al.*, 2004)

Table 5.1. Exploration and groundwater observation boreholes construction and testing details (after Entec, 1996).

Borehole Number	Grid Reference		Approx. Elevation of Casing Top (m, AMSL)#	Depth (m, BGL)	Geology (m, BGL)		Pumping Test Duration	Pumping Test Zone (m, BGL)	Transmissivity (m^2/d)
	Easting	Northing			Wadi Gravels	Ophiolite			
BHF 1	429000	2778500	56.5	77.0	0 - 66.0	66.0 - 77.0	-----	-----	-----
BHF 1B	429600	2778675	58.5	80.0	0 - 57.0	57.0 - 80.0	40 mins.	52.0 - 76.0	29.5
BHF 2	432250	2777500	28.5	70.0	0 - 54.0	54.0 - 70.0	4 hours	34.0 - 68.0	1200
BHF 3	432900	2779800	13.5	102.0	0 - 63.0	63.0 - 102.0	50 mins.	29.72 - 82.82	-----
BHF 3A	432900	2779800	13.5	54.0	0 - 54.0	-----	50 mins.	34.0 - 39.0	455
BHF 4	433800	2776550	12.5	70.0	0 - 57.0	57.0 - 70.0	2 hours	24.0 - 53.0	-----
BHF 4A	433800	2776550	12.5	30.0	0 - 30.0	-----	2 hours	? - 30.0	1650
BHF 5	432950	2773400	16.5	30.0	0 - 24.0	24.0 - 30.0	40 mins.	12.0 - 29.0	6959
BHF 7	431000	2773550	27.5	40.0	0 - 24.0	24.0 - 40.0	-----	-----	-----
BHF 9A	430300	2777850	46.5	56.0	0 - 49.0	49.0 - 56.0	-----	-----	-----
BHF 9B	430300	2777850	46.5	84.0	0 - 65.0	65.0 - 84.0	-----	-----	-----
BHF 10	431800.	2780250	21.5	57.0	0 - 48.0	48.0 - 57.0	5 hours	? - 57.0	56
BHF 11	430450	2781000	25.5	270.0	0 - 42.0	42.0 - 270.0	3 hours	?-51.0	305
							4 hours	51.0 - 270.0	52
BHF 12(R)	4 32200	2776070	26.5	204.0	0 - 72.0	72.0 - 204.0	2 hours	? - 102.0	550
							2 hours	102.0 - 204.0	490
BHF 13	427800	2774900	82.5	75.0	0 - 38.0	38.0 - 75.0	8 mins.	29.0 - 75.0	8.3
BHF 14	433900	2778750	9.5	113.0	0 - 99.0	99.0 - 113.0	40 mins.	22.3 - 113.0	3100
BHF 15(R)	428302	2778942	65.96^	60.0	-----	-----	-----	-----	-----
BHF 16(R)	428310	2777575	56.5*	62.0	-----	-----	-----	-----	-----
BHF 17A	433860	2776020	11.5*	-----	-----	-----	-----	-----	-----
BHF 17(R)	433860	2776020	11.5*	-----	-----	-----	-----	-----	-----
BHF 18(R)	431380	2778340	37.5*	47.0	-----	-----	-----	-----	-----
GWR No.6	432470	2778125	28.5*	-----	-----	-----	-----	-----	-----

Assumes level of casing top is 1.5m above ground level (Source: MAF).

^ Leveled top of casing.

* Estimated elevations

Several lithologic cross sections have been developed based on well lithology to help in examining the gravels thickness. Figure 5.3 shows a longitudinal cross section through Wadi Ham plain. This cross section shows that the thickness of wadi gravels and sand along Wadi Ham plain starts small upstream of the dam and increases downstream towards the Gulf of Oman. The thickness of wadi gravels along this section varies from 15 m to 63 m.

Figure 5.4 shows another cross section of Wadi Ham plain taken in transverse direction. This section shows that the thickness of wadi gravels varies also in transverse direction being more thick to the north than to the south. The thickness of wadi gravels along this section varies from 24 m to 99 m.

The wadi gravels are highly permeable, and also variable in hydraulic properties. Gravels tend to be unconsolidated at the ground surface, becoming better cemented and consolidated with depth. Some of the exploration boreholes through the wadi gravels showed presence of layers of well cemented gravel inter-bedded with poorly consolidated gravels which may form local confining layers (Entec, 1996).

Hydraulic conductivity values of the unconsolidated gravels tend to be very high, typically being 6 m/day to 17 m/day. Hydraulic conductivity of the cemented lower layer is in the range 0.086 to 0.86 m/day. In the unconsolidated gravels primary porosity is very high when compared to the cemented gravels. The specific yields typically range from 0.1 to 0.3 (Electrowatt, 1980). About 2 km wide and 3.5 km long section directly downstream of the dam, the saturated aquifer thickness ranges from 10 m to 40m with transmissivity ranging from less than 100 to about 200 m^2/day. The coastal plain, about 4.5 km in length and more than 8 km wide, has a saturated aquifer thickness from 50 m to 100 m with transmissivity more than 1000 m^2/day (ESCWA, 1997)

5.2.2 The Ophiolite

The stratum of gabbro and diorite of the Samail Ophiolite lies beneath the wadi gravels. The depth to the ophiolite layer varies from 15 m to 100 m. The gabbro/diorite layer is likely to be confined in some places by the cemented units within the base of the Wadi gravels. In other areas it may be semi-confined or even unconfined where fissured and fractured zones are encountered (Entec, 1996).

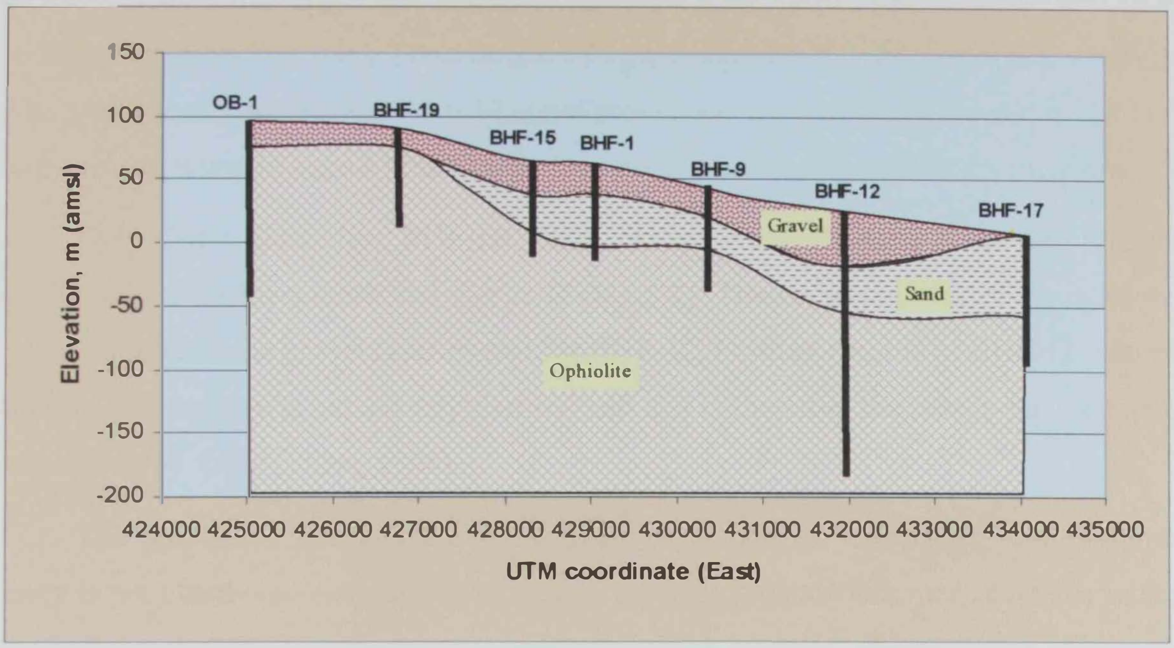

Figure 5.3. Longitudinal cross section through Wadi Ham plain (revised after Sherif et al., 2004).

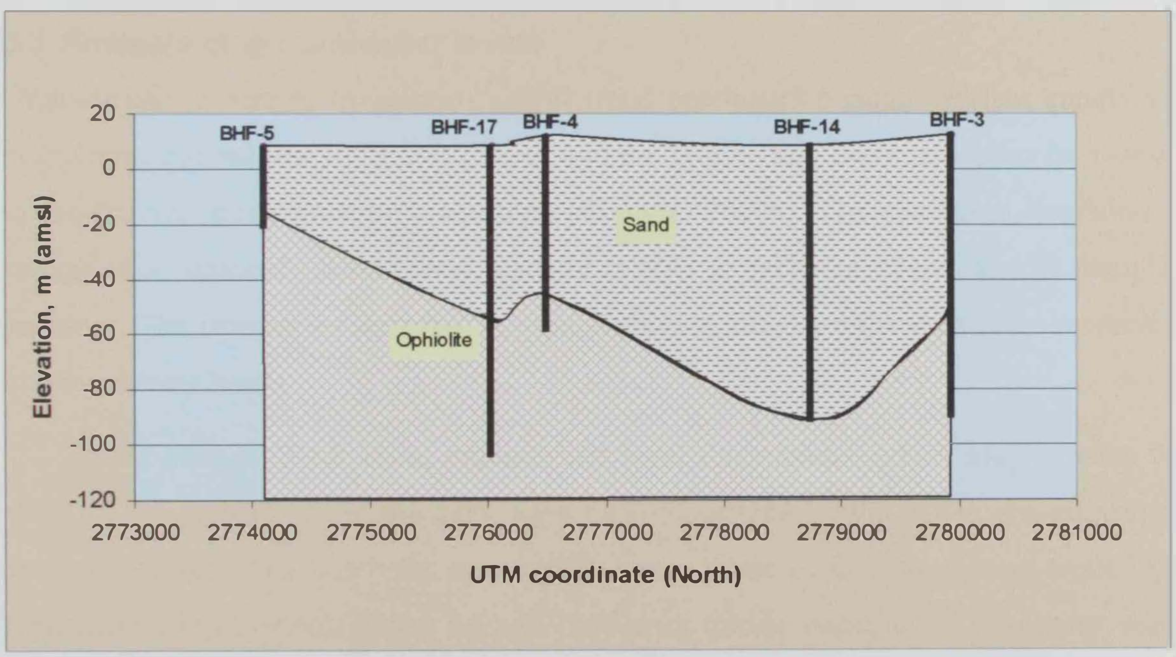

Figure 5.4. Transverse cross section through Wadi Ham plain (revised after Sherif *et al.*, 2004).

The study by Entec, 1996 discussed at length an eventual presence of a north west to south easterly trending major fault running through the Samail Ophiolite and attempted to locate the fault line using Time Domain Electromagnetic (TDEM) geophysical survey. The presence of fault was postulated but not proven and the study concluded that if a fault was present, it would have a limited effect on the groundwater system in the plain area.

Only three boreholes, BHF-1B, BHF-11 and BHF-12 have been test pumped exclusively from the gabbro and diorite (Table 5.1). Transmissivity values are relatively low compared to the gravel layer except for BHF-12. Drilling records of BHF-12 indicate that the gabbro here is extensively fissured and this is therefore the reason for the higher transmissivity value (Entec, 1996).

The interaction between the wadi gravel layer and the underlying gabbro/diorite layer is not clearly understood due to lack of separate groundwater measurements within the individual units. The groundwater modeling work by ESCWA (1997) has treated the wadi gravels as an unconfined aquifer without any hydraulic connection with the underlying ophiolite.

5.3 Analysis of groundwater levels

Groundwater is formed by excess rainfall (total precipitation minus surface runoff and evapotranspiration) that infiltrates deeper into the ground and eventually percolates down to the groundwater formations (aquifers) (Bouwer, 2000a). The change in groundwater storage is a seasonal phenomenon, usually related to recharge events and/or pumping practices. The increase or decrease in groundwater storage is reflected by rise or decline in groundwater levels.

In the case of Wadi Ham, groundwater level fluctuations, which may indicate the effect of the recharge from the dam, have been monitored by the MAF in a number of observation wells and boreholes on monthly basis. Some of the observation wells have groundwater level records dating back to 1977 prior to dam construction. However, some of the wells have been abandoned after being active for certain period. At the time of this study, monthly groundwater level data for 15 observation wells in the Wadi Ham plain area were available.

These observation wells, periods of data collection, and the minimum and maximum observed groundwater levels are listed in Table 5.2. There is a significant variation in groundwater level in each individual well with a fluctuation of water table ranging between 2 m and 50 m. The groundwater gradient in the plain area is very mild as compared to the gradient of groundwater within the area close to the dam. In some cases, water table levels fall below the mean sea level.

Table 5.2. Minimum and maximum water table levels in observation wells, m amsl

Observation well	Period	Max. water table		Min. water table		Remarks
		level	Month/yr	level	Month/yr	
BHF-1	1987-2003	53.066	5-1996	8.876	7-1994	Active
BHF-4	1988-2003	5.805	8-1996	2.585	7-2002	Active
BHF-4A	1990-2003	5.777	8-1996	2.347	7-2002	Active
BHF-9A	1987-2003	35.798	5-1996	0.728	6-1994	Active
BHF-9B	1990-2003	35.59	5-1996	3.02	7-2002	Active
BHF-12	1987-2003	11.329	7-1996	-1.191	8-2002	Active
BHF-15	1988-2002	64.106	4-1996	13.516	10-2001	Abandoned
BHF-16	1988-2003	54.317	5-1996	33.627	9-2002	Active
BHF-18	1988-2000	12.364	8-1996	-0.256	11-1989	Abandoned
BHF-19	1995-2003	86.618	3-1996	52.27	10-2002	Active
BHF-20	1995-2002	53.969	5-1996	21.559	9-2002	Abandoned
BHF-17R	1988-2003	3.153	7-1996	0.823	8-2000	Active
BHF-17A	1989-2003	3.897	6-1997	1.477	9-1999	Active
GWR-6	1977-2002	5.015	9-1996	-3.955	12-1984	Active
GWR-5	1977-2002	3.602	5-1996	-0.168	12-1980	Abandoned

The minimum water table fluctuation is found at the well number (BHF-17A) which is close to the farms at Kalba where groundwater is being intensively pumped. The maximum water table fluctuation is observed at the well number (BHF-15) which is very close to the dam in the downstream side. Except for well number (BHF-17A), the maximum water table levels were observed in the year 1996 during which heavy rainfall events have encountered and resulted in large floods. There is a very remarkable rise in groundwater levels after the heavy rainfalls and floods during the months of December, 1995 to February, 1996. The rise in groundwater levels is thus attributed to the recharge associated with this rainfall and floods.

The variation in groundwater levels is better understood and analyzed by plotting time series hydrographs of groundwater levels. Such hydrographs have been plotted for each observation well. The general trend line of groundwater fluctuations was fit on each hydrograph using simple regression technique. The monthly rainfall values were also plotted on the same graph to detect the response of groundwater levels to the recharge associated with rainfall and/or flood events.

Figure 5.5 shows the hydrograph of well number (BHF-1) which is located at downstream side close to the dam. Figure 5.6 shows the hydrograph of well number (BHF-4) which is located further away from the dam near the Gulf of Oman. In both wells the response to recharge is very clear. The three major peaks shown have originated from the recharge caused by heavy rainfalls in the same period with large flood volumes observed at the dam site. Small peaks are also noted during periods of low rainfall with or without small storage behind the dam.

Figures 5.7 and 5.8 show the hydrographs of wells number (GWR-5) and (GWR-6). These two wells are of specific importance as they have records of groundwater levels dating back to 1977 prior to the dam construction. The trend of groundwater levels was found declining over the years in all wells except for these two wells. However, when considering only the period since 1988, like in other wells, the trend becomes declining again. This clearly indicates that there was a building up in groundwater levels observed after dam construction compared to those observed before the construction of the dam (Figs. 5.7 and 5.8).

To detect the variation in well response to the recharge, groundwater levels for all wells were plotted together in Figure 5.9 for the water-year 1995-96 which was a wet year. Heavy rainfalls occurred during this year in the months of December 1995, through February 1996, with large storage accumulated behind the dam. There is a significant difference in the amount of rise and the time of response to the recharge for the different wells. Wells that are located close to the dam have shown higher and faster response to the recharge than those located further away from the dam (Fig. 5.9). For example, in the case of (BHF-15) which is close to the dam the level rose about 29 m during this period and the peak was observed in April, 1996. However, for (BHF-4) which is further away from the dam, the rise was about 2.5 m and the peak was observed in August, 1996.

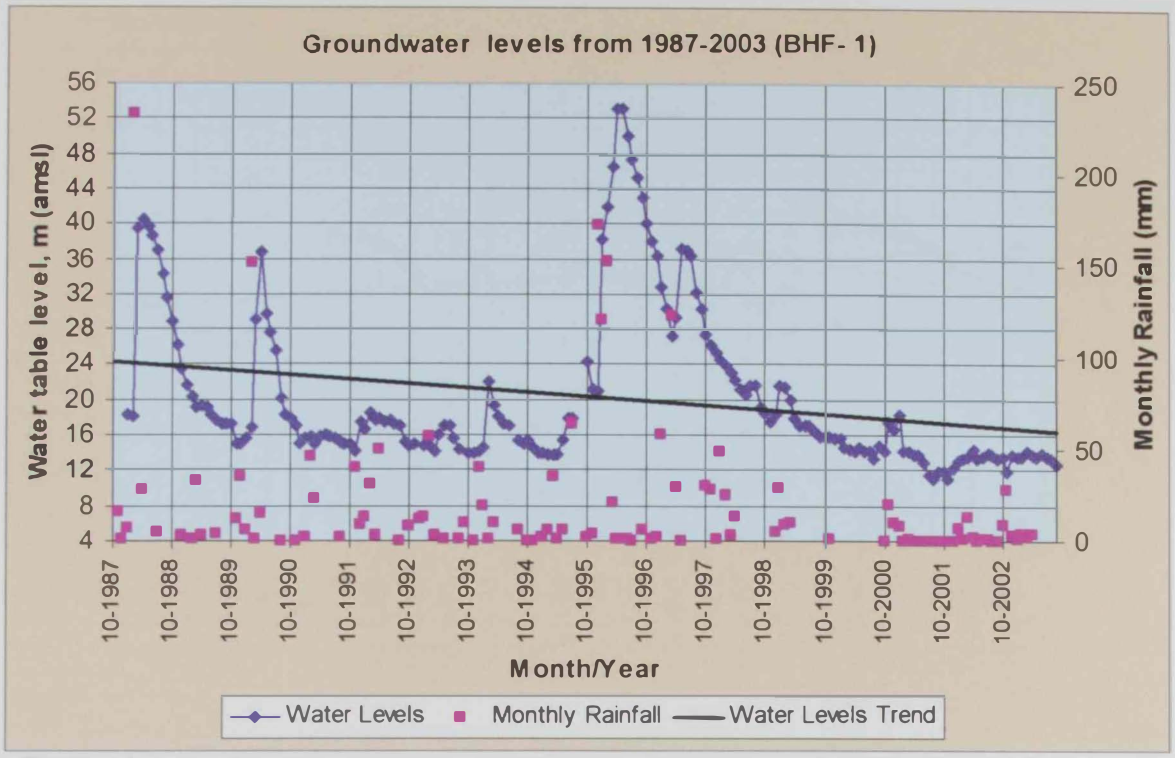

Figure 5.5. Variation of monthly groundwater levels for well number BHF-1 (revised after Sherif *et al.*, 2004).

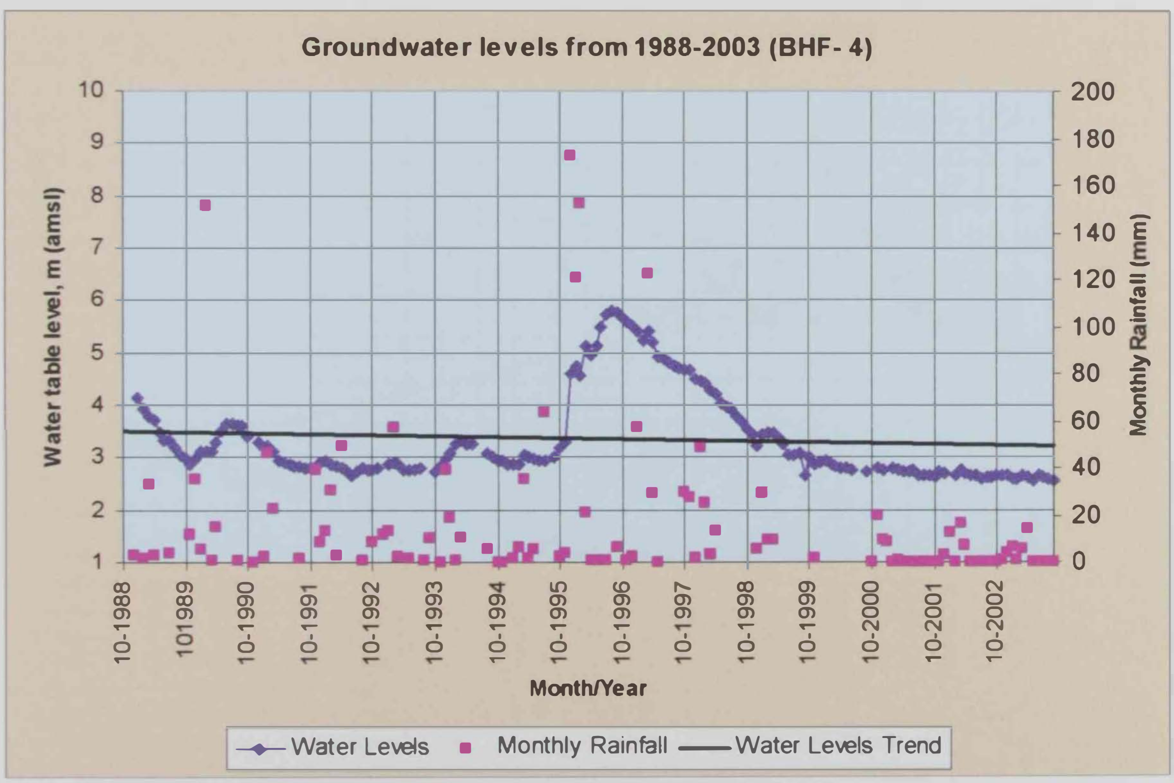

Figure 5.6. Variation of monthly groundwater levels for well number BHF-4 (revised after Sherif *et al.*, 2004).

