Integrated Water and Land Management to Deliver Sustainable Agriculture in Semi-arid Catchments

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degree of Doctor of Philosophy

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DECLARATION

I certify that except where due acknowledgement has been made, the work is that of the author alone; the work has not been submitted previously, in whole or in part, to qualify for any academic award; the content of the thesis is the result of work which has been carried out since the official commencement date of the approved research program; and, any editorial work, paid or unpaid, carried out by a third party is acknowledged.

Ammar Jarrar

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LIST OF ABBREVIATIONS

AGSM	Agricultural Sub-Model
Alt	Management Alternative
ARIJ	Applied Research Institute Jerusalem
AWBM	Australian Water Balance Model
BFI	Baseflow Index
BW	Total quantities of brackish water available (ML/Year)
CE	Coefficient of Efficiency
COST	Net costs associated with each management alternative ($)
CPF	Comprehensice Planning Framework
CROPWAT	FAO Crop Water Requirements Model
CWR	Crop Water Requirements
DEM	Digital Elevation Model
EQA	Environmental Quality Authority
ET_o	Reference Evapotranspiration
ET_a	Actual Evapotranspiration
FAO	Food and Agriculture Organization
GIS	Geographic Information System
GIUH	Geomorphological Instantaneous Unit Hydrograph
IOC	Importance Order of Criteria
IUH	Instantaneous Unit Hydrograph
K_c	Crop Coefficient
kg/ha	Kilogram per Hectar
KW-GIUH	Kinematic-Wave Geomorphological Instantaneous Unit Hydrograph
MCDSS	Multi Criteria Decision Support System
ML	Mega liter
MoA	Ministry of Agriculture
MODFLOW	U.S. Geological Survey Modular Finite-Difference Groundwater Flow Model
MoT	Ministry of Transportation
MSL	Mean sea level
M US$	Million US Dollar
NB	Total net benefit ($)
n_o	Overland roughness coefficient

n_c	Channel roughness coefficient
NR	Net return of water ($/m3)
OSLO II	Peace Agreement signed off between the Palestinians and Israel in 1994
PCBS	Palestinian Central Burea of Statistics
PU	Degree of political uncertainty
PWA	Palestinian Water Authority
SCWR	Degree of social conflict due to historical rights utilization
SY	Total quantities of abstracted groundwater above the safe yield (ML)
SW	Total quantities of winter surface water and wastewater available (ML/Year)
TIL	Total irrigated land (ha/Year)
UAWR	Percent utilization of available water resources (%)
UALR	Percent utilization of available land resources (%)
US$	US Dollar
US$/$m^3$	US Dollars per cubic meters
WHAT	Web-based Hydrograph Analysis Tool
WESI	Water and Environmental Studies Institute/An-Najah National University
WRC	Water Related Contribution
WSSPS	Water Sector Strategic Planning Study

ABSTRACT

Arid and semi-arid regions are generally characterized by water scarcity and low per capita water allocation. In such areas, intensive agricultural activities associated with high population growth cause a further exacerbation of this problem. The concept of integrated land and water management has been widely accepted as a practical solution for sustaining both the environment and agricultural production. Integrated catchment management provides an interdisciplinary framework that links physical, social and economical sciences into planning, policy and decision making. Specifically, prioritizing and evaluating different management alternatives using a multi-criteria decision analysis model is essential to achieve a long–term agricultural and natural resources sustainability in agriculture-dominated semi-arid catchments.

The main objective of the research is to develop a framework for a Multi Criteria Decision Support System (MCDSS) that provides planners and decision makers with a tool for planning integrated management of land and water. It also provides a soundly based analysis of agricultural water demand. The data from agriculture-dominated Faria catchment in the West Bank of Palestine was used for the study. The proposed approach integrates a rainfall-runoff model, a groundwater model, statistical analysis of spring discharges, a planning model and a multicriteria decision analysis model. Collectively they form the framework. These models were utilized to determine the optimal cropping pattern that maximizes net income of the catchment that could be sustained by its natural resources. Management alternatives can be introduced by determining the sustainable limits imposed by the limited natural resources. Management alternatives were developed to maximize the net benefit whilst sustaining the available water resources. To evaluate the overall efficiency of the introduced alternatives, decision criteria were developed to account for the economic and environmental consequences and a multi-criteria decision analysis was conducted to rank the land and water management alternatives.

The Kinematic-Wave Geomophological Instantaneous Unit Hydrograph (KW-GIUH) was used to estimate runoff from the catchment in ungauged situations. The model was calibrated and applied to the catchment and the results were compared with observed hydrographs. The simulated and recorded hydrographs were in good agreement. Sensitivity analysis was conducted for all catchment geomorphological parameters. The overland flow roughness

coefficient (n_o) and the channel flow roughness coefficient (n_c) were obtained from tables depending on the catchment cover and channel type. All other model parameters were obtained from the Geographic Information System (GIS). Peak flow values increased by 16% as the n_o decreased by 25% which reflects the land surface condition of the surface hydrologic system. However compared with the n_o, the n_c had a smaller effect on both simulated peak flow and time to peak. The stream order level of each subcatchment indicated that it is necessary to follow the stream network map in developing the KW-GIUH model.

The MODFLOW software package was utilized to estimate the amounts of groundwater that could be safely extracted under different management alternatives and climatic change. Groundwater recharge and pumping rates directly influenced output from MODFLOW. Groundwater recharge reflects the climate variability while groundwater pumping is directly reflective of the possible land area that could be managed. Results showed that, based on 50% allowable limit of drawdown percentage, the current groundwater wells are abstracting water above the safe yield with a total amount of 3200 ML/yr. A statistical analysis of the yield at Faria springs showed a considerable variation in the total annual yield. The reliability of each spring flow was tested and the results showed that the reliability of all the springs exceeded 50%. Hence yield from all the springs was used for planning purposes. Results from this analysis were used to supply the water resource data needed for the planning model as well as to formulate the management alternatives for the catchment.

The planning model (AGricultural Sub Model AGSM) integrates the outputs of the rainfall runoff and groundwater models to determine the total amounts of water available for irrigation. The AGSM computes the net income and the amounts of different qualities of water used to produce the optimal cropping pattern. Application of AGSM to data from the Faria Catchment showed that the model was capable of providing objective evaluation of the changes in management scenarios and water policies. Water demand curve that shows water demand as a function of water price was developed for the Faria catchment. A logarithmic demand curve was developed to derive the optimal water price. This curve approach provides the planners and decision makers with a simple and efficient method to combine the optimal area cropped and obtain the optimal water price that could be charged to farmers. The optimal price and price elasticity of agriculture water demand obtained through the curve enable policy makers to evaluate response to price changes, allow better trading opportunities and optimize agriculture water demand.

Management alternatives were developed for Faria catchment such that the optimal water and land utilization are met to maximize profits. Decision criteria were developed and utilized in a multi-criteria decision analysis. Each management alternative was evaluated in terms of different economic, environmental and social decision criteria. The importance order of criteria (IOC) method was employed in the multi-criteria decision analysis to rank the alternatives. The IOC method relies on the preference of the decision maker in stipulating of the decision criteria. Such an order reflects the importance of these criteria to the decision maker. Combining different management alternatives proved to be an efficient approach for maximizing net benefit and satisfying the yield limits. The latter is a very important issue in a resource constraint of semi-arid area. The ranking of the management alternatives indicates the successfulness of the alternatives for a specific importance order of the criteria. A combined management alternative that includes utilization of Jordan River, changing the cropping pattern by introducing high income crops, building surface water storage (dam) and implementing a groundwater pumping strategy proved to be the best alternative. The optimal management plan obtained from the developed approach addressed the current problems of the study area and met the triple bottom line of land use, governmental policies and water resources protection and development to optimize use of land and water resources. Compared to the “do nothing“ alternative the net benefit for the plan is more than double. The irrigated area under the optimal plan exceeded the current irrigated areas by 34%. Under this plan there is no groundwater abstracted above the safe yield. The amounts of surface water and wastewater that could be stored and made available for irrigation under the optimal plan are 6600 ML compared to none stored at present. The MCDSS developed under this study is a very useful tool that can be used by different decision makers and planners in various areas. It can be used to investigate the efficiency of different management alternatives in satisfying the decision criteria that ultimately aim at achieving the policies and strategies envisaged for the area under study.

CHAPTER 1

INTRODUCTION

1.1 GENERAL INTRODUCTION

Arid and semi-arid catchments are characterized by the scarcity of natural replenishment of water resources. This scarcity leads to the limited availability of water resources and the dire need to manage these resources carefully. The introduction of catchments as management units for optimal use of land and water resources has been demonstrated (Sharifi, 2003). It is more convenient to perform economic analysis and natural resources optimization at a catchment level thus enabling analysis that overlaps and integrates the sectoral concerns of foresters, hydrologists, economists, engineers, agriculturists and the community. The consideration of the whole catchment as an integrated unit in hydrological planning is necessitated by the fact that downstream interests are often affected due to development in the upstream (Singh, 2004).

In semi-arid catchments that are overlain by intensive agricultural areas and corresponding dense activities, water availability and shortage problems are further exacerbated. Degradation of water quantity becomes a major concern especially when setting up safe yield limits for the available water resources to guarantee the sustainability of these resources for the present and future utilization. Worldwide, many semi-arid catchments are under extreme stresses not only because of the limited replenishment of the water resources but also due to the increasing population and climate changes that may reduce the water availability further when considering the increasing need to boost agricultural production.

There is increasing pressure from engineers, water managers and policy makers to allocate less water for agriculture (which is considered the largest consumer of the currently available water supplies) and to increase the allocations to other sectors (e.g. urban use) and economic activities (Singh, 2004). This caused increasing conflict between the desire to grow the agriculture sector to sustain food production, the direct human and industry needs and water required to sustain freshwater ecosystems. Furthermore appropriate means of fair water allocation are necessary to achieve optimal water allocation between sectors of such a scarce resource. Even in a developed country such as Australia this is the case where in the state of Victoria 68% of the consumptive water is used in agriculture. As a result of rapid population growth and increasing concern about the environmental effects of surface water diversions, these water users are under increasing pressure to conserve water. Financial incentives, whether embodied in water trading opportunities or increased water rates, are widely considered by economists as an effective means of reducing water consumption in agriculture. However, it is sometimes postulated that the price of water delivered to farmers is so highly subsidized that there is no significant demand response to modest price changes. Missing from this important policy debate are sound estimates of the price elasticity of agricultural water demand.

Despite the rapid development in remote sensing, geographical information system (GIS) and information technology, there is still a missing link between all these technological developments and proper management of the water resources. The link could be established through the integration of all the relevant knowledge, experiences, technologies and information into a decision support system to support integrated management and decision making related to optimal use of natural resources. This study concentrates on developing management alternatives that ensure sustainable use of the available water resources both in quantity and quality, optimize the use of low quality water including treated effluent and brackish water and maximizing both the irrigated areas and the income of the local farmers. Since the decision criteria address conflicting objectives it is planned to employ a multi-criteria decision analysis to prioritize the management alternatives.

1.2 OBJECTIVES OF THE STUDY

The main objective of the study is to develop a framework to optimize the use of available land and all elements of water resources for an agriculture dominated semi-arid catchment. The development of an integrated natural resources management framework involves the inclusion of surface water and groundwater models to estimate the available water yield, a planning model for economic evaluation, a multi-criteria decision making model to decide between management alternatives and the use of Geographic Information System (GIS) technology to facilitate processing and visualization.

To achieve the main objective the following tasks will be carried out and reported:

1. Develop a simple rainfall-runoff model to determine stream flow yield from ungauged agriculture-dominated semi-arid catchments.

2. Determine the sustainable-yield limits of alternative sources including groundwater and spring water sources.
3. Develop a planning model for the study area to analyse agricultural water demand and determine the cropping pattern that maximizes the farmer's economic return under various quantities of water allocations for different qualities, timing and prices.
4. Conduct an economic cost analysis for evaluating the management alternatives that are needed to maintain the optimal land and water utilization and develop a multi-criteria decision analysis model for ranking the management alternatives after considering the corresponding socio-economical and environmental criteria.
5. Develop a Multi Criteria Decision Support System that can be used by different decision makers and planners in various areas to investigate the efficiency of different management alternatives.
6. Demonstrate the applicability of the developed approach by applying it to a real-case study through the examination of different alternatives that optimally manage the available land and water resources to maximize retun to users.

1.3 STUDY AREA

To deliver the above objectives, the study will be carried out using data from the semi-arid Faria catchment in West Bank, Palestine. The Faria catchment is located in the northeastern part of the West Bank (Figure 1-1) with a total area of about 330 km^2. The Faria catchment extends from the ridges of Nablus Mountains down the eastern slopes to the Jordan River and lies within the Eastern Aquifer Basin which is one of the three major groundwater basins that form the major water resources of the West Bank. Ground surface elevations in the catchment exceed 900 m above mean sea level (msl) in the western areas of the catchment and drop gradually down to 320 m below msl across the main surface water stream especially in the southern parts of the catchment. The climate is dominantly a Mediterranean, semi-arid climate, characterized by mild rainy winters that last about five months and moderately dry, hot summers. Average annual rainfall intensity in the study area reaches 600 mm in the northern and western portions of the catchment. Rainfall intensity decreases when one moves towards the east and the south.

Based on rainfall, location and altitudes three agroclimatic zones can be identified for the study area (MoA, 2005b; EQA, 2000; ARIJ, 1998; WSSPS, 2000; WESI, 1999) including:

A. ***Zone 1 - The upper zones:*** This zone includes two hydrologic subcatchments namely Faria and Badan subcatchments as shown in Figure 1-2. Slopes and mountain plateaus that extend throughout the western and northwestern parts of the catchment. The climate is semi-arid with a typical Mediterranean climate. Precipitation in this area exceeds 400 mm which makes rainfed agriculture feasible for olives, almonds and field crops. Precipitation exceeds evaporation in about 4-5 months of the year. Ground surface elevations in this area exceed 200 meters over mean sea level.

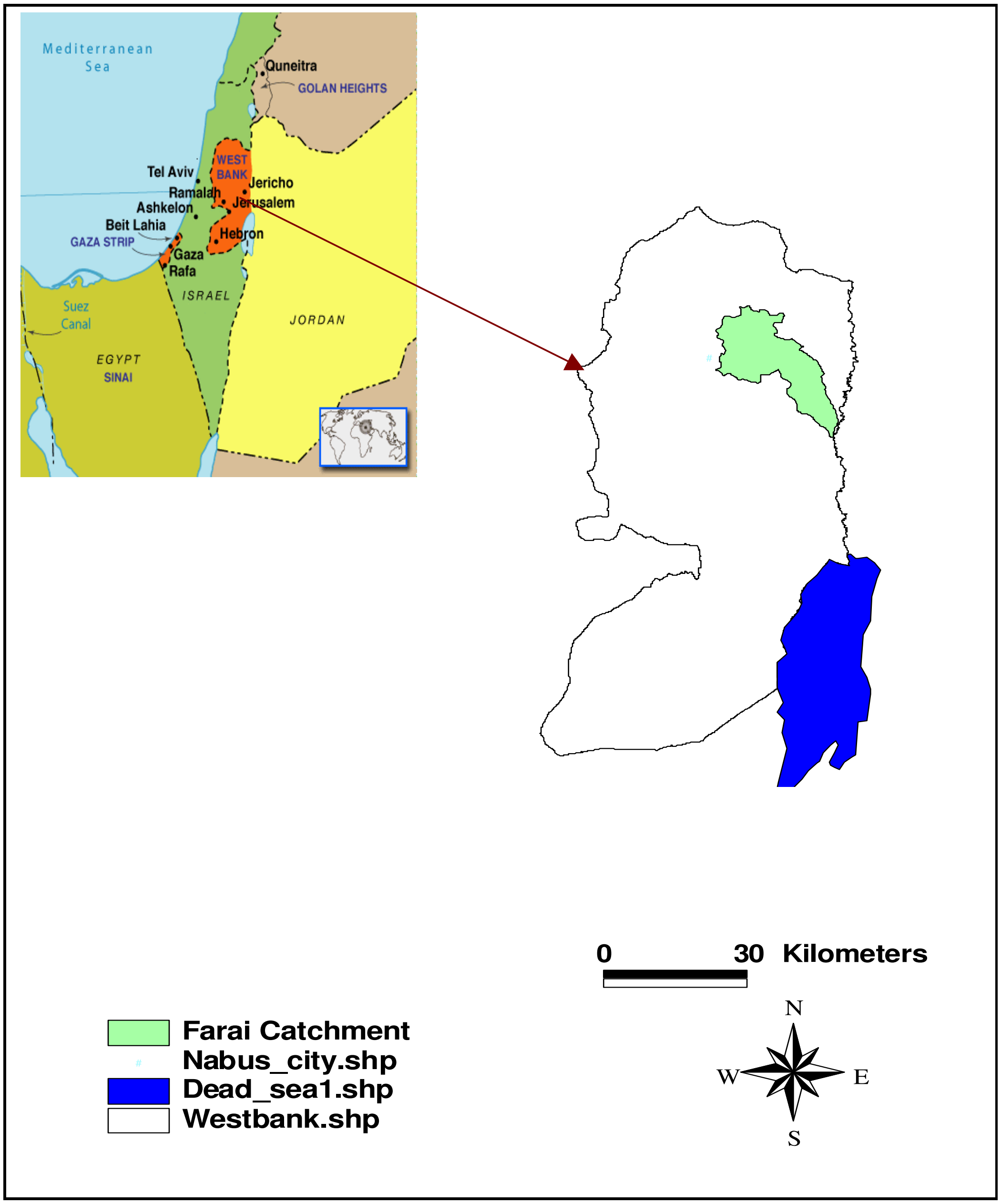

Figure 1-1 Map of West Bank and Faria catchment

B. ***Zone 2 - The central zone:*** The climate is semi-arid and the rainy season is shorter than the rainy season in the upper zone. Temperatures are higher in comparison to the upper zone. Precipitation ranges from 200 to 400 mm in this zone which makes rainfed agriculture feasible only in wet years. Precipitation exceeds evaporation during 3 months of the year on average. Ground surface elevations in this region range from 100 meters below sea level to about 200 meters above sea level.

C. ***Zone 3 - The lower zone:*** The climate in the lower zone is arid. Small amounts of precipitation fall during winter. The summer months are dry with eight months that are completely devoid of rain. Precipitation in this zone is usually less than 200 mm and thus irrigation is essential to achieve agricultural production. Precipitation exceeds evaporation in less than two months of the year in this zone. Ground surface elevations in this zone are usually less than 100 meters below sea level.

Zone 2 and Zone 3 constitute the Malaqi hydrologic subcatchment as depicted in Figure 1-2.

The irrigated agricultural sector is considered to be the backbone of the Palestinian economy and provides more than 80% of the employment in the study area. Irrigated agriculture constitutes about 70% of the total current agriculture in the study area, and reaches more than 90% in the lower part of the study area (Ministry of Agriculture, 2005a; PCBS, 2003). The study area is part of the Jordan Valley which is considered as the only potential area for agricultural development during the coming years, in addition to its current importance as the largest agricultural production area in the West Bank (WSSPS, 2000). The predominantly rural population in the catchment, estimated at about 21000, faces a series of environmental threats and poor economic conditions. The growth of rural population is about 3.5 % annually. This has resulted in increased demand for natural resources, mainly land and water. Lack of proper management of land and water resources has caused over abstraction of the scarce water resources and ineffective use of land. Surface runoff in the catchment is not utilized in winter as there are no dams in the catchment to store the excess water (WESI 2005, Abu Safat 1990). Existing cropping patterns are rigid in response to changing conditions including the ongoing shifts in the market demand and supply sides, available water quality, price and quantity. Unbalanced excessive abstraction of groundwater has increased salinity to unaccepted levels, especially in the lower areas (ARIJ, 1998). Living conditions for the population are probably the worst in the West Bank, with poverty levels exceeding 50 percent (WESI, 2005), especially in the wake of political uncertainity.

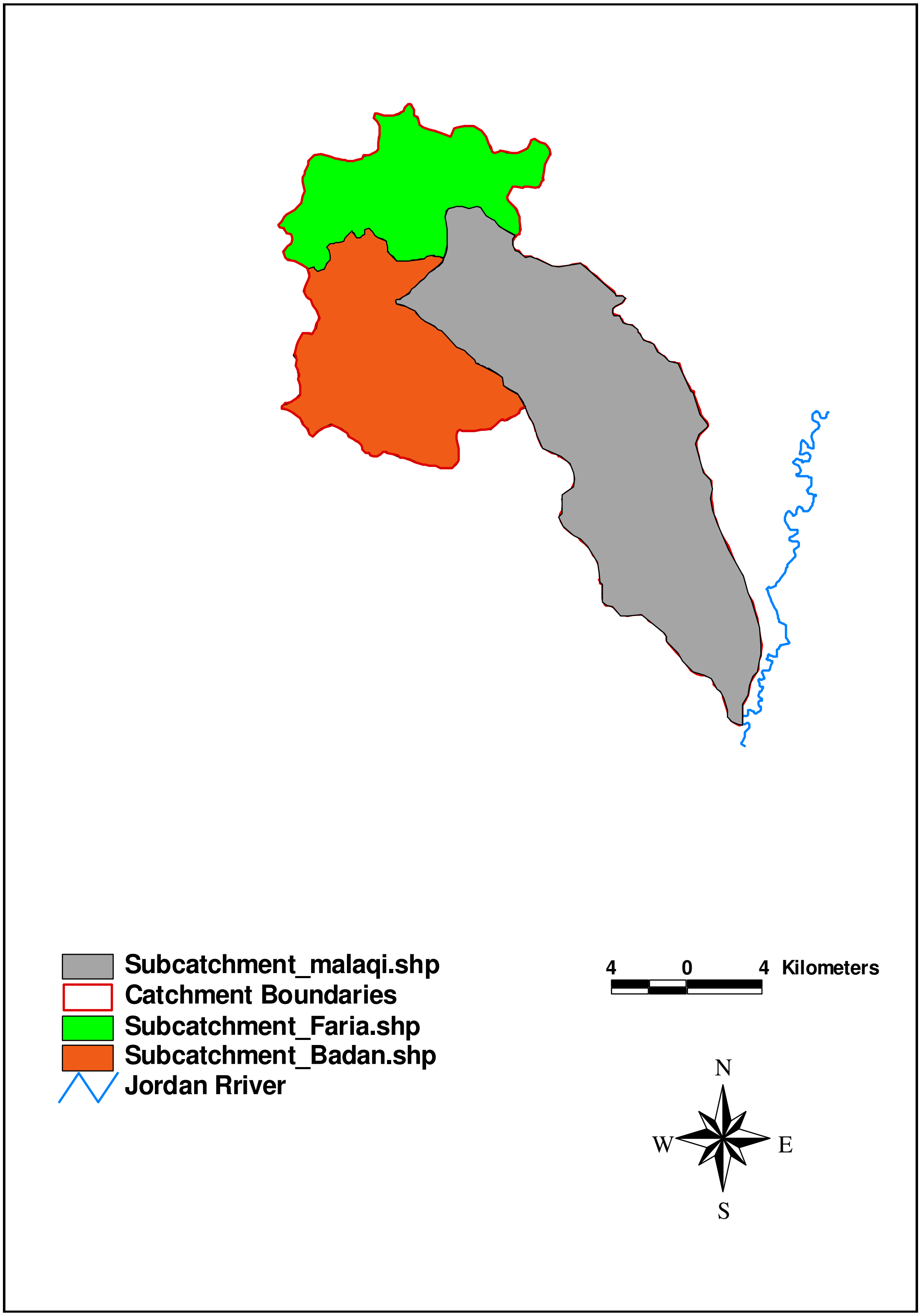

Figure 1-2 Faria Catchment with the three subcatchments Badan, Faria and Malaqi

Irrigated agriculture includes open field vegetables, greenhouses and irrigated trees. Open field vegetables cover about 1900 ha (Ministry of Agriculture, 2005a; PCBS, 2003). Protected agriculture under plastic occupies about 200 ha including about 450 Greenhouses (with a total area of about 45 ha). Greenhouses usually have more than 6 times returns than open field vegetables as the productivity under greenhouses is much more than that for open field crops. The most common irrigated trees in the catchment are citrus trees which cover about 300 ha.

The study area includes one unconfined upper and two confined lower aquifer systems. Groundwater is found in formations of Pleistocene to Lower Cenomanian age, at depth ranging from many meters to several hundred of meters (Ghanem, 1999). In the area under investigation, five sub-aquifers are located within unconfined and confined strata. Utilization of the groundwater aquifers is through springs and wells. There are 69 wells in the Faria catchment of which 61 agricultural wells, 3 domestic and 5 Israeli wells. The annual average abstraction of Israeli wells is 10000 ML. The annual average abstraction of Palestinian wells is 8300 ML, of which 75 % is utilized for agricultural purposes and the rest for domestic purposes (PWA, 2005, WESI, 2005). There are 11 fresh water springs in the study area most of which are located in the upper and middle parts of the stream (PWA, 2005, WESI, 2005). Water from irrigation wells is used in conjunction with spring discharge in most of the catchment.

Spring water is mixed with waste water from Nablus and Faria camp. No measurements of surface water quality are available for the Faria Stream. Chemical and biological analysis of the Palestinian wells in the upper areas of the study area did not show any serious quality problems (WESI, 2005) except for salinity which is a major concern that limits the utilization of groundwater.

Agricultural water supply is provided from springs and privately owned irrigation wells. There is no comprehensive system to supply water for irrigation. Each irrigation well has a separate pipe network to serve the farmers using the well. For springs, the only pipe network system is the Faria Irrigation Project which extends through the lower part of the catchment where water is conveyed through natural stream and open channels. There are no effective farmer unions or water user associations to manage and operate the irrigation systems. In addition, the water rights in the area for spring water are not clear and lack enforcement mechanisms. More detailed and specific data used for each area of the study are presented in Chapters 3, 4 and 5.

1.4 OUTLINE OF THE THESIS

Chapter 2 developes the methodology and provides a detailed explanation about decision support systems (DSS) and their role in the development of an integrated natural resources management framework. This chapter also gives a review of the previous research conducted on rainfall runoff modeling, groundwater modeling and planning model. Furthermore the selection of these models for the natural resources management framework is discussed within the scope of integrated land and water management.

Chapter 3 describes the development of the rainfall runoff model to estimate runoff from the ungauged catchments based on climatic data and catchment geomorphological characteristics used with the GIS tools. The runoff volume and peak flow are needed for infrastructure development for transport and storage of surface runoff or for recharge of groundwater. The sensitivity analysis was also conducted for geomorphological input parameters. The Kinematic Wave Geomorphological Instantaneous Unit Hydrograph KW-GIUH (Lee and Yen, 1997) was selected for this purpose. The model input parameters and theory behind the KW-GIUH model are described. Excess rainfall estimation needed for rainfall runoff modeling is also discussed. The developed KW-GIUH model was verified using a set of data from the Faria catchment in the West Bank.

The MODFLOW model (Harbaugh and McDonald, 1996) is a popular groundwater flow model used to estimate safe yield from catchments. The MODFLOW groundwater flow model has been utilized to estimate the amounts of groundwater that could be safely extracted under different management alternatives. Chapter 4 continues the work on application of the MODFLOW model and details the development of the groundwater module along with the statistical analysis of springs' yield. The application of the results from the MODFLOW model analysis and the statistical analysis of springs' yield in the development of the management framework will also be discussed.

Chapter 5 demonstrates the development and application of the planning model (AGricultural Sub Model AGSM) for the study area that involves the output of rainfall-runoff model, groundwater model and statistical analysis of springs' yield developed in Chapters 3 and 4. The application of the developed planning model to compute the net income and the total amounts of surface water and groundwater to produce the optimal cropping patterns are presented in this chapter. Further, the use of water demand curves to determine the optimal price of water and price elasticity is discussed.

Chapter 6 implements the methodology developed in Chapter 2 and utilizes the multi-criteria decision analysis for the selection of the feasible management alternatives. The Multi Criteria Decision Support System (MCDSS) is developed in this chapter. The chapter describes the importance order of criteria (IOC) method employed in the multi-criteria decision analysis to rank the management alternatives in terms of different social, economic and environmental decision criteria. The derived optimization approach is benchmarked against the current agricultural and water management practices in the area.

Finally summary, conclusions and recommendations from the study are provided in Chapter 7.

CHAPTER 2

DEVELOPING THE FRAMEWORK FOR THE INTEGRATED LAND AND WATER MANAGEMENT

2.1 INTRODUCTION

There are many aspects of water resources management including the optimal water allocation, quality assessment, conservation and the prediction of future water demands that require studying prior to develop strategies for water utilization, planning and decision making. A more practical and widely accepted approach is to apply the concept of integrated land and water management to sustain both the environment and agricultural production as a socio-economic resource base. As a preliminary step, these management aspects and others necessitate the characterization of the water resources in the area of interest. One of the established methods to carry out this assessment work is through modeling and statistical analysis of the spatial and temporal variability of water resources quantity. The catchment based approach to water resources management is the most logical basis for water resources management from environmental, social and economic point of view. In the following sections, the literature review covers studies pertaining to optimal water allocation for agriculture-dominated catchments, applications of decision support systems (DSS) and rainfall-runoff and groundwater modeling carried out to assist with catchment management. The information from these models and criteria for the DSS is also discussed when developing the framework for integrated land and water management. A brief methodology of the development of the multicriteria decision support systems (MCDSS) is also given in this Chapter.

2.2 OPTIMAL WATER ALLOCATION IN AGRICULTURE DOMINATED CATCHMENTS

Appropriate means of water allocation are necessary to achieve optimal allocation of a scarce resource. A number of criteria, which are often interrelated, come into play in planning and managing water systems. The emphasis placed on any one of these criteria by different countries varies (FAO, 1995). Such criteria include water use efficiency, socio-economics, environmental, policy reforms and politics, water pricing, public water allocation, water markets and mixed water allocation systems (ESCWA, 2003). Water resources have been allocated on the basis of social criteria maintaining the community welfare, by ensuring that water is available for human consumption, for sanitation, and for food production. Societies have invested capital in infrastructure to maintain this allocation. The demand for water and ability to control its location, timing, quality and quantity are becoming critical with the growing demand for municipal, industrial and agricultural uses in the arid region. The basic principle of treating water as an economic good is to allocate it to deliver best value. Various mechanisms are used for allocating water, including water pricing, social planning, user-based allocation, and water markets. Governments play an important regulatory role in water allocation systems, but how effectively they do so depends on the relative political influence of various stakeholders and segments of society. Examples from experience in several countries show that no single approach is suitable for all situations. Where there is strong demand for limited volumes of water and a history of cooperation, user-based allocation is generally more flexible than government allocation (Salman et al., 2001).