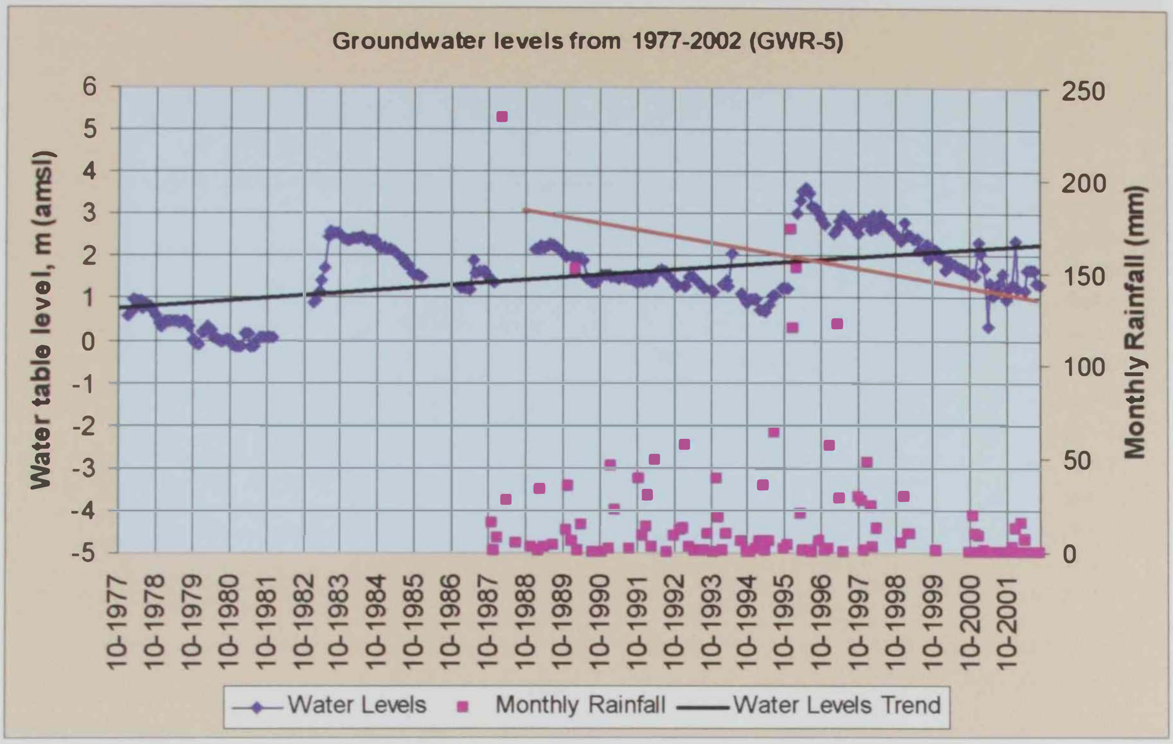

Figure 5.7. Variation of monthly groundwater levels for well number GWR-5 (revised after Sherif *et al.*, 2004).

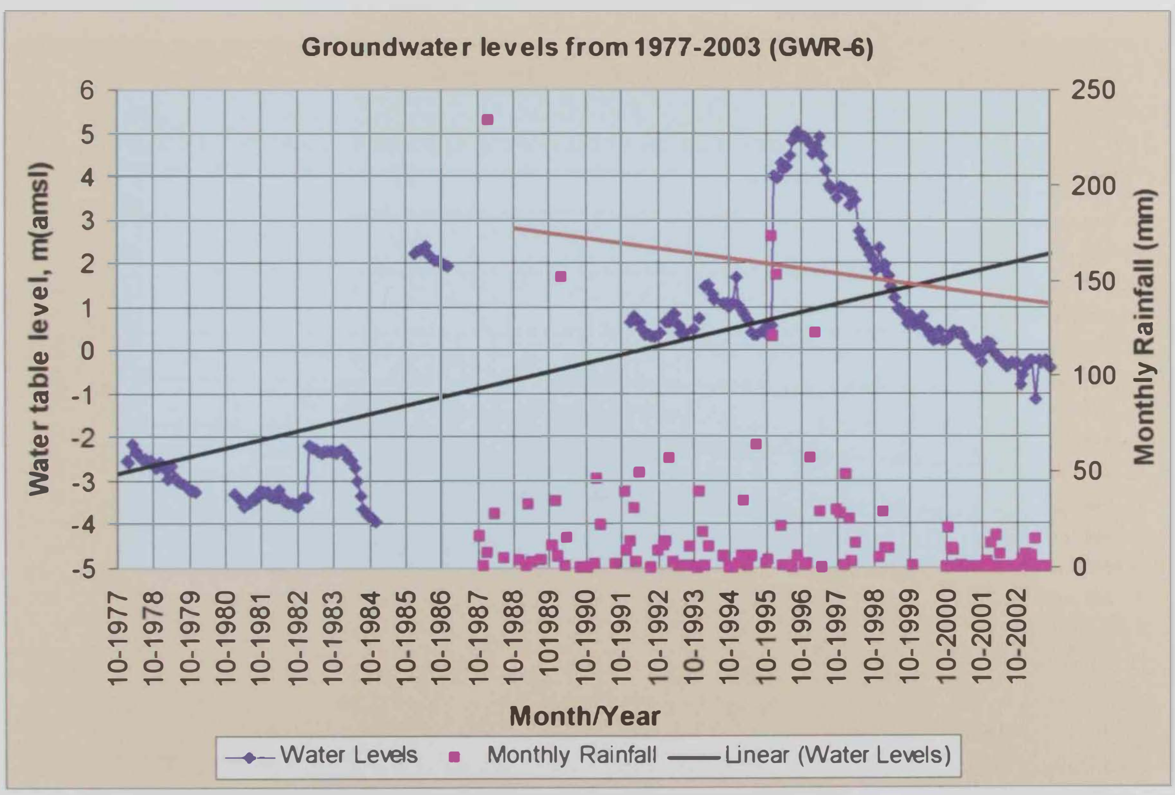

Figure 5.8. Variation of monthly groundwater levels for well number GWR-6 (revised after Sherif *et al.*, 2004).

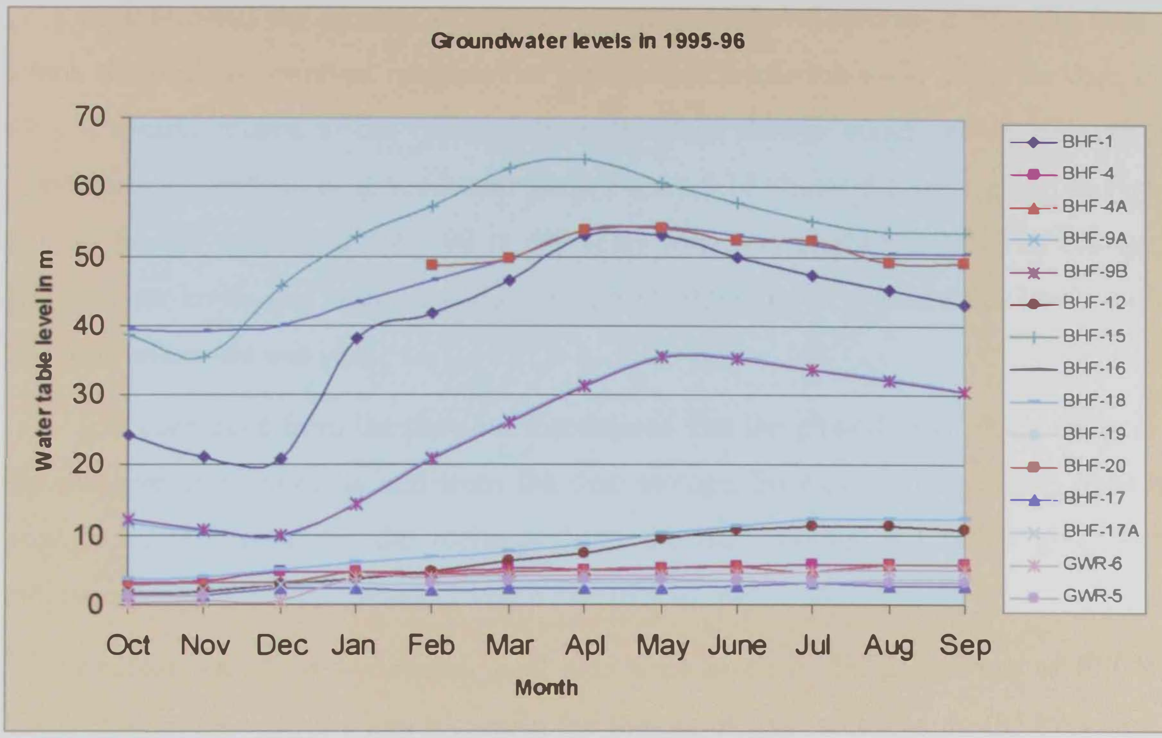

Figure 5.9. Comparison of monthly groundwater levels in wells in the year 1995-96 (after Sherif *et al.*, 2004).

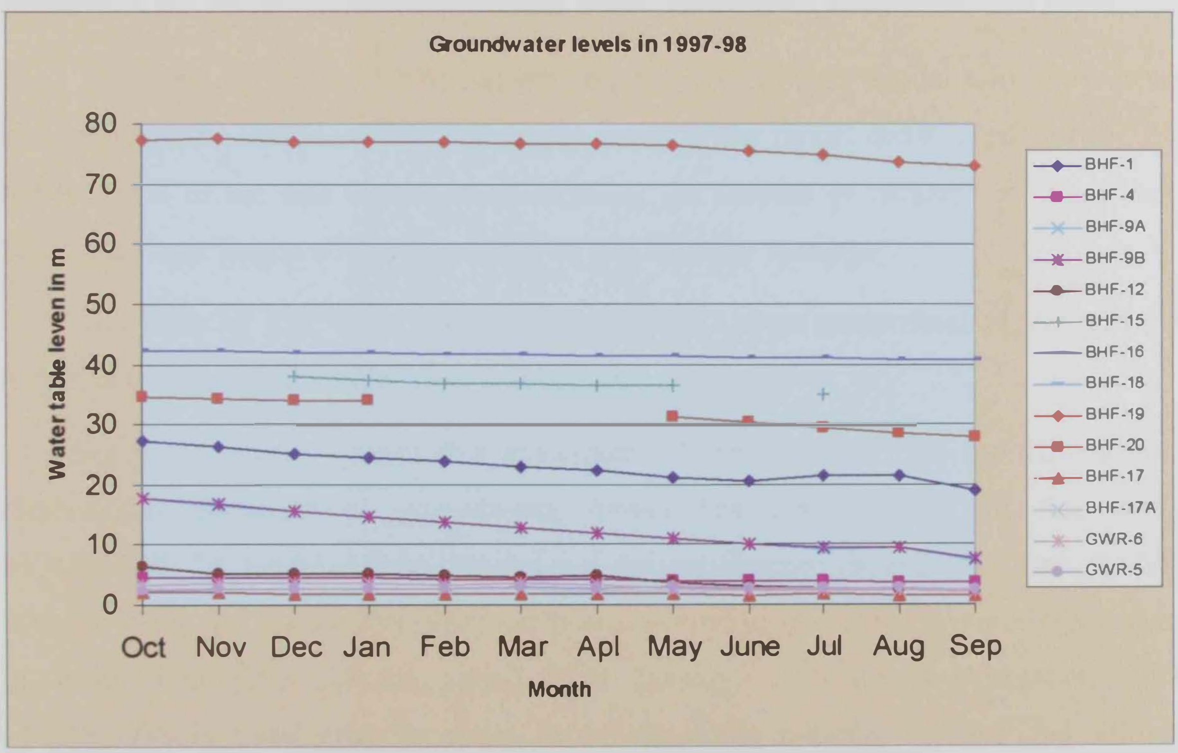

Figure 5.10. Comparison of monthly groundwater levels in wells in the year 1997-98 (after Sherif *et al.*, 2004).

In summary, the amount of rise in groundwater level decreases and the time at which the peak is observed increases as we move downstream away from the dam site. This is clearly related to the recharge from the dam storage which moves through the aquifer in the direction of groundwater flow. Figure 5.10 shows the same graph as Figure 5.9 but for the water year 1997-98 (a dry year) with very slight rainfall. The change in groundwater levels was steady and there was no building up of groundwater levels during this year unlike the wet year.

It is concluded from the previous discussions that the groundwater levels respond to the recharge from rainfalls and from the dam storage. However, this analysis does not provide quantification for the recharge from the dam storage but rather gives some indication on how the groundwater levels rise in response to recharge.

Evaluations of the hydrogeological data were made in the framework of ESCWA advisory services with the aim to assess the impact of dam recharge on the behavior of groundwater movement. Preliminary estimates of volume of dam recharge indicated a probability that a high percentage of flood water retained in the reservoir is producing recharge to the gravel aquifer (MacDonald, 1989 and Wagner, 1995).

The study by Entec (1996) has employed a groundwater model with only steady state phase and concluded that "the simple groundwater model constructed showed that the presence of the dam significantly enhances the amount of recharge in Wadi Ham, being the single largest controlling factor on groundwater recharge".

The study by ESCWA (1997) has constructed a more comprehensive groundwater model in which the subsurface flow was estimated at 2×10^6 m^3/yr.

For more reliable quantitative assessment of the recharge from the dam, a three dimensional mathematical groundwater model has been applied in this study. MODFLOW, "A modular three-dimensional finite-difference groundwater flow model", by McDonald and Harbaugh (1988) has been selected to model the groundwater system in Wadi Ham plain for the period from January, 1990 up to December, 1993. MODFLOW is considered by many to be the most reliable, verified and utilized groundwater flow model available (Kresic, 1997). A brief description of MODFLOW and the detailed modeling work are presented in the following section.

5.4 Modeling of Groundwater System in Wadi Ham

A three-dimensional mathematical groundwater model has been applied to model the sand and gravel aquifer system in the Wadi Ham plain area. The main objective of the model is to assess the role of Wadi Ham Dam in recharging the aquifer on quantitative basis. For this purpose, the period from beginning of January, 1990 up to the end of December, 1993 has been selected for the simulation. The selection of this period was based on the availability of groundwater level records and the occurrence of flood events. Most of the observation wells have records for this period including those that were abandoned after this date. A major flood has occurred during this period in February, 1990 in addition to small floods occurred in February, 1992 and February, 1993. The simulation was based on constant density model without consideration of density difference between the freshwater and the brackish water caused by seawater intrusion in the coastal plain.

5.4.1 Modeling code

The "modular three-dimensional finite-difference groundwater flow model" is a program for simulating confined or unconfined, saturated flow in one, two, or three dimensions. It allows both steady-state and transient simulations (McDonald and Harbaugh, 1988; Harbaugh and McDonald, 1996a,b). This model, known as MODFLOW, is developed by the United States Geological Survey (USGS) and it is in the public domain. At least with the public domain packages, it does not simulate unsaturated flow or solute and heat transport. However, MT3D (Zheng, 1990) and MOC3D (Konikow *et al.*, 1996) can simulate solute transport and both of them rely on MODFLOW by using the output of MODFLOW for the solute transport modeling.

MODFLOW is probably the most popular groundwater modeling program in existence. Some reasons for this popularity may be: (1) the program is applicable to most types of groundwater modeling problems, (2) the original packages in the program are well structured and documented, (3) the source code is in the public domain and thus can be checked for errors and modified by anyone with the necessary mathematical and programming skills, (4) the program is accepted by regulatory agencies and in litigation and (5) ongoing modifications of the program continue to increase its capabilities (Winston, 1999).

MODFLOW utilizes a numerical solution for the partial differential equation governing three-dimensional flow of groundwater of constant density through porous media:

$$\frac{\partial}{\partial x}\left(K_{xx}\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_{yy}\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_{zz}\frac{\partial h}{\partial z}\right)-W=S_s\frac{\partial h}{\partial t} \qquad (5.1)$$

where,

K_{xx}, K_{yy} and K_{zz} are values of hydraulic conductivity along the x, y and z coordinate axes which are assumed to be parallel to the major axes of hydraulic conductivity (Lt^{-1});

h is the potentiometric head (L);

W is a volumetric flux per unit volume and represents sources and/or sinks of water (t^{-1});

S_s is the specific storage of the porous material (L^{-1}); and

t is the time (t).

In general S_s, K_{xx}, K_{yy} and K_{zz} may be functions of space ($S_s = S_s(x,y,z)$, $K_{xx} = K_{xx}(x,y,z)$, etc.) and W may be a function of space and time ($W = W(x,y,z,t)$). Equation 5.1 describes groundwater flow under non-equilibrium conditions in a heterogeneous and anisotropic medium, provided the principal axes of hydraulic conductivity are aligned with the coordinates directions. Equation 5.1 together with specification of flow and/or head conditions at the boundaries of the aquifer system and specification of initial head conditions constitutes a mathematical representation of a groundwater flow system (McDonald and Harbaugh, 1988).

Except for very simple systems, analytical solution of Equation 5.1 is rarely possible so numerical methods must be employed to obtain approximate solutions. MODFLOW employs a finite-difference block-centered method in which a *grid* is formed by two sets of parallel lines that are orthogonal. The blocks formed by these lines are called *cells*. In the center of each cell is the *node* – at which the model calculates hydraulic head. The continuous system described by Equation 5.1 is then replaced by the finite set of discrete nodal points in space and time and the partial derivatives are replaced by terms calculated from the differences in head values at these points. The process leads to matrix systems of simultaneous linear algebraic difference equations; their solution yields values of head at specific points and times (Kresic, 1997).

MODFLOW offers choices of different solver packages to solve the matrix equations including Slice-Successive Over-relaxation Package (SOR), Strongly Implicit Procedure Package (SIP), Preconditioned Conjugate-Gradient Package (PCG4) and Link-Algebraic Multigrid Package (LMG).

MODFLOW possesses an open structure in which highly independent subroutines called "modules" are grouped into "packages" which deal with the specific hydrologic feature to be simulated. New modules and packages can be easily added to the program without modifying the existing packages or main code (Kresic, 1997). Packages of MODFLOW include, among others, the River Package, Recharge Package, Well Package, Drain Package, Evapotranspiration Package and the General Head Boundary Package. After its original release, MODFLOW was supplemented afterwards by various new packages and solvers either by the USGS or by other authorities.

Although MODFLOW is a powerful program, it can hardly be described as user-friendly. All input for the program is in the form of large text files that describe the grid structure, hydraulic properties, boundary conditions and transient data. These files must follow a strict format. If the format requirements are violated, MODFLOW will either not be able to run or will produce incorrect results. To alleviate this problem, a large number of pre- and post-processors have been developed that provide a graphical user interface for MODFLOW (Winston, 1999).

There are several integrated, user-friendly pre- and post-processing software packages that provide easy data input and visualization of modeling results in forms of contour maps and graphs. One of the most comprehensive and well structured modeling packages is the Visual MODFLOW, by Waterloo Hydrogeologic Inc., which combines MODFLOW, MODPATH, ZoneBudget, MT3Dxx/RT3D and the parameter estimation package, WinPEST. The logical menu structure and easy-to-use graphical tools allow to: easily dimension the model domain and select units, conveniently assign model properties and boundary conditions, run the model simulations, calibrate the model using manual or automated techniques (WinPEST), and visualize the results using 2D or 3D graphics.

The version, Visual MODFLOW Pro 3.1, (Waterloo Hydrogeologic, 2003) has been used in this study for construction of the groundwater model of Wadi Ham.

5.4.2 Model concept, construction and input data

Developing a model concept is the initial and the most important part of every modeling effort. It requires a thorough understanding of hydrogeology, hydrology and dynamics of groundwater flow in and around the area of interest. The final result is a computerized data base and simplified maps and cross sections that will be used in model design (Kresic, 1997). The model of the groundwater aquifer system in Wadi Ham was conceptualized and constructed based on the available information as follows.

Model Geometry and Grid

The model was set on an area of 110 km^2 that covers the aquifer system in Wadi Ham plain from upstream of dam to the Gulf of Oman and from south of Kalba to north of Fujairah. The model extends 11 km from east to west and 10 km from north to south between the UTM coordinates (425000E, 2772000N) and (436000E, 2782000N) (Fig. 5.11). The model area was discretized using uniform grid of 50 rows and 55 columns. The cells are thus 200 m by 200 m squares and the total number of cells is 2750 (Fig. 5.12).