Agriculture, especially in arid and semi-arid zones, requires water for irrigation. Under a situation where water and land are limited resources, competition among various stakeholders, such as industry and agriculture, for use of fresh water is bound to increase. There are substantial differences in the characteristics of water consumption between the various sectors. For example, compared to agriculture, water demands by households are not price sensitive, at least for the high priority uses necessary for human life. On the other hand, while agriculture can utilize low quality water types (recycled wastewater, brackish water and surface water) the household sector and some of the industrial sector can use mainly surface water of good quality. Another significant difference is that water supply to households and industry must be extremely reliable, whereas the reliance of the agricultural sector on a dependable supply of water may not be as important, especially when water is to be used for low cash field crops. As a result agriculture, although the main water-consuming sector, tends to be the most vulnerable to water shortages. Further, privately owned agriculture tends to require relatively low-cost water. This has led to considerable subsidization of water for agriculture (Fisher et al., 2005).

While agriculture productivity is subject to considerable uncertainty due to variability in water supply, it also has considerable flexibility, since it is often able to produce a large variety of crops farmed in the same area. This flexibility is mainly due to annual field crops that can be grown using different amounts and qualities of water during different growing seasons.

Agricultural planning methods to deal with such issues have been developed and used (Amir et al. 1991, 1992), but the sensitivity of agriculture to water remains an important issue for many countries in formulating water policies (Fisher et al., 2003).

A number of optimization and planning models (Vedula and Kumar, 1996; McKinney et al., 1999; McKinney and Cai, 1996; Schoengold et al., 2006; Arnold et al., 1998) have been developed to evaluate water resources and quality parameters affected by agricultural land management at both the field and at catchment scale. Of particular importance to basin-scale analyses are models of two fundamental types:

- models that simulate water resources behavior in accordance with a predefined set of rules (actual or hypothetical) governing water allocations and infrastructure operations, and
- models that optimize and select allocations and infrastructure based on an objective function (economic or other) and accompanying constraint.

Optimization models are more useful if improvement of the system performance is the main goal (McKinney et al., 1999). Vedula and Mujumdar (1992) and Vedula and Kumar (1996) described a stochastic dynamic programming model with numerous simplifications that solves for minimum crop yield reductions caused by water stress. In dynamic programming, transformation functions are determined, which link the values of the state variables in each time period to those in subsequent periods, to allow for the calculation of the objective function value. This disaggregation of the objective function into a series of recursively solved equations for multiple state variables introduces significant computational complexity.

McKinney and Cai (1996) developed hydrology inferred policy analysis tools to be used for water allocation decision making at the river basin scale. Their work involved the development of optimization models for the Amu Darya and Syr Darya basins in the Aral Sea basin of Central Asia using GAMS and ArcView GIS software. This hydrology-inferred approach has been extended recently to an economic optimization approach that considers cropping decisions and irrigation and drainage system improvements. The main limitation of using this model for river basin analysis arise from the fact that environmental impacts such as groundwater quality degradation and salinity problems could not be captured and therefore the results from this model do not wholly reflect conditions of sustainability of water management in irrigation-dominated river basins.

Lee and Howitt (1996) modeled water and salt balances in the Colorado river basin to determine salinity levels that maximize net returns to agriculture and municipal-industrial (MI) users at selected locations in the basin. Three scenarios were considered: economic optimality; no change in cropping patterns with subsidies for salinity control measures; and cropping changes with subsidies to maintain agricultural profits. The first-best, economically optimal scenario indicates major declines in cropped area with significant returns to MI uses. Of the two scenarios with subsidies, the cropping changes subsidized to maintain profits indicate marginally lower total subsidies with a minor, but significant reduction in salinity. Nonlinear crop production functions and MI costs per unit of salinity were derived for inclusion in the objective function and are specific for the basin conditions; these should be developed for the local conditions of other basins.

Schoengold et al. (2006) developed a model of agricultural water demand using data set from California's San Joaquin Valley and concluded that farmers respond in two ways to an increase in the marginal price of water, both by reducing their water applications and altering their land allocation.

Doppler et al. (2002) investigated the optimal water allocation and cropping patterns for the Jordan Valley in Jordan considering rising water prices and variations in income expected from agricultural production. The calculations were based on information available on water supplies, areas under irrigation and market conditions, and used linear programming to determine solutions that maximize gross margins and minimize potential variations in these margins. The results indicated that optimizing cropping patterns and the allocation of irrigation water has a substantial potential to increase the financial return to agriculture. Optimal solutions that considered the risk from varying gross margins reacted quite elastically, to demand for irrigation water due to rising water prices.

A mixed integer-programming model (MIP) had been used by Sharifi (2003) to simulate the farmers' reactions and decisions to the different subsidy schemes. This model, which was based on rational farmers who would like to maximize their profits, was used as a "planning model" to formulate and assess the impacts of various policy instruments. The MIP model had been applied to La Mancha catchment in Spain to study the effect of different governmental policies and scenarios on the overall water resources management and was capable of providing a useful tool for policy formulation but could not provide an optimal cropping pattern to maximize net benefits for the study area.

Soil and Water Assessment Tool (SWAT) is a catchment scale model (Arnold et al., 1998) developed to predict the impact of land use management practices on water, sediment, and agricultural chemical yields. The model combines land use practices with point source contributions, and performs flow and water quality routing in stream reaches. Srinivasan and Arnold (1994) used the SWAT model to simulate flow in the upper portion of the Seco Creek basin (114 km2) in Texas. The catchment was subdivided into 37 subbasins and the predominate land use was rangeland. Monthly simulated streamflow data were compared to measured streamflow data for a 20-month period. The authors reported that there were no general tendencies to over or underpredict surface runoff during certain seasons of the year. Simulated values compared well with measured values, with the average monthly predicted flows 12% lower than measured flows, and a coefficient of determination of 0.86. The model successfully predicted the effect of land use management on water yield but was not capable of optimally allocating the land and water resources to maximize catchment profits.

However, the common models used to assess water management and allocation issues are integrated hydrologic-economic models. Such models include (ESCWA, 2003):

(i) depiction of the entire system (at the basin or the country level);
(ii) integration of hydrologic and economic relationships in an endogenous system;
(iii) incorporation of water demands from all water-using sectors; and
(iv) the possibility of evaluating the economic benefits and costs of each of these demands.

A number of water management and allocation models have been developed using the above framework. One is the integrated hydrologic-economic model developed for the Mekong River Basin, which optimizes water allocation based on an objective function and its accompanying constraints (Ringler, 2001). The model focuses particularly on the economic component. The optimal allocation of water across water-using sectors is determined on the basis of the economic value of water in its various uses. Another model was developed by Wichelns (2002) to evaluate the trade-offs involved when Nile River water was allocated between competing regions and projects in Egypt. The goal of the model was to maximize the net social benefits generated with limited water resources. Other types of analysis have also been used; these include input-output models, which allow changes in the economy as a result of droughts or the reallocation of water away from agriculture to be analysed (Wolfenden et al., 2001). Although comprehensive, the models outlined in Ringler (2001), Wichelns (2002) and Wolfenden et al. (2001) are fairly complex and time-consuming. They require substantial amounts of data and advanced statistical programmes to analyse the data. Furthermore, most of these models are usually run on selected hypothetical situations. As an example, Wichelns limited his analysis to small-scale (10 hectares) catchments and one crop.

Cheesman (2005) proposed a non-linear model to define economically optimal agricultural cropland and irrigation allocations, using cropland area and irrigation volumes as the decision variables. The model used non linear programming modelling approach to define economically optimal crop area allocation and irrigation schedules based on empirical farm budget data and crop water yield simulations from evapotranspiration module that simulated vegetative growth responses to water application under well defined climatic and soil conditions. The model was applied to Dak Lak agricultural basin in Viet Nam to analyse the potential impact of switching out of coffee into more drought resistant perennial crops, as encouraged by the government, on the spatial and temporal availability of water and associated socio-economic outcomes. The proposed model focused on the region’s five main annual and seasonal crops but other perennial crops were not included.

Bielsa and Duarte (2001) proposed an optimization process for the optimal allocation of water among competing users. The specific application was carried out on the basis of real data for the Vadiello Reservoir in Spain. The analysis allowed to consider an example of how the allocation that maximized the objective function lead to a mitigation of the losses in dry periods, and to an increase in the joint profit when there was an extension to the surface area under irrigation. However the proposed approach did not consider the definition of both the water quantity and quality as part of the proposed process.

Adamson et al. (2005) incorporated all catchments of the Murray Darling Basin in Australia within a single modelling structure and incorporated risk and uncertainity in linear programming models for policy analysis. The approach used to develop the model in two software systems, GAMS and Excel, had been advantageous both for development and application view points. The results presented, though preliminary, implied that the worst case scenarios predicted under climate change would have differing implications on different parts of the Basin, with those catchments with higher salt loads were likely to face reduced options to manage, unless opportunities became available to reduce salt loads significantly. The industries already adopting near-precision agriculture enjoy the first mover advantage in securing water rights and allocating water to its best use. However, sustaining returns on their investment might rest on decisions by their upstream counterparts who could influence the salinity levels of the water they used.

Alternatively, agricultural demand for water and its optimal allocation can be viewed as part of the integrated catchment management approach through a proper decision support system. For that purpose, a model of agricultural response to water prices and policies, that is capable to predict the crop-choice decision of farmers, will be used as a planning tool within the proposed integrated land and water management model framework. Given the above complexities, and the data and time requirements of the above models, this study will develop a methodology to determine the optimal mix of water-consuming activities to maximize the net income of the agricultural production of a catchment and the water demands under various prices and water amounts and qualities. Section 2.6 gives a detailed discussion on the selection of the planning model needed to make decisions on optimized use of the land and water resources as part of the proposed framework.

2.3 DECISION SUPPORT SYSTEMS

Decision Support Systems (DSS) are powerful management tools that can support planners and decision-makers in the decision making process through generating and evaluating alternative solutions and scenarios to solve unstructured problems (Dutta, 2003). Given the development in management and planning, the growing number of qualitative and quantitative disciplinary models, and the advances in information technology, the main question is how to make effective and efficient use of all the information to achieve sustainable outcomes and development and how to share the limited water resources (Sharifi, 2003).

Singh (2004) reported that a DSS is composed of several models such as a database management system, data sources system, knowledge management system and includes query, display and analysis system for decision makers. Decision support models also involve modules that utilize analytical methods, decision analysis, optimization algorithms, and program scheduling routines. The above models can help decision makers to describe modeling process, formulate alternatives, analyze their impacts and select appropriate alternatives and if possible implement them (Adelman 1992).

Little (1970) proposed a DSS be a model based on a set of procedures for processing data and judgments to assist managers in their decisions making. A definition of a DSS or decision support models that is acceptable to everybody is not available at this moment mainly due to the fact that decision support systems are being studied by various scientific disciplines of different interests. In general the main goal of a DSS is to improve the quality of the decision to be made. Densham (1991) suggests that DSS has six characteristics including explicit design to solve unstructured problems; powerful and easy to use; ability to flexibly combine analytical models with data; ability to explore the solution space by building alternatives; capability of supporting a variety of decision-making styles and allowing interactive and recursive problem solves.

Andreu et al. (1996) designed a DSS for the planning stage associated with complex river basins. The modeling capability includes basin simulation and optimization modules. The Segura and Tagus river basins have been used as case studies in the development and validation phases. Dunn et al. (1996) developed DSS model to provide a quantitative description of the main economic and environmental impacts arising out of rural basin scale. The system integrates models of economics and hydrology with spatial databases, thereby permitting interactive evaluation of different future scenarios through a graphical user interface.

Decision support models are used to support decision processes like management information systems, data base management systems and some knowledge-based systems (Singh, 2004). The main difference between DSS and other information systems lies in the numeric model component. Formal quantitative models such as statistical simulation, logic and optimization models are an integral part of a DSS (Bell 1992). These models are used to represent the decision problem, their solutions are decision alternatives.

In Canada, Bender and Simonovic (1994) stated that DSS allowed reducing costs associated with promotion of strategies by seeking an adequate level consensus before proceeding with design and project implementation. They propose that DSS are likely to increase the efficiency of data collection through early clarification of issues in the planning process. Catchment decision support modeling is used by Hann et al. (1982) to simulate the hydrological process and system dynamics that takes place in a natural catchment to gain a better understanding of hydrological phenomena occurring in the catchment and to generate synthetic sequences of hydrologic data for forecasting future decision.

Spatial Decision Support Systems (SDSS), which are the integration of DSS and GIS are emerging as efficient tools for managing natural resources like land and water. Dutta (2003), applied AVSWAT tool in digitally delineating catchments in a block of Bankura district of West Bengal and then used it for estimating potential water, silt and crop yield from each of them. This was helpful in prioritising the catchments and presenting the results spatially for the district level decision makers. AVSWAT (Arc View- SWAT), a user- friendly PC based SDSS tool has been developed at the Black Land Research Center, Temple, Texas, USA integrating Soil and Water Analysis Tool (SWAT) and Arc View GIS version 3.0a software along with Spatial Analyst Version 1.1 extension. SWAT is a continuous time river basin or catchment scale model operating on daily time step.

However the emphasis of decision support models should focus on the use of existing knowledge and tools for sustainable catchment management and to reach an appropriate decision. Within a catchment there are a variety of stakeholders relying on water as natural resource and their activities have a direct influence on water quality and quantity. The needs and impacts of each stakeholder on the natural functions of a catchment need to be understood for making decisions regarding resource management and sustainable development. This also asks for a clear identification of the steps involved and the detailing of the requirements that adhere to specific legislation, regulation and policy all of which are a requirement for an effective management plan. Therefore, a key to effective water management is to have in place an enabling decision support system model, at a catchment scale and multi-stakeholder partnerships at all levels to facilitate the process of decision analysis. This model should reinforce the fact that nothing happens in isolation and that everything is connected by the land and water within the catchment. Accordingly a decision analysis model needs to be integrated within a decision support system framework.

2.4 MULTI-CRITERIA DECISION ANALYSIS FOR INTEGRATED LAND AND WATER MANAGEMENT

Traditionally, decision analysis in environmental planning and management scenarios is performed using cost-benefit analysis. However its use has decreased in solving environmental problems (Lahdelma et al., 2000). Khadam and Kaluarachchi (2003), investigated the limitations associated with cost-benefit analysis and proposed a methodology that aimed at reformulating the decision problem of groundwater remediation in a multi-criteria decision setting rather than the currently employed risk-cost-benefit analysis framework. According to above authors the main limitations of the cost-benefit analysis were its definition of risk, its definition of cost of risk, and its poor ability to communicate risk-related information.

In general, multi-criteria decision analysis evaluates a utility that expresses a decision maker's outcome preference in terms of multiple criteria. Multi-criteria decision analysis decomposes the complex problem of assessing a multiattribute utility function into one of assessing a series of unidimensional utility functions (Ascough et al., 1996). A criterion is a characteristic of the management alternatives that the decision maker considers important.

A large number of multicriteria decision making methods have been proposed in the past and applied to water resources planning. The development and application of systems analysis techniques to assist decision makers in evaluating project alternative having more than one objective in river basin planning is of recent origin (Raj, 1995). Zardari and Cordery (2006), considered water allocation as multicriterion decision problem and used ELECTRE I (Roy, 1991), a multicriterion decision aid, to determine preferences among five watercourses by considering socio-economic objectives. The use of ELECTRE I in this study is justified by the involvement of stakeholders, who assign weights to the criteria of their own choice. The results indicated a most efficient water user that should be given priority water allocations. Morais and Almeida (2006) described the application of multicriteria decision aid for choosing the priority city to receive a water supply system using the ELECTRE methodology integrating weighted qualitative judgment criteria, incorporating the concordance and discordance indices, specifying an efficient allocation of resources available and thus maximising gains. Raj (1995) used different ELECTRE techniques for water resources planning in one of the major river basins (Krishna river basin) in India and concluded that changing the weights assigned to each criterion had greater effect on the results than does the scales of criteria but neither effect was significant recommending the use of the ELECTRE for screening and ordering of alternatives. Abu-Taleb and Mareschal (1995) described the application of the PROMETHEE V multicriteria method to evaluate and select from a variety of potentially feasible water resources development options, so that the allocation of limited funds to alternative development projects and programs can proceed in the most efficient manner. Important policy issues such as environmental protection, water demand and supply management, and regional cooperation could be explicitly considered using the multicriteria procedure.

Warren (2004) reviewed the uncertainties associated with one of the most commonly used techniques for decision analysis, the Analytic Hierarchy Process (AHP) (Saaty, 1980) and concluded the AHP primary category of problems are: scale misinterpretation, comparison matrix evaluation, and multiple normalisations in hierarchical aggregation of priorities. Moreover, it has been shown that the axiomatic foundations of AHP are also questionable.

Multicriteria analysis had been adopted by Latinopoulos and Mylopoulos (2005) to maximize the farmer's welfare and the minimization of the consequent environmental burden. Weighted goal programming techniques were employed and implemented on a representative area in the Loudias River Basin in Greece to seek a compromising solution for area and water allocation under different crops. The analysis was undertaken under different policy scenarios and the results showed figures that were as close as possible to the decision maker's economic, social and environmental goals. Several weights were assigned based on the set goals according to the intentions of the decision maker that were likely to differentiate the final allocations of resources.

Almasri (2003) developed protection alternatives for management of nitrate contamination of Sumas-Blaine groundwater aquifer in the United States. Decision criteria were developed and utilized in a multi-criteria decision analysis. Each protection alternative was evaluated in terms of different decision criteria.

Khadam and Kaluarachchi (2003) explored two methods for ranking different alternatives for groundwater remediation, the approach of importance of the order of criteria (Yakowitz et al., 1993), and a fuzzy logic approach (Kaufman, 1975) based on fuzzy dominance and resemblance analysis (FDR). They found that the Importance Order of Criteria (IOC) method provided a ranking of alternatives comparable to that of an explicit two-stage decision analysis that has a filtering stage followed by a selection stage. The filtering stage rejects the alternatives that do not match the decision criteria and the selection stage ranks the filtered alternatives in a detailed manner for the final selection. The FDR method did not perform satisfactorily compared to the IOC method.

A major drawback of previous research conducted on different ranking methods of decision criteria is the complexity of computations and difficulty of implementation in addition to the associated assumptions of certain methods like the existence of fuzzy relation across the alternatives (Raj and Kumar, 1998), the biased weights of criteria given by decision-makers (Vincke, 1992).

In this study, a multi-criteria decision analysis methodology suggested by Khadam and Kaluarachchi (2003) that is based on the IOC will be adopted and utilized in finding the dominating alternative out of the set of alternatives. The IOC methodology is developed by Yakowitz et al., (1993) and is conceptually simple and provides the decision maker with clear graphical evidence if one alternative is strongly dominant over another. Almasri and Kaluarachchi (2005) reported that the IOC method in multi-criteria decision analysis is a straightforward and efficient method for decision analysis and allowed for ranking different alternatives for groundwater protection based on the preference order of the decision criteria.

2.5 DECISION SUPPORT SYSTEMS FOR INTEGRATED LAND AND WATER MANAGEMENT

Application of the concept of integrated land and water management has been widely accepted as practical solution for sustaining both the environment and agricultural production. Such an integrated approach aims to enhance income and food security in rural households, while maintaining a sustainable environmental and economic resource base. It will also characterize the interactions between human and natural resource systems, identify key constraints and opportunities with respect to resource use and existing production systems and design and test interventions for improved management of land and water resources.

Integrated water resources management (IWRM) calls for the integrated management of land and water resources and other related natural resources in a coordinated approach that aims to maximize socio-economic welfare in an equitable manner without sacrificing the sustainability of the ecological systems. According to ESCWA (2005), IWRM tools are grouped into three main categories:

- Enabling environment through policies, legislation and financing structures ;
- Institutional roles through organizational framework and institutional capacity building for developing human resources; and
- Management instruments through water resources assessment, plans for IWRM, demand management, social change instruments, conflict resolution, regulatory instrument, economic instruments and information management and exchange.

One of the main challenges confronting the water sector in arid and semi-arid regions is securing adequate food supply that could be addressed through optimal water allocation; better selection of crops in terms of economic return; water consumption; improve irrigation and application of water saving technology and determine water allocation among sectors according to needs, economic return and trade-offs between surface water, groundwater and treated wastewater (ESCWA, 2005).

Given the relationships and interdependencies that exist between land, water and various stakeholders, a comprehensive, all-inclusive approach to considering the factors affecting water resources within a catchment needs to be clearly understood. Any decisions regarding water resource management must be done in a socially, environmentally, and economically sustainable manner (Chander, 2005).

Integrated catchment management framework links physical, social and economical sciences into planning, policy and decision making. Specifically, prioritizing and evaluating different management alternatives through a multi-criteria decision analysis model is essential to achieve a long–term agricultural and natural resources sustainability in agriculture-dominated catchments. Such a framework forms the basis for an integrated land and water management model and will be developed under the scope of this study.

Despite the rapid development in remote sensing, GIS and information technology, there is still a missing link between all these development and proper management of the resources. The link could be established through integration of all the relevant knowledge, experiences, technologies and information into decision support system to sustain integrated management of natural resources. As discussed earlier, previous research focused on different approaches to optimal water allocation and decision support systems that utilize one or more physical models. The current study addresses integrated management of agriculture-dominated catchments in arid and semi-arid regions. Figure 2-1 shows a general pictorial illustration of the integrated natural resources management framework. As mentioned in Section 1.4 Faria catchment in West Bank, Palestine will be taken as a case study. The developed framework should be transferable and useful to other areas in the region as well as worldwide. Based on the developed framework, a multi criteria decision support system (MCDSS) will be developed tp provide decision makers with a tool to evaluate different management alternatives and ultimately select the best management alternative.

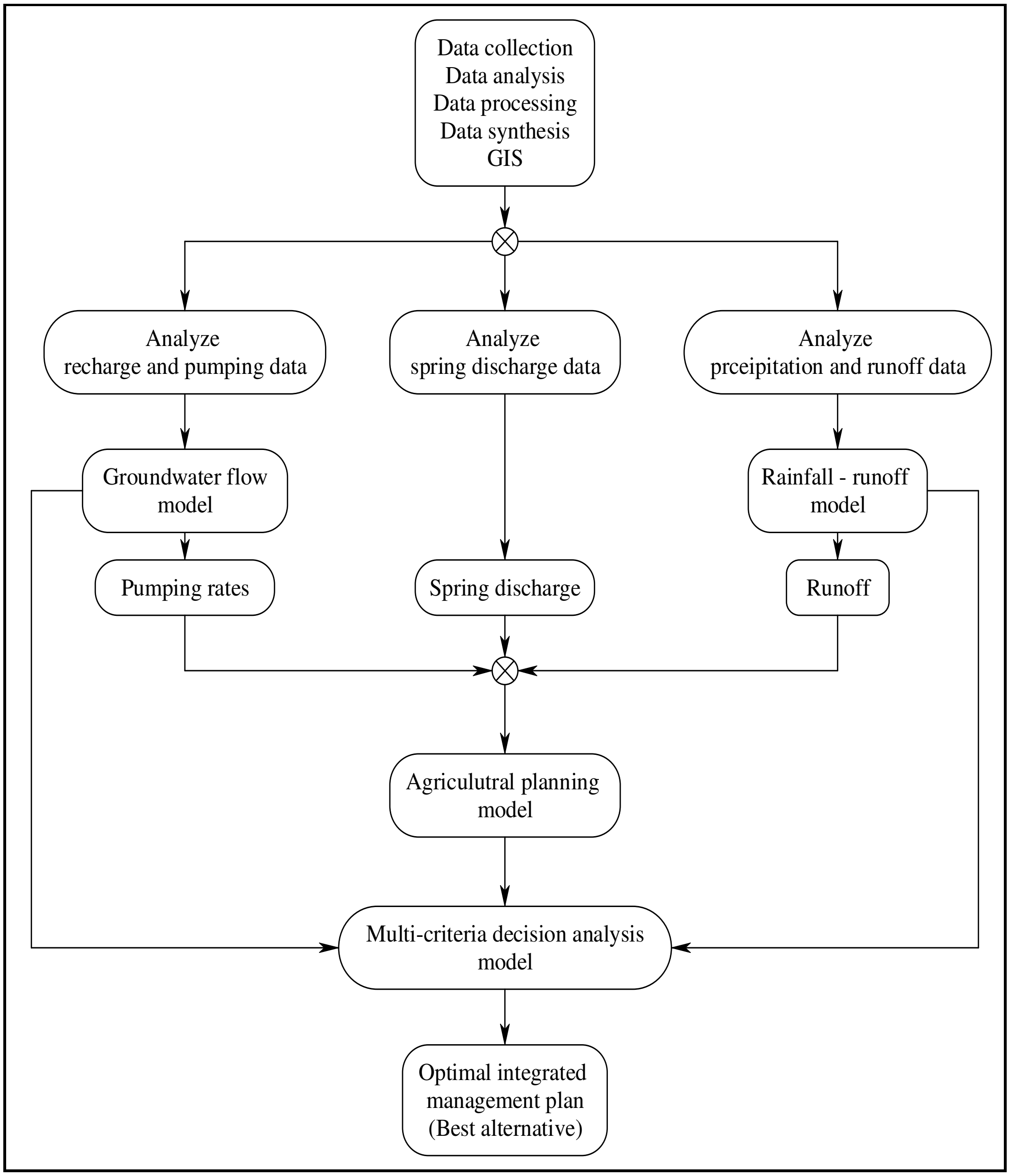

Figure 2-1 Conceptual illustration of the proposed integrated land and water management framework for agricultural dominated semi-arid catchments

Land and water resources are poorly and separately managed in the study area. Over abstraction of groundwater increased salinity levels. Surface runoff is not being utilized due the lack of storage facilities. Springs' water is a major source of irrigation water for the farmers. Existing cropping patterns are rigid in response to changing conditions including the ongoing shifts in the market demand and supply sides, available water quality, price and quantity. The decision-making processes for the management of these two resources are independent from each other. Importantly though, interaction between management of these resources does exist. An Integrated land and water management approach is proposed in this study to develop a multi-criteria decision analysis framework capable of addressing such an interaction in a way that sustains both the scarce natural resource and agricultural production. This framework involves all stakeholders, promotes co-operation and collaboration and builds a sense of community participation, helps reduce conflicts, increases commitment to actions that are necessary to meet the environmental goals, and ultimately improves the likelihood of successful and sustainable development programs.

In order to achieve an optimal use of land and water resources the study essentially needs to estimate available water resources including a quantification of runoff volume, estimation of the groundwater safe yield and investigating the reliability of springs' yield. Following a proper modeling of the available water resources, an analysis of agricultural water demand and optimal cropping patterns needs to be analyzed to maximize benefits out of the available resources. Further, the study concentrates on developing management alternatives that sustain the available water resources both in quantity and quality, optimize the use of low quality water including treated effluent and brackish water, and maximize both the irrigated areas and the income of the local farmers. Since the decision criteria involve conflicting objectives, a multi-criteria decision analysis is employed using the IOC method suggested by Yakowitz et al. (1993). Once this proposed management framework is implemented, it is expected that sustainable development conditions are created subject to the proper selection of the management alternatives. In the following sections, a detailed review of the models selection approach is presented.

2.6 SELECTION OF RAINFALL RUNOFF MODEL

Arid areas are those in which rainfall is not sufficient for regular crop production, semi-arid areas are those in which the rainfall is sufficient for short-season crops and where grass is an important element of the natural vegetation (Chow, 1964). In arid and semiarid regions, handling runoff problems as a resource management will improve the sustainability of available water resources by addressing water storage problems.

The nature of stream flow in a catchment is strongly related to the rainfall characteristics and catchment geomorphology. The excess rainfall depends on the temporal and spatial distribution of rainfall. The geomorphic characteristics are the channel network, topography, and surrounding landscape, which transform the rainfall input into an output hydrograph at the outlet of the catchment.

In deciding the management options for optimal use of land and water it is important to determine runoff volume that could be harvested from a storm for irrigation purposes. Furthermore, it is important to estimate the peakflow from a storm for infrastructure development for transport and storage of surface runoff or for recharge of groundwater. The basic approach will involve the application of a simple rainfall-runoff model to determine the streamflow yield as well as the peakflow based on climatic data and catchment geomorphological characteristics. The rainfall-runoff model developed for the study area is essentially needed for any water resources design work as part of proposed management options that would be applied to the study area.

In arid and semi-arid areas most of the catchments are ungauged. Accordingly it is important to develop a simple rainfall-runoff model to predict stream flow from agricultural catchments in arid and semi-arid areas. The discussion on model selection concentrates only on rainfall-runoff modeling from small ungauged catchments in arid and semi-arid areas.

Zhu et al. (1999) stated that the major barriers to modeling hydrological processes in semi-arid and arid areas are due to a lack of understanding and model representations of the distinctive features, processes associated with runoff generation and paucity of field data.

In general, continuous simulation models developed worldwide (Australia, USA, UK, Europe, others) need parametric calibration in order to work for local catchments conditions other than the areas where such models had been developed or calibrated. Furthermore, few studies have attempted to relate model parameters to catchment's physiographical characteristics (Jayasuriya, 1991). However, the lack of observed data required for the calibration of such models makes it difficult to successfully apply such models in ungauged situations.

Boughton (2005), reviewed different continuous water balance models applied in Australia for the last 45 years and concluded that almost all use of continuous models in Australia is where stream flow data are available for calibration. He also reported that a number of studies (Johnston and Pilgrim, 1976, Mein and Brown, 1978, Nathan and McMahon, 1990, Nathan et al., 1996, Weeks and Ashkanasy, 1985) have attempted to relate model parameters to land use and establish procedures for applying these models to ungauged catchments, with little success. Johnston and Pilgrim (1976) attempted to derive optimum values for the Boughton model (Boughton, 1966) parameters with an objective of relating the optimum values to measurable catchment characteristics, showing very clearly the problem of interrelationships among model parameters and the low sensitivity of some objective functions to a wide range of combinations of parameter values. Mein and Brown (1978) tested the sensitivity of each parameter in a modified version of the AWBM model (Boughton, 1966) to assess the potential for predicting the effects of land use change on runoff, reporting that, while the model performed well in relating runoff to rainfall in a calibration situation, the parameters could not be related to catchment characteristics with sufficient confidence to assess the effects of land use change. Nathan and McMahon (1990), calibrated the SFB model (Boughton, 1984) on 168 catchments in New SouthWales and Victoria, finding no correlations between values of the calibrated parameters and measurable characteristics of the catchments. Nathan et al. (1996) tested the 2-parameter MOSAZ model (Jayasuriya, 1991) on 195 catchments in Victoria, reporting the lack of physical basis of the developed equations to be able to predict model parameters in catchments that lie outside the geographic and physical limits used in their derivation. Weeks and Ashkanasy (1985) attempted to relate parameters of the Sacramento model (Burnash et al., 1973) to catchment characteristics in a group of 8 subcatchments, 88–880 km^2 in area, in the Lockyer Valley in southeast Queensland, with very variable results discouraging any further work.

The process of regionalization of parsimonious (fewer than about 7 parameters) rainfall-runoff model parameters involves considerable uncertainties emerging from the combined uncertainty in the relationship between catchment attributes and model parameters for the gauged catchments, together with the uncertainty in the catchment attributes and climatic data used for the ungauged catchments (Croke and Norton, 2005).

Yair and Kossovsky (2002) collected hydrological data from two instrumented catchments located in an arid rocky area and in a semi-arid soil covered area. They revealed that runoff generation rates in arid and semi-arid areas are primarily controlled by surface properties rather than by the absolute amounts of storm and annual rain amounts. Furthermore, they concluded that similar regional climatic conditions may have different effects on the hydrological response of different adjoining surfaces. The surface runoff generated was strongly controlled by the specific local surface conditions that prevailed in the area prior to the storm.

The US Department of Agriculture, Soil Conservation Services (SCS) runoff curve number (CN) (SCS, 1972) is widely used to estimate runoff from ungauged catchments. The SCS runoff equation was developed to estimate total storm runoff from total storm rainfall. The CN is an index based on physical parameters of the catchment. This method is able to reflect the effect of changes in landuse on runoff, known for its simplicity and can be applied to gauged as well as ungauged catchments.

In this study, the SCS method was initially applied for the study area to test the applicability of this method for runoff modelling as shown in Appendix I. The results showed that the SCS method overestimated the runoff amounts and is not suitable to be used for the estimation of excess rainfall for the study area. These results agree with other published research related to the use of SCS method (Mishra and Singh, 2004; SCS, 2004; Maidment, 1993). The SCS relationship generally did reasonably well where the runoff was a substantial fraction of the rainfall, but poorly in cases where the runoff was a small fraction of the rainfall; i.e., the CNs are low or rainfall values are small (SCS, 2004; Maidment, 1993). Curve numbers were originally developed from annual flood flows from experimental catchments in the United States, and their application to low flow conditions or for small peak flows is not recommended (SCS, 2004). Mishra and Singh (2004) applied the SCS CN method to a large set of event data in the Amicolala Creek catchment, the SCS CN method generally overestimates the runoff (Mishra and Singh, 2004). The drawbacks of the SCS curve number method for estimating yield from ungauged catchments were also reflected in results reported by many researchers (Jayasuriya, 1991, Boughton, 1984, Boughton, 2005, Boughton and

Droop, 2003). Australian Water Resources Council (1971) made a study of the SCS Curve Number method to test its ability for estimating runoff from ungauged catchments in Australia. The test results were not of sufficient accuracy to encourage further testing or development (Boughton, 2005) for Australian conditions.

Few studies (Boughton and Droop, 2003; Boughton et al., 2000; Singh and Woolhiser, 2002) have compared flood estimates by continuous simulation with those estimated by event-based models. Boughton et al., (2000) applied both continuous and event-based models to estimate flood peaks on three catchments of 62, 108, and 259 km^2, in Victoria, and concluded that the flood frequency estimates were rated as reliable for the 2-10 years average recurrence interval floods. Singh and Woolhiser, (2002) reported that except for the three World Meteorological Organization (WMO) studies on intercomparison of catchment hydrology models, no comprehensive effort has been made to compare models. The first WMO study dealt with conceptual models used in hydrologic forecasting, the second WMO study dealt with an intercomparison of models used for simulation of flow rates including snowmelt, and the third WMO study addressed models for forecasting streamflow in real time (Singh and Woolhiser, 2002).

RORB (Laurenson et al, 2005) is a flood hydrograph model in widespread use in Australia. The program provides an event-type modeling procedure, where rainfall is operated on by a loss model to produce rainfall-excess. In flood estimation applications, the program may be used on rural, urban or partly rural and partly urban catchments. It is mostly used for design flood investigations (Laurenson et al., 2005). Dyer et al. (1993), investigated the regionalization of the RORB parameters through determining prediction equations for the RORB parameters. Results indicated that the choice of loss model affected the model parameters and no relationship between catchment characteristics and the optimal values of the model parameters could be derived. Watershed Bounded Network Model (WBNM) is an event based nonlinear runoff routing model for calculating a flood hydrograph from a rainfall hyetograph (Boyd et al., 2001). The model structure is based on the geomorphology of the catchment and includes the storm temporal patterns for all Australian rainfall zones. The Urban Runoff and Basin System (URBS) runoff routing model is based on a network of subcatchments whose centroidal inflows are routed along a prescribed routing path to generate runoff (Carroll, 1994). The model requires that the stream length is specified to define the extent of catchment routing, and that catchment area is specified to determine excess rainfall.

However these models have been developed for Australian conditions and have not been checked in arid and semi-arid conditions. Furthermore they use a representative (single not probability distributed) value of losses in design flood estimation based on the Australian Rainfall and Runoff recommended loss values, which has a wide range and it is generally difficult to select a single representative value (Ilahi, 2005).

The unit hydrograph approach can be applied for rainfall-runoff modeling for all types of catchments including arid and semi-arid regions. Historical rainfall-runoff data for unit hydrograph derivation have been widely applied for the estimation of design flood hydrographs from gauged catchments where the unit hydrograph is derived from observed runoff data minus baseflow and records of rainfall minus abstractions (Maidment, 1993). For ungauged catchments, synthetic unit hydrographs may be derived by means of relationship between parameters of a unit hydrograph model and the physical characteristics of the catchment (Maidment, 1993). A number of synthetic unit hydrograph approaches are available including models of Snyder (1938,) Clark (1943), US Soil Conservation Service (SCS, 1972) and Geomorphological Instantaneous Unit Hydrograph (GIUH) (Lee, 1998).

The unit hydrograph is a surface runoff hydrograph resulting from one unit of rainfall excess uniformly distributed spatially and temporally over the catchment for the entire specified rainfall excess duration. If the duration of rainfall excess in infinetly small the unit hydrograph is called an instantaneous unit hydrograph (IUH) (Chow, 1964). Many investigators (Jain et al., 2000; Lee, 1998, Rodriguez-Iturbe and Valdes, 1979 and Gupta et al., 1980) have endeavored to relate the Instantaneous Unit Hydrograph (IUH) parameters to the catchment geomorphology and thus obtain the GIUH model. Jain et al. (2000), applied the GIUH for the estimation of design flood of the Gambhiri dam catchment in India, observing that the peak characteristics of the design flood are more sensitive to the various storm patterns as well as method of critical sequencing followed for the computation of design storm patterns. A unifying synthesis of the hydrologic response of a catchment to surface runoff through linking the Instantaneous Unit Hydrograph with the Geomorphologic parameters of a basin, had been studied by Rodriguez-Iturbe and Valdes (1979), concluding that the structure of the hydrologic response is intimately linked to the geomorphological parameters of a basin and that the IUH varies from storm to storm and throughout the same storm as a function of the velocity which occurs in the different instances of time throughout the basin. Gupta et al. (1980), employed the channel network of a river basin and the overland flow regions in a kinetic theoretic framework for obtaining an explicit mathematical representation for the IUH at the basin outlet. The theory provided excellent agreement for two basins with areas of the order of 1770 km^2 but underestimates the peak flow for the smaller basin with 483 km^2, which could be related to the linearity assumption in the rainfall runoff transformation embedded in their development.

A catchment can be separated into a series of runoff states, and the catchment hydrologic response can be considered to be a function of the runoff path probabilities and runoff travel time probabilities in different runoff states (Lee and Chang, 2005). Geomorphology based instantaneous unit hydrographs have been also applied by several engineers (Snell and Sivapalan, 1994; Sorman, 1995; and Hall et al., 2001) to predict runoff from rainfall for ungauged catchments. Snell and Sivapalan, (1994) used the GIUH capability to estimate floods for ungauged streams by using the information obtainable from topographic maps or remote sensing possibly linked with the Geographic Information Systems (GIS) and Digital Elevation Model (DEM). Sorman, (1995) applied GIUH model to predict the hydrograph characteristics of several semi-arid basins in the Kingdom of Suadi Arabia. IUH peak discharges were estimated for three basins with different sizes varying between 19.5 km^2 and 600 km^2. The errors were computed between the estimated peak discharges and time to peak, with the respective observed records, showing reasonable results with minimum errors varying between 18% and 34% for the peak discharge estimation and around 22% to 32% for the time to flow peaks. Hall et al. (2001), divided the rainfall excess duration into several (equal) time increments, with separate IUHs being generated for each interval. This quasi-linear approach was applied to 105 storm events from nine catchments in the south-west of England, ranging in size from 6 to 420 km^2. The results showed that, providing the time interval chosen is fine enough to capture the shape of the runoff hydrographs, a comparable level of goodness-of-fit can be obtained for catchments covering a range of about 1:75 in area, and recommended further investigation for the modified GIUH approach as described, and intercomparison with regression-based regionalisation methods.

Selection of rainfall runoff models for the study area

The West Bank, Palestine is considered as semiarid and has the Mediterranean type climate. Rainfall in the West Bank is limited to the winter and spring months from November to April. During the summer months, there is no rainfall. Rainfall excess drains into streams forming intermittent streams which go dry between rainfall events (ephemeral streams). The streams are much above the groundwater level and percolate to the groundwater while flowing. There are few continuous flowing streams that are fed by groundwater springs. These springs are mostly located and flowing in the eastern basin towards the Jordan River and the Dead Sea (Shaheen, 2002).

A number of studies have been carried out to analyze storm water problems in the West Bank. Husary et al. (1995) analyzed the rainfall data for the northern West Bank. They investigated the relationship between rainfall and runoff in Hadera catchment and found that the ratio of runoff to rainfall ranges from 0.1% to 16.3% with an average of 4.5% for the period 1982/83 to 1991/92. The spatial interpolation of the rainfall data of the northern West Bank was studied by Sabbobeh (1998). He investigated several interpolation techniques for the daily rainfall data from 28 stations. Al-Nubani (2000) studied the temporal characteristics of the rainfall data of Nablus meteorological stations. By correlating the occurrence of runoff in Rujeeb catchment east of Nablus to the total rainfall values, he concluded that runoff occurs when total rainfall exceeds 48 mm distributed over less than 15 hours duration. In the above study, the rainfall events that have caused runoff have a maximum intensity of about 10mm/hr. Al-Nubani (2000) reported that the runoff to rainfall ratio is 13.5%. The rainfall-runoff process of a 167 km^2 catchment in Jerusalem district has been studied by Barakat (2000). He analyzed the rainfall and runoff data of the upper Soreq catchment and developed the unit hydrograph related to four recorded events. The resulted runoff to rainfall ratio was averaged at 0.3%. Takruri (2003), developed synthetic unit hydrograph for Eastern Slope Areas of the West Bank and indicated that runoff is likely to occur in these area when total annual rainfall exceeds 200 mm and daily rainfall exceeds 50 mm in one day or 65-70 mm in two days.

As the main objective of the study is to develop a framework for integrated land and water management, the selected rainfall-runoff model need to be practical and readily usable in planning and decision making. It will also have to be interfaced with economic, social, political and administrative models. Since the major concern for the proposed decision support system is the optimal management of available water resources quantities throughout the year, runoff will be estimated, based on the available rainfall records. On average, only very few rainfall events produced runoff in semi-arid atmosphere such as the Faria catchment. The uncertainty that will be associated with parameter estimation in complex continuous models is seen to be too high due to the short data records of most of the catchments and the single events characteristic of the rainfall in semiarid areas. As such an event based rainfall runoff model which is applicable to ungauged catchments will be required in estimating runoff in the decision making process for the best management alternative. The KW-GIUH model has several characteristics that made it useful to predict runoff from ungauged catchments in arid and semi-arid area including;

(1) user-friendly program;

(2) a process- based physically sound model; and

(3) the model doesn’t require detailed parameters that are site specific and requires field investigation but rather utilizes generic parameters obtained from literature.

In this study the KW-GIUH model is selected to determine streamflow yield from Faria catchment. KW-GIUH rainfall-runoff model will require climatic data, and catchment geomorphological characteristics in estimating yield from an event. If available, catchment geomorphological characteristics could be obtained from GIS to apply into the KW-GIUH model.

2.7 SELECTION OF GROUNDWATER MODEL FOR THE STUDY AREA

Groundwater modeling can help analyze many groundwater problems. Groundwater models are useful for exploration studies preceding field investigations, for interpretive studies following the field program, and for predictive studies to estimate future field behavior. In addition, groundwater models improve the ability to reliably predict the rate and direction of groundwater flow and contaminant transport which is a critical issue in planning and implementing groundwater remediation. Therefore, groundwater models are tools that can aid in studying groundwater problems and can help increase our understanding of groundwater systems. Another valuable use of groundwater models is in the management of groundwater resources. The management of any system means making decisions aimed at achieving the system’s goals without violating specified technical and nontechnical constraints imposed on it. The output (e.g., minimize cost and maximize effectiveness of remediation or utilization) usually depends on both the values of the decision variables (areal and temporal distributions of pumpage) and on the response of the aquifer system to the implementation of these decisions. Constraints are expressed in terms of future values of state variables of the considered groundwater system such as water table elevations and concentrations of specific contaminants in the water. A typical constraint may be that the water level at a certain location should not drop below a specified level.

The equations that describe the groundwater flow and fate and transport processes may be solved using different types of models (Welsh, 2007). Some models may be exact solutions to equations that describe very simple flow or transport conditions (analytical model) and others may be approximations of equations that describe very complex conditions (numerical models). In selecting a model for use at a site, it is necessary to determine whether the model equations account for the key processes occurring at the site. Analytical models are an exact solution of a specific, greatly simplified, groundwater flow or transport equation. The equation is a simplification of more complex three-dimensional groundwater flow or solute transport equations. This resulted in changes to the model equations that include one-dimensional uniform groundwater flow, simple uniform aquifer geometry, homogeneous and isotropic aquifers, uniform hydraulic and chemical reaction properties, and simple flow or chemical reaction boundaries. Analytical models are typically steady-state and one-dimensional. Well hydraulics models, such as the Theis or Neumann methods, are examples of analytical one-dimensional groundwater flow models (Domenico and Robbins, 1985). Because of the simplifications inherent with analytical models, it is not possible to account for field conditions that change with time or space. This includes variations in groundwater flow rate or direction, variations in hydraulic or chemical reaction properties, changing hydraulic stresses, or complex hydrogeologic boundary conditions.

Numerical models are capable of solving the more complex equations that describe groundwater flow and solute transport. These equations generally describe multi-dimensional groundwater flow, solute transport and chemical reactions, although there are one-dimensional numerical models. Numerical models use approximations (e.g. finite differences, or finite elements) to solve the differential equations describing groundwater flow or solute transport. The approximations require that the model domain and time be discretized. In this discretization process, the model domain is represented by a network of grid cells or elements, and the time of the simulation is represented by time steps. The accuracy of numerical models depends upon the accuracy of the model input data, the size of the space and time discretization (the greater the size of the discretization steps, the greater the possible error), and the numerical method used to solve the model equations. In addition to complex three-dimensional groundwater flow and solute transport problems, numerical models may be used to simulate very simple flow and transport conditions, which may just as easily be simulated using an analytical model. However, numerical models are generally used to simulate problems which cannot be accurately described using analytical models (Anderson and Woessner, 2002).

The Murray-Darling Basin Commission (MDBC, 2000) provided detailed guidelines for best-practice groundwater flow modeling. A groundwater model provides a scientific means to draw together the available data into a numerical characterization of a groundwater system. The model represents the groundwater system to an adequate level of detail, and provides a predictive scientific tool to quantify the impacts on the system of specified hydrological, pumping or irrigation stresses.

The U.S. Geological Survey modular finite-difference groundwater flow model, which is commonly called MODFLOW, can simulate groundwater flow in a three-dimensional medium. The revised MODFLOW is referred to as MODFLOW-96 (Harbaugh and McDonald, 1996).

MODFLOW is a computer program for simulating common features in groundwater systems (Harbaugh and McDonald, 1996). The program was constructed in the early 1980's and has continually evolved since then with development of many new packages and related programs for groundwater studies. Currently, MODFLOW is the most widely used program in the world for simulating groundwater flow (Almasri, 2003). The model is public domain, and is applicable to a variety of field conditions. It has been widely accepted by practitioners and researchers alike and was applied in numerous field-scale modeling studies (Almasri, 2003). The popularity of the program is attributed to the following factors:

- The finite-difference method used by MODFLOW is relatively easy to understand and apply to a wide variety of real-world conditions.
- MODFLOW works on many different computer systems ranging from personal computers to super computers.
- MODFLOW can be applied as a one-dimensional, two-dimensional, or full three-dimensional model.
- Each simulation feature of MODFLOW has been extensively tested.
- Data input instructions and theory are well documented.

- The modular program design of MODFLOW allows for new simulation features to be added with relative ease.

MODFLOW is designed to simulate aquifer systems to simulate a wide variety of hydrologic features and processes. Steady-state and transient flow can be simulated in unconfined aquifers and confined aquifers. MODFLOW simulates groundwater flow in aquifer systems using the finite-difference method. In this method, an aquifer system is divided into rectangular blocks by a grid. The grid of blocks is organized by rows, columns, and layers, and each block is commonly called a "cell". For each cell within the volume of the aquifer system, the user must specify aquifer properties. Also, the user specifies information relating to wells, rivers, and other inflow and outflow features for cells corresponding to the location of the features. MODFLOW uses the input to construct and solve equations of groundwater flow in the aquifer system. The solution consists of head (groundwater level) at every cell in the aquifer system as well as outflow from the aquifer system (Harbaugh and McDonald, 1996).

In a number of studies the MODFLOW model had been applied to investigate groundwater responses under irrigation districts and from groundwater pumping schemes (Almasri, 2003). Jagelke and Barthel (2005) utilized MODFLOW for groundwater modeling within a framework of integrated regional model that included joint modelling of surface and subsurface systems to assess the quantity and quality of the water resources in Neckar catchment in Germany. The developed MODFLOW groundwater model results gave a better understanding of the groundwater recharge processes and evaluated the groundwater resources on catchment scale.

Shigidi (2000) utilized MODFLOW to develope an iterative Artificial Neural Network (ANN) to solve the inverse problem in groundwater flow in heterogeneous confined aquifers. MODFLOW was used to compute the corresponding potentiometric head distribution. Al-Murad (2002) utilized MODFLOW for groundwater modeling in order to prepare the input-output response patterns where dispersivity was generated randomly from a prespecified range and the corresponding concentration distribution was simulated. Results showed that the model successfully gave the required output needed to estimate the dispersivity values with high accuracy.

Yusoff (2002) coupled a MODFLOW numerical groundwater model with a climate change model created at the United Kingdom's Hadley Centre for Climate Prediction and Research to evaluate changes in river baseflow, groundwater recharge, and groundwater levels in the Chalk aquifer of west Norfolk, United Kingdom. The MODFLOW successfully simulated the head levels and groundwater flows under climatic change.

The study area includes one unconfined upper and two confined lower aquifer systems. Groundwater is found in formations of Pleistocene to Lower Cenomanian age, at depth ranging from several hundred of meters to many meters. In the area under investigation, five sub-aquifers are located within unconfined and confined strata. These aquifers are the unconfined Pleistocene, Neogene and Eocene and the confined upper and lower Cenomanian sub-aquifers. The Neogene sub-aquifer consists of well cemented conglomerates and contains a small amount of fresh water. It is composed of Beida formation and conglomerate lenses, marl and clay of the Lower Tertiary. The Eocene sub-aquifer consists mainly of nummulitic limestone with chalks, chert bands and marl. The limestone is thin bedded with chalk, chert and marl intercalation (Rofe and Raffety, 1965).

Groundwater aquifers are usually utilized through springs and wells. There are 11 springs in Faria catchment (WESI, 2005). Most of these springs are located in the upper and middle parts of the stream. There are 69 wells in the study area; of which 61 are agricultural wells. Based on the data available the average total utilization of Palestinian wells is 8300 ML/year (WESI, 2005). Water from irrigation wells is used in conjunction with spring discharge in most of the catchment. Palestinian agricultural wells are usually small wells with shallow depths. Data for water levels for Faria wells showed that there is a large variability of water table elevations which could be attributed to variations in rainfall and pumping rates. Some wells in the study area showed significant reductions in water table elevations in the order of 10 to 20 meters for the last 30 years (WESI, 2005).

As part of the integrated land and water management framework, analysis of springs' yield allows for fine detailed analysis and aims at capturing any seasonal trends. In addition, analysis of yearly springs' yield will be carried out to provide an overview and generic insight to the potential yield of the springs. Reliability of springs' yield is also investigated to determine the amount of springs' yield to be used for planning purposes.

The main intent for the utilization of the groundwater flow model in this management framework is the aid in development of management alternatives for the study area. The amounts of groundwater that could be safely extracted under different management alternatives and climate variability need to be estimated in order to evaluate the management alternatives and select an optimal plan for the management of the study area.

2.8 SELECTION OF THE AGRICULTURE PLANNING MODEL FOR THE STUDY AREA

In Palestine, agriculture is currently the largest water consumer accounting for about 70% of total current available water supplies (WSSPS, 2000). Furthermore, agriculture had historically been the major productive sector in the Palestinian economy, accounting for about 30 % of GDP (Gross Domestic Product) (WSSPS, 2000). Feeding a growing population requires the optimum utilization of available natural resources including water, land and labour. The improvement of the contribution of agriculture to the economy will come from a contribution of improved farming practices and irrigation.

WSSPS (2001), CPF (1998), ARIJ (1998), WESI (1998) and WESI (1999) analysed national agriculture water demand problems in the West Bank. However, no studies had been conducted on a catchment level for optimal cropping patterns. Above studies investigated the estimation of the current agricultural water demand and predicted amounts of agricultural water demand for the future based on certain assumptions and political scenarios. Results showed that at present the necessary water is not available to extend irrigation. Sufficient supply can be shown to be potentially available if maximum practical use is made of surface runoff and suitably treated wastewater.

As discussed earlier in Section 2.2, agricultural demand for water and its optimal allocation is a very important factor in the integrated catchment management and for any proposed decision support system. For that purpose a model of agricultural response to water prices and policies, modeling the crop-choice decision of farmers, is needed as a planning tool within the proposed framework.

AGricultural Sub-Model (AGSM) (Fisher et al., 2005) is an optimizing model for planning and management of water and land resources. AGSM provides national, district or catchment level planners with a tool for planning agricultural production under various water allocations and prices. It uses data on available land, water requirements per unit land area for different crops, and net revenues per unit of land area generated by growing of those crops (gross income less direct expenses such as labor, materials, machinery, fertilizers). These net revenues do not include payments for water, which are handled separately. In addition, these net revenues do not include fixed costs (such as invested capital, land value). The AGSM takes prices of water or quantity allocations for water and generates cropping pattern which maximizes agricultural income. By varying water prices demand functions for each water type can be constructed. The model can also be used to examine the effects of water quantity allocations or changes in the prices of water on agricultural production (Fisher et al., 2005).

The AGSM will be used to obtain the optimum mix of land and water allocation to maximize the profit for each management alternative. The AGSM has several characteristics that made it useful to provide the optimal mix of water-consuming activities to maximize the net income of the agricultural production of a catchment and the water demands under various prices and water qualities. This includes: (1) user-friendly program; (2) successful application to other catchments in the region; (3) AGSM can serve as a planning tool suggesting to planners what crop patterns are likely to prove optimal under various conditions and relating these to different water policies (Fisher et al., 2005).

AGSM had been successfully applied to different districts in Israel to analyse agricultural water demand and investigate the response of agricultural production to water policy (Amir and Fisher, 1999, Amir and Fisher, 2000). Above authors have analysed the effects of water prices, administrative water quotas and limitation of certain crops as water policy-making factors, and concluded that the combined policy of water quotas and pricing may contradict the basic intentions of the decision-makers. The quotas and prices are two policy instruments acting on the same goal of overall water consumption. Their joint use is therefore likely to lead to a situation in which one of them is redundant and might result in unintended effects. One way of avoiding such unintended effects would be to use AGSM to calculate the prices that should be charged at the margin in order to accomplish the desired rationing.

Application of the AGSM to Jezreel Valley in Israel (Amir and Fisher, 1999, Amir and Fisher, 2000) revealed that the response of agricultural production systems to water limitations should be evaluated by the decision-makers by analysing the marginal reduction of income. In the

Jezreel Valley there is no change in agricultural production as a result of reduction in water allocation due to the presence of rainfed winter crops. In this case the administrative water reduction may well be justified by allocating the water to another district who is willing to pay a higher price for water to maximize the agricultural production.

Salman et al. (2001) used the AGSM to analyze inter-seasonal allocation of irrigation water in terms of both quantity and quality, and its impact on agricultural production and income. The main aim was to highlight water-scarcity issues as a problem arising when water is not found in the right quantities and of the right quality at the appropriate place and time. The application of the model to data from Jordan Valley in Jordan suggested that the model outputs closely relate the actual response of farmers to water prices and can serve planners as an approximation to the real world. It will assist agricultural planners to allocate scarce water resources among agricultural activities by time, space and different water qualities.

2.9 CLIMATE CHANGE

A global-scale scenario cannot be reliably applied to quantitatively describe the climatic change within the study area or even to the country, because of the small size of the country, the coarse resolution of current models and the great spatial inaccuracy of global models (Pe'er and Safriel, 2000). Changes in climate already detected in the region may be instructive in assessing the exposure to those changes predicted by different scenarios. Observed trends in Israel and the region do not always support a scenario of warming and drying. The discrepancies between model predictions and the actual observations may be only partly due to the complexity of modeling climate change in this region (Pe'er and Safriel, 2000).

Based on the climate scenario of Dayan and Koch (1999) and evaluation of the observations and models, the following scenario for Israel is the currently most likely one (Pe'er and Safriel, 2000) to represent the study area, indeed this scenario has been also adopted by Palestine Academy for Science and Technology in their study regarding climatic change (WESI, 2005) for the region:

1. Warming:
    - 0.7-0.8° C by 2050
    - 1.6-1.8° C by 2100

2. Precipitation:
    - -4 to -2% by 2050
    - -8 to -4% by 2100
3. A 10% increase in evapotranspiration with an increased temperature of 1.5°C anticipated around 2100.
4. Delayed winter rains.
5. Increased rain intensity and shortened rainy seasons.
6. Increased frequency and severity of extreme climatic events.
7. Greater spatial and temporal climatic uncertainity.

A change in rainfall pattern will result in different amounts of runoff. Increases in seasonal temperature variability, storminess and frequency of temperature extremes may endanger cold-sensitive and heat-sensitive crops. A delayed growing season will cause a loss of the special advantage over countries in colder climates in early exports of flowers, fruits and vegetables (Pe'er and Safriel, 2000).

Time horizons are not available for the climate changes in the country, so the information is based on a qualitative evaluation of anticipated trends and effects. Nevertheless the effect of climate change has been addressed in developing the decision support system framework such that this system can integrate possible scenarios on the long term planning horizons (50 or 100 years) that are beyond the planning horizon of 10 years used for this study.

2.10 SUMMARY

Lack of proper water allocation and optimal cropping systems accompanied with prolonged drought periods negatively affect the obtainable surface water and groundwater resources compelling the need for developing optimal water allocation policies that consider the available water resources in the catchments such that the socioeconomic revenue is maximized. The current study proposes a framework for integrated management of agriculture-dominated catchments in arid and semi-arid regions.

The main aim of this study is to develop an integrated natural resources management framework that involves diverse modules of surface water and groundwater models, yield from natural springs, a planning model for economic evaluation, a multi-criteria decision analysis model, and a GIS technology to facilitate processing and visualization.

In this study the Geomorphological Instantaneous Unit Hydrograph model is selected to determine streamflow yield from ungauged agricultural-dominated catchments. The basic approach will involve the application of a KW-GIUH rainfall-runoff model that is capable to determine runoff based on climatic data, and catchment geomorphological characteristics through GIS tools. MODFLOW is selected as groundwater flow model for the study area to aid in development of management alternatives for the study area. As part of the land and water management framework, analysis of monthly springs' yield is used to provide an overview and generic insight to the potential yield of the springs and comprehend the temporal springs' yield performance. AGSM will be used to plan the land and water allocation to farmers in the study area. The model can provide the optimal mix of water-consuming activities to maximize the net income of the agricultural production of a catchment and the water demands under various prices and water qualities.

The study concentrates on developing management alternatives that sustain the available water resources both in quantity and quality, optimize the use of low quality water including treated effluent and brackish water, and maximize both the irrigated areas and the income of the local farmers. Since the decision criteria involve conflicting objectives, a multi-criteria decision analysis is employed to prioritize the management alternatives using the Importance Order of Criteria (IOC) method. The proposed management framework involves all stakeholders, promotes co-operation and collaboration and builds a sense of community participation, helps reduce conflicts, increases commitment to actions that are necessary to meet the environmental goals, and ultimately improves the likelihood of successful and sustainable development programs. Faria catchment in West Bank, Palestine will be taken as a case study. The developed framework should be transferable and useful to other areas in the region as well as worldwide.

CHAPTER 3

RAINFALL-RUNOFF MODELLING

3.1 INTRODUCTION

The main objective of the study is to develop a framework for integrated land and water management from agriculturally dominated ungauged catchments. In deciding the management options for optimal use of land and water it is important to determine runoff volume that could be harvested from a storm for irrigation purposes. Furthermore, it is important to estimate the peak flow from a storm for infrastructure development for transport and storage of surface runoff or for recharge of groundwater. The basic approach will involve the application of a simple rainfall-runoff model to determine the streamflow yield as well as the peak flow based on climatic data and catchment geomorphological characteristics. Furthermore the rainfall runoff model developed for the study area is essentially needed for any water resources design work as part of proposed management options that would be applied to the study area. Faria catchment is an arid catchment and rainfall is limited to winter and spring seasons only. Streams in Faria catchment are ephemeral streams which go dry in between storms except for the main stream that receives water from springs. As a result it was decided to use the GIS based Kinematicwave Geomorphological Unit Hydrograph (KW-GIUH) to obtain the surface runoff hydrograph from the Faria catchment. In the KW-GIUH approach, excess rainfall is assumed to follow different paths on overland areas and in channels of different stream orders to reach the catchment outlet. In applying the KW-GIUH model to any water resources design project for an ungauged catchment, the design storm can be determined from the depth-duration-frequency relationship of rainfall.

This Chapter exemplifies the development of the KW-GIUH model for the Faria catchment based on climatic data, and catchment geomorphological characteristics obtained via GIS tools. Verification of the developed model against recorded streamflow and rainfall data from the study area will be conducted. Baseflow will be separated to obtain surface runoff hydrograph and Baseflow Index (BFI) will be calculated. The spatial average rainfall over each subcatchment in the study area will be estimated using the Thiessen polygon-GIS tools. Excess rainfall estimation needed for rainfall runoff modeling is also analyzed in this chapter. Sensitivity of the hydrograph to all input parameters will be also conducted and the applicability of the KW-GIUH model in optimizing the land and water management model will be discussed.

Figure 3-1 depicts the overall conceptual functionality of the surface water module of the management framework. The amounts of runoff generated under rainfall are needed to proceed with optimal management of land and water resources for the catchment under study. The rainfall data together with the KW-GIUH model (that is developed from catchment characteristics) are used to predict the runoff volume that will be later used by the planning model (AGSM) for the optimal catchment management.