Boundary Conditions

The model of Wadi Ham was made of one layer representing the wadi gravels aquifer. The aquifer was treated as unconfined aquifer assuming the underlying Ophiolite to be impermeable. Although the Ophiolite is fissured and fractured in some areas and thus permeable, a two-layer representation is not recommended when taking in consideration the available information. Such two-layer model would require known geometry and separate distribution of hydrogeologic parameters and hydraulic heads for each layer.

The model area and the aquifer boundaries were delineated and digitized based on the remote sensing image of Wadi Ham. The aquifer boundaries include the Gulf of Oman which is treated as constant head boundary. Out of the total number of cells (2750), 178 cells were located in the sea and thus assigned a constant head of zero (Fig. 5.12).

The aquifer is also bounded by the hard rocks of the Ophiolite sequence in the mountain ranges. The Ophiolite sequence rocks also outcrop in the model area as isolated hills near the dam and downstream of the dam in the plain area. Cells that are occupied by these Ophiolite rocks were all marked as inactive (no flow) cells.

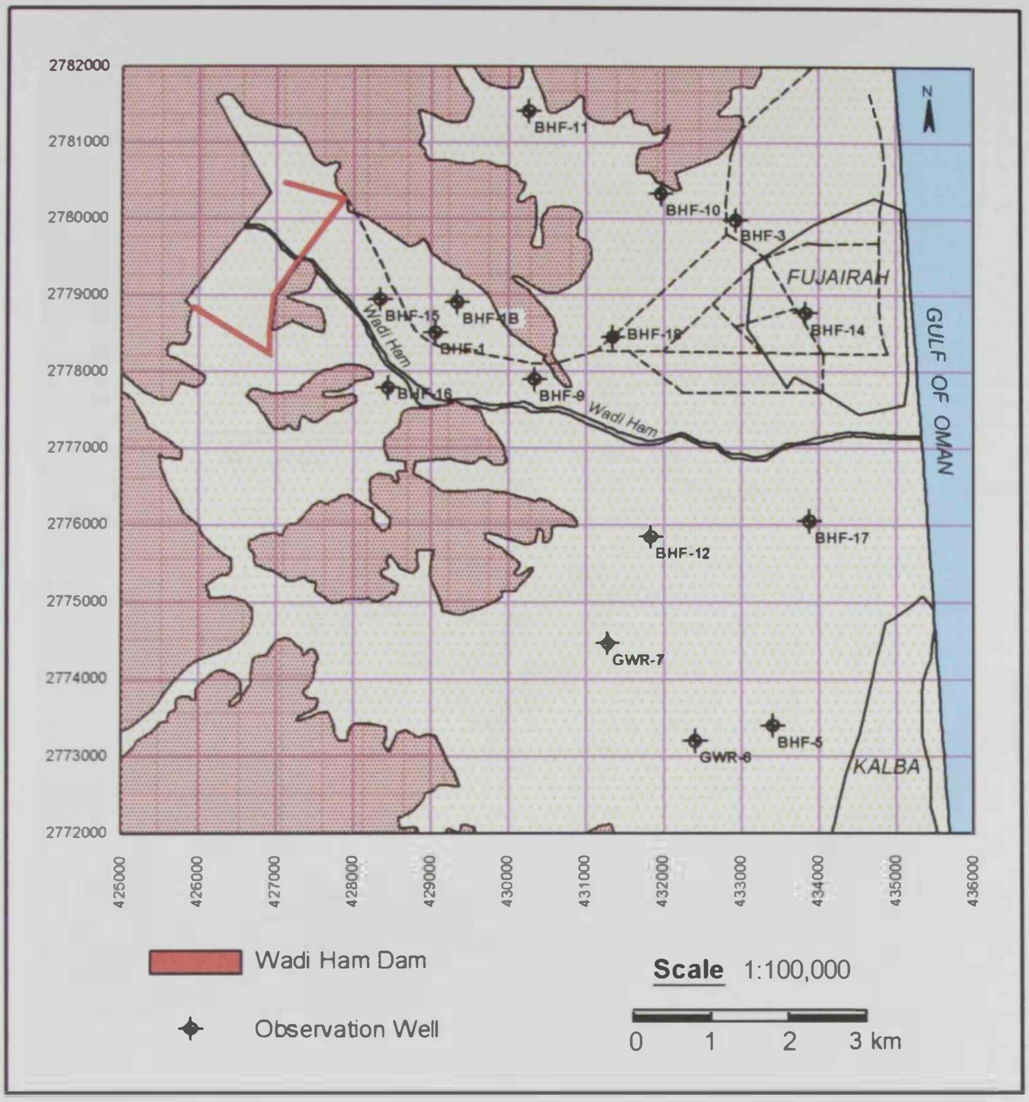

Figure 5.11. Wadi Ham model area.

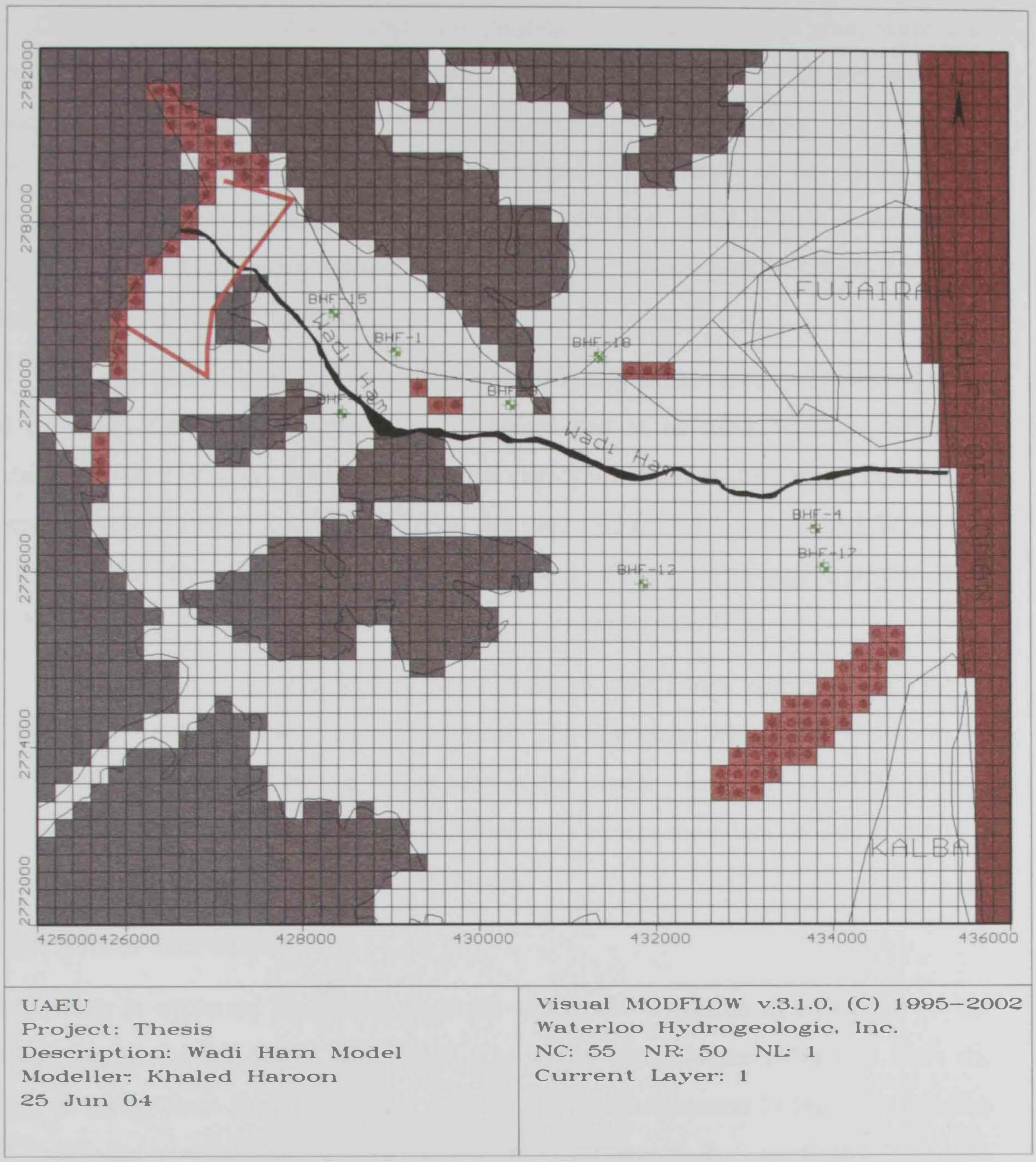

Figure 5.12. Model grid and boundaries.

Cells in the area that lies upstream (beyond the dam retention area) were also marked as inactive cells. There was no need to extend the model to this area where no sufficient information about lithology or head exists. The total number of the inactive cells in the model is 806 cells. Other boundary cells at the edges of the model (in the north and south) were not assigned any boundary condition. MODFLOW automatically assumes no flow boundaries around the edges of the model domain unless otherwise specified by a boundary condition.

To compensate for the inflow to the aquifer from upstream area and from the upper part of Wadi Yabsah, a specified flow boundary had to be used. In finite-difference models, like MODFLOW, specified flow boundaries are simulated by using injection or pumping wells to inject or extract water at the specified rates (Anderson and Woessner, 1992). For Wadi Ham model, 31 cells were assigned inflow rates through injection wells which are simply pumping wells but with positive pumping rates (Fig. 5.12).

Recharge to the aquifer was assigned through two zones corresponding to the recharge from rainfall and from the dam storage. The first zone, which corresponds to the recharge from rainfall, covers the whole area of the model. However, MODFLOW will assign recharge to "active" cells only. The other zone covers the retention area behind the dam where 30 cells have been assigned recharge.

Groundwater Extraction

Groundwater is exploited intensively from the aquifer for irrigation in farms and for the domestic supply in the MEW well fields. The major groundwater extraction from the aquifer is taking place from the Sha'ara well field 2 km downstream of the dam, Fujairah well field in the west of Fujairah and wells in the palm trees farms near Kalba. Estimates for the pumping from these locations were reported by ESCWA (1997) as the following:

a. Fujairah well field: 3.2 million m^3/year until 1988, very limited groundwater extraction since 1988;
b. Sha'ara well field: about 1 million m^3/year since 1988; and
c. New well field near Kalba, about 6 million m^3/year, partly replaced by new desalination plant in 1995.

Pumping was assigned to 45 cells in the model, three for Sha'ara well field; three for Fujairah well field and the rest (39 cells) for wells near Kalba (Fig. 5.12).

Hydrogeological Data

The hydrogeological input data to Wadi Ham model included aquifer geometry, hydraulic conductivities and storativities. To input aquifer geometry, data from 20 boreholes and observation wells with known lithology were used. Aquifer geometry was introduced to the model in terms of ground surface and aquifer bottom elevations from. Visual MODFLOW can import and interpolate external data files (Surfer™ .GRD or ASCII .TXT format) to create variable bottom and top elevations for each cell in the model. The interpolated surfaces of ground and aquifer bottom are presented in the form of contour maps in Figures 5.13 and 5.14.

The model area was divided into different hydraulic conductivity and specific yield zones and initial guesses of these were quoted from the values reported in previous studies (Electrowatt, 1980 and Entec, 1996) as described previously in section 5.2. However, these values have been readjusted during model calibration.

Hydrological Data

The hydrologic data considered in the model included rainfall data and flood occurrences and volumes. These data helped in assigning recharge rates to the aquifer. Assumed percentages of rainfall were input to the model on monthly basis. However, flood volumes were not used directly in the model but rather assumed recharge (infiltration) rates in the retention area were used. Both recharge rates from rainfall and from the dam were readjusted during model calibration.

Observed Groundwater Levels

The Waterloo Hydrogeologic Inc. version of MODFLOW contains an enhancement designed to make model calibration more efficient. This Calibration Package saves the calculated heads at the locations of specified *observation wells* every time step in a .HVT file (Head versus Time). This allows the comparison of simulated heads with observed heads and produces calibration statistics and time series graphs at observation wells (Waterloo Hydrogeologic, 2003). The observation wells and boreholes available in Wadi Ham were merged in the model and available records of monthly groundwater levels were specified for each individual well.

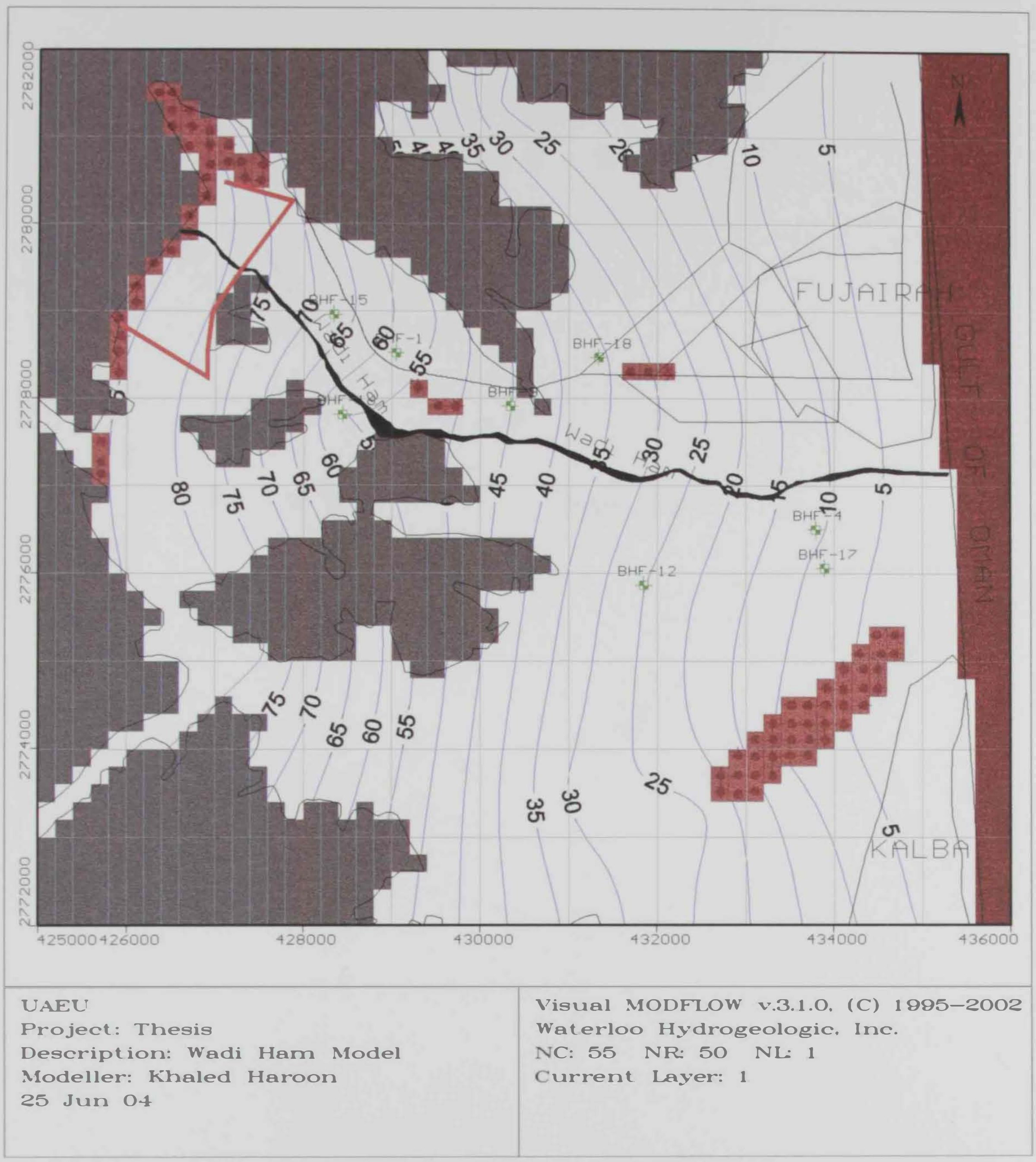

Figure 5.13. Contour map of ground surface levels (input to model).

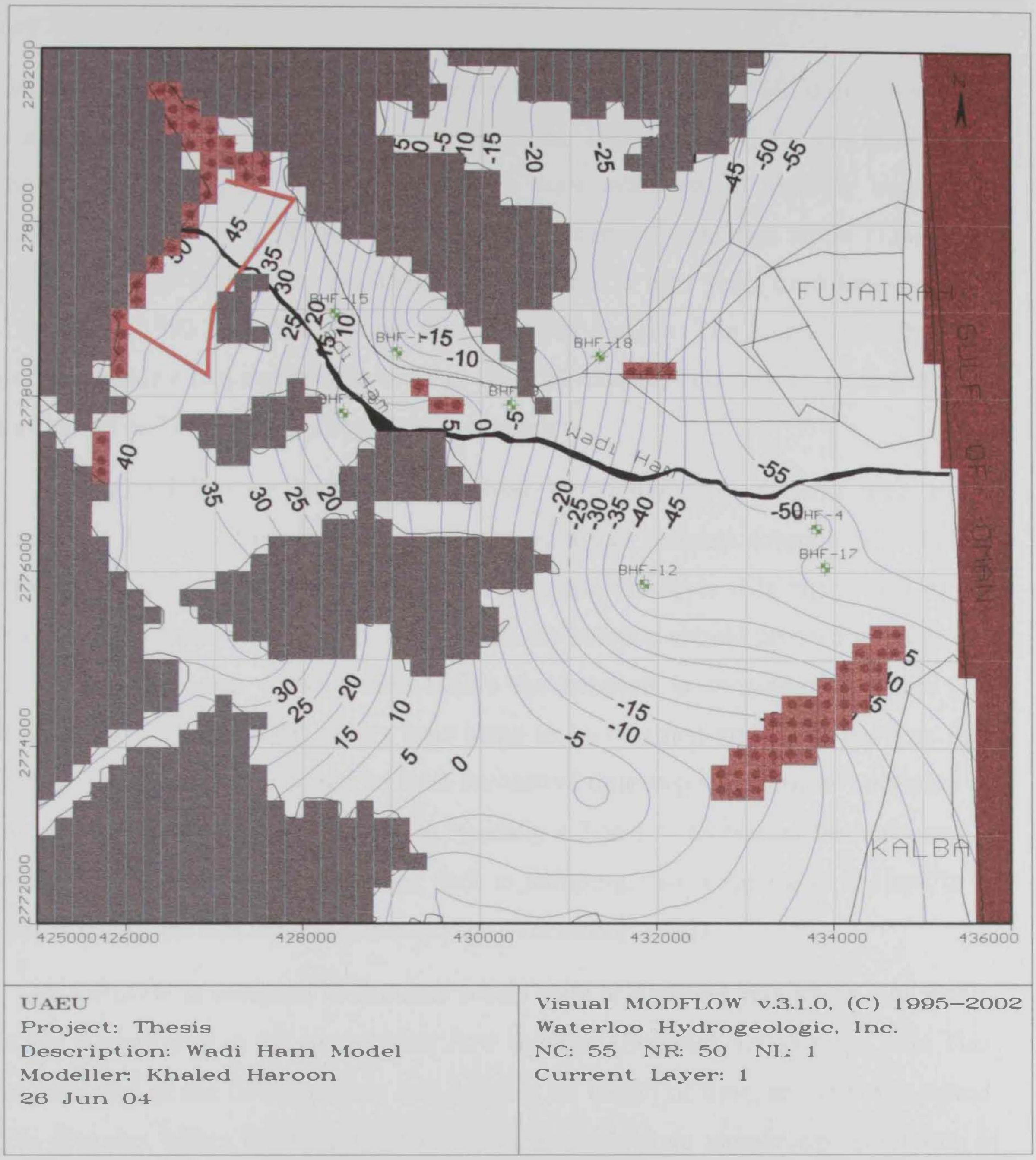

Figure 5.14. Contour map of aquifer bottom levels (input to model).

Time Discretization

In MODFLOW terminology, a *stress period* is defined as a time period during which all the stresses (boundary conditions, pumping rates, recharge, etc.) on the system are constant. Since most of data for Wadi Ham were available on monthly basis (like observed heads) or even yearly (like groundwater extractions), each stress period was taken equal to one month. The simulation period which is **four** years from January, 1990 to December, 1993 was thus divided into 48 stress periods. The length of each stress period was made a real month; that is either 28 or 29 days for February depending on year and either 30 or 31 days depending on actual month.

The finite-difference technique employed by MODFLOW requires each *stress period* to be discretized in several *time steps* to obtain an accurate solution. The smaller the time step, the more accurate the solution obtained. However it is impractical to use extremely small time steps. As a rule of thumb, the solution should proceed through five time steps during each stress period before the solution is considered accurate (De Marsily, 1986). MODFLOW allows time steps to be of equal size or to increase in a geometric series; that is the length of each successive time step is a constant multiplier of the previous time step. The multiplier is typically 1.2 to 1.5. Increasing the time step is recommended when simulating a stress, such as pumping, that is applied to the aquifer at the beginning of the simulation (Anderson and Woessener, 1992).

MODFLOW is designed to simulate steady state or transient conditions. For steady state, the storage term in the groundwater flow equation (Equation 5.1) is set to zero. This is the only part of the flow equation that depends on length of time, so the stress-period length does not affect the calculated heads in a steady-state simulation. Therefore, a single time step is all that is required for steady-state stress periods (Harbaugh *et al*., 2000). For the model of Wadi Ham, each stress period was discretized into 10 time steps with a multiplier of 1.2.

5.4.3 Simulated processes, calibration and results

Calibration of a flow model is the process of finding a set of boundary conditions, stresses and hydrogeologic parameters which produces the result that most closely matches field measurements of hydraulic heads and flows. It is a demonstration that the model is capable of producing field-measured heads and flows which are the *calibration values*.