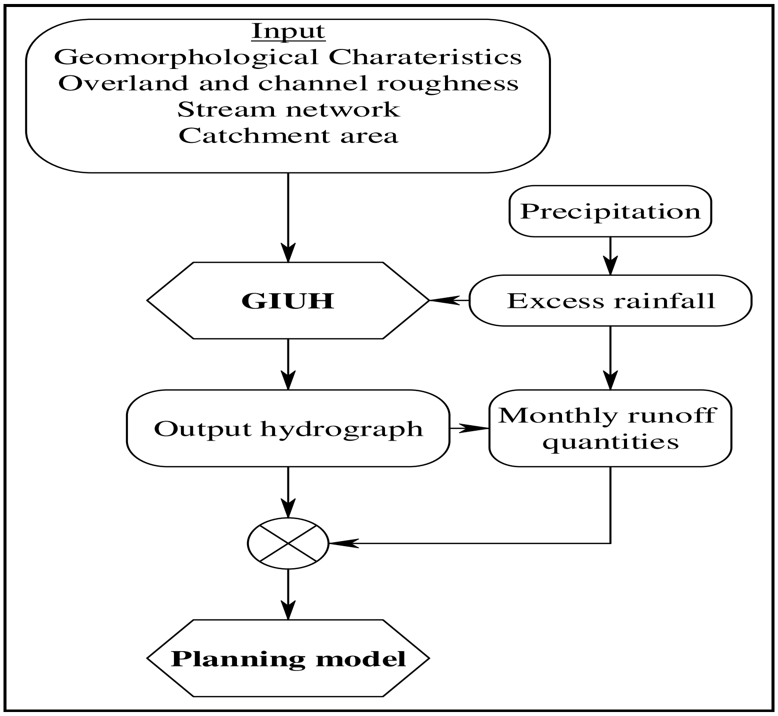

Figure 3-1 Overall conceptual functionality of the surface water module of the land and water management framework

As described in Section 2.9 quantitative describtion of climatic change cannot be reliably applied to the study area. However the effect of climate change has been addressed in developing the decision support system framework such that this system can integrate possible quantitatively described scenarios on the long term planning horizons (50 or 100 years) that are beyond the planning horizon of 10 years used for this study. A change in the precipitation pattern will be captured through the resulting amounts of excess rainfall and ultimately the amounts of runoff and peak flows. Increased rain intensity combined with a reduction in overall precipitation will diminish vegetation cover and increase surface runoff, leading to desertification. The resulting soil erosion, salinization, and loss of vegetation will further increase surface runoff. Agricultural fields (mainly rainfed ones) will become more saline from increased evapotranspiration. Increased surface runoff will increase flash floods during peak waterflows (Pe’er and Safriel, 2000).

3.2 DATA COLLECTION

For the purpose of developing and applying the KW-GIUH rainfall runoff model different data are needed including rainfall, streamflow, climatic data and catchment geomorphological characteristics. Available data were obtained from different sources, including relevant institutions and governmental departments as well as available literature and reports, and compiled in a composite database and placed in a format that is readable by GIS to signify the spatial distribution of the relevant data. In addition, the database was put up in a format that is accessible by MS Excel for ease of analysis and manipulation. GIS maps were available (WESI, 2005) for the area to get the geomorphological characteristics of the catchment such as the stream length, stream order, subcatchment area and slope. Available data for the purpose of rainfall runoff modelling included long term average rainfall (yearly, monthly and daily), climatic data, landuse data, soil data, and GIS maps that can be used to find the geomorphological characteristics of the catchment. The available data can be used to develop the KW-GIUH for the catchment. However for purposes of application and verification of the developed KW-GIUH model, hourly rainfall intensity and streamflow data were measured for the rainy season of years 2004/2005. The following sections give a detailed description of data collection. Section 3.2.1 describes the climatic conditions, digital elevation model and some catchment’s characteristics as obtained from GIS maps. Section 3.2.2 gives a detailed description on rainfall intensity and streamflow data that will be used for model application and verification.

3.2.1 Climatic Data

As discussed earlier in Section 1.4 the study area is considered as the most important agricultural catchment in Palestine (Figure 3-2). The climate is dominantly a Mediterranean, semi-arid climate. Mean temperature in the Faria catchment ranges between 14.4 ºC in winter and 31.4 ºC in summer. Mean maximum temperature in the Faria catchment ranges between 19.5 ºC in winter and 39.4 ºC in summer. Mean relative humidity ranges between 43% and 73%. Topography is a unique factor in the catchment that starts at an elevation of about 900 m above mean sea level in the upper part and descends drastically to about 250 m below sea level at the point where the Faria stream meets the Jordan River. Figure 3-3 depicts the digital elevation model of the study area.

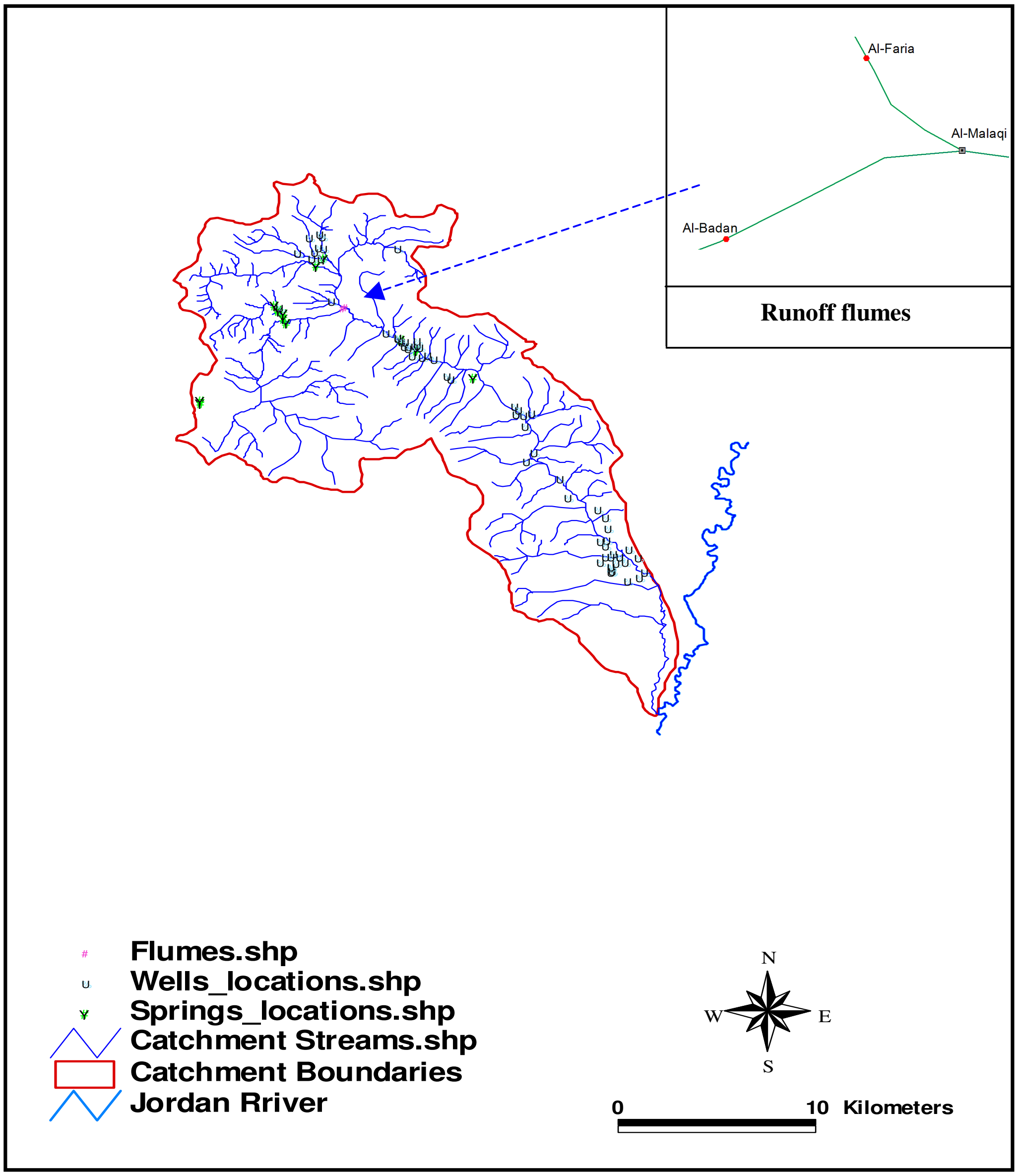

Figure 3-2 Faria catchment with a depiction of the spatial locations of the rainfall stations, springs, groundwater abstraction wells, runoff gauging stations and major surface streams

Summary of the available rainfall data is presented in Table 3-1. The Table shows that Nablus and Talluza stations have the largest average annual rainfall and Faria station has the lowest. The location of rainfall stations and the rainfall isoheights for the long term avarage rainfall are shown Figure 3-4.

The Faria catchment under study is divided into three subcatchments which are Faria subcatchment, Badan subcatchment and Malaqi subcatchment. The areas of the subcatchments are 64km^2, 85km^2 and 185km^2 respectively as shown in Figure 3-5.

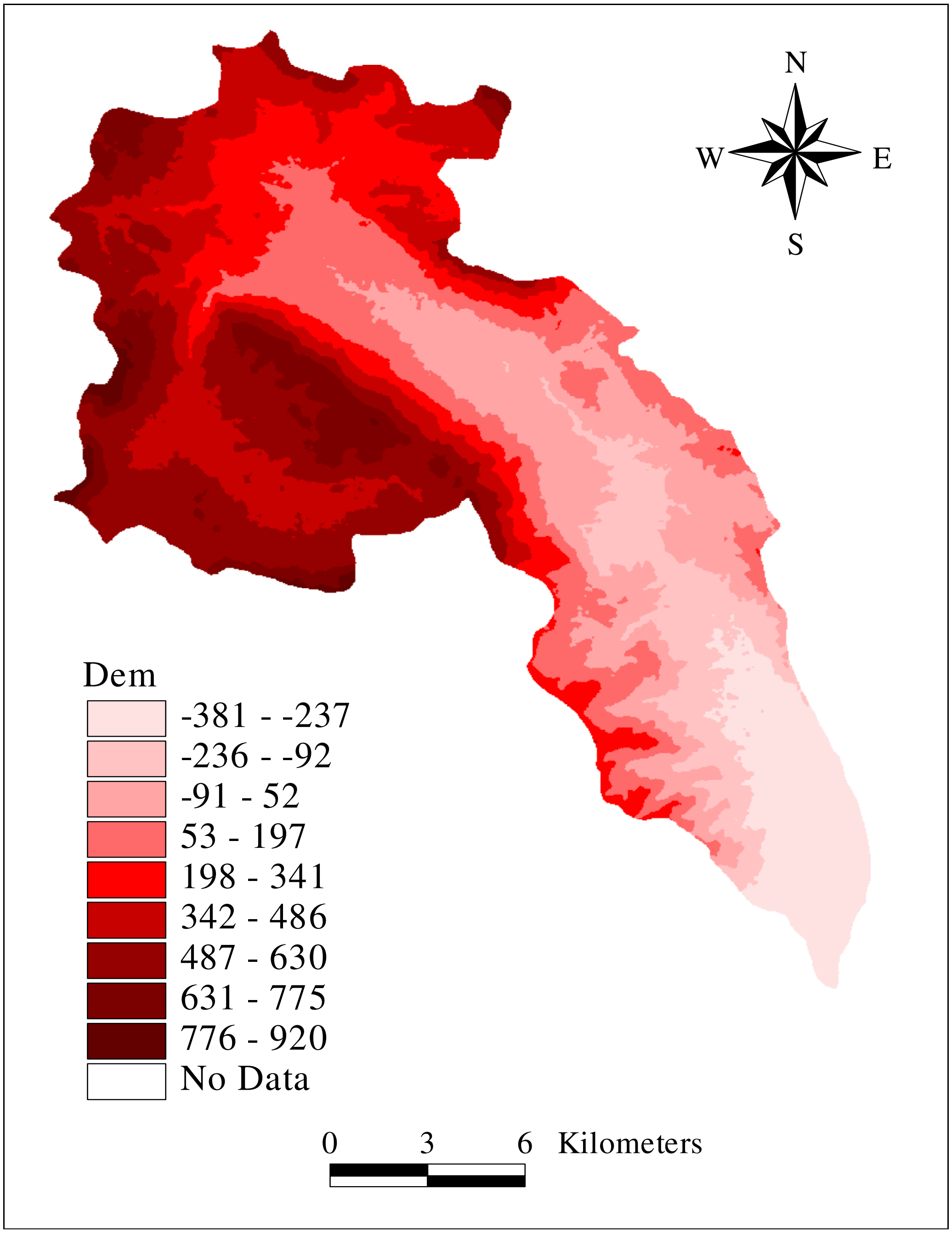

Figure 3-3 Digital Elevation Model (DEM) of Faria Catchment

Table 3-1 Altitude and rainfall information at the different stations in the study area

Station Name	Altitude	Rainfall			
		Period	Mean (mm)	Max (mm)	Min (mm)
Al Faria	-237	1952-1989	198.6	424.0	30
Nablus	570	1946-2003	642.6	1387.6	315.5
Tubas	375	1967-2003	415.2	889.5	201.5
Tammun	340	1966-2003	322.3	616.1	124.2
Talluza	500	1963-2003	630.5	1303	292.2
Beit Dajan	520	1952-2003	379.1	777	141

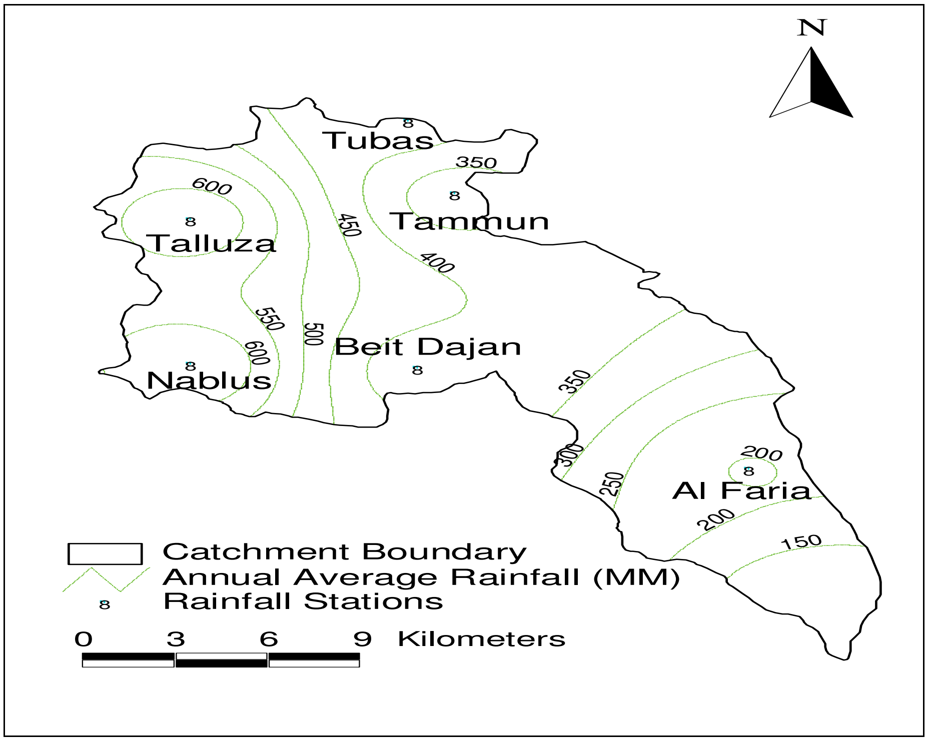

Figure 3-4 Rainfall stations and rainfall distribution within the Faria Catchment

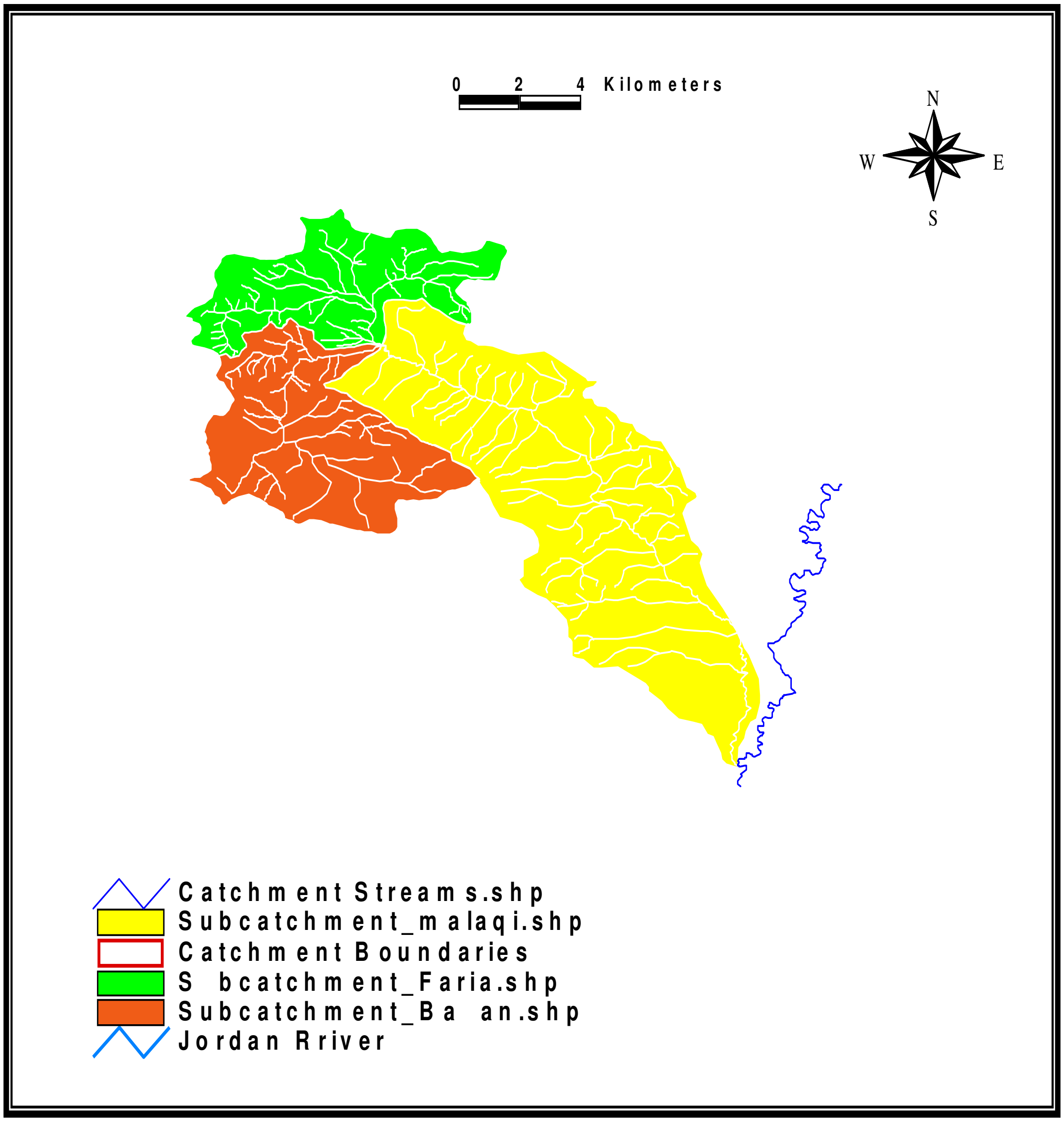

Figure 3-5 The three subcatchments of the Faria Catchment

Reference Evapotranspiration (ET_o) was estimated by using the CROPWAT version 4.2 model (FAO, 1998a) based on the average climatic data for the upper and lower parts of the catchment (MoT, 1998; WESI, 2005; ARIJ, 1998). CROPWAT is a computer program for irrigation planning and management, developed by the Land and Water Development Division of FAO (FAO, 1998a). Its basic functions include the calculation of reference evapotranspiration, crop water requirements, and irrigation water requirements. The calculation of reference evapotranspiration (ET_o) is based on the FAO Penman-Monteith method (FAO, 1998a) shown in Equation 3-1. Input data include monthly temperature (maximum and minimum), humidity, sunshine, solar radiation and wind-speed.

$$ET_O = \frac{0.408\Delta(R_n - G) + \gamma \frac{900}{T+273} u_2 (e_s - e_a)}{\Delta + \gamma(1 + 0.34 u_2)} \qquad (3\text{-}1)$$

where,

ET_o reference evapotranspiration [mm day^{-1}],

R_n net radiation at the crop surface [MJ m^{-2} day^{-1}],

G soil heat flux density [MJ m^{-2} day^{-1}],

T mean daily air temperature at 2 m height [°C],

u_2 wind speed at 2 m height [m s^{-1}],

e_s saturation vapour pressure [kPa],

e_a actual vapour pressure [kPa],

e_s - e_a saturation vapour pressure deficit [kPa],

Δ slope vapour pressure curve [kPa $°C^{-1}$],

γ psychrometric constant [kPa $°C^{-1}$].

Input data for the study area as well as results of the ET_o values for different months for the upper and lower parts of the catchment are shown in Table 3-2 and Table 3-3 respectively.

Table 3-2 ET_o values for different months for the upper part of the catchment

Month	MaxTemp (deg.C)	MiniTemp (deg.C)	Humidity (%)	WindSpd. (Km/d)	SunShine (Hours)	Solar Rad. (MJ/m^2/d)	ET_o (mm/d)
January	13.1	6.2	67.0	156.3	4.7	9.6	1.58
February	14.4	6.7	67.0	169.8	4.8	11.5	2.01
March	17.2	8.8	62.0	179.5	6.4	16.0	2.93
April	22.2	12.0	53.0	182.5	8.2	20.8	4.35
May	25.7	14.9	51.0	192.2	8.9	23.1	5.3
June	27	17.4	55.0	215.4	8.4	22.7	5.65
July	29.1	19.3	61.0	223.7	9.6	24.2	5.84
August	29.4	19.5	65.0	210.2	10.9	25.0	5.65
September	28.4	18.5	64.0	184.8	10.2	21.8	4.77
October	25.8	16.2	57.0	137.6	9.8	18.2	3.56
November	20.2	12.1	57.0	139.1	7.0	12.3	2.31
December	14.6	7.8	67.0	137.6	4.8	9.0	1.49
Average	**22.3**	**13.3**	**60.5**	**177.4**	**7.8**	**17.9**	**3.79**

Table 3-3 ETo values for different months for the lower part of the catchment

Month	MaxTemp (deg.C)	MiniTemp (deg.C)	Humidity (%)	WindSpd. (Km/d)	SunShine (Hours)	Solar Rad. ($MJ/m^2/d$)	ETo (mm/d)
January	19.5	9.3	73.0	110.4	5.7	10.6	1.69
February	20.2	9.2	73.0	156.0	6.0	12.9	2.34
March	24.3	12.1	63.0	146.4	7.5	17.4	3.53
April	29.1	14.4	63.0	86.4	8.7	21.5	4.28
May	34.6	19.0	52.0	79.2	10.3	25.1	5.53
June	37.1	21.1	51.0	86.4	11.6	27.4	6.30
July	39.4	22.7	51.0	163.2	11.7	27.3	7.55
August	38.5	24.2	52.0	156.0	11.0	25.2	6.89
September	36.6	22.9	43.0	120.0	9.9	21.5	5.50
October	33.5	20.2	54.0	60.0	8.5	16.7	3.32
November	27.9	16.8	55.0	60.0	7.3	12.7	2.16
December	21.5	11.9	67.0	50.4	6.2	10.4	1.37
Average	**30.2**	**17.0**	**58.1**	**106.2**	**8.7**	**19.1**	**4.20**

3.2.2 Rainfall Intensity and Stream Flow

Tipping Bucket rainfall gauges installed at three locations namely Tammon, Tubas, and Deir El Hatab (Beit Dajan) within the catchment were used to measure rainfall intensity. Runoff data were measured through two Parshall flumes with data loggers at Al-Malaqi Bridge (Figure 3-2) that measure the flows at the two main streams of upper Faria subcatchments, Faria and Badan.

Catchment rainfall intensity

Rainfall intensity measured from the three rainfall stations were used to calculate the rainfall intensity of other rainfall stations using weighted average rainfall given in Equation 3-2.

$$Rn=[(R1/LTA1)+ (R2/LTA2)+ (R3/LTA3)]*(LTAn/3) \qquad (3\text{-}2)$$

where,

Rn=Rainfall intensity for unknown location

R1, R2, R3=Measured rainfall intensity from rainfall station locations 1,2,3, respectively

LTA1, LTA2, LTA3, LTAn = Measured long term average rainfalls at rainfall stations 1,2,3 and n respectively.

For hydrologic applications it is often necessary to compute estimates of mean areal precipitation for a catchment from rain gauge observations. The spatial average rainfall over each subcatchment in the study area was estimated using the Thiessen polygon. The Thiessen polygon method is based on the assumption that for any point in the catchment, rainfall is equal to the observed rainfall at the closest gauge. The station weights are specified by the relative areas of the Thiessen polygon network, the boundaries of the polygons being formed by the perpendicular bisectors of the lines joining adjacent gauges. This method is well suited to graphical determination of weights (Maidment, 1993). Arc View GIS was used to prepare the Thiessen polygons for the study area. The resulting weight for each station contributing to the spatial subcatchment rainfall was used to calculate the subcatchment rainfall using Excel sheets.

Rainfall intensity measured from the three locations within the catchment consisted of 10872 hourly rainfall readings distributed as 3624 readings for each of the three locations during the 2004/2005 rainy season (starting from November, 2004 till April, 2005). The resulting Excel-database was used to calculate the rainfall intensity for the other three locations namely Nablus, Talluza, and AlFaria using Equation 3-2 adding a further 10872 readings for the other three locations.

The spatial average rainfall over each sucatchment in the catchment was estimated using the Thiessen polygon-GIS tools as shown in Figure 3-6. The resulting weight for each station contributing to the spatial subcatchment rainfall was used to calculate each of the three subcatchment hourly rainfall intensity using Excel sheets with a total of 3624 readings for each subcatchment during the rainy season.

Table 3-4 shows both the 2004/2005 rainfall and the long-term average yearly rainfall for different stations, the corresponding Thiessen polygons weight for each station contributing to the spatial subcatchment rainfall as prepared using ArcView GIS capabilities. The subcatchment rainfall for both the 2004/2005 rainy season and on long-term basis is shown in Table 3-4.

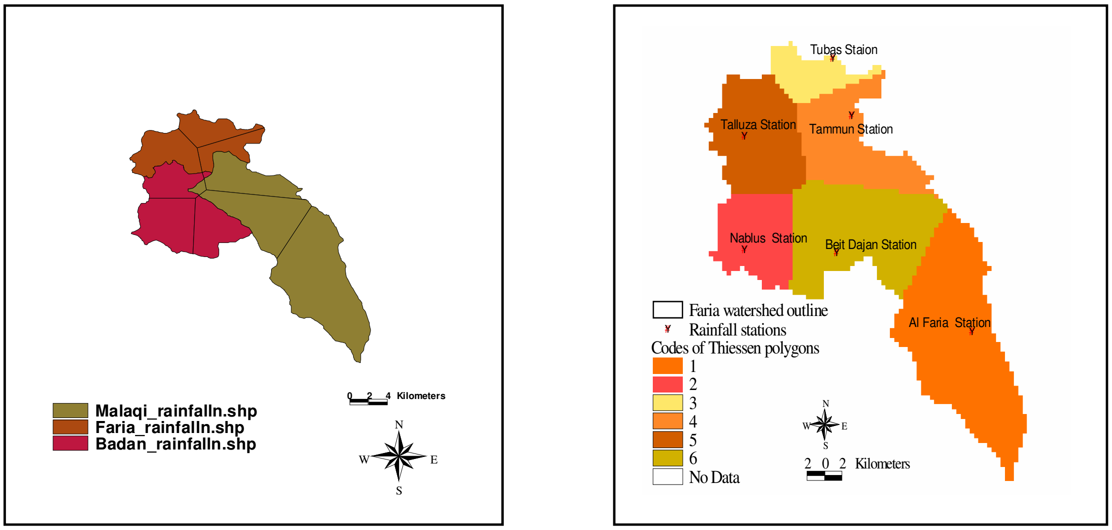

Figure 3-6 a)Subcatchemts with Thiessen polygons b) The spatial distribution of rainfall stations and the corresponding Thiessen polygons as prepared using ArcView GIS capabilities.

The results showed that the long-term average yearly subcatchment rainfall was 551.6 mm, 492.7 mm, and 270.7 mm, for the Badan, Faria, and Malaqi respectively. The lower catchment has the lowest long-term average rainfall of less than 300 mm indicating the less potential for generating runoff. This is consistent with the previous studies (Moe et al., 1998, Takruri, 2003, Rofe and Fafety, 1965) and field observations showing that most of the runoff is generated in the upper catchment, while the lower Malaqi subcatcment has no runoff due to the dominant aridity of this area. For the rainy season of 2004/2005 the subcatchment rainfall was 619 mm, 604 mm and 303 mm for the Badan, Faria and Malaqi subcatchments.

During the rainy season of 2004/2005, a major storm was recorded during February. Hourly rainfall and runoff data recorded for this event will be used to verify the developed KW-GIUH model for the study area as detailed in Section 3.4.3.

Table 3-4 Subcatchments' rainfall stations and the corresponding Thiessen polygons as prepared using ArcView GIS capabilities

Subcatchment	Area, km^2	Contributing rainfall station (RN)	% Weight of RN	Long-term		2004/2005	
				Average Rainfall (mm)	Sub-catchment rainfall (mm)	Rainfall (mm)	Sub-catchment Rainfall (mm)
Badan	85	Nablus	0.4239	642.6	272.4	757.5	321.1
		Tammoun	0.0064	322.3	2.1	434.2	2.8
		Talluza	0.2431	630.5	153.3	743.2	180.7
		BeitDajan	0.3266	379.1	123.8	350.3	114.4
Total					551.6		619
Faria	64	Tubas	0.2969	415.2	123.3	525.3	155.9
		Tammoun	0.2398	322.3	77.3	434.2	104.1
		Talluza	0.4634	630.5	292.1	743.2	344.4
Total					492.7		604.4
Malaqi	185	Faria	0.5544	198.6	110.1	234.1	129.8
		Tammoun	0.1760	322.3	56.7	434.2	76.4
		Talluza	0.0102	630.5	6.4	743.2	7.6
		BeitDajan	0.2573	379.1	97.5	350.3	90.1
Total					270.7		303.9

3.3 MODEL DEVELOPMENT

A significant advance in modeling runoff from ungauged catchments was initiated by Rodriguez-Iturbe and Valdes (1979) and Gupta et al. (1980), who rationally interpreted the runoff hydrograph in the framework of travel time distribution explicitly accounting for geopmorphological structure of the basin. They developed the instantaneous unit hydrograph (IUH) using geomorphic stream-order information of the catchment. In this stream-order-based IUH approach, each of the channels is assigned an order following the Strahler stream-ordering system (Strahler, 1957). For basins of any order, the peak discharge q_p and the time to peak t_p, which are the most important characteristics of the GIUH, are worked out from the derived functional relationship of the GIUH as given below (Rodriguez-Iturbe and Valdes, 1979; Jain et al., 2000):

$$q_p = 1.31 \quad R_L^{0.43} \left(\frac{V}{Li} \right) \tag{3-3}$$

$$t_p = 0.44 \quad R_L^{-0.38} \left(\frac{R_B}{R_A} \right)^{0.55} \left(\frac{Li}{V} \right) \tag{3-4}$$

$$R_A = \frac{\overline{A_i}}{\overline{A_{i-1}}} \tag{3-5}$$

$$R_B = \frac{\overline{N_i}}{\overline{N_{i-1}}} \tag{3-6}$$

$$R_L = \frac{\overline{L_{C\,i}}}{\overline{L_{C\,i-1}}} \tag{3-7}$$

$$\overline{L}_{ci} = \frac{1}{N_i} \sum_{i=1}^{N} L_{ci} \tag{3-8}$$

where,

- q_p = the peak discharge (h^{-1})
- t_p = time to peak (hr)
- R_B, R_L and R_A represent the bifurcation ratio, the length ratio and the area ratio, respectively.
- L_i = the total length of the highest order stream (km)
- V flow velocity (m/s)
- N_i is the number of *i*th-order channels
- $\overline{L_{c\,i}}$ is the mean *i*th-order stream length
- $\overline{A_i}$ is the *i*th-order contributing area

The above equations represent general relationships which allow the estimation of the peak and time to peak of the IUH for any catchment.