Calibration is the most important, critical and time consuming stage in any modeling task. Tens to hundreds of model runs are typically needed to achieve calibration (Kresic, 1997; Anderson and Woessener, 1992).

Model calibration can be performed to steady-state or transient data sets. Most calibrations are performed under steady-state conditions but may also involve a second calibration to a transient data set. The most common type of transient calibration begins the simulation from the calibrated steady-state solution. For example, initial conditions for the transient calibration may represent steady-state conditions prior to development of the aquifer. The model is then calibrated to a time series of water level changes caused by pumping (Anderson and Woessener, 1992).

There are two methods of calibration; trial-and-error (manual) calibration and automated calibration. Manual calibration was the first technique applied in groundwater modeling and is still preferred by most users. Although it is heavily influenced by the user's experience, it is always recommended to perform this type of calibration, at least in part. It will sometimes be necessary to change the input and run the model tens of times before reaching the target. Automated calibration is a recent technique developed in order to minimize uncertainties associated with the user's subjectivity. Most computer codes for automated calibration search an optimal parameter set for which the sum of squared deviations between calculated and measured values is reduced to minimum. One of the highly regarded codes for parameter estimation developed for MODFLOW is the PEST. The efficiency of PEST and similar codes, coupled with the trial-and-error input from user, is the most appropriate calibration method available (Kresic, 1997).

The calibration of Wadi Ham model was performed in two stages, steady-state and transient conditions. The steady-state calibration was performed to produce initial heads for the transient calibration. Combinations of manual trial-and-error calibration and automated calibration, using the PEST package integrated with Visual MODFLOW, have been used. Initially, several runs were performed using manual calibration until a rather agreement was obtained. The manual calibration gave an idea about accepted ranges of values of parameters which were used as guidance for automated calibration. The calibration was then continued using automated calibration in alternation with manual calibration until the best possible results were achieved.

Steady-State Simulation

In order to perform a transient simulation, the system must be initially at a balanced state. A stable piezometric surface must be used as initial head condition. The available groundwater level records, however, were not sufficient to derive such a stable surface. Therefore, a steady-state calibration had to be used. Although such a steady-state condition may not exist and groundwater levels appear to be significantly fluctuating, it provides a balanced initial head necessary for the transient simulation.

The month of January, 1990 was selected as steady-state condition. This month came after a prolonged dry period from May through November, 1989 with very slight water level fluctuations. The calibration parameters used for steady state calibration were hydraulic conductivities, boundary inflows, pumping rates from well fields and the rainfall from the month of December, 1989 which provided initial recharge to the system.

The model was calibrated to match the observed heads at observation wells. The calibrated values of hydraulic conductivity were around 5 m/day at dam site and downstream up to BHF-1, 8 to 10 m/day in the area between BHF-1 and BHF-9, and between 20 to 28 m/day within the coastal plain. The water balance for the steady-state simulation, as extracted from the output file, is listed in Table 5.3.

Table 5.3. Water balance for steady-state simulation.

STRESS	IN (m^3/day)	OUT (m^3/day)
STORAGE	0.00	0.00
CONSTANT HEAD	8921.09	2020.69
WELLS	1600.00	17300.00
RECHARGE	8710.23	0.00
TOTAL	19231.30	19320.72
IN - OUT = -89.41		
PERCENT DISCREPANCY = -0.46%		

In the water balance totals, release of water from storage is counted as inflow and uptake is counted as outflow. The difference between total inflow and outflow is divided by either inflow or outflow (inflow in MODFLOW) to yield error in water balance. An error of around 1% is usually considered acceptable (Anderson and Woessener, 1992). For the model of Wadi Ham this was 0.46% and thus the accuracy is satisfactory.

The recharge caused by December rainfall is 8710.23 m^3/day. This recharge was assigned at a rate of 0.1233 mm/day (3.452 mm/month) and infiltrated through an active area of 70.64 km^2 (total model area minus inactive and constant head cells). Knowing that the average areal rainfall for this month is 38 mm, the recharge is thus about 9 % of monthly rainfall.

The inflow from the upper part of the aquifer (shown as WELLS, IN) was simulated as being 1600 m^3/day. Groundwater extraction (WELLS, OUT) was 17300 m^3/day of which 3000 m^3/day and 300 m^3/day were pumped from Sha'ara and Fujairah well fields respectively. The pumping from Sha'ara well field is equivalent to 1.095 MCM per year which is slightly higher than the estimated 1 MCM per year. However, the pumping from wells near Kalba, which is equivalent to 5.11 MCM per year, is relatively less than the estimated 6 MCM per year.

The only constant head boundary in the model is the Gulf of Oman. Therefore, the constant head component of the water balance (IN and OUT) refers to the interaction between the aquifer and the Gulf of Oman. The water balance has shown that about 2020 m^3/day of groundwater was discharged to the Gulf whereas about 8921 m^3/day of seawater intruded the aquifer. This seawater intrusion was recognized in the field most notably in the area of BHF-4, BHF-17, BHF-5 and the farms near Kalba.

The distribution of heads produced by steady-state calibration in January, 1990 was used for the transient simulation. January, 1990 head distribution is presented in the form a contour map in Figure 5.15.

Transient Simulation

The period from beginning of January, 1990 to end of December, 1993 was simulated on transient basis. The head in January, 1990 was transferred directly from the steady-state phase and assigned as initial head condition. The same hydraulic conductivity zones created in the steady-state simulation were used in the transient calibration. The final calibrated values were similar to those of the steady-state except for the area from the dam up to BHF-1 where the conductivity was decreased slightly. A new parameter was added to the calibration process; that is the specific yield of the aquifer. This parameter is not used by MODFLOW in steady-state simulations.

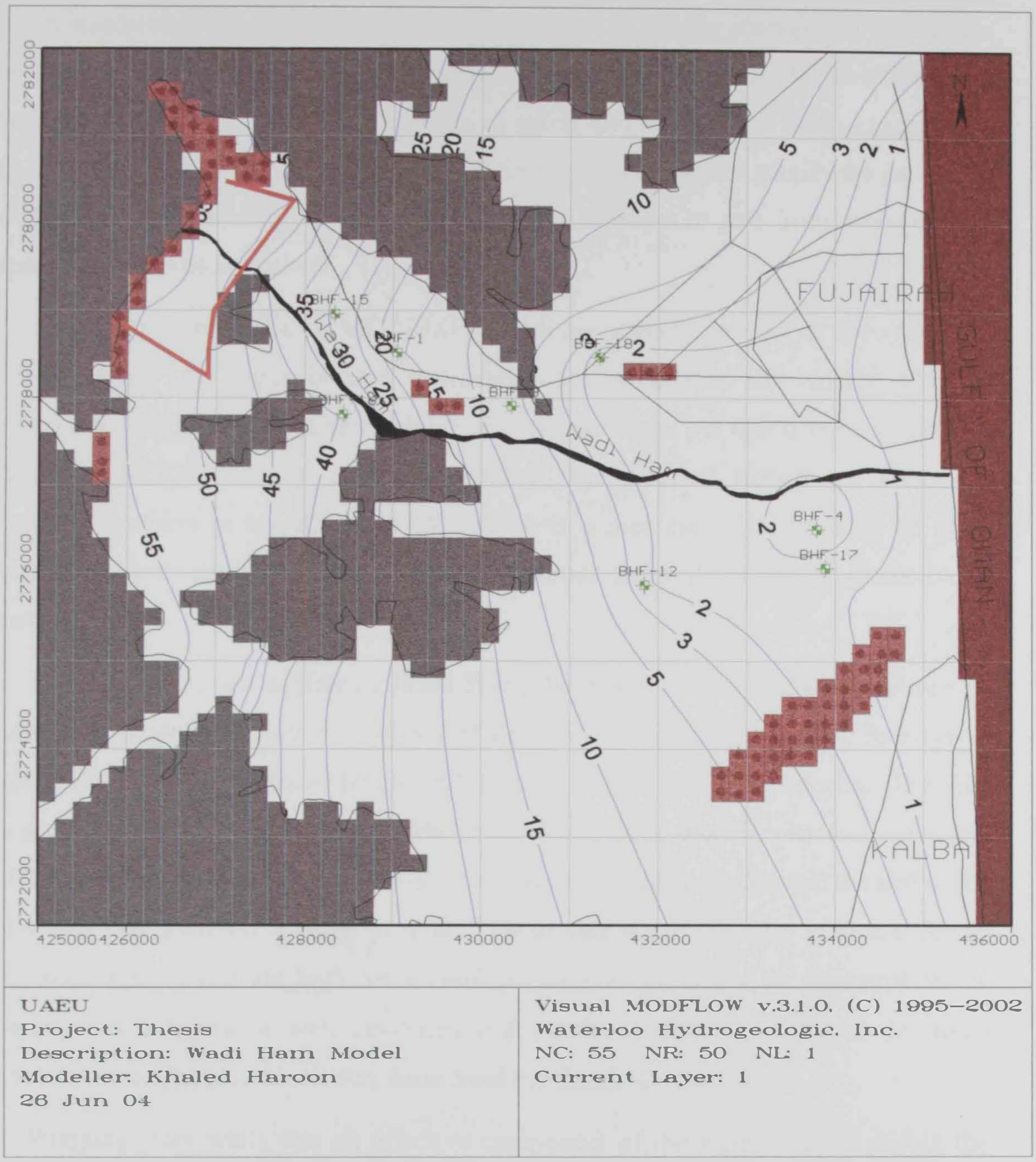

Figure 5.15. Head distribution in January,1990 (output of steady-state simulation, input for transient simulation).

In steady-state flow condition, there is no change in aquifer storage and the storage term in the groundwater flow equation (Equation 5.1) is set to zero. However, during a transient simulation, water is released from or taken into storage and heads change with time as a result of this transfer of water. Therefore, it is necessary to specify the parameter that describes the capacity of the aquifer to transfer water to and from storage. This property is known as *storativity*.

Storativity is specified to MODFLOW by either *specific storage* or *specific yield*. Specific storage (S_s) which is used in Equation 5.1 is equal to the volume of water released from storage within a unit volume of porous material per unit decline in head. S_s is used by MODFLOW for confined aquifers. The relevant storage parameter for unconfined aquifers is specific yield (S_y) which is a measure of the volume of water released by gravity drainage per unit volume of porous material in response to decline of the water table.

For the unconfined aquifer of Wadi Ham, the model calibration was achieved by dividing the model area into four zones of different specific yield values. In order to simulate the high rise in water levels, rather small values had to be adopted. The area between the dam and BHF-15 was divided into two zones of specific yields of 0.012 and 0.0078; the area downward up to BHF-9 was assigned a value of 0.005 and the rest of the model area was assigned a value of 0.042. These values are typically very low for an unconfined aquifer and are high for a confined aquifer. This may be attributed to the presence of the layers of well cemented and poorly consolidated gravels, described previously in section 5.2, which may form local confining layers.

Pumping from wells was an effective component of the water balance during the transient calibration. The pumping rates were varied from one month to another. The wells were simulated with very low rates during months with heavy rainfall in accordance with the fact that lower quantities of water would be required for irrigation. All wells were shut off in the simulation during the month of February, 1990 except for Sha'ara well field which was kept operating with a very low rate. For the other months, Sha'ara well field was simulated as producing between 2000 m^3/day to 4300 m^3/day with the cumulative production being about 1 MCM per year. The inflow from the upper part of the aquifer was simulated as being 3800 m^3/day on average.

The recharge to the groundwater system was of specific concern during the transient simulation. The recharge was treated as a space and time dependent process. The system was simulated as receiving recharge from both the rainfall and the retention area of the dam. The recharge was also varied with time from one month to another. The recharge from the dam was assigned during only three months among the simulated 48 months. These are February, 1990; February, 1992 and February, 1993. The month of February, 1990 has witnessed a major flood with large flood volume whereas small floods have occurred in the other two months. There was no information about the dwelling time of the storage in the reservoir. Therefore, the recharge was assumed to infiltrate through the whole month. Recharge rate was thus variable being much higher in February, 1990 than the other two months. The calibrated recharge rates are 0.0738, 0.0155 and 0.0351 m/day respectively.

Recharge from rainfall was assigned to the model during several months according to the historical records of monthly rainfall. The average areal rainfall (Chapter 3) was considered for this purpose assuming percentage of this rainfall to infiltrate and recharge the aquifer. This percentage was adjusted during model calibration and the final percentage accepted was 10% of the rainfall.

The transient calibration was performed on basis of both time and space. The calibration was performed in time by matching observed head values in observation wells using time series graphs. It should be emphasized that water level records were available on monthly basis and exact recording days were unknown. However, the rise of water level was found to appear in the reading of the month next to the month in which the flood and/or rainfall occurred. Knowing that and considering that recharge was assigned to the model during the same month of flood or rainfall, water level records were assumed to be taken in the beginning of each month. The calibration in space was performed by matching contours of observed head. The time series graphs of calculated versus observed heads for BHF-1, BHF-15 and BHF-9 are shown in Figures 5.16 through 5.18. Figures 5.19 through 5.21 show contour maps matching between calculated and observed heads at selected times (before, during the peak and after the major flood). Considering the nature of information available and the matching shown in this figures, the model is believed to be calibrated to an acceptable degree.

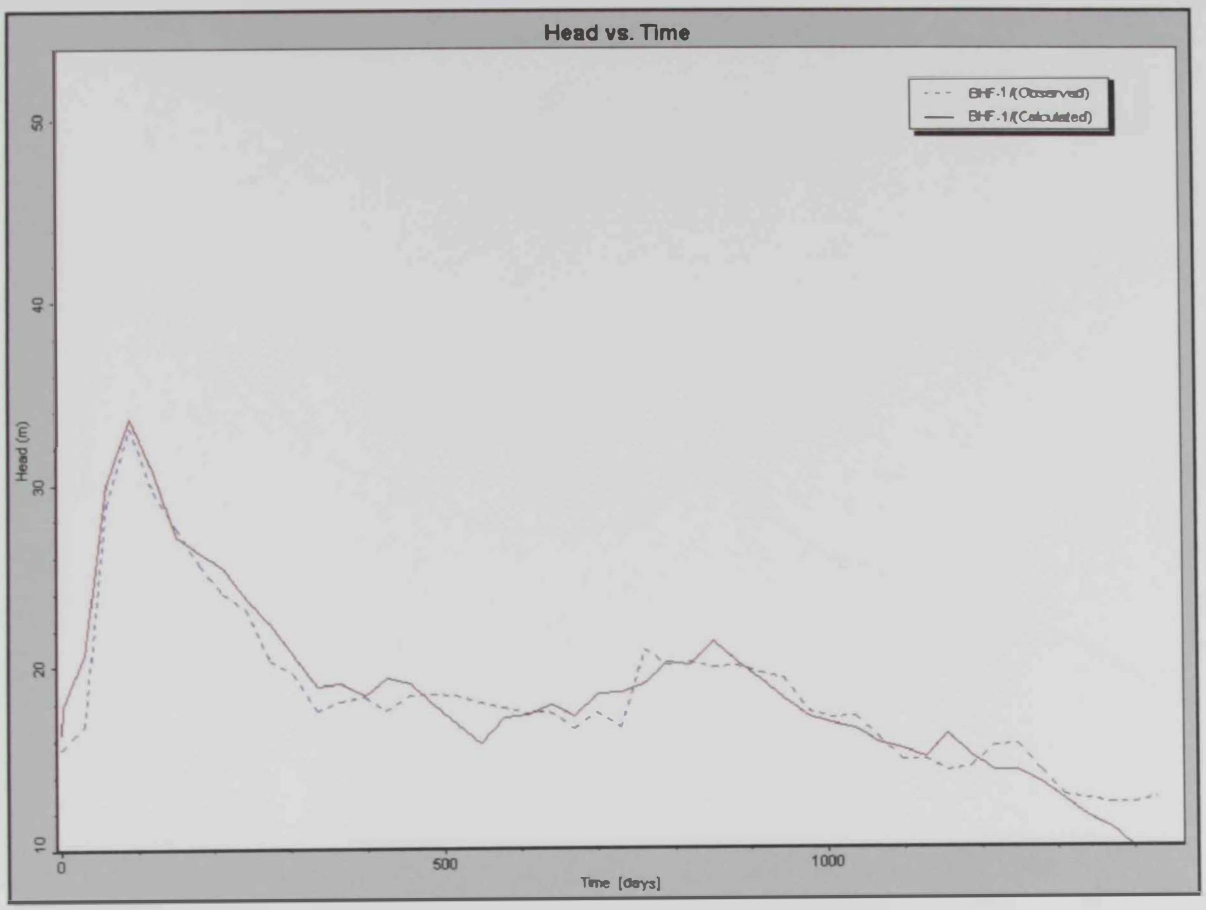

Figure 5.16. Simulated versus observed heads (BHF-1).

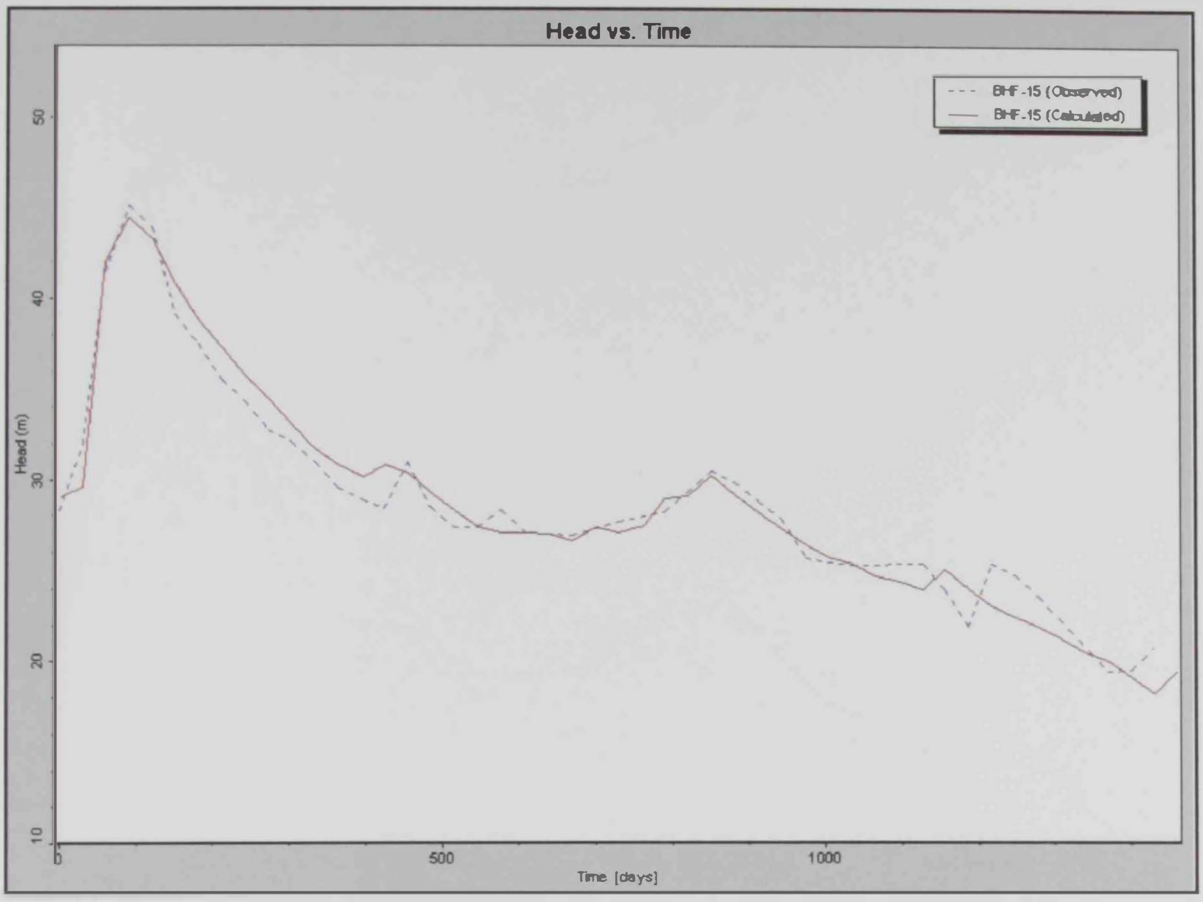

Figure 5.17. Simulated versus observed heads (BHF-15).

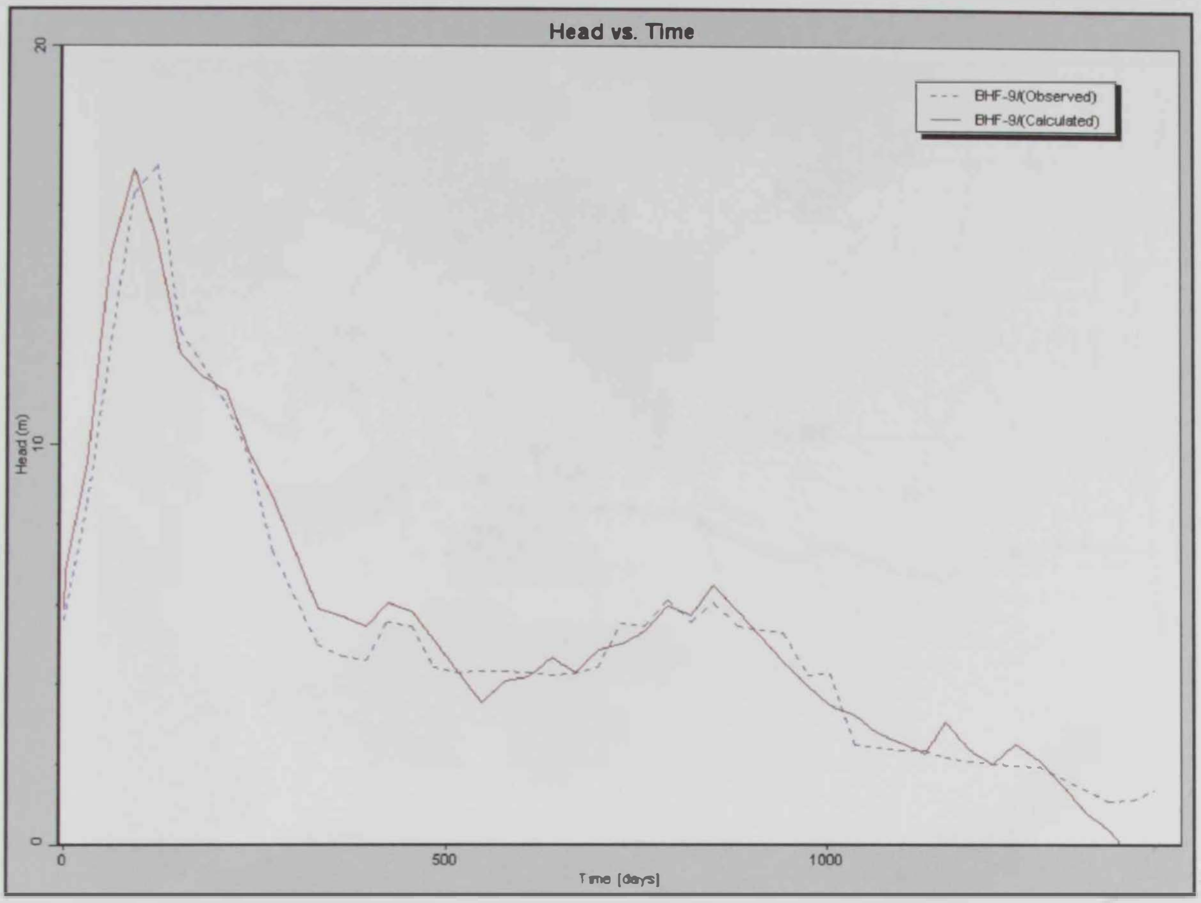

Figure 5.18. Simulated versus observed heads (BHF-9).

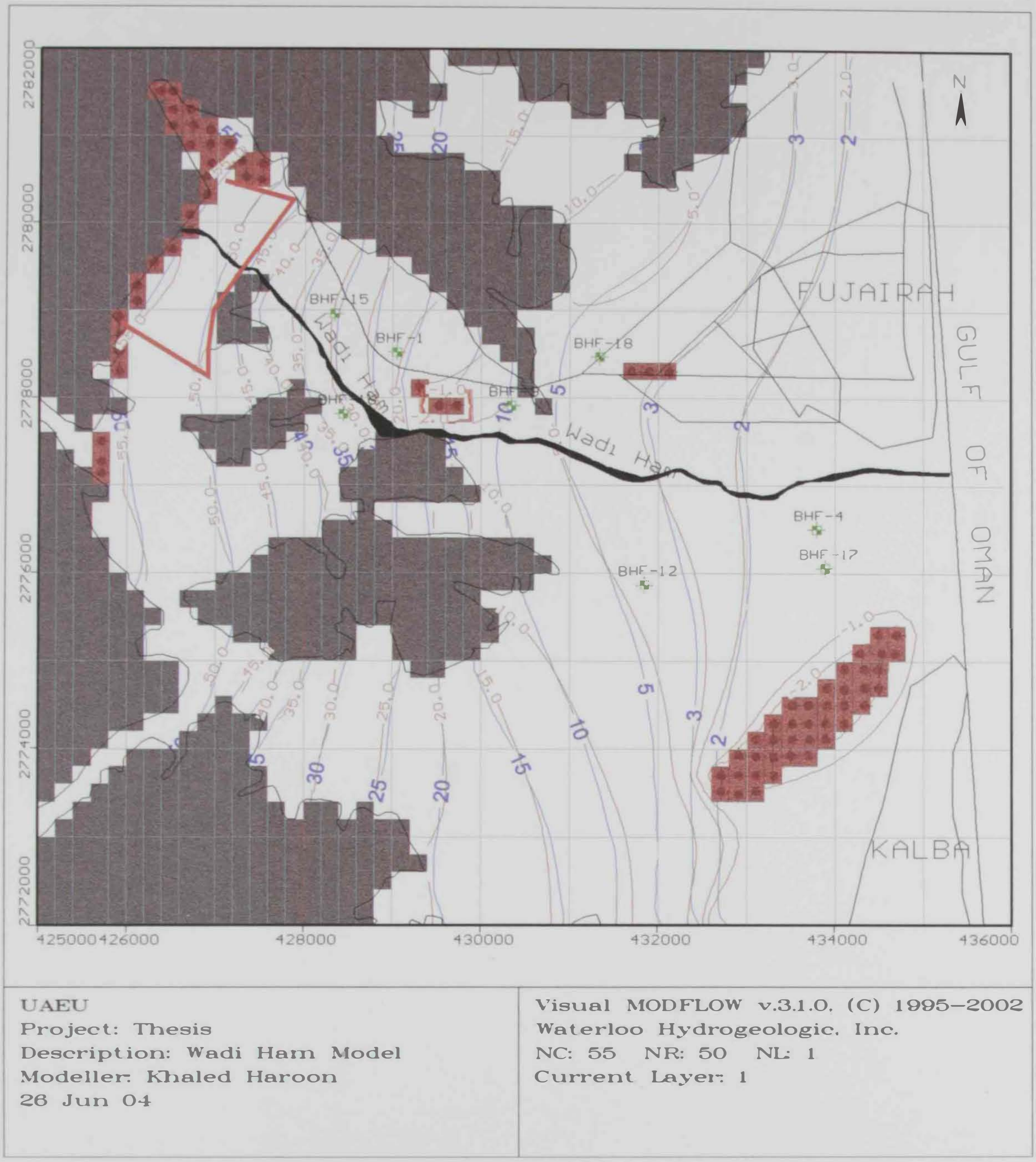

Figure 5.19. Simulated versus observed heads (February, 1990).

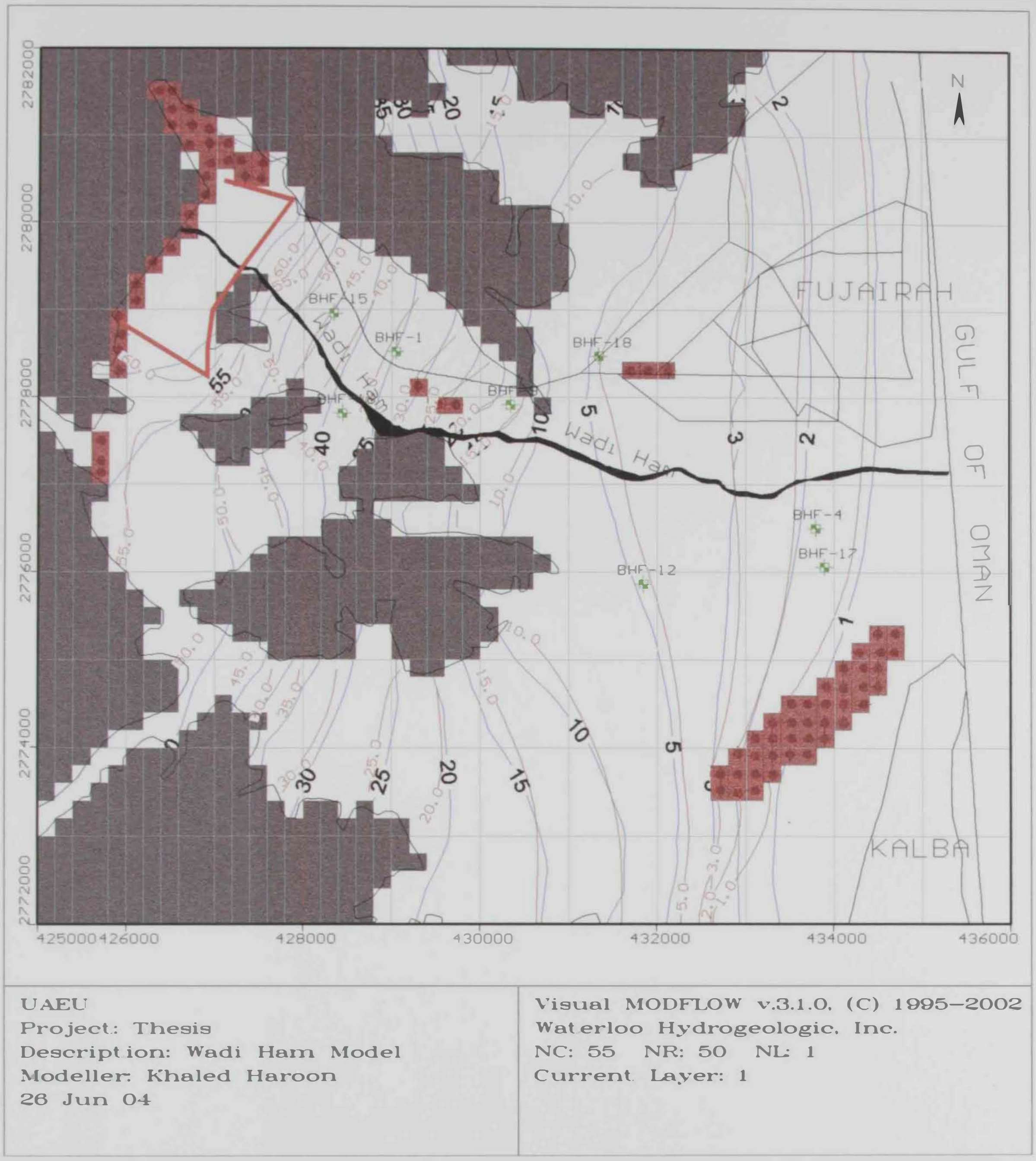

Figure 5.20. Simulated versus observed heads (April, 1990).

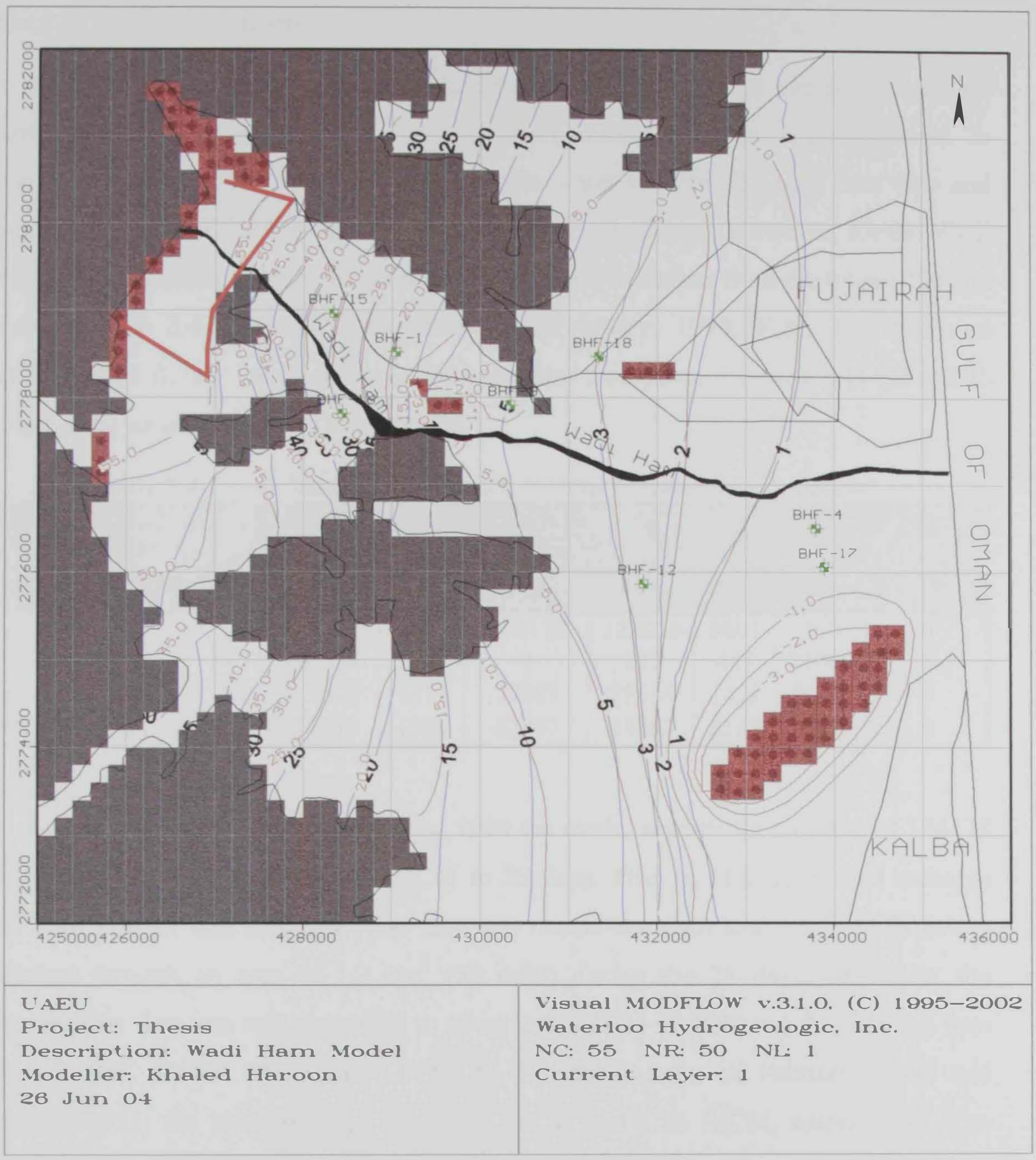

Figure 5.21. Simulated versus observed heads (January, 1991).

Effect of Wadi Ham Dam

The effect of Wadi Ham Dam in recharging the groundwater system can be assessed on quantitative basis by study of the water balance during the transient simulation. In transient simulations, MODFLOW computes the water balance for each time step and saves it to the output file for each time step or for selected steps. However, for the Wadi Ham model, the water balance for selected months was extracted from the output file and listed in Table 5.4. These are the months of February, 1990; February, 1992 and February, 1993 during which recharge from the dam took place in addition to the month of May, 1991 as an example of a dry period.

Table 5.4. Water balance for selected months in transient simulation.

Month	Stress period	IN (m^3/day)				OUT (m^3/day)			
		Storage	Head	Wells	Recharge	Storage	Head	Wells	Recharge
Feb - 90	2	0	873	3990	124178	123050	5807	300	0
May -91	17	5967	11650	3990	0	125	445	21000	0
Feb - 92	26	406	1309	1472	27893	27536	1452	2044	0
Feb - 93	38	780	1020	1500	62867	58467	5520	2100	0

During the major flood of February, 1990 the model accepted a recharge of 124178 m^3/day which is equivalent to 3.48 MCM in 28 days. This is, of course, total recharge from both rainfall and dam site. The recharge rate at the dam site was 0.0738 m/day infiltrated through an area of 1.2 km^2 (30 cells) during the 28 days. Therefore, the recharge from dam site was amounted to about 2.48 MCM ($0.0738 \times 1.2 \times 28$) and thus recharge from rainfall was about 1.0 MCM. For the months of February, 1992 and February, 1993; the total recharge was 0.81 MCM and 1.76 MCM, respectively. The recharge from dam site was 0.54 and 1.18 MCM respectively, provided that recharge rates were 0.0155 and 0.0351 m/day. Noting that the month of February in 1992 was 29 days, therefore, the recharge from rainfall in these two months is 0.27 and 0.58 MCM, respectively.

The recharge from dam site was compared with the recorded flood volumes. The recorded volume in February, 1990 was 4.87 MCM. No records were available for the months of February, 1992 and February, 1993. The HEC-HMS rainfall/runoff model developed previously (Chapter 4), was used to generate flood volumes at dam site for these two months and flood volumes were found 1.69 and 3.27 MCM, respectively.

With these volumes, the recharge from the dam was 51% in February, 1990; 32% in February, 1992 and 36% in February, 1993. The high percentage of recharge in February, 1990 may be attributed to error in the recorded volume. The record was taken on 11^{th}, February while the rainfall continued for further two days and added more water to the storage. The simulation of this flood with HEC-HMS gave a flood volume of 5.76 MCM at the dam site. With this volume the percentage of recharged water is around 43 %.

Another effect of the recharge from dam is the enhancement of groundwater quality by fighting back the seawater intrusion. This effect of recharged water appears clear in the water balance (Table 5.4). The inflow from the sea (Head, IN) is reduced to minimum during flood months. During the dry month of May, 1991; the flow of seawater intrusion was about 11650 m^3/day compared to only 873 m^3/day during the flood month of February, 1990. Water losses to the sea (Head, OUT) also took place with variable rates being more obvious during flood months. However, the amount of water added to storage exceeds by far the water losses.

The flow directions at selected times (before, during the peak and after the major flood) are presented graphically in Figures 5.22 through 5.24. It appears that water losses to sea are taking place in the area to the north of Fujairah and the seawater intrusion occurs most notably in the area of Kalba. The recovery of depression cone in the farms area in response to February, 1990 flood appears clearly in Figure 5.23.

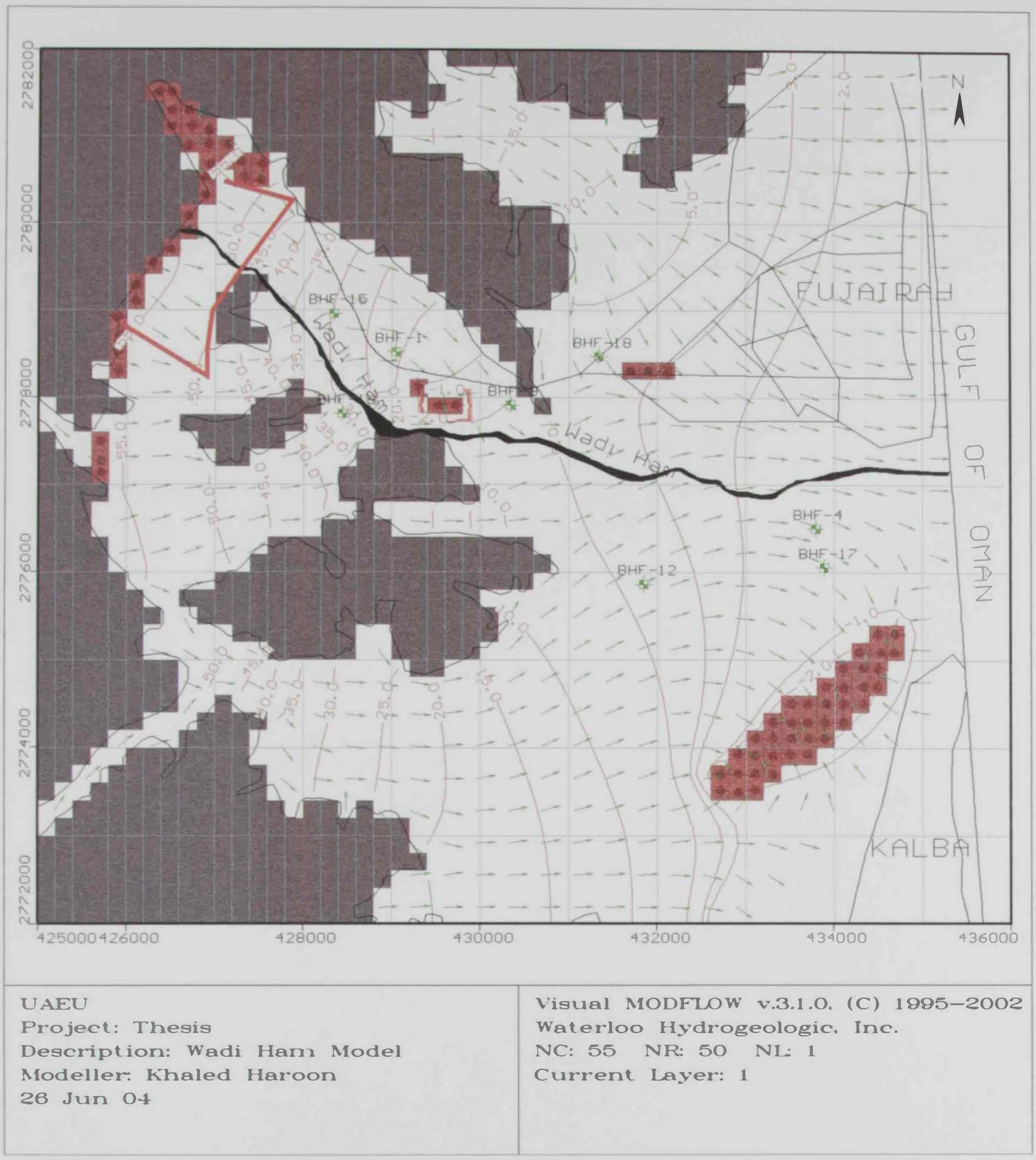

Figure 5.22. Groundwater flow directions (February, 1990).