A major problem in applying the stream-order IUH model is the travel time determination that depends on the flow velocity. Gupta et al. (1980) estimated the travel time on overland areas and in channels by assuming exponential and uniform probability distribution functions respectively. Jin (1992) suggested gamma distribution to yield better results. Rodriguez-Iturbe and Valdes (1979) estimated the travel time from discharge records by regression methods. These empirical equations vary from one catchment to another and are not applicable beyond the catchment condition from which they were developed, such a drawback hindered practical applications of the stream-order IUH approach to ungauged catchments. Rodriguez-Iturbe et al. (1982) utilized geomorphologic laws to relate the travel time for the first-order channel, estimated using a kinematic-wave approximation, to that of higher-order channels, without considering the travel time for the overland flow.

Kinematic-wave GIUH model (KW-GIUH)

As stated above a difficulty in applying the geomorphology-based unit hydrographs lies in the determination of travel time that is a hydraulic problem. As an alternative approach, Lee and Yen, (1997) and Yen and Lee, (1997) used the kinematic-wave theory to analytically determine the travel times for overland and channel flows in a stream-ordering sub basin system, and then substituted into the GIUH model to develop a kinematic-wave based GIUH model (KW-GIUH) for catchment runoff simulation. The resultant instantaneous unit hydrograph is a function of the time rate of water input (intensity of rainfall excess in application). In applying the instantaneous unit hydrographs for hydrograph simulation, the model deals with temporally nonuniform rainfall through convolution integration of the instantaneous unit hydrographs applied to the rainfall excess of varying intensities with time. After choosing appropriate values of the overland and channel roughness coefficients, the runoff process of unit rainfall excess can be predicted based simply on catchment geomorphology obtained from a topographic map or GIS.

As detailed in Lee and Yen (1997) the travel times Tx_{oi} and Tx_i and the water depth h_{coi} are computed from Equations 3-9 and 3-10 and aided by Equations 3-11 to 3-15. More details of the model are available in Appendex II.

$$T_{x_{oi}} = \left(\frac{n_o \bar{L}_{oi}}{\bar{S}_{Oi}^{1/2} \, q_L^{m-1}} \right)^{1/m} \tag{3-9}$$

$$T_{xi} = \frac{B_i}{2 q_L \bar{L}_{oi}} \left[\left(h_{coi}^{m} + \frac{2 q_L n_c \bar{L}_{oi} \bar{L}_{ci}}{B_i \, \bar{S}_{ci}^{1/2}} \right)^{1/m} - h_{coi} \right] \tag{3-10}$$

$$\bar{L}_{oi} = \frac{A P_{OAi}}{2 N_i \bar{L}_{ci}} \tag{3-11}$$

$$h_{coi} = \left[\frac{q_L n_c (N_i \bar{A}_i - A P_{OAi}}{N_i B_i \bar{S}_{ci}^{1/2}} \right]^{1/m} \tag{3-12}$$

$$B_i = \frac{B_\Omega \sum_{l=1}^{i} \bar{L}_{c_l}}{\sum_{l=1}^{\Omega} \bar{L}_{c_l}} \tag{3-13}$$

$$\bar{A}_i = \frac{1}{N_i} \sum_{j=1}^{N_i} A_{ij} \tag{3-14}$$

$$P_{OA_i} = \frac{1}{A} \left(N_i \bar{A}_i - \sum_{l=1}^{i-1} N_l \bar{A}_l P_{x_l x_i} \right) \tag{3-15}$$

where,

- T_{x_i} is the rainwater travel time for the ith order channel
- $T_{x_{oi}}$ is the travel time through the ith order overland plane
- x_{oi} denote the *i*th-order overland flow regions
- x_i denote the *i*th-order channels
- i=1, 2, …, Ω channel order

- h_{co_i} is the water depth at the entrance of the ith-order channel
- $\overline{L}_{oi}$ is the mean length of the *i*th-order overland flow planes
- N_i is the number of *i*th-order channels
- $\overline{L}_{ci}$ is the mean *i*th-order stream length
- $\overline{A}_i$ is the *i*th-order sub catchment contributing area
- P_{OA_i} is the ratio of *i*th-order overland area to the watershed area
- $\overline{S}_{ci}$ is the mean *i*th-order channel slope
- $\overline{S}_{oi}$ is the mean *i*th-order overland slope
- n_o is the overland flow roughness
- n_c is the channel flow roughness
- B_i is the *i*th-order channel width
- q_L is the spatially uniform intensity of rainfall excess
- B_Ω is the channel width at watershed outlet
- m is an exponent
- A is the catchment area
- $P_{x_i x_j}$ is the stream network transitional probability
- A_{ji} is the area of the overland flow regions that drains directly into the jth channel of order i, and also includes overland areas draining into the lower order channels tributary to this jth channel of order i

The instantaneous unit hydrograph of the catchment can be expressed (Rodriguez-Iturbe and Valdes, 1979; Gupta et al., 1980) as given in Equation 3-16.

$$u(t) = \sum_{w \in W} [f_{x_{oi}}(t) * f_{x_i}(t) * f_{x_j}(t) * \ldots * f_{x_\Omega}(t)]_w . P(w) \qquad (3\text{-}16)$$

where,

- $w \in W$, W is the path space given as $\langle x_{oi}, x_i, x_j, \ldots, x_\Omega \rangle$
- * denotes a convolution integral
- $f_{x_j}(t)$ is the travel-time probability-density function in state x_j, with a mean value of T_{xj}, and obtained using the Laplace Series.

- P(w) = probability of a drop of rainfall excess adopting this path and is calculated using Equation 3-17.

$$P(w) = P_{OA_i} . P_{x_{oi} x_i} \dots P_{x_i x_j} \dots P_{x_k x_\Omega} \qquad (3\text{-}17)$$

where

- w = specified flow path
- P(w) = probability of a drop of rainfall excess adopting this path
- i, j, … k, Ω are stream order numbers
- $Px_{oi}x_j$ = transitional probability of the raindrop moving from the ith-order overland region to the ith-order channel; and
- Px_ix_j is the transitional probability of the raindrop moving from an ith-order channel to a jth-order channel and is computed as given in Equation 3-18.

The ratio of *i*th-order overland area to the catchment area (P_{OA_i}) is estimated using Equation 3-15. The stream network transitional probability of the raindrop moving from an *i*th-order channel to a *j*th-order channel is computed as recommended by Lee and Yen (1997) and given in Equation 3-18.

$$P_{x_i xj} = \frac{N_{i,j}}{N_i} \qquad (3\text{-}18)$$

where,

- $N_{i,j}$ is the number of the ith-order channels contributing to jth-order channels

As applied in a linear response system, the system output generated by using the KW-GIUH model can be determined using the convolution integral of the rainfall input and the IUH (Equation 3-16), which can be expressed as in Equation 3-19 (Lee and Yen, 2000). In traditional hydrology the IUH of a catchment is unique. However, the IUH u(t) in Equation 3-19 temporally varies with the rainfall excess intensity. The dynamic nature of the catchment hydrologic response function is regarded as the major merit of the KW-GIUH model (Lee and Yen, 2000).

$$Q(t) = \int_0^t R_e(\tau)u(t-\tau)d\tau \qquad (3\text{-}19)$$

where,

Q=the direct runoff at the catchment outlet

R_e=the input (rainfall excess) of the catchment

$u(t)$=the IUH generated by KW-GIUH model, and

τ = a dummy variable

The catchment runoff response characteristics are expressed as a set of instantaneous unit hydrographs for different amount of water in the flow (Lee and Yen, 2000). Linear superposition is applied to combine the component hydrographs of hourly rainfall excess to produce the complete surface runoff hydrograph.

The proposed kinematic-wave GIUH (KW-GIUH) method has been tested on several catchments in the United States and Taiwan (Yen and Lee, 1997). The application of the KW-GIUH model to two hilly catchments in the eastern United States (Otego Creek and Wills Creek) and two Great Plain catchment in Illinois (Salt Creek and Kaskaskia River) indicate that the model generates hydrographs in good agreement with recorded hydrographs, demonstrating that the method is a potentially useful tool for hydrograph generation for ungauged and inadequately-gauged catchments (Yen and Lee, 1997). Lee and Yen (1997) also applied the KW-HIUH model to two selected rainstorms on the Keeling River catchment at WuTu, Taiwan. According to above authors the simulated and observed results were in good agreement for these two storms. Lee and Chang (2005) applied the GIUH model in Heng-Chi catchment in China, to consider both the surface and subsurface flow processes. Kinematic-wave approximation was used to estimate the mean value of the travel-time probability distributions for runoff in surface flow regions and channels. The simulated hydrographs obtained using the GIUH model were in good agreement with the observed hydrographs (Lee and Chang, 2005).

Geographical Information System (GIS) provide a digital representation of the catchment characterization used in hydrologic modeling. GIS also provide the basis for hydrologic modeling of ungauged catchments and for studying the hydrologic impact of physical changes within a catchment. The integration of GIS into hydrologic models follows one of the two approaches: (i) develop hydrologic models that operate within a GIS framework, (ii) develop GIS techniques that partially parameterize existing hydrologic models. Jain et al. (2000) has applied the second approach to Gambhiri river catchment in India and concluded that the peak characteristics of the flows are more sensitive to the various storm patterns as well as methods of critical sequencing followed for the computation of the design storm.

A GIS-based KW- GIUH approach will be developed and applied for the estimation of flow hydrographs for the semiarid catchment of Faria, West Bank. With the given geomorphic properties of the catchment, the unit hydrograph can be determined hydraulically without using any recorded data of past rainfall or runoff events, and thus attaining the rainfall-runoff process in arid and semiarid regions. As discussed earlier in the GIUH approach, excess rainfall is assumed to follow different paths on overland areas and in channels of different stream orders to reach the catchment outlet.

The KW-GIUH program (version 1.2) has been developed by Lee and Chang (2001). The above software package was used to develop the KW-GIUH model for the Faria catchment. Input data to the KW-GIUH model are discussed in Section 3.3.1

3.3.1 Geomorphological Factors for Development of the KW-GIUH Model

For the KW-GIUH model, the geomorphic characteristics of the catchment were estimated using the Digital Elevation Model (DEM) shown in Figure 3-3 and Arc-View GIS tools. The catchment was divided into three subcatchments, two of them considered as the upper Faria subcatchments including Badan and Faria, and the third subchatchment is Malaqi which is considered as the lower Faria subcatchmnet. Table 3-5 below provides a list of input geomorphic parameters for the KW-GIUH model and the methodology followed to calculate them. The stream order for each of the three subcatchments of the Faria catchment as prepared using ArcView GIS capabilities is shown in Figure 3-7. The stream network order of Badan and Faria subcatchments is fourth order and for Malaqi subcatchment third order as shown in Figure 3-7. The overflow contributing areas as prepared using ArcView GIS capabilities for each of the stream orders of the Faria catchment as a whole, and for the three subcatchments

Faria, Badan and Malaqi are shown in Figure 3-8, Figure 3-9, Figure 3-10 and Figure 3-11 respectively.

Table 3-5 KW-GIUH Input parameters and the methods of determination

Parameter	Description	Method of Determination
N_i	number of ith-order channels	ArcView GIS capabilities
$\overline{L}_{ci\ (m)}$	mean ith-order stream length	ArcView GIS capabilities
$\overline{A}_i\ (km^2)$	ith-order sub catchment contributing area	ArcView GIS capabilities
P_{OAi}	ratio of ith-order overland area to the catchment area	ArcView GIS capabilities
$\overline{S}_{ci\ (m/m)}$	mean ith-order channel slope	ArcView GIS capabilities
$\overline{S_{oi}}\ (m/m)$	mean ith-order overland slope	ArcView GIS capabilities
Area (km^2)	Subcatchment area	ArcView GIS capabilities
n_o	overland flow roughness	Literature and Field Investigations
n_c	channel flow roughness	Literature and Field Investigations
$B_\Omega\ (m)$	channel width at catchment outlet	Field Investigation

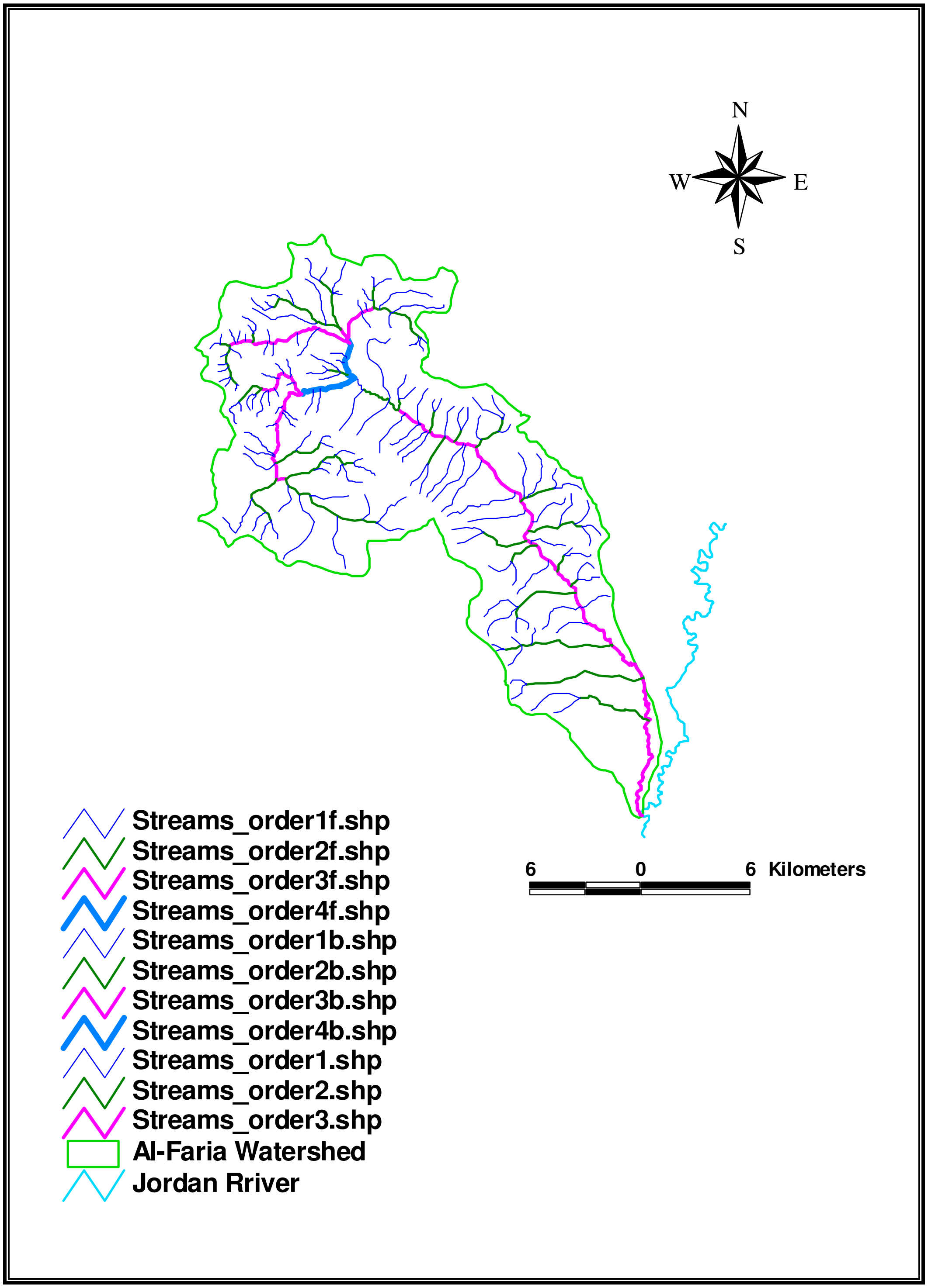

Figure 3-7 Stream order for each of the three Subcatchments (Faria f, Badan b and Malaqi) of the Faria Catchment as prepared using ArcView GIS capabilities

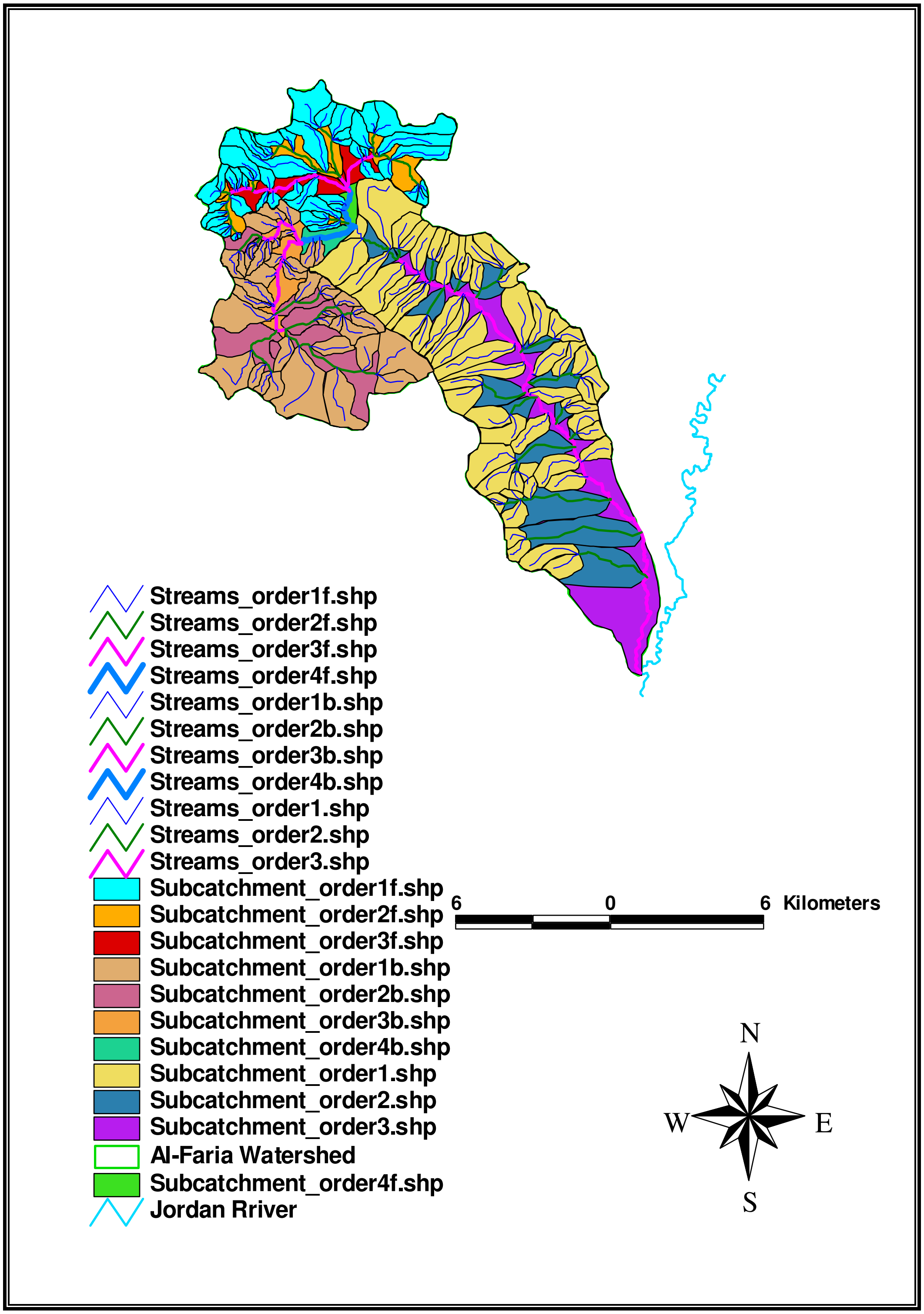

Figure 3-8 Overflow contributing areas for each of the stream orders of the three Subcatchments (Faria f, Badan b and Malaqi) of the Faria Catchment as prepared using ArcView GIS capabilities

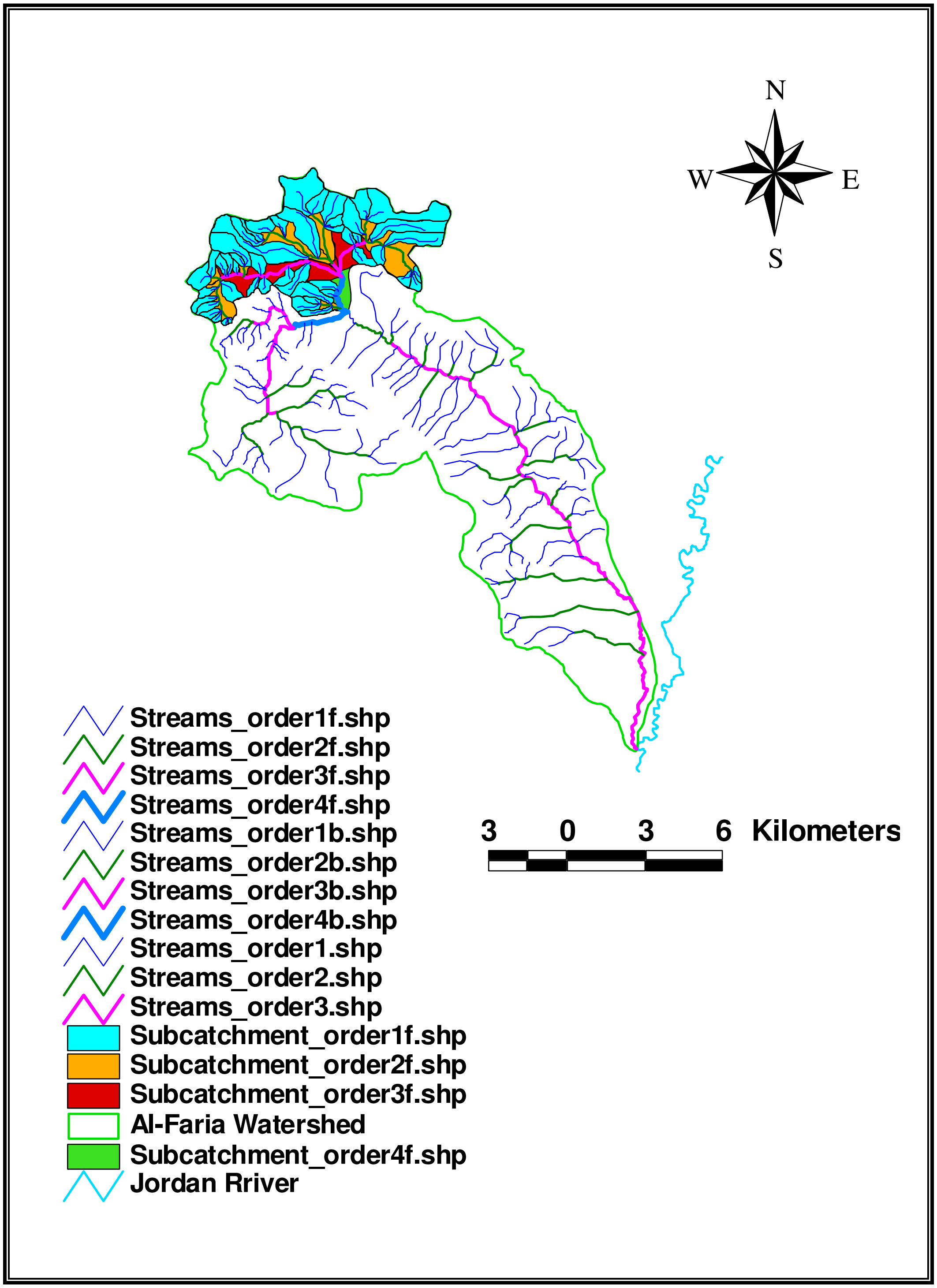

Figure 3-9 **Overflow contributing areas for each of the stream orders of Faria Subcatchment as prepared using ArcView GIS capabilities**

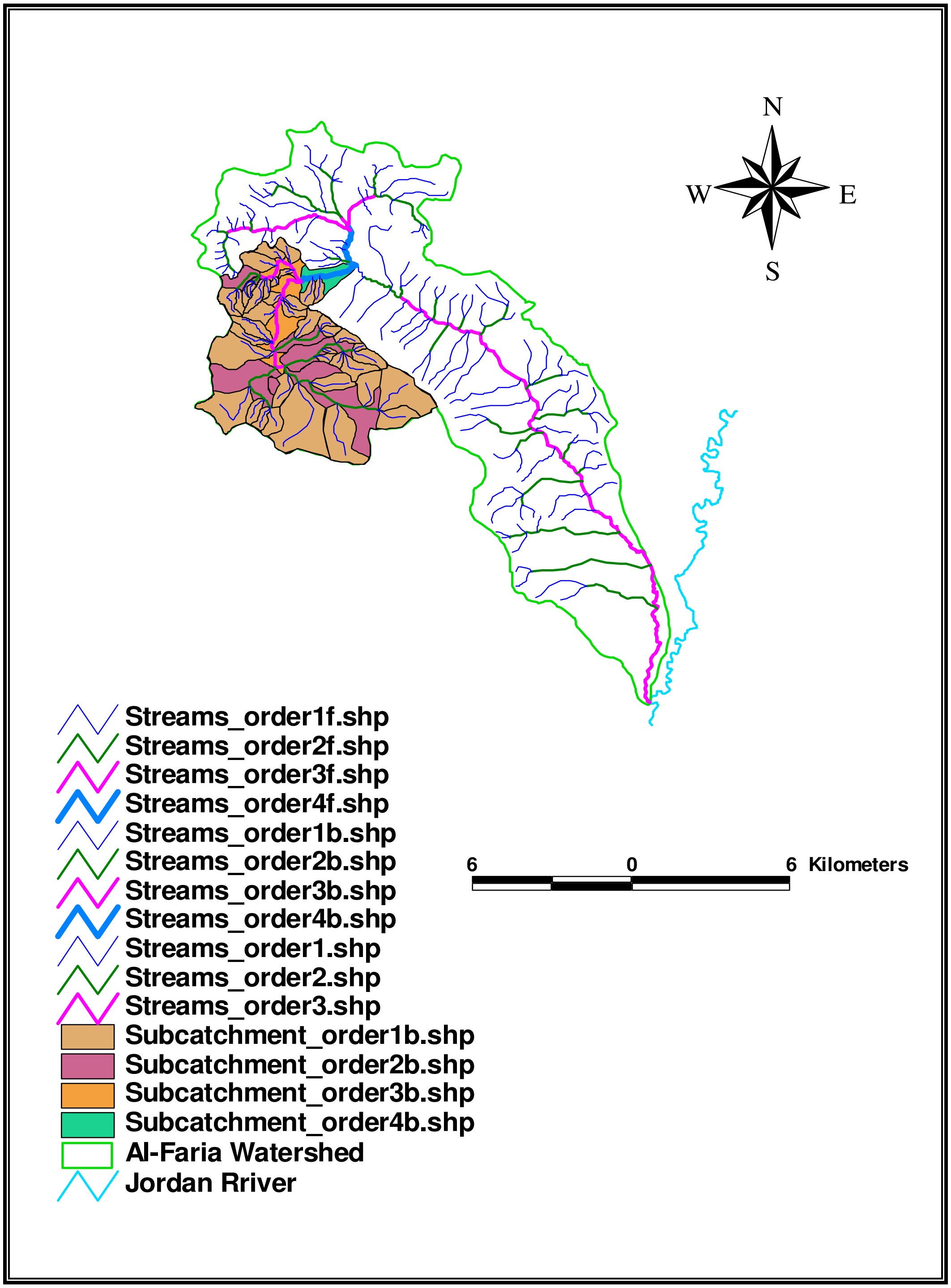

Figure 3-10 **Overflow contributing areas for each of the stream orders of Badan Subcatchment as prepared using ArcView GIS capabilities**

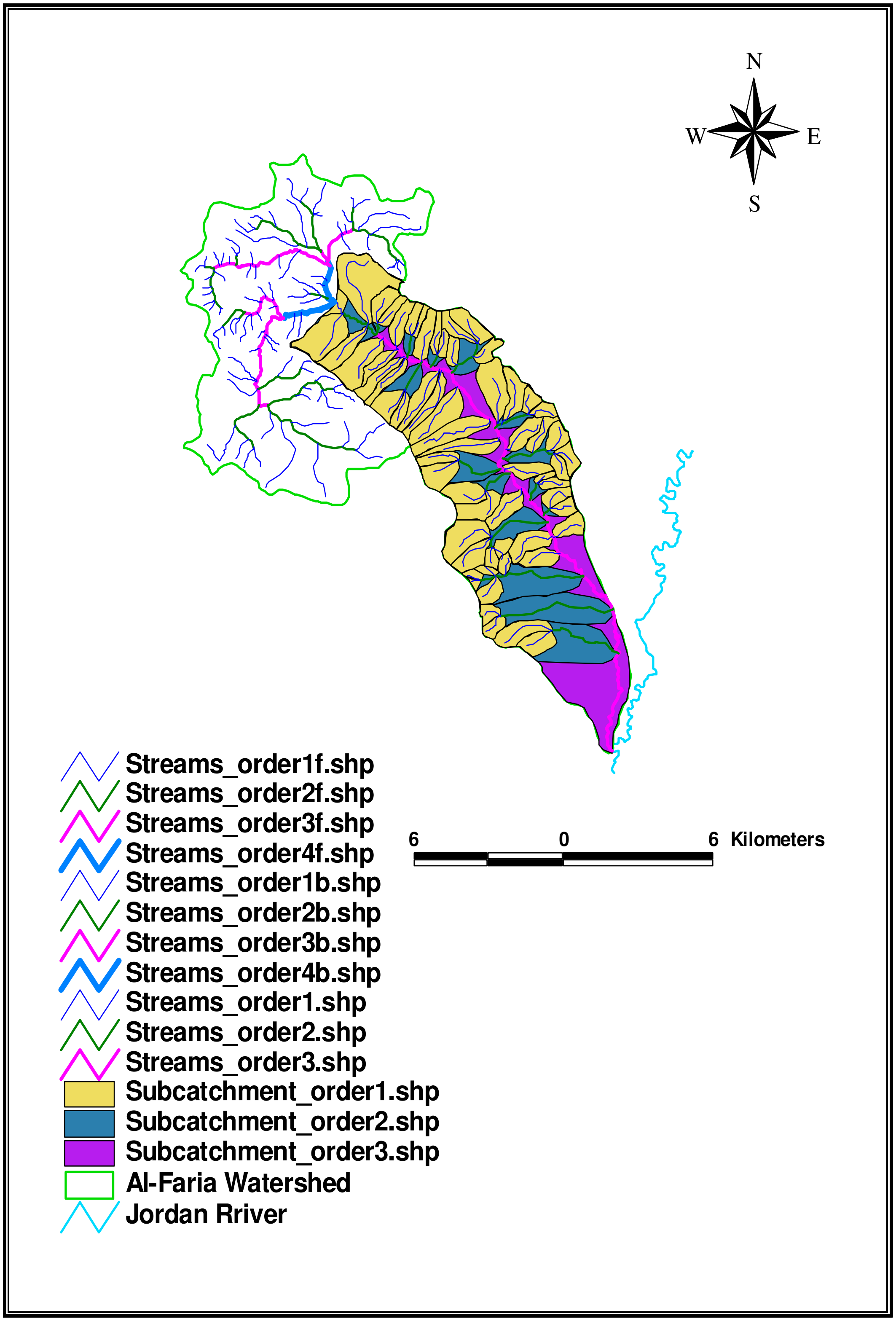

Figure 3-11 Overflow contributing areas for each of the stream orders of Malaqi Subcatchment as prepared using ArcView GIS capabilities