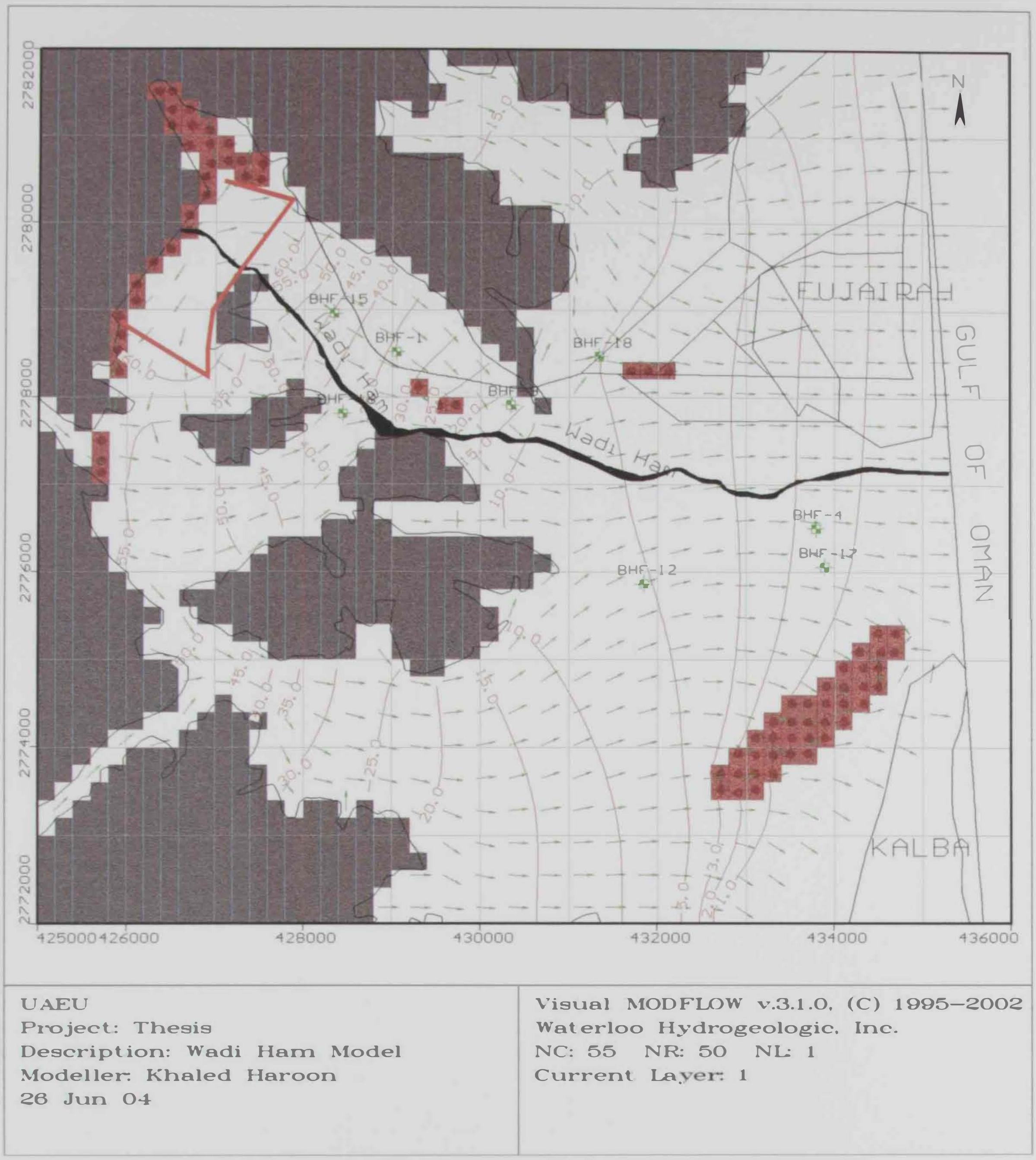

Figure 5.23. Groundwater flow directions (April, 1990).

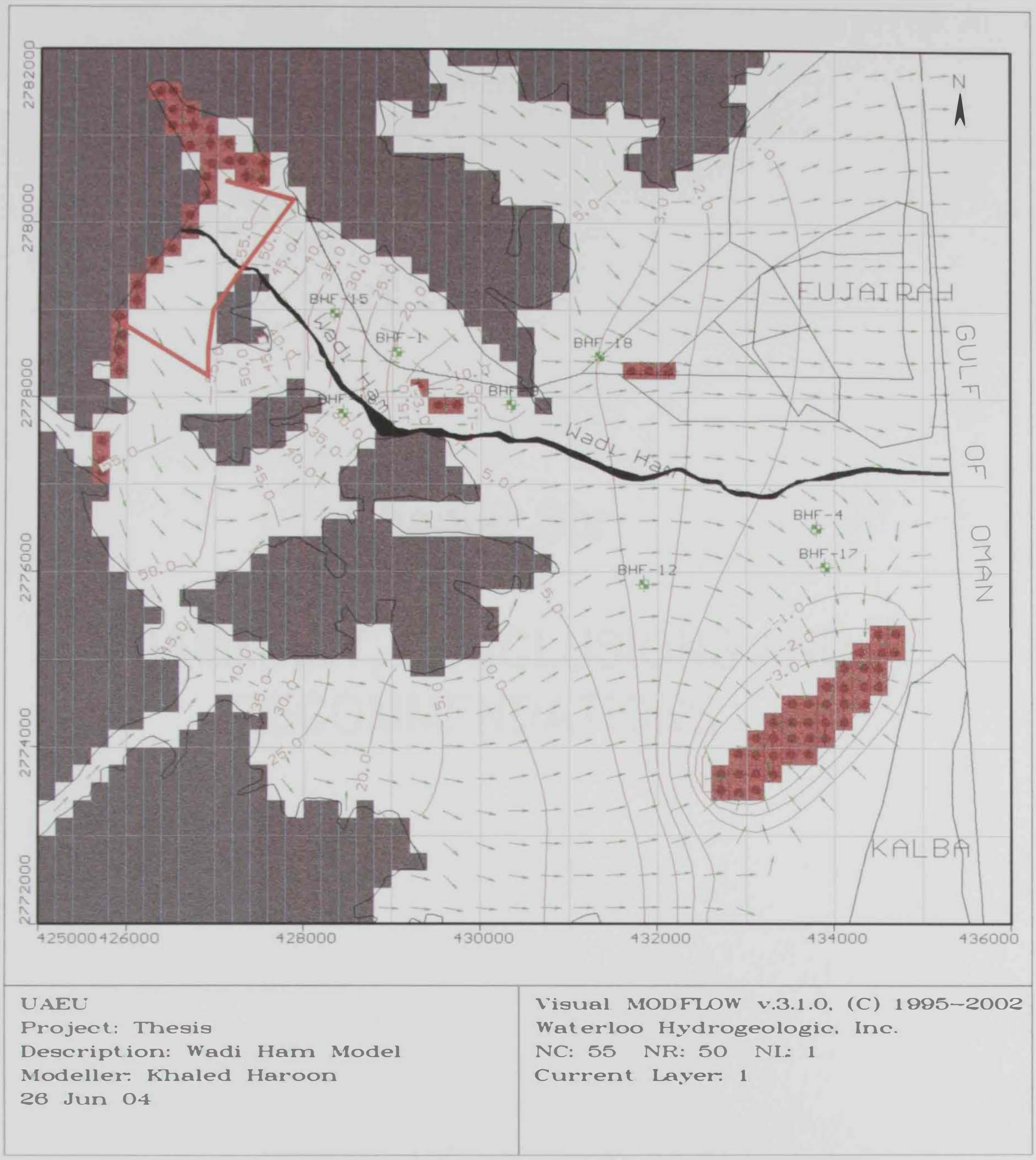

Figure 5.24. Groundwater flow directions (January, 1991).

Chapter Six

SUAMMARY, CONCLUSIONS AND RECOMMENDATIONS

CHAPTER SIX:

SUAMMARY, CONCLUSIONS AND RECOMMENDATIONS

6.1 Summary

In arid and semi-arid regions, surface water resources are scarce and, in most cases, groundwater is the only available natural resource of freshwater. Being the only freshwater resource, groundwater is intensively pumped in quantities that exceed by far the natural recharge. Such exploitation of groundwater often results in a remarkable depletion and quality deterioration. Sustainable management of groundwater is thus a key issue and requires implementation of appropriate technologies to augment groundwater resources.

Arid and semi-arid regions are characterized by infrequent rainfalls which are generally of short durations and high intensities resulting in flash floods which flow as ephemeral streams. Such floods can be harvested and used to recharge groundwater resources. Artificial recharge augments the natural movement of surface water into the underground formations using some means of construction whereby surface water from streams or lakes is made to infiltrate into the ground. Artificial recharge of groundwater has become a common practice in arid regions and is widely used as a conservative technique to save water in times of water surplus for use in times of water shortage.

The UAE is known by its arid conditions and its limited renewable freshwater resources. A long hot summer and short mild winter characterize the climate. Surface water in UAE is very limited and of a little significance in the water budget of the country. Groundwater represents a vital natural resource and although it may not be suitable, in most cases, for drinking and other potable purposes; it represents the main source for irrigation. About 85% of the total water consumption in UAE is groundwater.

The sustainability of groundwater resources is of prime concern for authorities in the UAE and major actions have been undertaken toward the implementation of appropriate rainwater harvesting technologies. A large number of detention and retention dams has been constructed during the last two decades across the main Wadis to harvest the surface water runoff and artificially recharge the groundwater.

The importance of this study evolves from the need to assess surface water runoff in the main wadis and its role in recharging the groundwater which is of vital role in the sustainable development of UAE, specifically, the agricultural development.

This study aims at the quantitative assessment of surface water runoff accumulated at the dam site of Wadi Ham and the associated groundwater recharge. The study provides a methodology that can be followed in other sites of similar hydrological and hydrogeological conditions.

A comprehensive literature review of all previous studies and investigations related to Wadi Ham was carried out. Related information about geology, hydrology and hydrogeology; and historical records of rainfall, surface water flow and groundwater levels were collected from the MAF and other available studies. The data were checked for consistency and unreliable or erratic records were eliminated.

The general location of Wadi Ham and the catchment to the dam were presented on illustration maps. The catchment boundary to the dam and the drainage network were delineated based on remote sensing image and the different tributaries of Wadi Ham were identified. Stream lengths were measured and catchment area was identified.

The geology and climatology of Wadi Ham area were discussed. Rainfall data were analyzed in both time and space. The average areal rainfalls were calculated using the Thiessen polygon method. Representative raingauges were identified and corresponding weights were calculated. Average areal rainfalls were analyzed on quantitative and probabilistic basis using statistical and frequency analyses. The statistical values and distribution characteristics of total annual, mean monthly and one-day annual maximum rainfall were presented. The annual rainfall data were fitted with appropriate distributions and exceedance probabilities and return periods were calculated.

The thesis work included the assessment and modeling of surface water runoff in Wadi Ham for the period 1979 to 1989. The available historical records of surface water flow were analyzed to detect its variation on monthly and annual basis.

Surface water flow data were critically analyzed in conjunction with rainfall data. Based on available information, four flood events were selected for the calibration of the model. A rainfall/runoff model was developed using the HEC-HMS package. The model was conceptualized with five subbasins and the input data were presented. The model was calibrated against historical records and calibrated basin parameters were presented.

Prior to the modeling of groundwater system, the hydrogeology of Wadi Ham area was studied and the existing hydrogeologic layers were identified. Thicknesses of different layers were examined based on lithologic cross sections and hydrogeologic properties were discussed. Preliminary assessment of groundwater recharge was carried out based on historical records of groundwater levels. The variation in groundwater levels was illustrated on water level hydrographs. The water levels at various observation wells were compared and the variation in wells' response to the recharge was discussed.

The aquifer system at Wadi Ham plain area was modeled using the three-dimensional groundwater model, MODFLOW. The model was conceptualized with single layer representing the main sand and gravels aquifer. Based on available information, the period from January, 1990 up to December, 1993 was selected for the calibration of the model. The simulation started with steady-state simulation for the month of January, 1990. The steady-state simulation provided the stable head configuration necessary as initial head condition for the transient simulation. The model accounted for different pumping activities and dealt with recharge as a space and time process. The model was calibrated against historical groundwater levels on space and time basis. A quantitative assessment for the recharge from rainfall and from the dam was made based on the model water balance. The flow directions were studied and the water losses to the sea and the seawater intrusion were assessed. The model was limited to the calibration phase due to the lack of long term data.

6.2 Conclusions

Based on the various discussions and results of this research study, the following conclusions are made:

1- Sustainable development in arid and semi-arid regions requires proper management of groundwater resources through implementation of artificial recharge techniques.

2- The total catchment area to the dam is approximately 195 km^2. This includes Wadi Ham itself and the catchment of Wadi Al-Farfar system. The two wadis flow separately but they join with each other about one kilometer upstream of the dam.

3- The precipitation input to Wadi Ham is solely via rainfall. Rainfall distribution is intermittent and highly scattered. The mean annual rainfall estimated for 23 years is 154 mm with about 50% of it occurs in the winter months of February and March.

4- Surface water flow is variable reflecting the intermittent nature of rainfall. Most flood events are over within a relatively short period of time, with many being over in less than a day. Wadi Al-Farfar system has great contribution to the total runoff and accounts for about 40% of the total runoff yield accumulated at the dam site.

5- The hydrogeology of the Wadi Ham plain area consists of two units, namely, wadi gravels and the Samail Ophiolite. The wadi gravels form the major aquifer system with its thickness varies from about 15 m to 100 m.

6- The Wadi Ham Dam has an effective role in groundwater recharge and its effect is clearly reflected in groundwater levels rise, being more clear in the area immediately downstream of the dam. The recharge from the dam ranged from 32% to 43% of the dam storage. Although water losses to the sea are increased, the amount of water added to storage exceeds by far the water losses. Water losses to the Gulf appear in the area around Fujairah.

7- The recharge from dam enhances groundwater quality by fighting back the seawater intrusion. The seawater intrusion is reduced to low levels during recharge events from the dam. Seawater intrusion appears in the area around Kalba.

6.3 Recommendations

Based on the various discussions and results of this study, the following recommendations are made for future investigations:

1- Historical records of rainfalls, surface water flow and groundwater levels are of great importance for any research. The continuity of this data is highly recommended.

2- Installation of a flow gauge close to dam site will help in measuring the total flows to the dam and provide better estimations of the total runoff volumes. It is also recommended to reinstall the flow gauge at Bithnah.

3- The groundwater model constructed in this thesis is of preliminary nature. A more comprehensive model that considers different pumping scenarios and future predictions is recommended. Such a model, however, requires additional data including new drillings and pumping tests of longer durations to arrive at more accurate values of hydraulic conductivities and storativities.

4- It is recommended to perform field inventory of the pumping wells operating in the area to have better estimations of groundwater extraction.

5- Additional observation wells in the area north of Fujairah are recommended and groundwater level monitoring in BHF-3 and BHF-10 northwest of Fujairah; and BHF-13 and BHF-5 west of Kalba, which were abandoned, should be resumed. It is also recommended to register the measurement day.

6- Regular cleanup of silt sedimentations in the retention area should be made to enhance the infiltration process.

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Appendix (A)
Monthly Rainfall Data

Monthly Rainfall Data

Month/ Year	Monthly Rainfall (mm)			
	Farrah	Bithna	Masafi	Average Aerial*
Oct-1980	0	0	0	0.0
Nov	0	8	10.2	5.9
Dec	0	0	1	0.2
Jan	6.8	19	3.6	11.4
Feb	0	0.6	2.2	0.8
Mar	15.6	18.6	22.6	18.5
Apr	57.8	34	13.2	37.1
May	22.6	24	41.8	27.6
Jun	0	0	0	0.0
Jul	0	0	0	0.0
Aug	0	0	0	0.0
Sep	0	0	0.4	0.1
Oct-1981	0	0	6.6	1.5
Nov	0	0.2	0.4	0.2
Dec	0	0	0.6	0.1
Jan	9.6	11.9	6.8	10.0
Feb	185.8	173.5	168.2	176.3
Mar	112.4	122.2	165.6	129.0
Apr	5	1.6	0	2.4
May	0	0	0	0.0
Jun	0	0	0	0.0
Jul	0	0	0	0.0
Aug	0	0	0	0.0
Sep	0	0	0	0.0
Oct-1982	4	12.8	4	7.9
Nov	42.2	23.8	31.2	31.6
Dec	25.2	30.8	28.6	28.4
Jan	3.4	8	21.6	9.6
Feb	171.6	138.2	48	128.4
Mar	72	57.2	41.9	58.6
Apr	62.6	56.8	35.8	53.9
May	0	0	0	0.0
Jun	0	0	0	0.0
Jul	0	0	0	0.0
Aug	13.2	22.4	38.4	23.1
Sep	0	0	0	0.0
Oct-1983	0	0	0	0.0
Nov	0	0	0.6	0.1
Dec	0.8	2	1.8	1.6
Jan	0.4	1.4	5.2	1.9
Feb	1.2	9.2	0.2	4.5
Mar	14.6	18.8	27.4	19.4
Apr	0	0	0	0.0
May	24.6	39.4	2.6	26.0
Jun	0	0	0	0.0
Jul	0	0.2	36.2	8.4
Aug	1.2	0.4	29.8	7.5
Sep	0.6	4.6	6.6	3.7

* Thiessen average = 0 331× Farah + 0.438 × Bithna + 0.231 × Masafi

Monthly Rainfall Data

Month/ Year	Monthly Rainfall (mm)			
	Farah	Bithna	Masafi	Average Aerial*
Oct-1984	0	0	0	0.0
Nov	0	0	0.2	0.0
Dec	6	4.2	7.8	5.6
Jan	12.4	17	17.6	15.6
Feb	0	0	0	0.0
Mar	1.8	1.6	1.8	1.7
Apr	0	0	0.8	0.2
May	1	0	0	0.3
Jun	0	0	0	0.0
Jul	1.2	0.4	0	0.6
Aug	0	0	0	0.0
Sep	0	0	5.4	1.2
Oct-1985	0	0	0	0.0
Nov	0	6.8	0.2	3.0
Dec	0	0	1.4	0.3
Jan	37	20.8	25	27.1
Feb	16.2	12.6	23.8	16.4
Mar	19.4	23	8.8	18.5
Apr	0.2	0	2	0.5
May	0	0	0	0.0
Jun	0	0	1.8	0.4
Jul	0	0	3.2	0.7
Aug	0	0	7	1.6
Sep	0	0.6	4.4	1.3
Oct-1986	0	1.6	5.2	1.9
Nov	0.2	0	1	0.3
Dec	8.6	6.4	14.8	9.1
Jan	0	0	0.4	0.1
Feb	7.2	7	9.8	7.7
Mar	146.4	125.2	97.8	125.9
Apr	9.4	17.2	18.8	15.0
May	8	0.2	9.8	5.0
Jun	1	6.6	0	3.2
Jul	0	0	0	0.0
Aug	0.6	1.8	0.4	1.1
Sep	0	0	0.8	0.2
Oct-1987	0	0	0	0.0
Nov	16	6.8	17.2	12.2
Dec	1.2	2	5.6	2.6
Jan	7.4	10.4	4.6	8.1
Feb	233.2	185.8	168.9	197.6
Mar	0	0.4	2.6	0.8
Apr	28	34.8	29.5	31.3
May	0	0	0	0.0
Jun	0	0	0	0.0
Jul	4.8	12	14	10.1
Aug	0	0	0	0.0
Sep	0	0	0	0.0

* Thiessen average = 0.331 × Farah + 0.438 × Bithna + 0.231 × Masafi

Monthly Rainfall Data

Month/ Year	Monthly Rainfall (mm)			
	Farah	Bithna	Masafi	Average Aerial*
Oct-1988	0	0	0	0.0
Nov	0	0	0	0.0
Dec	3.4	19.8	2.4	10.4
Jan	0	0	0	0.0
Feb	1.4	7	8.4	5.5
Mar	33.2	30.4	39.6	33.5
Apr	3.2	5	9	5.3
May	0	0	0	0.0
Jun	0	0	0	0.0
Jul	3.8	0.4	5.6	2.7
Aug	0	0	0	0.0
Sep	0	0	0	0.0
Oct-1989	0	0	3.8	0.9
Nov	11.8	15.8	0.4	10.9
Dec	35.2	22.4	71.8	38.0
Jan	5.8	3.8	20.2	8.3
Feb	151.2	147	150.2	149.1
Mar	0.8	0.4	4	1.4
Apr	15.2	31.4	15.6	22.4
May	0	0	0	0.0
Jun	0	8	8	5.4
Jul	0	0	17	3.9
Aug	0.4	0	0	0.1
Sep	0	0	0	0.0
Oct-1990	0	3.4	14.2	4.8
Nov	0.2	2.6	1.4	1.5
Dec	0	0	0	0.0
Jan	2.2	7.6	22	9.1
Feb	46	13.2	6.4	22.5
Mar	22.6	15.8	35.6	22.6
Apr	0	0	0	0.0
May	0	0	0	0.0
Jun	0	0	0	0.0
Jul	0	0	0	0.0
Aug	1.6	0	0	0.5
Sep	0	0	0.2	0.0
Oct-1991	0	0	0.2	0.0
Nov	39.6	43.8	5.2	33.5
Dec	8.8	22.6	5.6	14.1
Jan	13.4	33.2	49.4	30.4
Feb	30.6	44	40.4	38.7
Mar	3.2	3.4	3.4	3.3
Apr	49.6	57.2	38	50.2
May	0	0	0	0.0
Jun	0	0	0	0.0
Jul	0	0.6	0.6	0.4
Aug	0.4	0	0.4	0.2
Sep	0	0	0	0.0