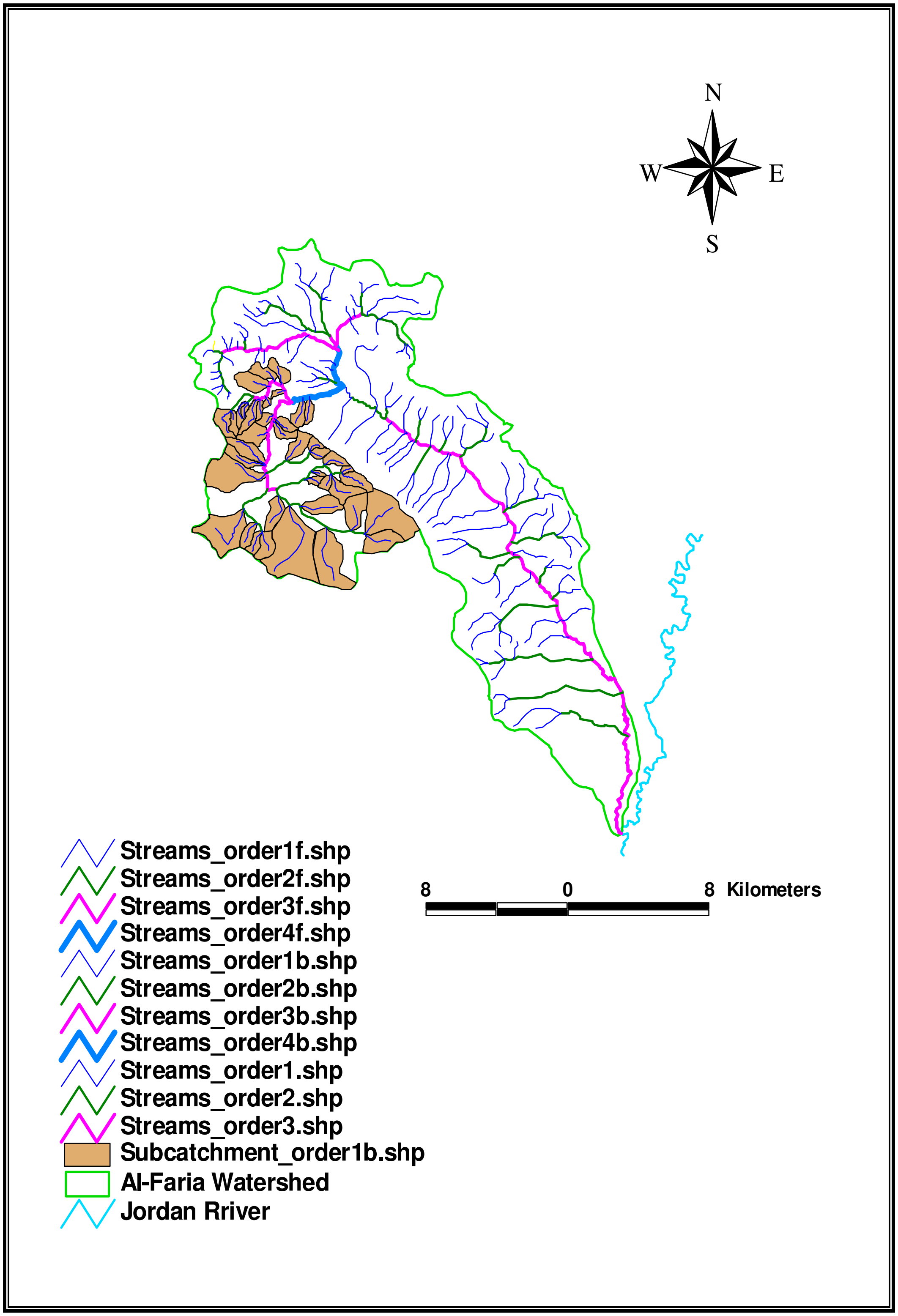

Figure 3-12 **Overflow contributing areas for stream order 1 of Badan Subcatchment as prepared using ArcView GIS capabilities**

As an example, the visual and tabulated geomorphic input parameters for stream order 1 of Badan subcatchment as extracted from the ArcView GIS capabilities is shown in Figure 3-12 and Table 3-6 respectively. The above information together with Equations 3-14, 3-15 and 3-18 were used to calculate Ni, $\bar{L}_{ci}$, $\bar{A}_i$, P_{OA_i}, $\bar{S}_{ci}$ and $\bar{S}_{oi}$ as given in Table 3-7. Similar methodology was followed for determining the KW-GIUH model input parameters for other stream orders and for other subcatchments.

The selection of the other model input parameters namely overland flow roughness coefficient n_o and channel flow roughness coefficient n_c was from previously published research (Engman, 1986; Australian Rainfall and Runoff, 1987; Storm Water Management Manual, 1990; Chow, 1964; Chow et al., 1988; Maidment, 1993; Lee and Yen, 1997; Yen and Lee, 1997; Lee and Chang; 2005; Yen, 1986). Based on vegetation cover, surface roughness and catchment characteristics the overland roughness coefficient values vary between 0.5 to 1 and can reach up to 5 especially if interflow is a dominant part of the runoff in the catchment (Lee, 2005, Personal communications with the model developer). This is clear in the Faria catchment which is characterized as a karstic area with dominant interflow (Rofe and Rafety, 1965, Ghanem, 1999). Emmett (1978) studied overland flow on natural rangeland hillslopes and reported extreme variability in flows due to topographic and vegetation irregularities that yielded high overland roughness coefficient values of 1 or more. Engman (1986) recommended values in the range of 0.39 to 0.63 for grass covered lands. In general, vegetation retards overland flow allowing more time for water to enter the soil. Vegetables and field crops dominate in Faria subcatchment and as a result n_o value was taken as 1 (Table 3-7). The total cultivated area in Badan subcatchment is about two times that of Faria subcatchment. In addition trees are dominant in this subcatchment. A mix of all crops exists in the Malaqi subcatchment (Ministry of Agriculture, 2005a, PCBS, 2003). A n_o values of 2 and 1.5 were taken for Badan and Malaqi subcatchments respectively.

The value of the channel flow roughness coefficient was taken as 0.03 (Table 3-7) based on literature and field investigation of the channel conditions (Linsely et al., 1982, Australian Rainfall and Runoff, 1987, Chow, 1964, Chow et al. 1988, Maidment, 1993, Lee and Yen, 1997, Yen and Lee, 1997, Lee and Chang, 2005).

Table 3-6 Badan Subcatchment stream order 1 geomorphological data as prepared using ArcView GIS capabilities

Channel Length (m)	Elevation Difference (m)	Channel Slope (m/m)	Area (m^2)
952	123	0.129	658943
536	108	0.201	1192569
1245	70	0.056	841965
1316	52	0.040	4236927
1761	251	0.143	4120573
1090	360	0.330	939644
967	377	0.390	1469173
692	157	0.227	1301462
2151	323	0.150	905341
810	144	0.178	1180918
1058	134	0.127	471548
718	151	0.210	335427
658	45	0.068	392658
1125	39	0.035	236204
2182	252	0.115	672993
1349	104	0.077	609173
1766	89	0.050	1242968
1436	84	0.058	360425
584	105	0.180	435192
1181	139	0.118	1475248
1720	267	0.155	1208170
859	167	0.194	227612
1184	184	0.155	765421
1617	192	0.119	1458663
911	79	0.087	830279
3311	152	0.046	907761
537	103	0.192	436695
496	97	0.196	440912
693	72	0.104	486797
444	100	0.225	155850
492	29	0.059	191561
834	144	0.173	217998
4531	164	0.036	1999416
2142	197	0.092	5034478
1751	131	0.075	1838732
3157	157	0.050	4697354
1651	53	0.032	7900747
2299	121	0.053	1290577
1538	97	0.063	1636814
1480	595	0.402	798499
1304	432	0.331	593398

Table 3-7 provides the geomorphological characteristics of the three subcatchments needed for the KW-GIUH model. The calculated stream network transitional probability $P_{x_i x_j}$ (Equation 3-18) for the three sub catchments are shown in Table 3-8. The 1mm-GIUH hydrographs for the three subcatchments are plotted from the KW-GIUH model (Lee and Chang, 2001) output results as shown in Figure 3-13. The model uses the equations given in Appendix II to obtain the KW-GIUH.

Table 3-7 KW-GIUH Input parameters for Faria, Badan, and Malaqi Subcatchments

Parameter	Faria subcatchment				Badan subcatchment				Malaqi subcatchment		
	Order				Order				Order		
	1	2	3	4	1	2	3	4	1	2	3
N_i	49	8	3	1	41	6	2	1	62	16	1
$\overline{L}_{ci\,(m)}$	1031	2120	3496	2621	1379	3202	5027	3172	1920	2611	32084
$\overline{A}_i\,(km^2)$	0.937	5.099	18.365	64.0	1.370	10.12	40.73	85.28	1.81	8.38	184.96
P_{OA_i}	0.717	0.11	0.142	0.031	0.66	0.31	0.018	0.012	0.606	0.285	0.109
$\overline{S}_{ci}$ (*m/m*)	0.117	0.058	0.033	0.031	0.14	0.062	0.051	0.029	0.14	0.063	0.01
$\overline{S}_{oi}$ (*m/m*)	0.107	0.085	0.161	0.093	0.17	0.092	0.14	0.135	0.146	0.122	0.081
Area (km^2)	64.0				85.28				184.96		
n_o	1.0				2.0				1.5		
n_c	0.03				0.03				0.03		
B_Ω (*m*)	3.70				4.60				5		

Table 3-8 Stream network transitional probability for the three Subcatchments

Description	Badan	Faria	Malaqi
$P_{1,2}$	0.61	0.74	0.73
$P_{1,3}$	0.34	0.22	0.27
$P_{1,4}$	0.05	0.04	0
$P_{2,3}$	1	0.87	1
$P_{2,4}$	0	0.13	0
$P_{3,4}$	1	1	0

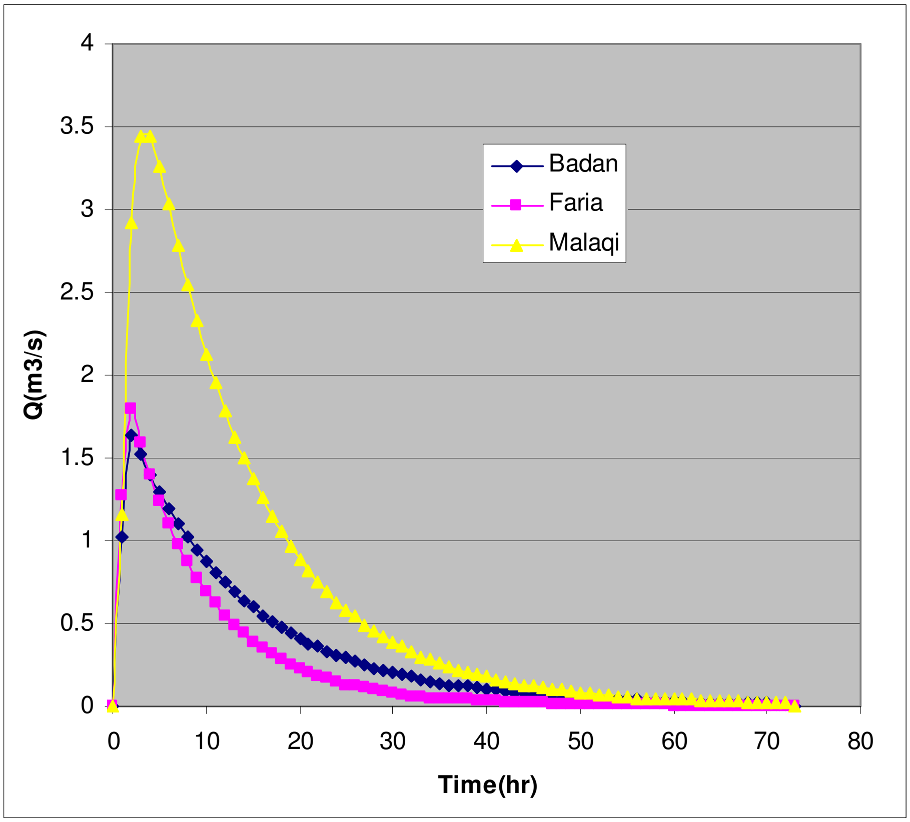

Figure 3-13 Theoretical 1mm-KW-GIUH for Faria, Badan and Malaqi Subcatchments

The rainfall excess for a unit time (rainfall excess intensity q_L) is taken as 1 mm/hr when producing graphs in Figure 3-13. The most important feature of the KW-GIUH model is a set of IUHs for different amounts of rainfall excess for a unit of time (q_L). Application of the KW-GIUH model to generate IUHs for different q_L was investigated. Rainfall excess intensity q_L values were 0.1 mm/hr, 0.5 mm/hr and 1 mm/hr and IUHs were calculated for all three subcatchments. IUHs for Badan, Faria and Malaqi subcatchments are shown in Figure 3-14, Figure 3-15 and Figure 3-16 respectively. The peak of the IUH for the three subcatchments increases with increasing rainfall excess intensity while the time to peak decreases as shown in Table 3-9. It is important to note that the increase in peak (u_p) is not linearly proportional to the increase in rainfall excess intensity.

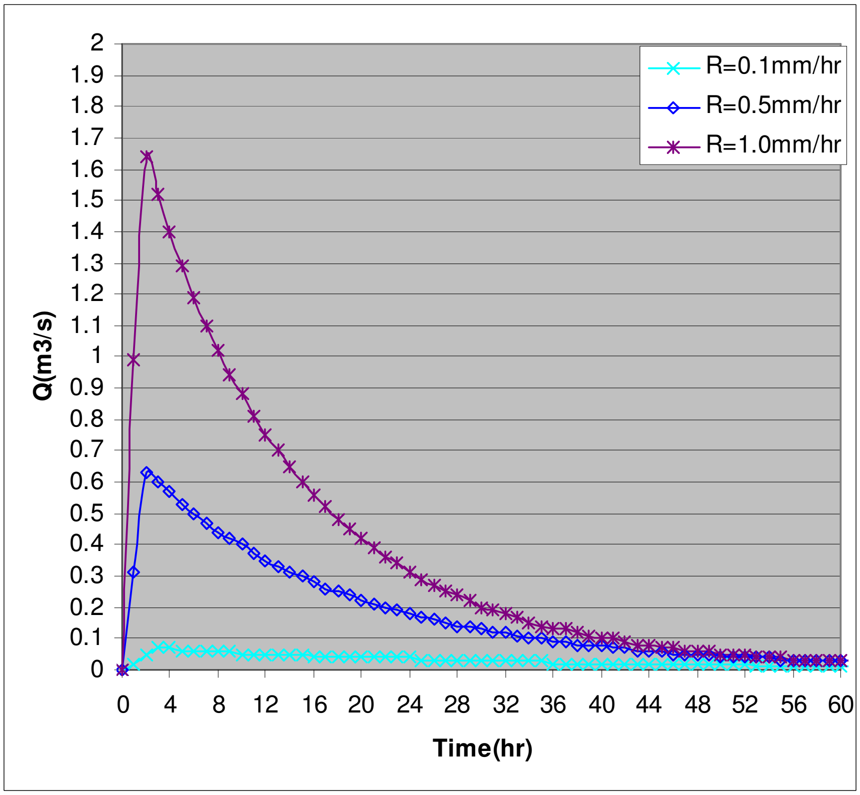

Figure 3-14 Variation of Badan subcatchment KW-GIUH with flow rate

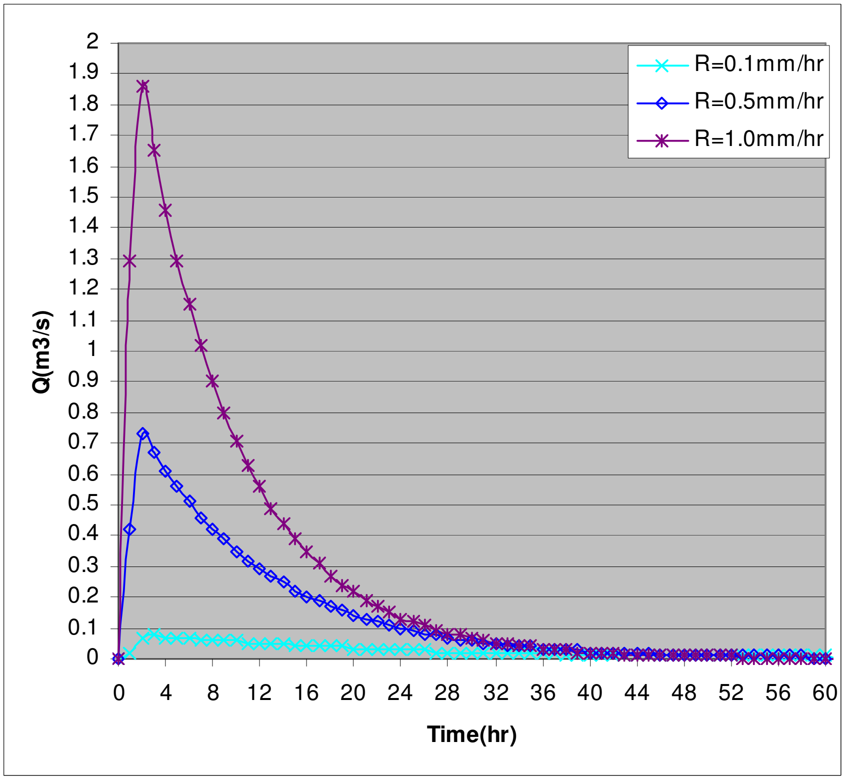

Figure 3-15 Variation of Faria subcatchment KW-GIUH with flow rate

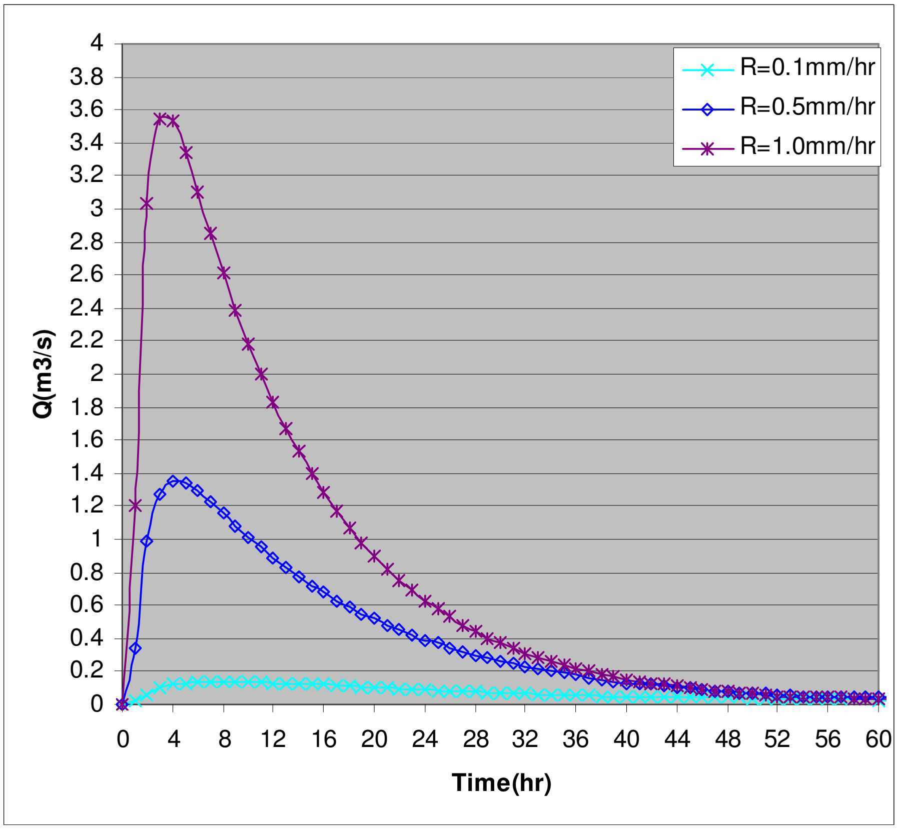

Figure 3-16 Variation of Malaqi subcatchment KW-GIUH with flow rate

Table 3-9 Peak discharge values and time to peak for different values of rainfall excess for Badan, Faria and Malaqi subcatchments

Sub-catchment	Value	0.1 mm	0.5 mm	1.0 mm
Badan	Qpeak (m^3/s)	0.07	0.63	1.64
	Time to Peak (hr)	3	2	2
Faria	Qpeak (m^3/s)	0.08	0.73	1.86
	Time to Peak (hr)	3	2	2
Malaqi	Qpeak (m^3/s)	0.14	1.35	3.54
	Time to Peak (hr)	6	4	3

In application of the KW-GIUH model to any rainwater storm, it is necessary to calculate IUH for different amounts of q_L prior to linear superposition is applied to combine the component hydrographs of hourly rainfall excess to produce the complete surface runoff hydrograph (Lee and Yen, 1997). The dimensionless relationships for the IUH peak (u_p^*) and time to peak (t_p^*) with q_L as generated by the KW-GIUH model in the Faria catchment are shown in Equations 3-20 to 3-25. The nonlinearity of the IUH is shown as the exponents in Equations 3-20 to 3-25.

Badan Subcatchment

$$\frac{u_p}{u_p^*} = 1.002\left(\frac{q_L}{q_L^*}\right)^{0.731} \qquad (3\text{-}20)$$

$R^2=1$

$$\frac{t_p}{t_p^*} = 0.707\left(\frac{q_L}{q_L^*}\right)^{-4.824} \qquad (3\text{-}21)$$

$R^2=0.92$

Faria Subcatchment

$$\frac{u_p}{u_p^*} = 0.996\left(\frac{q_L}{q_L^*}\right)^{0.73} \qquad (3\text{-}22)$$

$R^2=1$

$$\frac{t_p}{t_p^*} = 0.707\left(\frac{q_L}{q_L^*}\right)^{-4.824} \qquad (3\text{-}23)$$

$R^2=0.92$

Malaqi Subcatchment

$$\frac{u_p}{u_p^*} = 0.998\left(\frac{q_L}{q_L^*}\right)^{0.712} \qquad (3\text{-}24)$$

$R^2=1$

$$\frac{t_p}{t_p^*} = 1.107\left(\frac{q_L}{q_L^*}\right)^{-3.364} \qquad (3\text{-}25)$$

$R^2=0.98$

where u_p and t_p are the peak and time to peak respectively, u_p^* and t_p^* are the peak and the time to peak of the IUH for $q_{L=}\ q_L^* = 1.0$ mm/hr.

3.4 APPLICATION AND VERIFICATION OF KW-GIUH MODEL TO FARIA CATCHMENT

3.4.1 Baseflow Separation

In arid areas the surface runoff is very precious. For validation of hydrology components of models, direct runoff and baseflow components of the stream flow hydrograph typically need to be separated, because direct runoff and baseflow are usually simulated separately in computer model (Srinivasan and Arnold, 1994). The KW-GIUH gives only the surface runoff while the observed hydrograph consists of both surface runoff and baseflow. The total flow in the streams is divided into two parts, storm or direct runoff and baseflow. The distinction is actually on the basis of time arrival in the stream rather than on the path followed. Direct runoff is presumed to consist of surface runoff and a substantial portion of the interflow (water moving laterally through the upper soil layers until entering a stream channel), whereas baseflow is considered to be largely groundwater (Linsely et al., 1982, Wilson, 1990, Australian Rainfall and Runoff, 1987).

The shape of the hydrograph varies depending on physical and meteorological conditions in a catchment (Bendient and Huber, 2002), thereby complicating hydrograph analysis. The first step of separating the baseflow is to identify the starting and ending points of direct runoff. Direct runoff starts when the flow starts to increase while the ending point can be identified when a plot of log flow rate against time becomes a straight line (Chapman, 1999). There are several graphical methods to define baseflow between these starting and ending points (Chow et al., 1988). However, graphical methods might not be very efficient when separating baseflow for long time periods and can result in inconsistent results (Lim et al, 2005).

In this study, it is planned to use the recursive digital filters (Eckhardt, 2004) incorporated in the Web-based Hydrograph Analysis Tool (WHAT) system; available at http://pasture.ecn.purdue.edu/~what (Lim et al, 2005) to separate baseflow from observed flow. The WHAT system provides an efficient tool for baseflow separation. Data from fifty gauging stations in Indiana, with drainage area ranging from 33 km^2 to 313,933 km^2, indicated typical matching of baseflow using the recursive digital filters (Eckhardt, 2004) with those obtained from manual separation and measured baseflow. For all the fifty gauging stations coefficient of efficiency values were over 0.91 and the coefficient of determination values were over 0.98 (Lim et al., 2005).

The Australian Water Balance Model (AWBM) rainfall runoff model developed by Boughton (2004) has been used successfully to separate baseflow from streamflow. The AWBM model of the Rainfall Runoff Library (RRL) toolkit (CRC for Catchment Hydrology, 2004) was used for comparison of the baseflow separation results obtained from the WHAT system. The baseflow index (BFI) calculated from both above methods were compared. Following sections describe the WHAT model and AWBM model and their application to separate the baseflow from the observed data obtained from Badan and Faria subcatchments.

Application of the WHAT model

As stated above the WHAT model incorporates recursive digital filters (Eckhardt, 2004) for baseflow separation. Filtering direct runoff from baseflow is similar to signal analysis and processing (Eckhardt, 2004). The digital filter method has been used in signal analysis and processing to separate high frequency signal from low frequency signal. This method has been used in baseflow separation because high frequency waves can be associated with the direct runoff and low frequency waves can be associated with the baseflow (Eckhardt, 2004).

The recursive digital filtering of hydrographs serves to partition the streamflow into two components, direct runoff and baseflow as given in Equation 3-26 (Eckhardt, 2004).

$$y_k = f_k + b_k \tag{3-26}$$

where,

y = total streamflow,

f = direct runoff,

b = baseflow,

k = time step number

Eckhardt (2004) proposed the general form of a digital filter considering a digital filter parameter and a maximum value of Baseflow Index BFI_{max} (maximum value of long-term ratio of baseflow to total stream flow) as given in Equation 3-27.

$$b_k = [\ (1 - BFI_{max}) * a * b_{k-1} + (1 - a) * BFI_{max} * y_k\ ] / (1 - a * BFI_{max}) \qquad (3\text{-}27)$$

where,

b_k= Baseflow at time step k,

b_{k-1}= Baseflow at time step k-1,

y_k=Total streamflow at time step k,

a =Filter parameter ,

BFI_{max}= Maximum value of Baseflow Index. Representative BFI_{max} values were estimated by Eckhardt (2004) for different hydrological and hydrogeological situations as follows:

- 0.80 for perennial streams with porous aquifers,
- 0.50 for ephemeral streams with porous aquifers,
- 0.25 for perennial streams with hard rock aquifers.

Average daily measured runoff data for Badan and Faria subcatchments were calculated and entered to the program to obtain the baseflow values for each subcatchment. Faria catchment has a porous aquifer (Ghanem, 1999; WESI, 2005; Rofe and Raffety, 1965). Accordingly a value of 0.8 was used for BFI_{max} while a filter parameter value of 0.98 was used as recommended by Eckhardt (2004). Figure 3-17 and Figure 3-18 show the baseflow separation for the Badan and Faria subcatchments using the WHAT model.

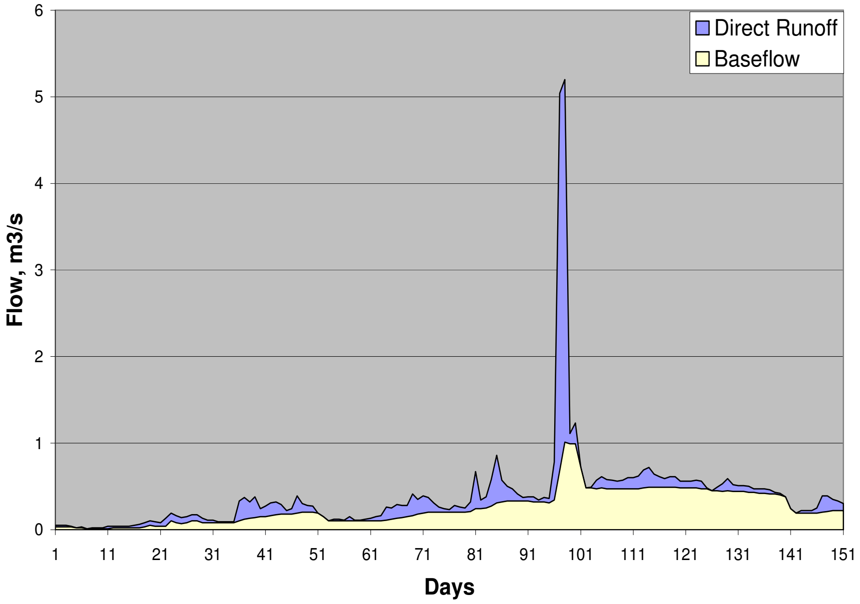

Figure 3-17 Baseflow separation for Badan subcatchment using the WHAT model

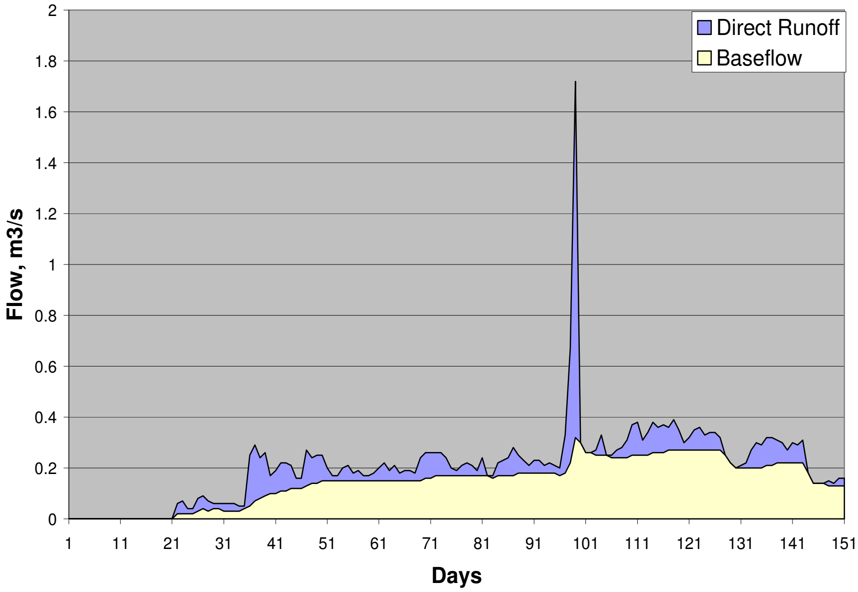

Figure 3-18 Baseflow separation for Faria subcatchment using the WHAT model

Application of the AWBM model

The Australian Water Balance Model (AWBM) is a catchment water balance model that relates runoff to rainfall to produce flood hydrographs on daily basis (Boughton, 2004). It is based on overland flow generation and surface storage of catchment areas. The model was developed in the early 1990s and is now one of the most widely used rainfall-runoff models in Australia. Recently the model was adapted for use on ungauged catchments with calibrations on 221 catchments in mainland Australia (Boughton, 2005).

As detailed in Boughton (2004), the model consists of three parameters namely surface storage capacity, baseflow recession constant and baseflow index. The AWBM model requires three input data files, daily rainfall data, average daily actual evapotranspiration (ETa) data and daily flow data. Figure 3-19 shows the schematic diagram of the AWBM model. There are three surface moisture stores that allow for partial area runoff generation. When runoff is generated from one or more of the stores, it is divided into surface runoff and baseflow recharge. There is a surface runoff store that attenuates the surface runoff component and, similarly, the baseflow store attenuates the baseflow component of streamflow. The division of generated runoff into surface runoff and baseflow is determined by the baseflow index (BFI ranges from 0 to 1). The recharge of the baseflow store is BFI*Excess, where Excess is the amount of generated runoff, and the recharge of the surface runoff store is (1-BFI)*Excess.

The daily discharge from the baseflowstore into streamflow is (1-Kb)*BS, where BS is the amount of moisture in the baseflow store and Kb is the daily baseflow recession constant. Similarly, the daily discharge from the surface runoff store is (1-Ks)*SS, where SS is the amount of moisture in the surface runoff store, and Ks is the daily surface runoff recession constant. There are five parameters for the generation of runoff, three capacities of the surface stores and two partial areas. The three partial areas must sum to 1, so only two can be evaluated and the third is automatically determined. There are three other parameters, the BFI and the two daily recession constants Kb and Ks. The surface store parameters determine the amount of runoff and the other three parameters determine the timing of the runoff. A significant feature of the AWBM has been the development of calibration procedures based on the structure of the model rather than using trial and error testing of different sets of parameter values. In operation, AWBM assumes default values for the baseflow parameters, BFI and Kb and the surface runoff recession constant (Ks) to make a preliminary calibration of the surface stores. The preliminary calibration of the surface storage parameters makes total calculated runoff equal to the total actual runoff. After this preliminary calibration, the BFI, Kb, and Ks are calibrated in that order.

Daily measured rainfall and runoff data for both subcatchments were entered to the program. Actual daily evapotranspiration for the study area is calculated from potential monthly evapotranspiration input values (Tables 3-2 and 3-3) and crop coefficient values as shown in Equation 3-28.

$$ET_a=ET_o*K_c \qquad (3\text{-}28)$$

where,

ET_a = Daily Actual evapotranspitration, mm/day

ET_o = Daily Reference (potential) evapotranspitration, mm/day

K_c = Crop coefficient

The average daily actual evapotranspiration was estimated as a spatially averaged evapotranspiration rate of the catchment based on the types of major crops cultivated in the catchment, and as given by Equation 3-29. Crop coefficient varies according to the type of crop and the stage of growth. Values of K_c were based on the revised FAO methodology for crop water requirements (FAO, 1998b).

The actual evapotranspiration for each subcatcment (ET_c) was estimated using the weighted average equation given by Equation 3-29.

$$ET_c = \sum_{i=1}^{n} (ET_{ai} * A_i)/A_n \qquad (3\text{-}29)$$

where,

ET_c = Weighted average actual evapotranspitration (mm/day)

ET_{ai} = Actual evapotranspitration of crop I (mm/day)

A_i = Area of crop i (ha)

A_n = Total cultivated area (ha)

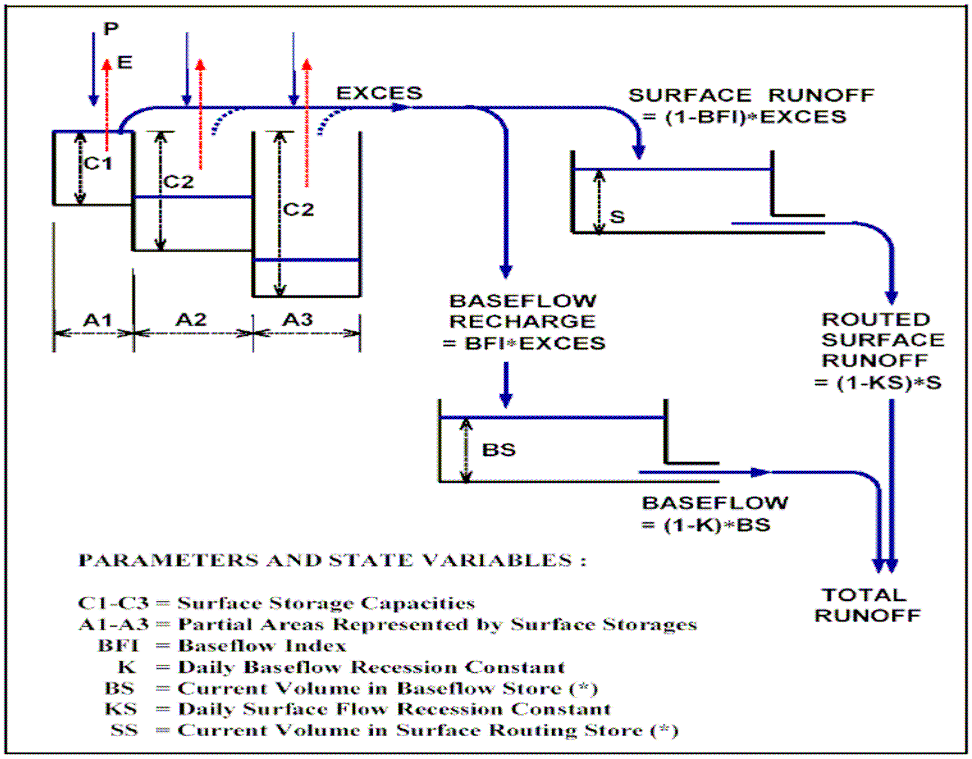

Figure 3-19 Schematic diagram of the AWBM model (Boughton, 2004)

The resulting ET_c values for each subcatchment for the rainy months are shown in Table 3-10. Further details on the methodology for estimating crop water requirements and crop coefficients as well as the cropping patterns are given in Chapter 5 which discusses the AGSM.

Figure 3-20 and Figure 3-21 show the baseflow separation for the Badan and Faria subcatchments using the AWBM model

Table 3-10 Weighted average actual evapotranspitration (ET_c) for each subcatchment for the rainy months

Month	Weighted average actual evapotranspitration (ET_c) (mm/day)	
	Badan subcatchment	Faria subcatchment
November	1.6	2.5
December	1	0.8
January	1	0.8
February	1.3	1.8
March	1.9	2.8

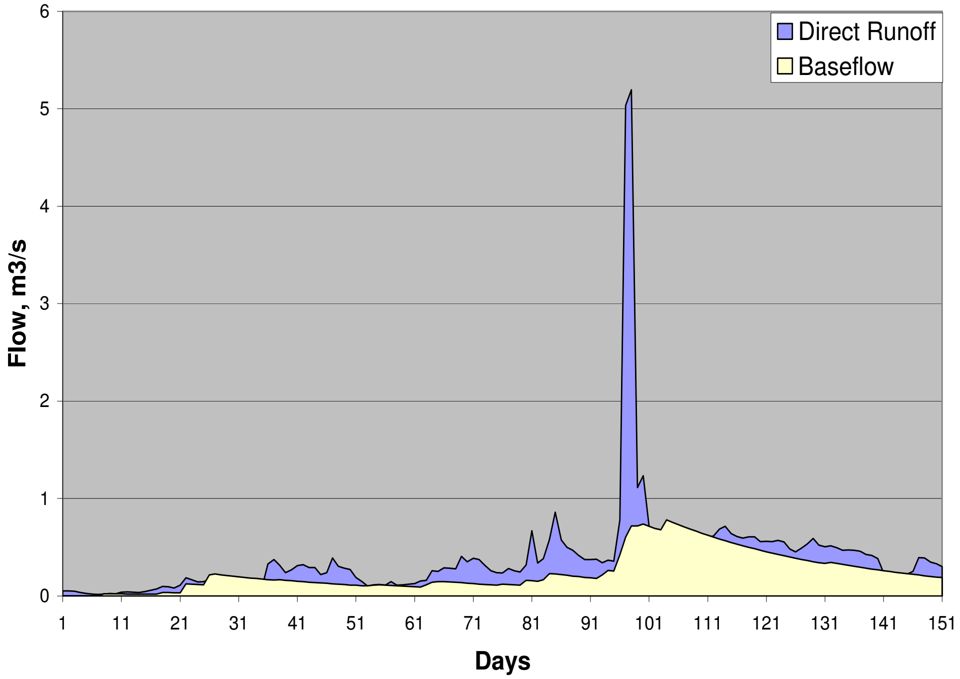

Figure 3-20 Baseflow separation for Badan subcatchment using AWBM model

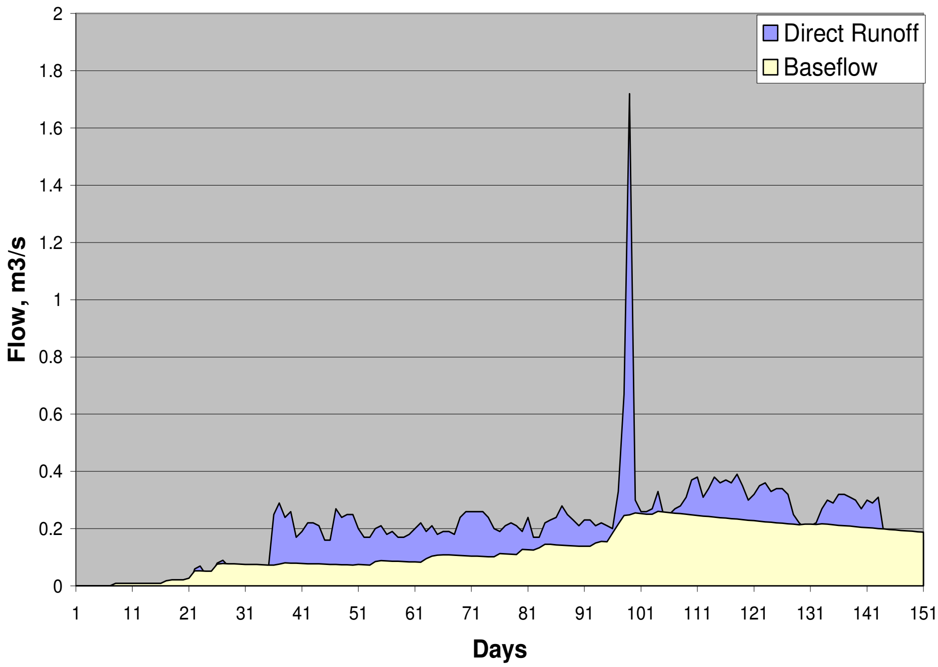

Figure 3-21 Baseflow separation for Faria subcatchment using the AWBM model

Comparison of BFI obtained from WHAT and AWBM models

Baseflow Index (BFI) defined as the ratio of baseflow to total stream flow obtained from AWBM model was compared to the value obtained from the WHAT model. Table 3-11 summarizes the breakdown of the baseflow separation process as well as BFI calculated using WHAT and AWBM models. The two models showed almost equal amounts of baseflow and surface runoff. The baseflow values for Badan subcatchment obtained from the WHAT and AWBM models were 3270 ML and 3180 ML respectively; and those for Faria subcatchment were 1890 ML and 1720 ML respectively. The daily average surface runoff peak values were 5.2 m^3/s and 1.7 m^3/s for Badan and Faria streams respectively.

Table 3-11 Baseflow volume and Baseflow Index (BFI) from WHAT and AWBM models

Subcatchment	Baseflow ML		Direct Runoff ML		Total Flow ML		BFI	
	WHAT	AWBM	WHAT	AWBM	WHAT	AWBM	WHAT	AWBM
Badan	3270	3180	1830	1840	5100	5020	0.64	0.63
Faria	1890	1720	790	810	2680	2530	0.70	0.68

The two models also showed good agreement with regard to the direct runoff volumes from both Badan and Faria subcatchments (Table 3-11). The results of the AWBM and WHAT models will be further analyzed in relation to KW-GIUH results in the following sections. The WHAT model will be used to separate baseflow from a hydrograph to obtain the surface runoff.

3.4.2 Excess Rainfall

In order to generate the surface runoff hydrograph, hourly values of rainfall excess from the hyetograph should be applied to the developed IUH. Excess rainfall or effective rainfall is that rainfall which is neither retained on land surface nor infiltrated into the soil. After flowing across the catchment surface, excess rainfall becomes direct runoff at the catchment outlet. The graph of excess rainfall versus time or excess rainfall hyetograph is a key component of the study of rainfall-runoff relationships. The difference between the observed total rainfall hyetograph and the excess rainfall hyetograph is termed abstractions or losses. Losses are primarily water absorbed by infiltration with some allowance for interception and surface storage (Chow et al., 1988). While a number of models have been proposed for estimating rainfall excess, the most commonly used models are the index models and the USDA Soil Conservation Service (SCS, 1972) runoff curve number model (Maidment, 1993). In this study, the SCS method was developed for the study area to test the applicability of this method for runoff modelling as previously discussed in Section 2.7 and presented in Appendix I. Results indicated that the SCS method overestimated the excess rainfall and is not suitable to be used for the estimation of runoff amounts for such conditions. A widely used model in hydrologic modeling is the Horton infiltration model (Horton, 1942). In this study, the Horton model was used to estimate the amounts of excess rainfall that are needed for the rainfall-runoff modeling.

Excess rainfall-Horton model

The Horton three-parameter empirical infiltration model (Equation 3-30) was used to estimate excess rainfall (Horton, 1942) from the observed total rainfall.

$$f_p=f_c+(f_o-f_c)e^{-kt} \quad (3\text{-}30)$$

where,

- f_p is the infiltration capacity (mm/hr),

- f_o is the maximum infiltration rate at the beginning of a storm event (mm/hr),
- f_c is the final infiltration rater(mm/hr), and
- k is a constant that controls the rate of decrease in the infiltration capacity

The maximum infiltration rate (f_o) at the beginning of a storm event reduces to a low and approximately constant rate (f_c) as the infiltration continues and the soil becomes saturated (Maidment, 1993). The f_o, f_c and k are rarely measured locally but more commonly they are obtained from values published in the literature for different types of soils (Pit et al., 1999). Values for the parameters f_o, f_c and k were 15 mm/hr, 4.5 mm/hr and 0.2 respectively and were obtained from available literature (WESI, 2005; Pit et al., 1999; Mishra and Singh, 2004; Mishra et al., 2004; Horton, 1942). Figure 3-22, Figure 3-23 and Figure 3-24 show the total rainfall and Horton infiltration rate for the major storm event for Faria, Badan and Malaqi subcatchments respectively that was recorded in February during the rainy season 2004/2005. Excess rainfall was calculated by subtracting the infiltration capacity f_p from the total rainfall intensity at a particular time.

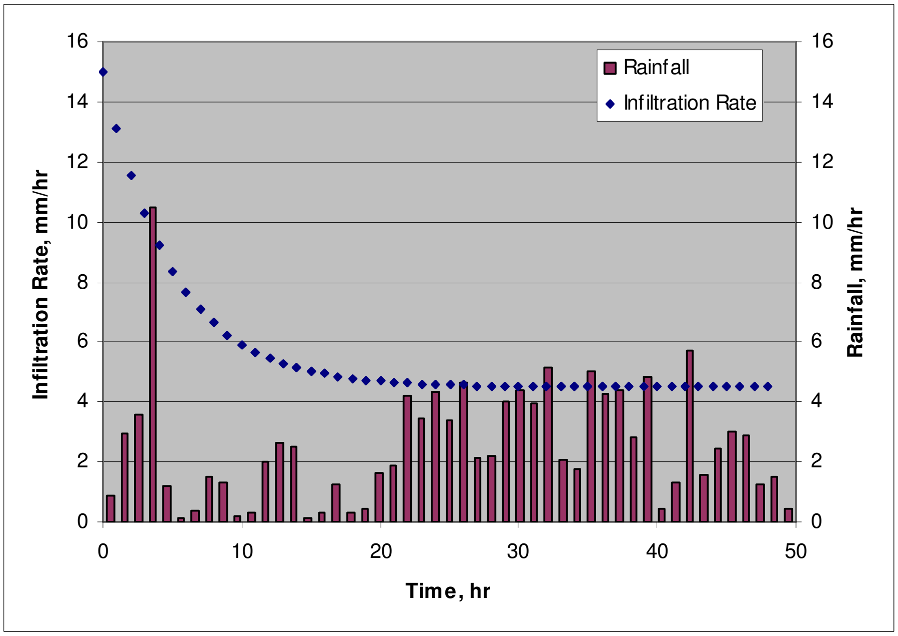

Figure 3-22 Total rainfall and Horton's infiltration rate for the major storm event for Faria subcatchment

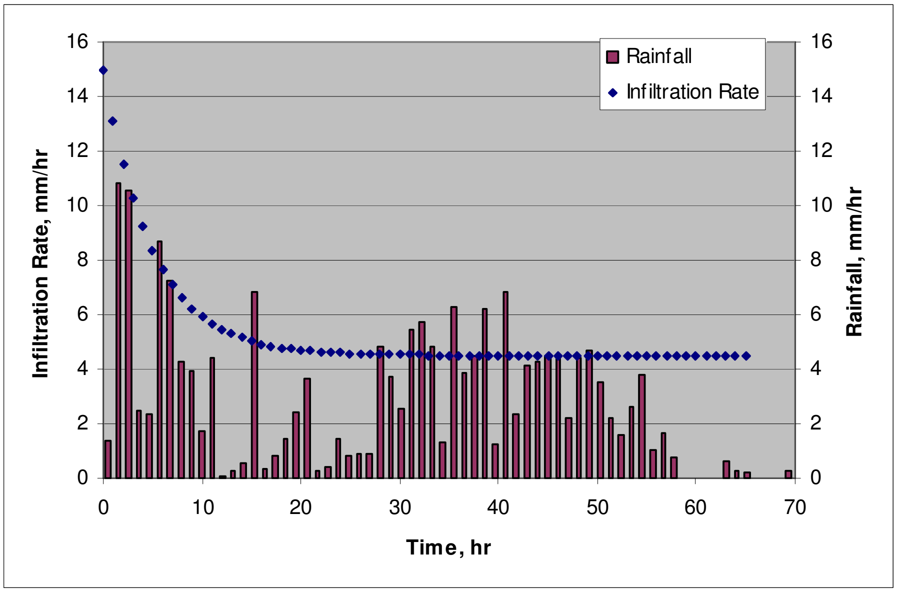

Figure 3-23 Total rainfall and Horton's infiltration rate for the major storm event for Badan subcatchment

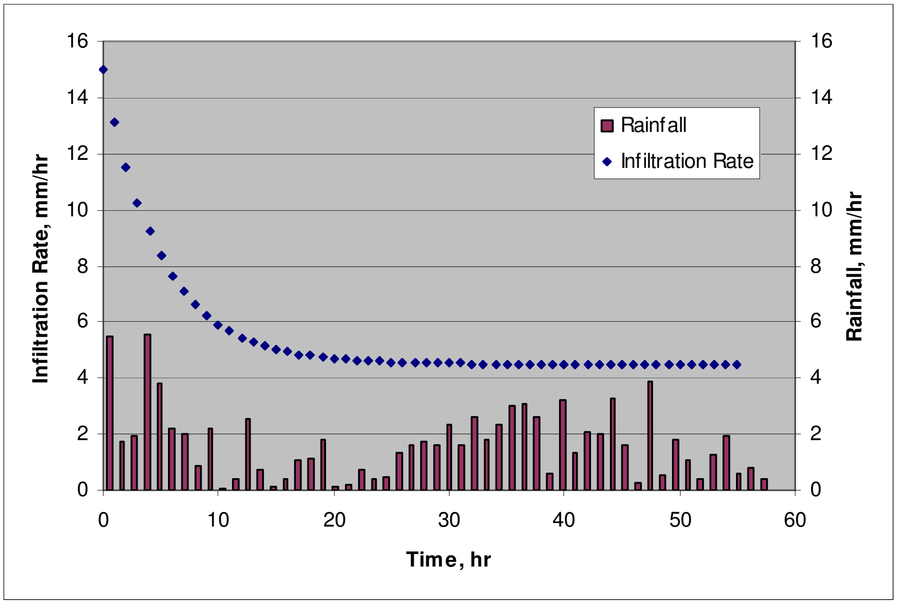

Figure 3-24 Total rainfall and Horton's infiltration rate for the major storm event for Malaqi subcatchment

Table 3-12 shows the results of the excess rainfall estimated by the Horton method for each rainfall event in Badan, Faria and Malaqi subcatchments. The accuracy of the infiltration parameters could be also checked by comparing the excess rainfall volume obtained from the Horton (excess rainfall*catchment area) with the surface runoff volume obtained after the baseflow separation from observed storm data using the WHAT model. The excess rainfall volume obtained from the Horton is 1757 ML and 702 ML for Badan and Faria subcatchments respectively. The percent error between the excess rainfall and surface runoff volume is 3.9 % and 10.9 % for Badan and Faria subcatchments respectively.

Table 3-12 Excess rainfall estimated by the Horton method for each rainfall event in Badan and Faria subcatchments

Subcatchment	Date	Event Rainfall (mm)	Excess Rainfall- Horton (mm)
Badan	17/11/04	21.8	0.00
	21/11	40.62	0.00
	26/11	46.85	0.00
	7/12	12.21	0.00
	24/12	19.28	0.60
	2/01/05	27.46	0.00
	15/01	15.16	0.00
	19/01	21.51	0.00
	23/01	27.08	2.30
	1/02	42.85	1.92
	4/02	185.9	10.70
	11/02	28.73	3.19
	10/03	11.24	1.96
Faria	17/11/04	24.37	0.00
	21/11	55.05	0.00
	26/11	48.19	0.00
	7/12	15.08	0.52
	24/12	26.19	1.44
	2/01/05	36.48	0.00
	15/01	20.31	0.00
	19/01	28.9	0.00
	23/01	21.8	1.45
	1/02	37.08	1.23
	4/02	123.4	2.90
	11/02	24.91	1.59
	10/03	17.38	1.88
Malaqi	17/11/04	10.46	0
	21/11	18.5	0
	26/11	20.48	0
	7/12	5.2	0
	24/12	9.6	0
	2/01/05	14.44	0
	15/01	7.88	0
	19/01	11.08	0
	23/01	18.41	0
	1/02	25.85	0
	4/02	88.98	0
	11/02	9.78	0
	10/03	8.01	0

Sensitivity of excess rainfall on Horton model parameters was investigated. Table 3-13 and Table 3-14 depict the variation in rainfall excess for Badan and Faria subcatchments respectively with ±5% change in Horton infiltration parameters. No excess rainfall was obtained for Malaqi subcatchment. The results indicated that the estimation of excess rainfall was sensitive to all the Horton infiltration model parameters. The excess rainfall was more sensitive to f_c as compared to f_o and k. In arid and semi arid catchments it is important to estimate the excess rainfall accurately for planning purposes. Data regarding soil physical properties and infiltration parameters should be collected at different locations of the catchments. Such a huge project requires governmental funding and support of different research organizations. However and depending on the information available for each catchment, other methods could be used to estimate the excess rainfall such as Initial-Constant loss method (Australian Rainfall and Runoff, 1987), Green and Ampt model (1911) and SCS-CN model (SCS, 1972).

Table 3-13 Excess rainfall values for the major storm event at different values of f_c, f_o and K for Badan subcatchment (Dark column sells represent actual used values)

fo	**14.25**	**15**	**15.75**
Excess Rainfall (mm)	11.04	10.7	10.37
% Error	3.2		3.1
fc	**4.275**	**4.5**	**4.725**
Excess Rainfall (mm)	13.5	10.7	8.6
% Error	26.2		19.6
k	**0.19**	**0.2**	**0.21**
Excess Rainfall (mm)	10.35	10.7	11.02
% Error	3.2		3.0

Table 3-14 Excess rainfall values for the major storm event at different values of f_c, f_o and K for Malaqi subcatchment (Dark column sells represent actual used values)

f_o	**14.25**	**15**	**15.75**
Excess Rainfall (mm)	3.32	2.9	2.69
% Error	14.4		7.2
f_c	**4.275**	**4.5**	**4.725**
Excess Rainfall (mm)	4.32	2.9	1.8
% Error	48.9		37.9
k	**0.19**	**0.2**	**0.21**
Excess Rainfall (mm)	2.68	2.9	3.09
% Error	7.5		6.5

3.4.3 Model Verification

Since all models and their parameters are approximations to reality, there is a general need for verification of these models against observed data. In addition, sensitivity analysis was conducted for model parameters to test the effect of each parameter on the model results.

As described in Section 3.2.2, during the rainy season of 2004/2005 a major storm recorded during February was used to verify the model. Runoff data were measured through two Parshall flumes with data loggers that measure the flows at the two main streams of upper Faria subcatchments, Faria and Badan. Runoff measuring devices are not available for Malaqi and accordingly runoff data for Malaqi subcatchment could not be measured. Hourly rainfall and runoff data were used to verify the developed KW-GIUH model by comparing the simulated and measured runoff during this event for Faria and Badan subcatchments. Surface runoff obtained from baseflow separation was utilized in the verification process. Sensitivity analysis was conducted for model input parameters to test the effect of each parameter on the model results.

Various criteria may be used for evaluating the suitability of the model for the catchment of interest and therefore judging the fit of a simulated to an observed hydrograph. Common measures are the differences between peak magnitudes, a measure of overall fit and different time measures (Chow, 1964; Chow et al., 1988; Maidment, 1993; Lee and Yen, 1997; Lee and Chang; 2005). Five criteria were chosen to analyze the degree of goodness of fit. These criteria can be defined as follows:

1. Runoff Volume Error (EQ_v) defined as

   $$EQ_v(\%)=[(Q_v)_{sim}-(Q_v)_{rec}/(Q_v)_{rec}]*100 \qquad (3\text{-}31)$$

   where $(Q_v)_{sim}$ is the simulated total runoff volume, and $(Q_v)_{rec}$ is the recorded total runoff volume.

2. The coefficient of determination (R^2) (Aitken, 1973 cited in Jayasuriya, 1991), which is a measure of the degree of association between the observed and the predicted values, indicates the deviation of the estimated values from the line of best fit or the regression line. R^2 is defined as

$$R^2=[\{\sum_{t=1}^{n}(Q_{rec}(t)-\overline{Q}_{rec})^2-\sum_{t=1}^{n}(Q_{rec}(t)-Q_{sim}(t))^2\}/\sum_{t=1}^{n}(Q_{rec}(t)-\overline{Q}_{rec})^2] \qquad (3\text{-}32)$$

where $Q_{rec}(t)$ is the recorded discharge at time t, $Q_{sim}(t)$ is the estimated discharge obtained from the regression line, $\overline{Q}_{rec}$ is the average recorded discharge during the storm event, and n is the number of discharge records during the storm event. The coefficient of determination will always be less than unity. A value of R^2 close to unity is an excellent result while a low value indicates inadequate modeling.

3. The coefficient of efficiency (CE) (Nash and Sutcliffe, 1970) defined as

$$CE=1-[\sum_{t=1}^{n}(Q_{rec}(t)-Q_{sim}(t))^2/\sum_{t=1}^{n}(Q_{rec}(t)-\overline{Q}_{rec})^2] \qquad (3\text{-}33)$$

where $Q_{rec}(t)$ is the recorded discharge at time t, $Q_{sim}(t)$ is the simulated discharge at time, $\overline{Q}_{rec}$ is the average recorded discharge during the storm event, and n is the number of discharge records during the storm event. The better the fit, the closer CE is to one.

4. The error of peak discharge EQ_p defined as

$$EQ_p(\%)=[(Q_p)_{sim}-(Q_p)_{rec}/(Q_p)_{rec}]*100 \qquad (3\text{-}34)$$

where $(Q_p)_{sim}$ is the peak discharge of the simulated hydrograph, and $(Q_p)_{rec}$ is the recorded peak discharge.

5. The error of the time to peak discharge ET_{pk} defined as

$$ET_{pk}=(T_p)_{sim}-(T_p)_{rec} \qquad (3\text{-}35)$$

where $(T_p)_{sim}$ is the simulated time to peak discharge, and $(T_p)_{rec}$ is the recorded time to peak discharge.

Table 3-15 summarizes the goodness-of-fit parameters between recorded and simulated hydrographs for both Badan and Faria subcatchments. Figure 3-25 and Figure 3-26 depict the recorded and simulated hydrographs for both subcatchments.

Table 3-15 Simulated and recorded hydrograph results of the major storm event for Badan and Faria subcatchments

Sub-catchment	Recorded			Simulated			Evaluation Criteria				
	Qv 1000m3	Qp (m3/s)	Tp (h)	Qv 1000m3	Qp (m3/s)	Tp (h)	EQv	R^2	CE	EQp (%)	ET_{pk} (h)
Badan	917	12.59	38	901	13.17	39	1.74	0.81	0.80	4.6	1
Faria	186	3.21	43	179	3.41	42	3.76	0.81	0.80	6.23	-1

As shown in Figure 3-25, Figure 3-26 and Table 3-15, the simulated and recorded hydrographs and flow values are in good agreement. The error of runoff volume is less than 5%. The values of the coefficient of determination and the coefficient of efficiency are 0.81 and 0.80 respectively. The error of peak discharge is less than 10%, and the error of time to peak discharge is limited to one hour.