* Thiessen average = 0.331 × Farah + 0.438 × Bithna + 0.231 × Masafi

Monthly Rainfall Data

Month/ Year	Monthly Rainfall (mm)			
	Farah	Bithna	Masafi	Average Aerial*
Oct-1992	8.4	0	0.8	3.0
Nov	0	0.6	0	0.3
Dec	12	13.2	17.4	13.8
Jan	13.2	18.6	21.8	17.6
Feb	57	87.6	108.4	82.3
Mar	2.6	1.8	5.4	2.9
Apr	0	0.2	0	0.1
May	1.2	0	0	0.4
Jun	0	0	0	0.0
Jul	0	0	0	0.0
Aug	0.6	6.6	1	3.3
Sep	10.2	1.4	0.2	4.0
Oct-1993	0	0	14	3.2
Nov	0.2	0	0	0.1
Dec	39.6	90.8	11	55.4
Jan	18.6	16.8	29.6	20.4
Feb	0.8	1.6	2.6	1.6
Mar	10.2	13.4	11.8	12.0
Apr	0	0	1	0.2
May	0	0.6	0	0.3
Jun	0	0	5	1.2
Jul	0	0	0	0.0
Aug	5.6	0.2	0	1.9
Sep	0	0	0	0.0
Oct-1994	0.2	1.6	14.6	4.1
Nov	0.2	0	0.4	0.2
Dec	0	0	0	0.0
Jan	1.8	0.6	0	0.9
Feb	6	12.6	35	15.6
Mar	35	186.8	100.8	116.7
Apr	1.2	0.8	5	1.9
May	5.8	4.4	19.2	8.3
Jun	0	0	0	0.0
Jul	63.8	68	91.6	72.1
Aug	0	0	0	0.0
Sep	0	0	0	0.0
Oct-1995	2.4	2.2	19.8	6.3
Nov	4.2	0.2	0	1.5
Dec	172.8	192	159	178.0
Jan	120.4	122	75.4	110.7
Feb	152.2	34.8	48.8	76.9
Mar	20.8	135.8	169.2	105.5
Apr	1	0.4	0	0.5
May	0	0	0	0.0
Jun	0.8	16.4	3	8.1
Jul	0.4	5.2	17.8	6.5
Aug	0	0	0	0.0
Sep	6	0	13.8	5.2

* Thiessen average = 0.331 × Farah + 0.438 × Bithna + 0.231 × Masafi

Monthly Rainfall Data

Month/ Year	Monthly Rainfall (mm)			
	Farah	Bithna	Masafi	Average Aerial*
Oct-1996	0	0	1.4	0.3
Nov	0.6	1.8	7.2	2.7
Dec	2.4	2.8	4.2	3.0
Jan	57.4	58	61.6	58.6
Feb	0	0	0.2	0.0
Mar	122.2	123.6	133.9	125.5
Apr	29	16	5.8	17.9
May	0.2	0	0.8	0.3
Jun	0	0	23	5.3
Jul	0	0	0	0.0
Aug	0	0	0	0.0
Sep	0	0	6.5	1.5
Oct-1997	30	96.2+	96.2	74.3
Nov	27.8	27+	27	27.3
Dec	1.4	5.6+	5.6	4.2
Jan	48.4	80.4+	80.4	69.8
Feb	24.8	38.2+	38.2	33.8
Mar	3.2	20+	20	14.4
Apr	13	3.2+	3.2	6.4
May	0	0+	0	0.0
Jun	0	0.8+	0.8	0.5
Jul	0	9.4+	9.4	6.3
Aug	0	5.8+	5.8	3.9
Sep	0	6.4+	6.4	4.3
Oct-1998	0	3+	3	2.0
Nov	0	0+	0	0.0
Dec	5.2	4.4+	4.4	4.7
Jan	29.2	20.8+	20.8	23.6
Feb	9.4	11.6+	11.6	10.9
Mar	9.6	18.8+	18.8	15.8
Apr	0	0+	0	0.0
May	0	0+	0	0.0
Jun	0	0+	0	0.0
Jul	0	2.8+	2.8	1.9
Aug	0	9.8+	9.8	6.6
Sep	0	30.2+	30.2	20.2
Oct-1999	0	0+	0	0.0
Nov	1.2	0+	0	0.4
Dec	0	1.8+	1.8	1.2
Jan	0	0.2+	0.2	0.1
Feb	0	0.2+	0.2	0.1
Mar	0	0+	0	0.0
Apr	0	0+	0	0.0
May	0	0.2+	0.2	0.1
Jun	0	0+	0	0.0
Jul	0	3.2+	3.2	2.1
Aug	0	5.2+	5.2	3.5
Sep	0	0+	0	0.0

* Thiessen average = 0.331 × Farah + 0.438 × Bithna + 0.231 × Masafi

\+ Missing record, assumed equal to Masafi record

Monthly Rainfall Data

Month/ Year	Monthly Rainfall (mm)			
	Farah	Bithna	Masafi	Average Aerial*
Oct-2000	3.4+	3.4+	3.4	3.4
Nov	13.6+	13.6+	13.6	13.6
Dec	12.4+	12.4+	12.4	12.4
Jan	22+	22+	22	22.0
Feb	0.4+	0.4+	0.4	0.4
Mar	0.6+	0.6+	0.6	0.6
Apr	0+	0+	0	0.0
May	0+	0+	0	0.0
Jun	0+	0+	0	0.0
Jul	1.2+	1.2+	1.2	1.2
Aug	0+	0+	0	0.0
Sep	16.8+	16.8+	16.8	16.8
Oct-2001	0	0	1.8	0.4
Nov	0	0	0	0.0
Dec	0	0.4	0.8	0.4
Jan	7	10.4	3	7.6
Feb	1.4	0	0.4	0.6
Mar	13.2	14.2	30.6	17.7
Apr	1.8	5.2	1.4	3.2
May	0	0	0	0.0
Jun	1.4	2.4	2.6	2.1
Jul	0.6	0	0	0.2
Aug	0	0	0	0.0
Sep	0	0	0	0.0
Oct-2002	8.6	28.4	19	19.7
Nov	27.4	0.8	0.8	9.6
Dec	3	0.8	5.4	2.6
Jan	1.2	2.6	5	2.7
Feb	3.6	0	7.4	2.9
Mar	2.4	0	8.6	2.8
Apr	4.4	0.2	30.8	8.7
May	0	0	0	0.0
Jun	0	0	0	0.0
Jul	0	0	0	0.0
Aug	0	0	0	0.0
Sep	0	0	0	0.0

* Thiessen average = 0.331 × Farah + 0.438 × Bithna + 0.231 × Masafi
\+ Missing record, assumed equal to Masafi record

Appendix (B)
Bithna Flow Gauge Data

Bithna flow gauge daily discharge readings 1979 to 1990

Date	Total Daily Flow (1000 m^3/d)	Date	Total Daily Flow (1000 m^3/d)
12/03/79	38.12	02/04/82	49.16
13/03/79	330.60	03/04/82	45.71
30/10/79	146.22	04/04/82	39.31
30/12/79	177.19	05/04/82	31.80
30/04/81	2.68	06/04/82	22.98
01/05/81	3.82	07/04/82	18.66
02/05/81	52.72	08/04/82	18.66
13/02/82	0.30	09/04/82	18.66
14/02/82	3622.90	10/04/82	18.23
15/02/82	1296.00	11/04/82	17.28
16/02/82	208.30	12/04/82	16.68
17/02/82	103.70	13/04/82	16.68
18/02/82	77,80	14/04/82	16.68
19/02/82	27.65	15/04/82	16.68
20/02/82	2.77	16/04/82	15.38
21/02/82	2.07	17/04/82	14.08
22/02/82	2.07	18/04/82	14.08
23/02/82	1.99	19/04/82	10.54
24/02/82	1.40	20/04/82	6.91
25/02/82	1158.50	21/04/82	6.91
26/02/82	18.10	22/04/82	6.91
27/02/82	3.10	23/04/82	5.53
28/02/82	2.02	24/04/82	4.15
01/03/82	2.07	25/04/82	4.15
02/03/82	2.07	26/04/82	4.15
03/03/82	2.07	27/04/82	4.15
04/03/82	1.90	28/04/82	4.15
05/03/82	1.73	29/04/82	3.63
06/03/82	1.21	30/04/82	2.59
07/03/82	0.17	01/05/82	2.07
13/03/82	11.20	02/05/82	1.90
14/03/82	121.40	03/05/82	1.21
15/03/82	17.80	04/05/82	0.69
16/03/82	15.40	02/11/82	40.15
17/03/82	14.00	14/11/82	6.08
18/03/82	14.00	12/02/83	1078.39
19/03/82	14.00	13/02/83	318.60
20/03/82	9.10	14/02/83	36.29
21/03/82	4.15	15/02/83	28.51
22/03/82	4.15	16/02/83	21.60
23/03/82	4.15	17/02/83	14.69
24/03/82	4.15	18/02/83	11.23
25/03/82	3.63	19/02/83	10.80
26/03/82	2.60	20/02/83	10.37
27/03/82	2.10	21/02/83	10.37
28/03/82	1928.10	22/02/83	9.07
29/03/82	3069.50	23/02/83	6.91
30/03/82	723.80	24/02/83	6.05
31/03/82	51.54	25/02/83	6.05
01/04/82	49.16	26/02/83	5.70

Bithna flow gauge daily discharge readings 1979 to 1990

Date	Total Daily Flow (1000 m^3/d)	Date	Total Daily Flow (1000 m^3/d)
27/02/83	5.70	28/04/83	1.47
28/02/83	4.32	29/04/83	1.47
01/03/83	2.94	30/04/83	0.69
02/03/83	2.94	15/03/84*	80.00
03/03/83	2.94	28/03/87	307.31
04/03/83	2.94	06/04/87	270.97
05/03/83	2.51	01/06/87	55.90
06/03/83	2.07	17/02/88	5355.72
07/03/83	2.07	18/02/88	1548.80
08/03/83	2.07	19/02/88	12.96
09/03/83	2.07	20/02/88	9.50
10/03/83	2.07	21/02/88	6.48
11/03/83	1.73	22/02/88	2.16
12/03/83	1.47	27/04/88	192.38
13/03/83	2.17	08/12/88	250.00
14/03/83	1.47	18/03/89	280.00
15/03/83	1.47	10/02/90	213.05
16/03/83	1.47	11/02/90	1543.95
17/03/83	1.47		
18/03/83	1.47		
19/03/83	0.69		
30/03/83	294.55		
31/03/83	64.68		
01/04/83	7.69		
02/04/83	6.05		
03/04/83	5.88		
04/04/83	5.70		
05/04/83	349.41		
06/04/83	544.32		
07/04/83	296.35		
08/04/83	111.46		
09/04/83	34.13		
10/04/83	27.91		
11/04/83	60.69		
12/04/83	44.76		
13/04/83	36.12		
14/04/83	28.34		
15/04/83	21.00		
16/04/83	14.60		
17/04/83	10.54		
18/04/83	8.73		
19/04/83	6.74		
20/04/83	5.88		
21/04/83	5.70		
22/04/83	4.32		
23/04/83	2.51		
24/04/83	2.07		
25/04/83	2.07		
26/04/83	2.07		
27/04/83	1.73		

* Exact date unknown

Bithna flow gauge monthly discharge readings 1979 to 1990

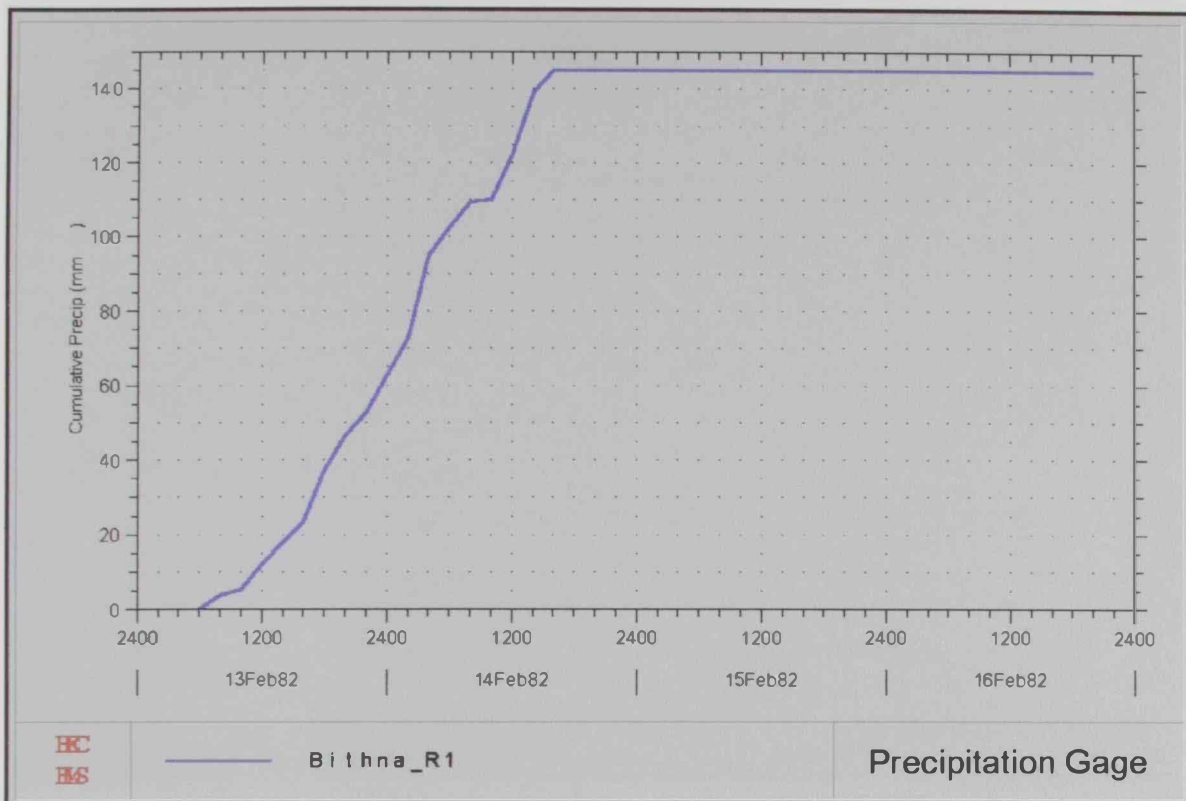
Year	Month	Total Monthly Discharge (1000 m^3/d)	Year	Month	Total Monthly Discharge (1000 m^3/d)
1979	Jan	-	1983	Jan	0.00
	Feb	-		Feb	1548.65
	Mar	-		Mar	397.26
	Apr	-		Apr	1650.62
	May	-		May	0.00
	June	-		June	0.00
	July	-		July	0.00
	Aug	-		Aug	0.00
	Sep	-		Sep	0.00
	Oct	146.22		Oct	0.00
	Nov	0.00		Nov	0.00
	Dec	177.19		Dec	0.00
1980	Jan	0.00	1984	Jan	0.00
	Feb	0.00		Feb	0.00
	Mar	0.00		Mar	80.00
	Apr	0.00		Apr	0.00
	May	0.00		May	0.00
	June	0.00		June	0.00
	July	0.00		July	0.00
	Aug	0.00		Aug	0.00
	Sep	0.00		Sep	0.00
	Oct	0.00		Oct	0.00
	Nov	0.00		Nov	0.00
	Dec	0.00		Dec	0.00
1981	Jan	0.00	1985	Jan	0.00
	Feb	0.00		Feb	0.00
	Mar	0.00		Mar	0.00
	Apr	2.68		Apr	0.00
	May	56.54		May	0.00
	June	0.00		June	0.00
	July	0.00		July	0.00
	Aug	0.00		Aug	0.00
	Sep	0.00		Sep	0.00
	Oct	0.00		Oct	0.00
	Nov	0.00		Nov	0.00
	Dec	0.00		Dec	0.00
1982	Jan	0.00	1986	Jan	0.00
	Feb	6528.67		Feb	0.00
	Mar	6025.59		Mar	0.00
	Apr	503.64		Apr	0.00
	May	5.87		May	0.00
	June	0.00		June	0.00
	July	0.00		July	0.00
	Aug	0.00		Aug	0.00
	Sep	0.00		Sep	0.00
	Oct	0.00		Oct	0.00
	Nov	46.23		Nov	0.00
	Dec	0.00		Dec	0.00

Bithna flow gauge monthly discharge readings 1979 to 1990

Year	Month	Total Monthly Discharge (1000 m^3/d)	Year	Month	Total Monthly Discharge (1000 m^3/d)
1987	Jan	0.00	1989	Jan	0.00
	Feb	0.00		Feb	0.00
	Mar	307.31		Mar	280.00
	Apr	270.97		Apr	0.00
	May	0.00		May	0.00
	June	55.90		June	0.00
	July	0.00		July	0.00
	Aug	0.00		Aug	0.00
	Sep	0.00		Sep	0.00
	Oct	0.00		Oct	0.00
	Nov	0.00		Nov	0.00
	Dec	0.00		Dec	0.00
1988	Jan	0.00	1990	Jan	0.00
	Feb	6935.63		Feb	1757.00
	Mar	0.00		Mar	Flow Gauge
	Apr	192.38		Apr	Destroyed in Feb 1990
	May	0.00		May	
	June	0.00		June	
	July	0.00		July	
	Aug	0.00		Aug	
	Sep	0.00		Sep	
	Oct	0.00		Oct	
	Nov	0.00		Nov	
	Dec	250.00		Dec	

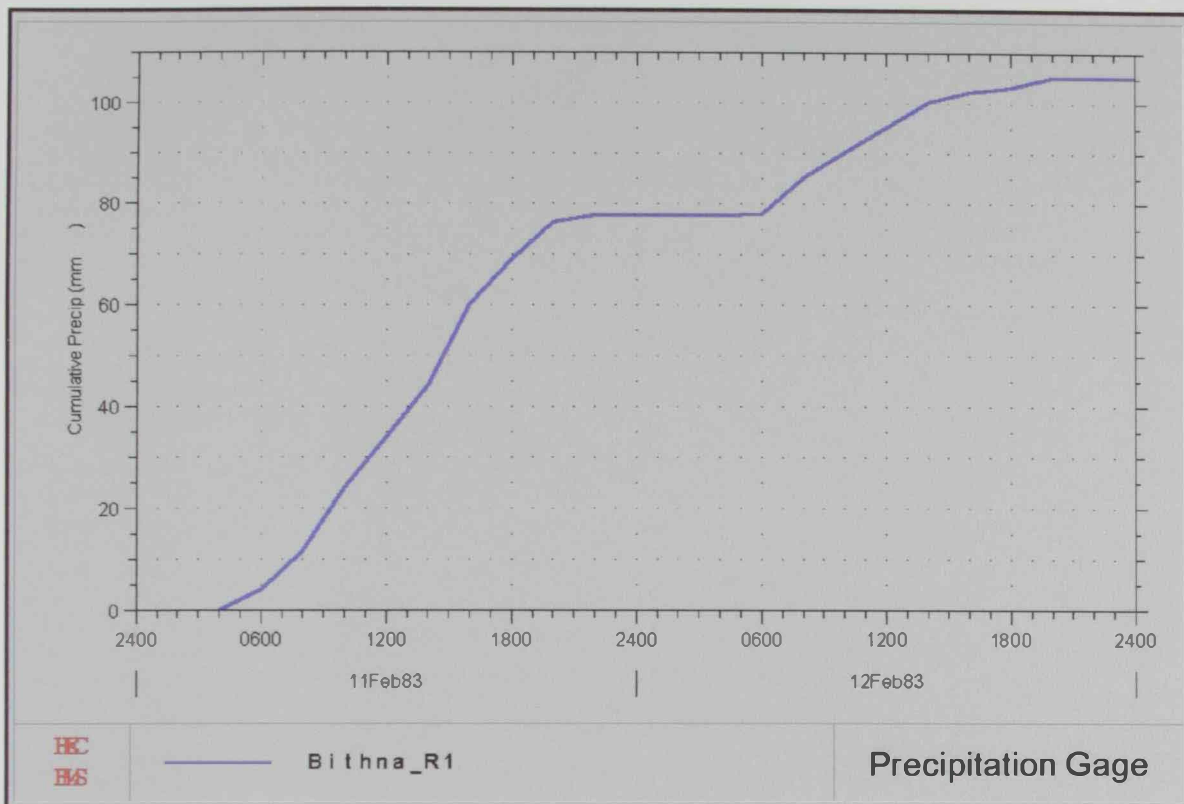
Appendix (C)
Rainfall Patterns for Simulated Storms

Rainfall pattern for storm on 13.02.1982



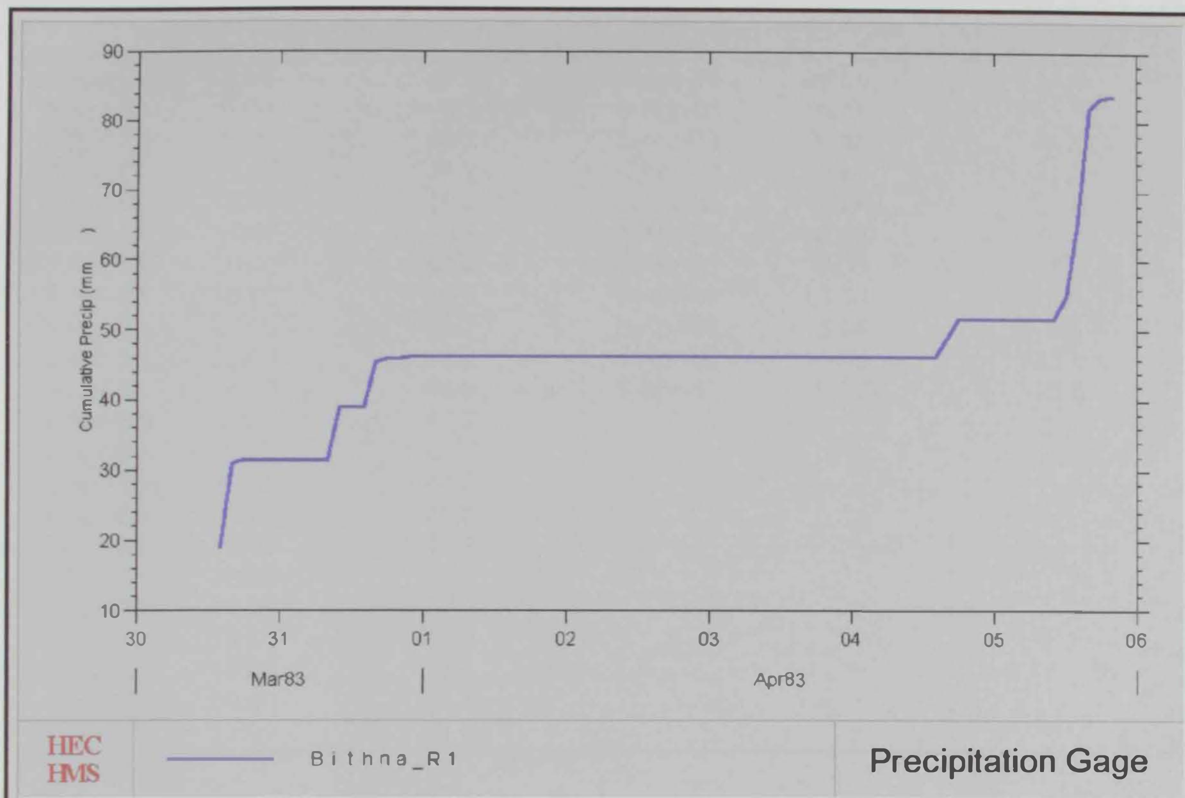
Date	Time	Cumulative Precipitation (mm)	Date	Time	Cumulative Precipitation (mm)
13-Feb-82	6:00	0	15-Feb-82	2:00	145
13-Feb-82	8:00	3.8	15-Feb-82	4:00	145
13-Feb-82	10:00	5.4	15-Feb-82	6:00	145
13-Feb-82	12:00	12.2	15-Feb-82	8:00	145
13-Feb-82	14:00	18.2	15-Feb-82	10:00	145
13-Feb-82	16:00	23.2	15-Feb-82	12:00	145
13-Feb-82	18:00	37.2	15-Feb-82	14:00	145
13-Feb-82	20:00	46.6	15-Feb-82	16:00	145
13-Feb-82	22:00	52.6	15-Feb-82	18:00	145
13-Feb-82	24:00	63	15-Feb-82	20:00	145
14-Feb-82	2:00	73	15-Feb-82	22:00	145
14-Feb-82	4:00	95	15-Feb-82	24:00	145
14-Feb-82	6:00	102.5	16-Feb-82	2:00	145
14-Feb-82	8:00	109.4	16-Feb-82	4:00	145
14-Feb-82	10:00	110	16-Feb-82	6:00	145
14-Feb-82	12:00	122.6	16-Feb-82	8:00	145
14-Feb-82	14:00	139.1	16-Feb-82	10:00	145
14-Feb-82	16:00	145	16-Feb-82	12:00	145
14-Feb-82	18:00	145	16-Feb-82	14:00	145
14-Feb-82	20:00	145	16-Feb-82	16:00	145
14-Feb-82	22:00	145	16-Feb-82	18:00	145
14-Feb-82	24:00	145	16-Feb-82	20:00	145

Rainfall pattern for storm on 11.02.1983



Date	Time	Cumulative Precipitation (mm)	Date	Time	Cumulative Precipitation (mm)
11-Feb-83	4:00	0	12-Feb-83	24:00	105.3
11-Feb-83	6:00	4.2			
11-Feb-83	8:00	11.6			
11-Feb-83	10:00	24.2			
11-Feb-83	12:00	34.2			
11-Feb-83	14:00	44.2			
11-Feb-83	16:00	60.2			
11-Feb-83	18:00	69.2			
11-Feb-83	20:00	76.5			
11-Feb-83	22:00	77.7			
11-Feb-83	24:00	77.7			
12-Feb-83	2:00	77.7			
12-Feb-83	4:00	77.7			
12-Feb-83	6:00	77.9			
12-Feb-83	8:00	85.3			
12-Feb-83	10:00	90.3			
12-Feb-83	12:00	95.3			
12-Feb-83	14:00	100.3			
12-Feb-83	16:00	102.3			
12-Feb-83	18:00	103.3			
12-Feb-83	20:00	105.3			
12-Feb-83	22:00	105.3			

Rainfall pattern for storm on 30.03.1983

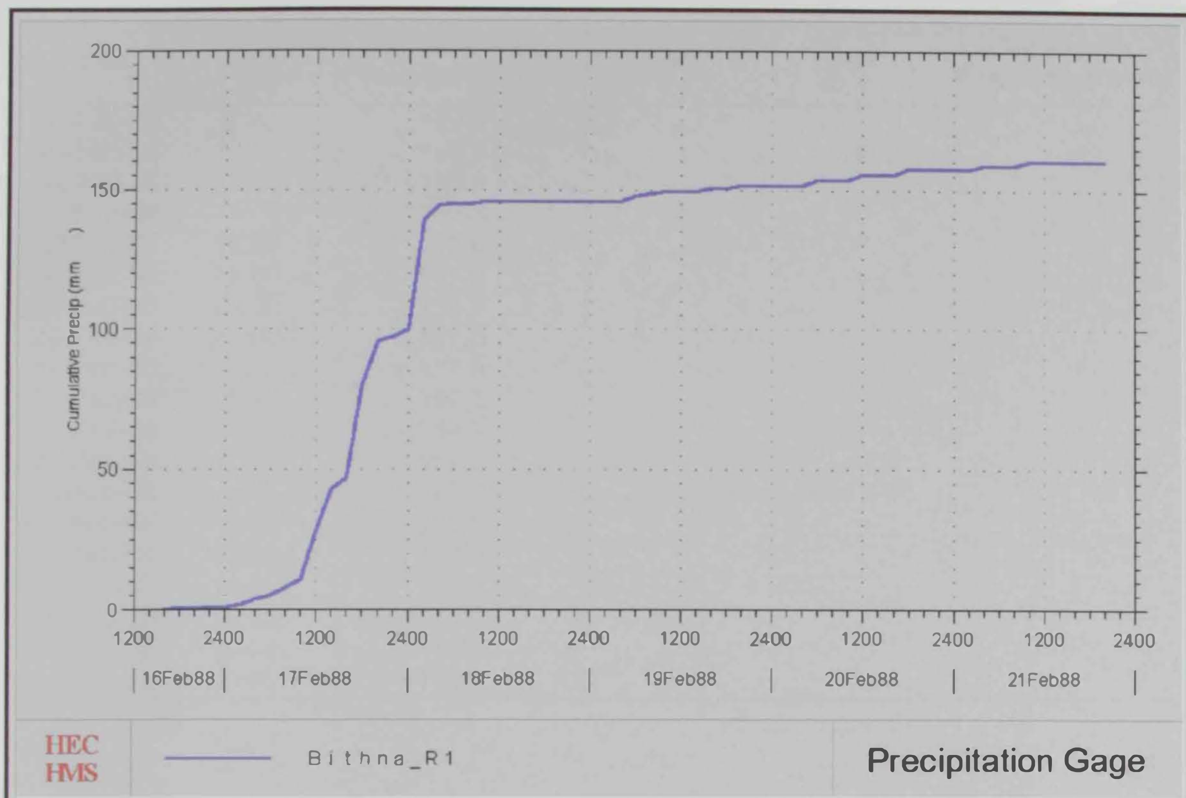


Date	Time	Cumulative Precipitation (mm)	Date	Time	Cumulative Precipitation (mm)
30-Mar-83	14:00	19	1-Apr-83	10:00	46.2
30-Mar-83	16:00	31	1-Apr-83	12:00	46.2
30-Mar-83	18:00	31.4	1-Apr-83	14:00	46.2
30-Mar-83	20:00	31.4	1-Apr-83	16:00	46.2
30-Mar-83	22:00	31.4	1-Apr-83	18:00	46.2
30-Mar-83	0:00	31.4	1-Apr-83	20:00	46.2
31-Mar-83	2:00	31.4	1-Apr-83	22:00	46.2
31-Mar-83	4:00	31.4	1-Apr-83	0:00	46.2
31-Mar-83	6:00	31.4	2-Apr-83	2:00	46.2
31-Mar-83	8:00	31.4	2-Apr-83	4:00	46.2
31-Mar-83	10:00	39	2-Apr-83	6:00	46.2
31-Mar-83	12:00	39	2-Apr-83	8:00	46.2
31-Mar-83	14:00	39	2-Apr-83	10:00	46.2
31-Mar-83	16:00	45.4	2-Apr-83	12:00	46.2
31-Mar-83	18:00	45.9	2-Apr-83	14:00	46.2
31-Mar-83	20:00	46	2-Apr-83	16:00	46.2
31-Mar-83	22:00	46.2	30-Mar-83	14:00	19
31-Mar-83	0:00	46.2	2-Apr-83	18:00	46.2
1-Apr-83	2:00	46.2	2-Apr-83	20:00	46.2
1-Apr-83	4:00	46.2	2-Apr-83	22:00	46.2
1-Apr-83	6:00	46.2	2-Apr-83	0:00	46.2
1-Apr-83	8:00	46.2	3-Apr-83	2:00	46.2

Rainfall pattern for storm on 30.03.1983...continued

Date	Time	Cumulative Precipitation (mm)	Date	Time	Cumulative Precipitation (mm)
3-Apr-83	4:00	46.2	4-Apr-83	24:00	51.6
3-Apr-83	6:00	46.2	5-Apr-83	2:00	51.6
3-Apr-83	8:00	46.2	5-Apr-83	4:00	51.6
3-Apr-83	10:00	46.2	5-Apr-83	6:00	51.6
3-Apr-83	12:00	46.2	5-Apr-83	8:00	51.6
3-Apr-83	14:00	46.2	5-Apr-83	10:00	51.6
3-Apr-83	16:00	46.2	5-Apr-83	12:00	55
3-Apr-83	18:00	46.2	5-Apr-83	14:00	66
3-Apr-83	20:00	46.2	5-Apr-83	16:00	82
3-Apr-83	22:00	46.2	5-Apr-83	18:00	83.6
3-Apr-83	24:00	46.2	5-Apr-83	20:00	83.8
4-Apr-83	2:00	46.2			
4-Apr-83	4:00	46.2			
4-Apr-83	6:00	46.2			
4-Apr-83	8:00	46.2			
4-Apr-83	10:00	46.2			
4-Apr-83	12:00	46.2			
4-Apr-83	14:00	46.2			
4-Apr-83	16:00	48.8			
4-Apr-83	18:00	51.6			
4-Apr-83	20:00	51.6			
4-Apr-83	22:00	51.6			

Rainfall pattern for storm on 16.02.1988



Date	Time	Cumulative Precipitation (mm)	Date	Time	Cumulative Precipitation (mm)
16-Feb-88	16:00	0	18-Feb-88	12:00	145.6
16-Feb-88	18:00	0.6	18-Feb-88	14:00	145.6
16-Feb-88	20:00	0.6	18-Feb-88	16:00	145.6
16-Feb-88	22:00	0.8	18-Feb-88	18:00	145.6
16-Feb-88	0:00	1	18-Feb-88	20:00	145.6
17-Feb-88	2:00	1.8	18-Feb-88	22:00	145.6
17-Feb-88	4:00	3.8	18-Feb-88	0:00	145.6
17-Feb-88	6:00	5	19-Feb-88	2:00	145.6
17-Feb-88	8:00	8	19-Feb-88	4:00	145.6
17-Feb-88	10:00	11	19-Feb-88	6:00	147.6
17-Feb-88	12:00	28	19-Feb-88	8:00	148.6
17-Feb-88	14:00	43	19-Feb-88	10:00	149.6
17-Feb-88	16:00	47	19-Feb-88	12:00	149.6
17-Feb-88	18:00	80	19-Feb-88	14:00	149.6
17-Feb-88	20:00	96	19-Feb-88	16:00	150.6
17-Feb-88	22:00	97.4	19-Feb-88	18:00	150.6
17-Feb-88	0:00	100	19-Feb-88	20:00	151.6
18-Feb-88	2:00	139.2	19-Feb-88	22:00	151.6
18-Feb-88	4:00	144.6	19-Feb-88	0:00	151.6
18-Feb-88	6:00	145	20-Feb-88	2:00	151.6
18-Feb-88	8:00	145	20-Feb-88	4:00	151.6
18-Feb-88	10:00	145.6	20-Feb-88	6:00	153.6

Rainfall pattern for storm on 16.02.1988...continued

Date	Time	Cumulative Precipitation (mm)	Date	Time	Cumulative Precipitation (mm)
20-Feb-88	8:00	153.6			
20-Feb-88	10:00	153.6			
20-Feb-88	12:00	155.6			
20-Feb-88	14:00	155.6			
20-Feb-88	16:00	155.6			
20-Feb-88	18:00	157.6			
20-Feb-88	20:00	157.6			
20-Feb-88	22:00	157.6			
20-Feb-88	0:00	157.6			
21-Feb-88	2:00	157.6			
21-Feb-88	4:00	158.9			
21-Feb-88	6:00	158.9			
21-Feb-88	8:00	158.9			
21-Feb-88	10:00	160.4			
21-Feb-88	12:00	160.4			
21-Feb-88	14:00	160.4			
21-Feb-88	16:00	160.4			
21-Feb-88	18:00	160.4			
21-Feb-88	20:00	160.4			

ملخص الرسالة

تعاني المناطق الجافة عادة من ندرة المياه السطحية حيث تشكل المياه الجوفية في معظم الأحيان المصدر الوحيد للمياه العذبة في تلك المناطق مما يؤدي إلى ضخ المياه الجوفية بكميات كبيرة تفوق كثيرا التغذية الطبيعية للخزانات الجوفية. وهذا الإستنزاف المفرط يؤدي بدوره إلى إنخفاض كبير في المخزون الجوفي أو نضوبه تماما كما يؤدي إلى تدني نوعية المياه الجوفية و ارتفاع ملوحتها. لذلك فإن الحفاظ على مصادر المياه الجوفية وتنميتها تمثل قضية محورية في هذه المناطق. و تتطلب الإدارة المستدامة لهذه المصادر العديد من الإجراءات الإ رية و التنظيمية و منها استخدام التغذية الاصطناعية من أجل زيادة مستويات المياه الجوفية و تحسين نوعيتها.

و تعاني دولة الامارات العربية المتحدة – كغيرها من المناطق الجافة– من نقص موارد المياه المتجددة، فالمياه السطحية نادرة جدا و ذات أهمية محدودة في الموازنة المائية للدولة. و على العكس من ذلك تمثل المياه الجوفية مصدرا حيويا للمياه، فبالرغم من التوسع المستمر في بناء محطات التحلية إلا أن ذلك لا يمثل حلا بعيد المدى للمشكلة وما زالت المياه الجوفية بالرغم من تدني نوعيتها تمثل المصدر الرئيس لمياه الري حيث تقدر نسبة استهلاكها بحوالي ٨٥٪ من الاستهلاك الإجمالي للمياه في الدولة. لذلك فإن الجهات المسئولة في الدولة تولي اهتماما كبيرا للحفاظ على مصادر المياه الجوفية و تنميتها. و من أجل ذلك تم إنشاء العديد من السدود على مخارج الوديان الرئيسة خلال العقدين الماضيين بهدف تجميع مياه السيول و تغذية المياه الجوفية.

و تنبع أهمية هذه الدراسة من الحاجة إلى تقييم جريان المياه السطحية في الوديان و دورها في تغذية المياه الجوفية، و تهدف الدراسة إلى عمل تقييم كمي لمقدار المياه المتجمعة في موقع السد في وادي حام بدولة الإمارات العربية المتحدة و مقدار تغذية المياه الجوفية الناتجة عن ذلك باستخدام نماذج رياضية لمحاكاة جريان المياه السطحية والجوفية. ونظرا لعدم توفر بيانات متصلة و كافية فقد اقتصرت معايرة نموذج المياه السطحية على البيانات المتوفرة في الفترة من عام ١٩٧٩ إلى عام ١٩٨٩م كما اقتصرت معايرة نموذج المياه الجوفية على البيانات المتوفرة في الفترة من بداية عام ١٩٩٠ وحتى نهاية عام ١٩٩٣م. وقد تم توفير كافة البيانات الخاصة بهذه الدراسة من خلال مشروع (تقييم فاعلية سدود البيج – الطويين – حام في تغذية المياه الجوفية باستخدام النماذج الرياضية) والذي تم تمويله من قبل وزارة الزراعة والثروة السمكية.

وقد استعملت الخرائط الطبوغرافية و صور الأقمار الاصطناعية من أجل تحديد حدود الوادي و تخطيط شبكة التصريف و تحديد الروافد الرئيسة في الوادي و قياس أطوالها و تم عمل تحليل شامل للتغيرات في كميات الأمطار و الجريان و مناسيب المياه الجوفية اعتمادا على البيانات المسجلة في محطات الرصد و في آبار المراقبة و تم دراسة الأوضاع الجيولوجية والهيدروجيولوجية لمنطقة الدراسة. و قد تضمنت الدراسة استخدام نموذج رياضي لدراسة الجريان السطحي و عمل تقييم لكميات الجريان الكلية كما تضمنت استخدام نموذج رياضي ثلاثي الأبعاد لدراسة حركة المياه الجوفية و عمل تقييم كمي لتغذية المياه الجوفية من السد و قد ساعد النموذج أيضا في دراسة إتجاهات حركة المياه الجوفية و الموازنة المائية للمياه الجوفية في الخزان.

و قد خلصت الدراسة إلى عدة إستنتاجات حيث تبين أن مساحة التصريف إلى السد تبلغ حوالي ١٩٥كم^{٢} و تشمل وادي حام و وادي الفرفار كما تبين أن الأمطار ذات طبيعة متقطعة و غير منتظمة و يقدر المتوسط السنوي بحوالي ١٥٤ مم و تنعكس طبيعة هذه الأمطار على مقدار الجريان الذي هو بدوره غير منتظم. كما استنتج من نتائج الدراسة أن وادي الفرفار يشارك بنسبة كبيرة في حجم السيول المتجمعة عند السد حيث تصل هذه النسبة إلى أكثر من ٤٠ ٪ في بعض الأحيان.

من خلال نتائج الموازنة المائية للنموذج الرياضي للمياه الجوفية تبين أن السد يقوم بدور فعال في تغذية المياه الجوفية حيث تروواح ما يترشح منه إلى الخزان الجوفي ما بين ٣٢٪ إلى ٤٣٪ من مخزون السد و هذا يظهر بشكل واضح و ملموس في ارتفاع مناسيب المياه الجوفية و تبين أيضا من خلال دراسة إتجاهات حركة المياه الجوفية و الموازنة المائية أن التغذية من السد تساهم بشكل واضح في تقليل تداخل مياه البحر في الخزان الجوفي و بالرغم من وجود بعض الفواقد إلى الخليج إلا أنها محدودة قياسا إلى ما تضيفه التغذية من السد إلى الخزان الجوفي.

و بناءا على نتائج هذه الدراسة فقد تم طرح بعض التوصيات للدراسات المستقبلية منها ضرورة رصد البيانات بصورة مستمرة واقتراحات بإنشاء محطات رصد و آبار مراقبة إضافية كما أن نموذج المياه الجوفية المعد في هذه الدراسة يمكن تحسينة و زيادة إمكانياته بعمل حصر شامل لآبار الضخ في منطقة الدراسة إضافة إلى إجراء المزيد من اختبارات الضخ.

المصطلحات : المناطق الجافة ، الإمارات العربية المتحدة، وادي حام، التغذية الاصطناعية ، المياه الجوفية ، جريان سطحي ، نماذج رياضية.

جامعة الإمارات العربية المتحدة
عمـــــادة الدراســـات العليـــــا

عنوان الرسالة

تقييم جريان المياه السطحية و تغذية المياه الجوفية باستخدام النماذج الرياضية

إسم الباحث

خالد عمر محمد هارون

المشرف

أ.د. محسن شريف (أستاذ موارد المياه - كلية الهندسة- جامعة الإمارات العربية المتحدة)

جامعة الإمارات العربية المتحدة
عمـــادة الدراســـات العليـــا

تقييم جريان المياه السطحية و تغذية المياه الجوفية باستخدام النماذج الرياضية

إعداد

خالد عمر محمد هارون

رسالة مقدمة إلى
عمادة الدراســـات العليا
جامعة الإمارات العربية المتحدة

لإستكمال متطلبات الحصول على درجة ماجستير العلوم فى موارد المياه

عمادة الدراســـات العليا
جامعة الإمارات العربية المتحدة

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بسم الله الرحمن الرحيم

وَأنزلنا منَ السَّماءِ ماءاً بِقدرٍ فأسكنّاهُ
فِي الأرْضِ وإنّا على ذهابٍ بِهِ لقادِرُونَ

صَدَقَ اللهُ العَظِيمُ
المؤمنون - 18