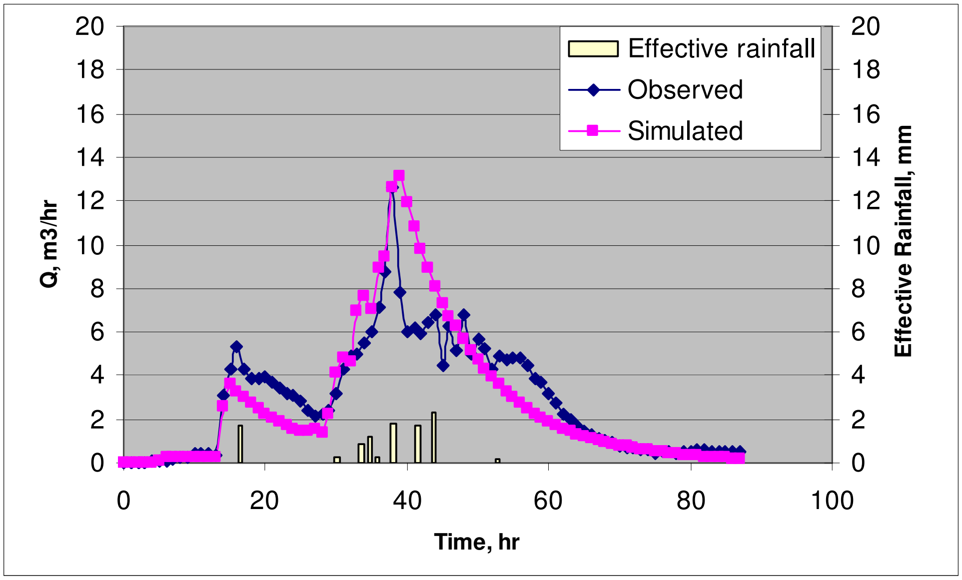

Figure 3-25 Recorded and simulated Direct Runoff Hydrograph for Badan Subcatchment

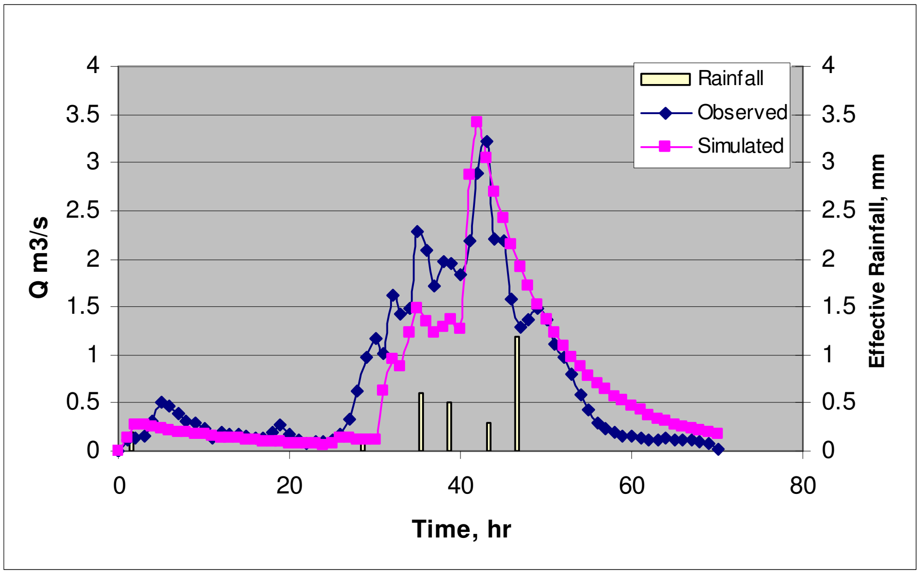

Figure 3-26 Recorded and simulated Direct Runoff Hydrograph for Faria Subcatchment

3.4.4 Sensitivity Analysis

All model parameters except, overland flow roughness coefficient (n_o), channel flow roughness coefficient (n_c) and channel width at the subcatchment outlet (B_Ω) were obtained from GIS. The channel width at the subcatchment outlet (B_Ω) can be obtained from field investigation. Two parameters, overland flow roughness coefficient (n_o) and channel flow roughness coefficient (n_c) were obtained from published literature based on catchment and channel conditions. The resulting values of n_o were 2.0, 1.0, and 1.5 for the Badan, Faria, and Malaqi subcatchments respectively. The n_c was 0.03 for each subcatchment. The criteria for selection of these values were given in Section 3.3.1. Sensitivity analysis was conducted to investigate the effect of each parameter on the produced 1mm-GIUH model.

The n_o and n_c values were changed to investigate the effect on the simulated hydrograph.Table 3-16, Figure 3-27, Figure 3-28 and Figure 3-29 show the variation for Badan, Faria and Malaqi subcatchments' IUHs for a range of overland flow roughness coefficients respectively. The channel flow roughness coefficient was fixed at 0.03. Results showed that the peak values produced by the IUHs increase as the n_o values decrease, which explicitly reflects the land surface condition of the surface hydrologic system. However, the n_o value has little impact on the time to peak. These results are consistent with those of Lee and Chang (2005). They found that the surface flow IUH is influenced by the variation of the surface roughness conditions. It is worth mentioning that the inverse relation of IUH peaks to the n_o value could be attributed to the fact that the storage effect in the KW-GIUH model is mainly incorporated in the overland flow routing that is represented by an exponential distribution (Lee and Yen, 1997). Table 3-16 presents the peak discharge values and the % errors when n_o values were changed by ± 25 % from the original value for all three subcatchments. It is possible for an error of such a magnitude to occur as n_o and n_c values are obtained from published research. A change in n_o value from 1.0 to 0.75 for Faria subcatchment, 2.0 to 1.5 for Badan subcatchment and 1.5 to 1.125 for Malaqi subcatchment can cause an error of 16 % to 17 % in the estimation of the peak flow.

Table 3-16 Peak discharge values and time to peak for different values of n_o at n_c=0.03 for Faria subcatchments 1mm- IUH (Dark column sells represent actual used values)

Sub-catchment	Value	n_o=1.5	n_o=1.75	n_o=2.0	n_o=2.25	n_o=2.5
Badan	Qpeak (m^3/s) % Error	1.91 16.5	1.76 7.3	1.64	1.53 6.7	1.45 11.6
	Time to Peak (hr)	2	2	2	2	2
Sub-catchment	Value	n_o=0.75	n_o=0.875	n_o=1.0	n_o=1.125	n_o=1.25
Faria	Qpeak (m^3/s) %Error	2.16 16.1	1.99 7.0	1.86	1.75 5.9	1.66 10.8
	Time to Peak (hr)	2	2	2	2	2
Sub-catchment	Value	n_o=1.125	n_o=1.31	n_o=1.5	n_o=1.688	n_o=1.875
Malaqi	Qpeak (m^3/s) %Error	4.1 15.8	3.79 7.1	3.54	3.34 5.6	3.17 10.5
	Time to Peak (hr)	3	3	3	3	4

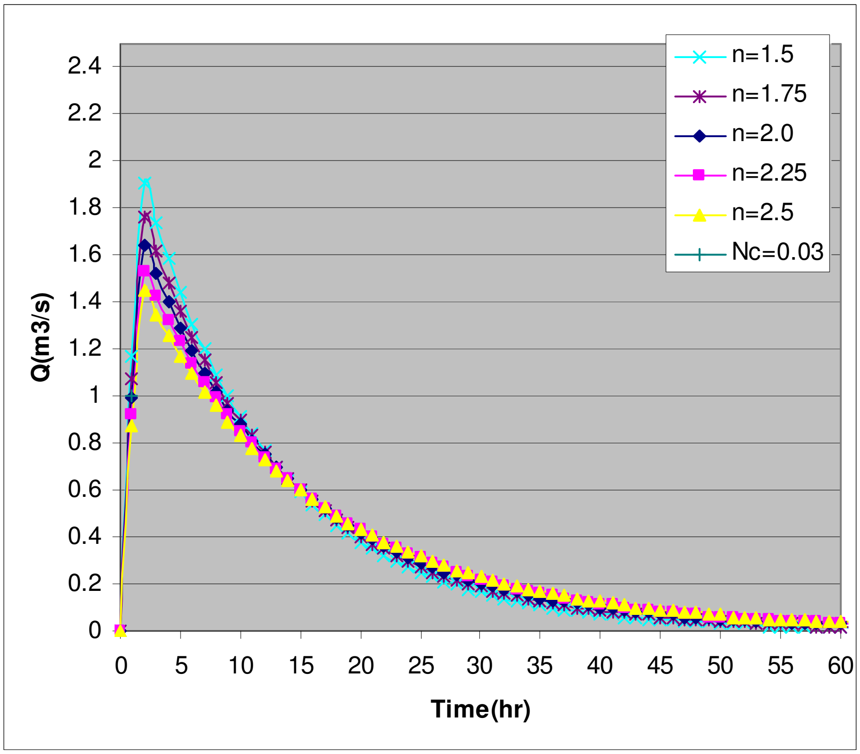

Figure 3-27 1 mm-GIUHs with different overland roughness coefficient values for Badan

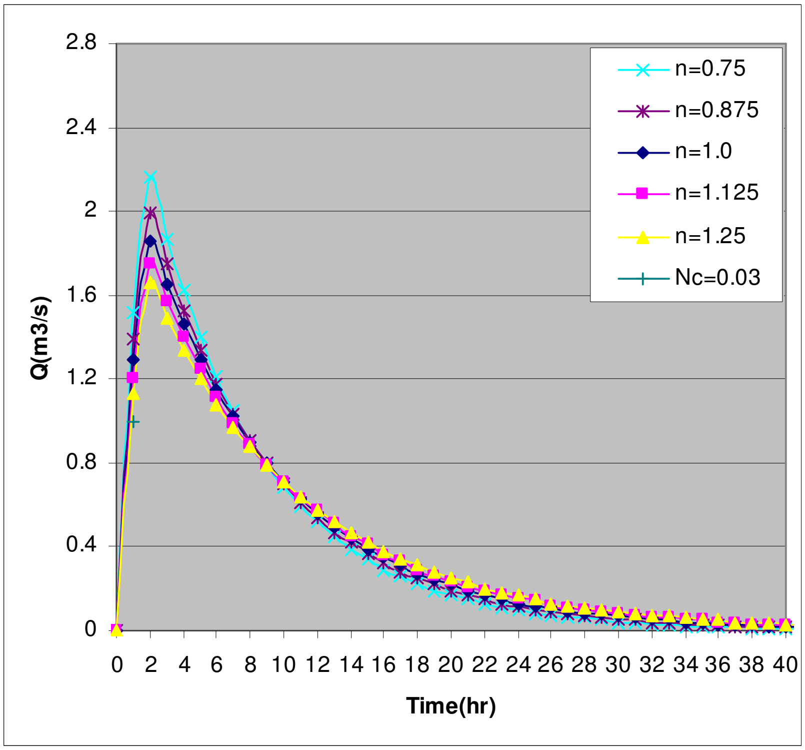

Figure 3-28 1 mm-GIUHs with different overland roughness coefficient values for Faria

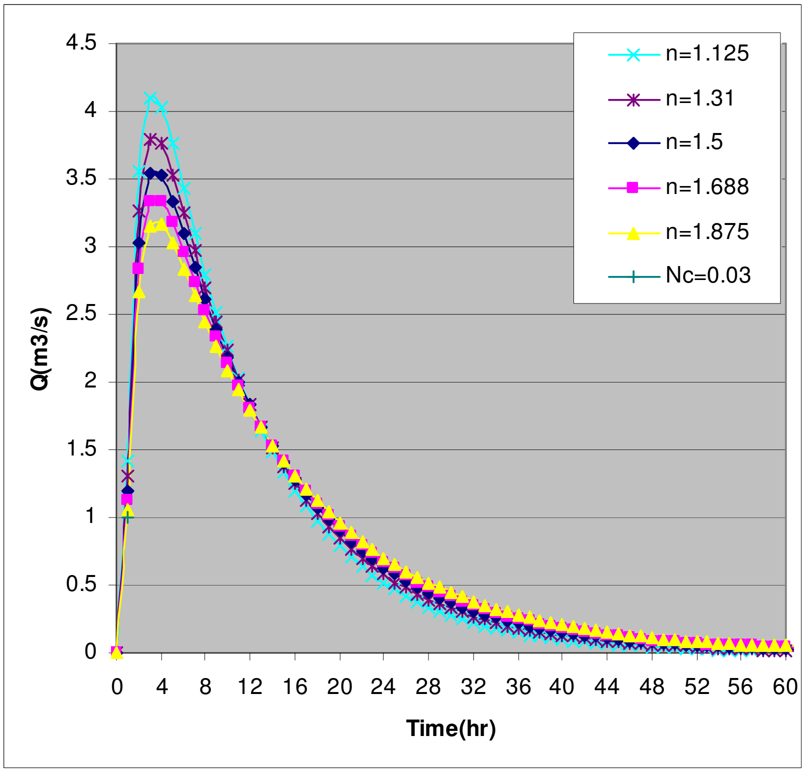

Figure 3-29 **1 mm-GIUHs with different overland roughness coefficient values for Malaqi**

Similar analysis was performed (Table 3-17, Figure 3-30, Figure 3-31 and Figure 3-32) for the channel flow roughness coefficient (n_c) indicating that increasing the channel roughness coefficient value resulted in slightly reducing the IUH peak value for Faria subcatchments as well as delaying the time to IUH peak in Malaqi subcatchment as shown in Table 3-17. In the KW-GIUH the storage and translation effects are incorporated in the channel flow routing that are represented by an exponential distribution and a uniform distribution respectively (Lee an Yen, 1997). Table 3-17 summarizes the peak discharge and time to peak values produced by the unit hydrograph for different values of n_o and n_c. Table 3-17 depicts the peak discharge values when n_c is 0.01, 0.03 and 0.05. The possible % error compared to the original n_c value is also presented in Table 3-17. Results showed that increasing the channel flow roughness coefficient from 0.03 to 0.05 (or 67%) resulted in only a slight decrease of 1.8 % in the peak value from 1.64 m^3/s to 1.61 m^3/s.

In the KW-GIUH model, the travel time is a function of the amount of water in the flow that is represented by the spatially uniform rainfall excess (q_L) to be applied to the IUH. For a given catchment there is a set of IUHs corresponding to different values of q_L that in application is the temporally varying intensity of the rainfall excess where the peak of the IUH increases with increasing rainfall excess intensity while the time to peak decreases (Lee and Yen, 1997). In the KW-GIUH the nonlinear behaviour is considered through the travel time in the channels and on overland surfaces. The travel time for the overland flow region is assumed to follow an exponential distribution. The travel time for the storage component of a channel is assumed to follow an exponential distribution, but the translation component of a channel is also assumed to follow a uniform distribution (Lee and Yen, 2000).

Table 3-17 Peak discharge values and time to peak for different values of n_c at n_o = 2, 1, 1.5 for Badan, Faria, and Malaqi subcatchments' 1mm- IUH respectively (Dark column sells represent actual used values)

Sub-catchment	Value	n_c=0.01	n_c=0.02	n_c=0.03	n_c=0.04	n_c=0.05
Badan	Qpeak (m^3/s) % Error	1.64 0	1.64 0	1.64	1.63 0.6	1.61 1.8
	Time to Peak (hr)	2	2	2	2	2
Faria	Qpeak (m^3/s) % Error	1.87 0.5	1.86 0	1.86 0	1.85 0.5	1.83 1.6
	Time to Peak (hr)	2	2	2	2	2
Malaqi	Qpeak (m^3/s) % Error	3.83 8.2	3.71 4.8	3.54	3.46 2.3	3.38 4.5
	Time to Peak (hr)	2	3	3	4	4

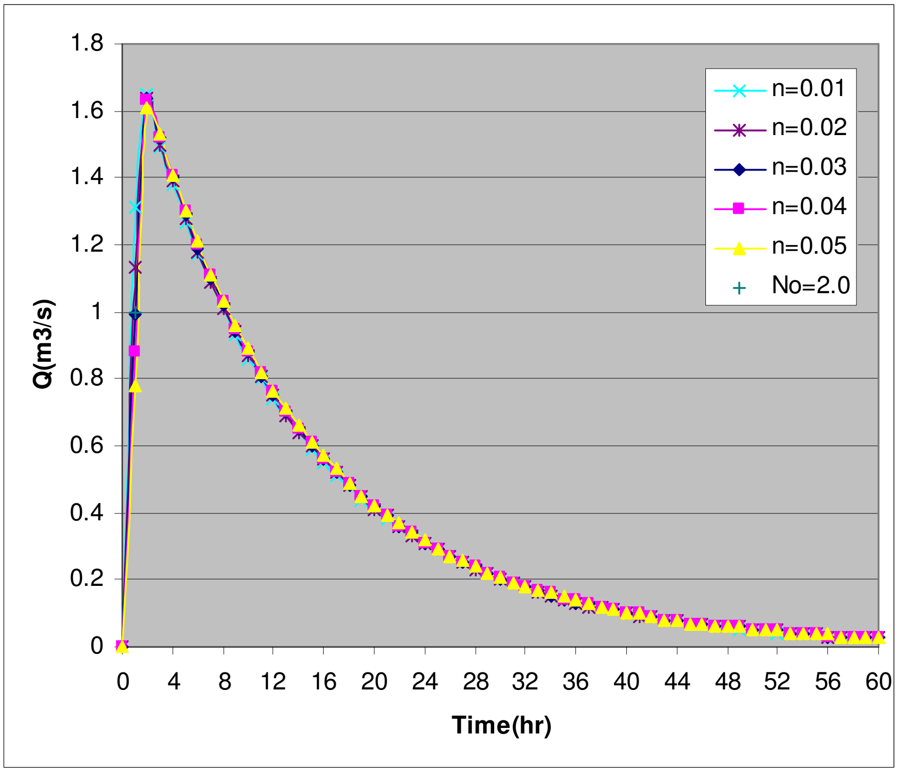

Figure 3-30 1 mm-GIUHs with different channel roughness coefficient values for Badan

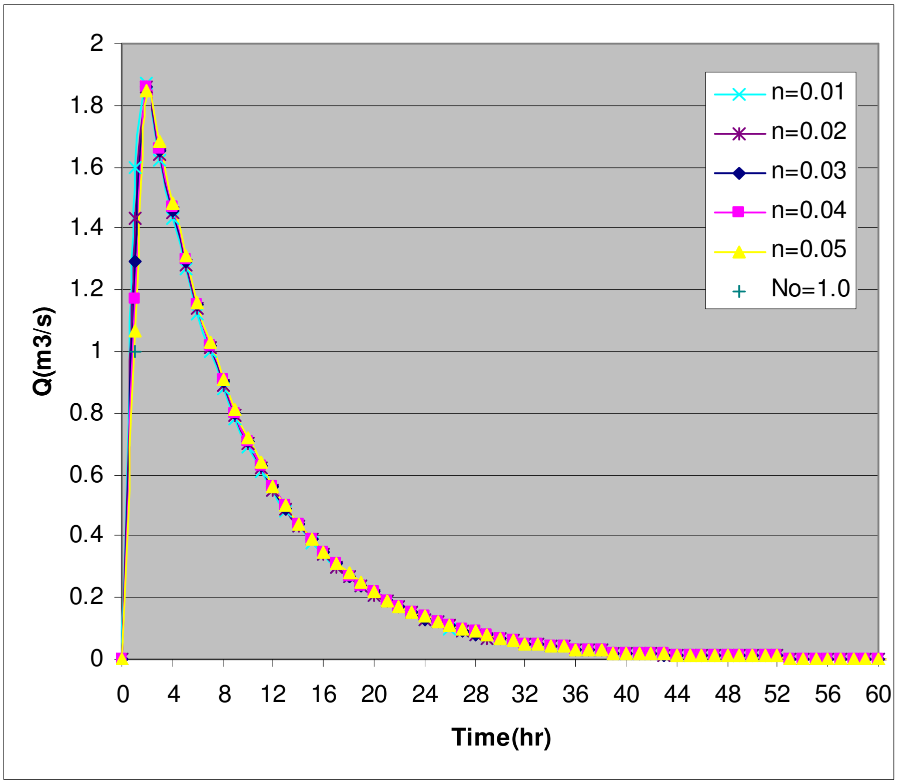

Figure 3-31 1 mm-GIUHs with different channel roughness coefficient values for Faria

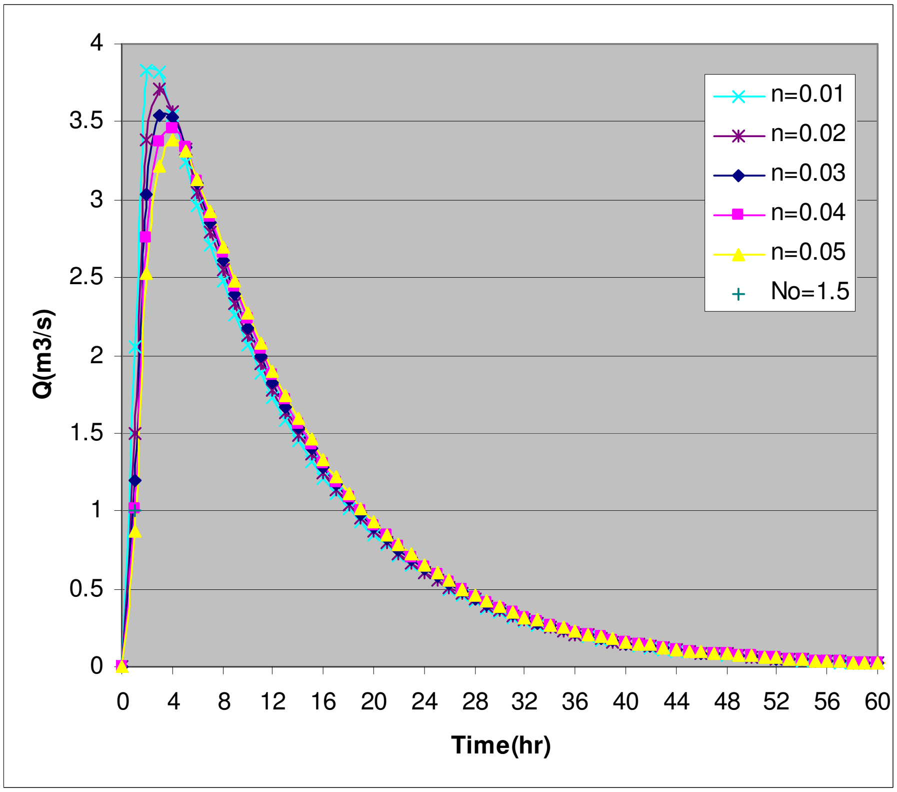

Figure 3-32 1 mm-GIUHs with different channel roughness coefficient values for Malaqi

All geomorphic characteristics were obtained from GIS tools. Although GIS tools estimate these parameters fairly accurately it was decided to conduct a sensitivity analysis on these parameters as well.

Generally, the catchment stream network map includes all intermittent and permanent flow lines. The first sensitivity test was conducted on the stream order of each subcatchment. The stream order was decreased by one order to investigate the effect of the stream order level on the model results. As shown in Table 3-18, Figure 3-33, Figure 3-34 and Figure 3-35 decreasing the Badan subcatchment stream order from fourth level to third level resulted in decreasing the peak discharge from 1.64 m^3/s to 0.87 m^3/s or 47 %. Similar effect was noticed for Faria and Malaqi subcatchments where decreasing the stream order level resulted in decreasing the peak value from 1.86 m^3/s to 1.09 m^3/s (41 %) and from 3.54 m^3/s to 2.04 m^3/s (42%) for Faria and Malaqi subcatchments respectively. Therefore it is important to follow the stream network map in rainfall runoff modelling.

Table 3-18 Peak discharge values for different stream order for Badan, Faria, and Malaqi subcatchments' 1mm- IUH (Dark column sells represent actual used values)

Sub-catchment	Value	Fourth level	Third level
Badan	Qpeak (m^3/s) % Error	1.64	0.87 47
Sub-catchment	Value	Fourth level	Third level
Faria	Qpeak (m^3/s) %Error	1.86	1.09 41
Sub-catchment	Value	Third level	Second level
Malaqi	Qpeak (m^3/s) %Error	3.54	2.04 42

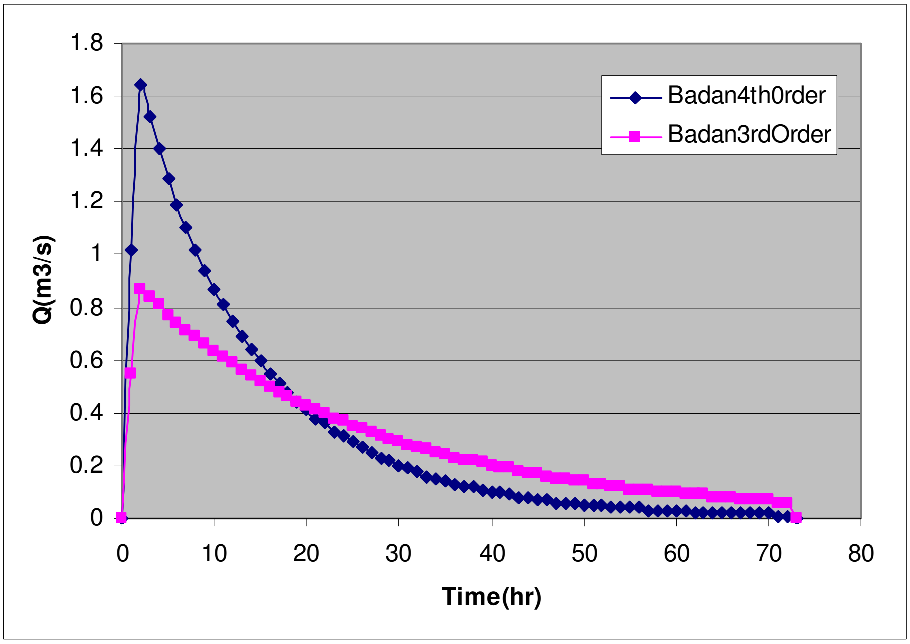

Figure 3-33 1 mm-GIUH produced with 3rd and 4th stream order level for Badan

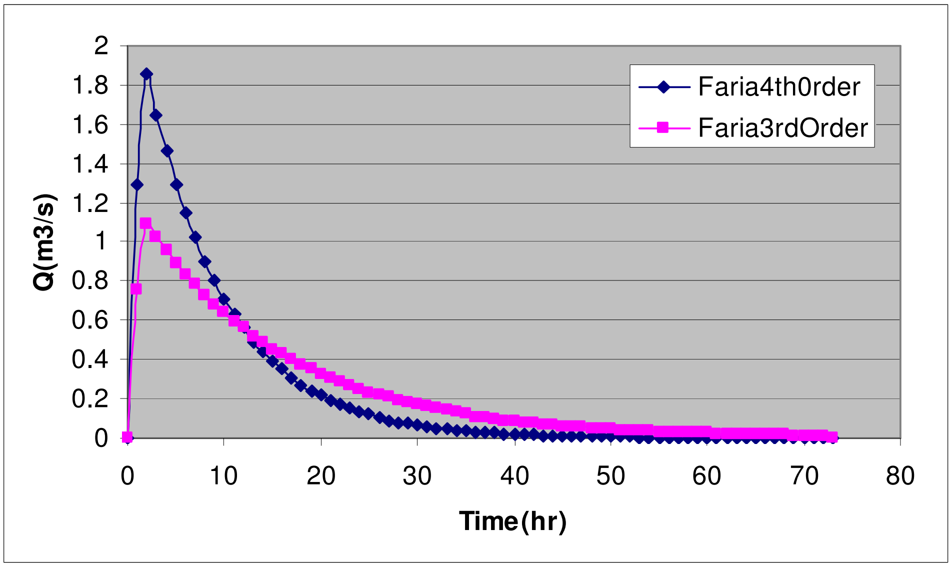

Figure 3-34 1 mm-GIUH produced with 3rd and 4th stream order level for Faria

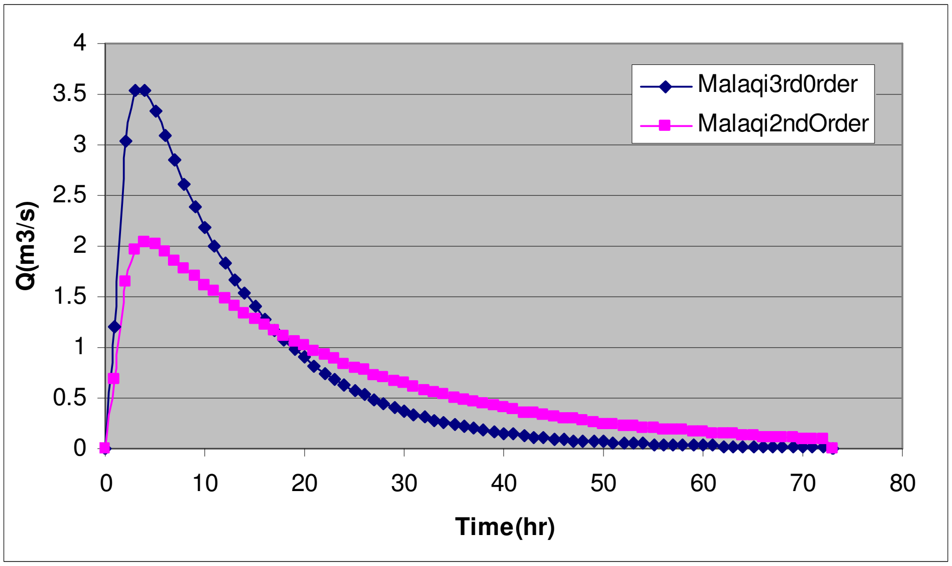

Figure 3-35 1 mm-GIUH produced with 2nd and 3rd stream order level for Malaqi

Each parameter (channel width at catchment outlet B_{Ω}, number of ith-order channels N_i, mean ith-order stream length $\overline{L}_{ci}$, ith-order sub catchment contributing area $\overline{A}_{i}$, mean ith-order overland slope $\overline{S_{oi}}$, mean ith-order channel slope $\overline{S}_{ci}$ and subcatchment area) was changed to investigate the effect on the unit hydrograph.

The channel width at the catchment outlet (B_Ω) is the only geomorphic parameter that cannot be obtained from a topographic map or GIS maps. Therefore a sensitivity analysis for B_Ω is important. As shown in Table 3-19, a change of 25 % in B_Ω resulted only in 0.6 % change of the peak discharge of Faria subcatchment. Similar results were observed for Badan and Malaqi subcatchments.

Tables 3-20 to 3-25 show that changing the value of each of the independent geomorphological parameters by 10 % resulted in a change in the peak discharge value up to 7.9 %. The value of the ratio of ith-order overland area to the catchment area P_{OAi} is dependent on the values of number of ith-order channels N_i, ith-order sub catchment contributing area $\overline{A}_i$ and subcatchment area. Accordingly, a change in the value of N_i or $\overline{A}_i$ will automatically result in a change in the value of P_{OAi} for the three subcatchments. It is also worth mentioning that testing the sensitivity of the model output to changing the catchment area (A) will necessarily mean a consistent and similar ratio change of the different orders sub catchment contributing area $\overline{A}_i$ (in order to maintain the same ratio of ith-order overland area to the catchment area P_{OAi}). However, with the GIS maps the geomorphologic properties of catchments could be measured accurately.

Table 3-19 Peak discharge values and time to peak for for different values of channel width at catchment outlet B_Ω (*m*) for Faria subcatchments 1mm- IUHs (Dark column sells represent actual used values)

Subcatchment	**Value**	**3.45**	**4.14**	**4.6**	**5.06**	**5.75**
Badan	Qpeak (m^3/s)	1.63	1.64	1.64	1.64	1.63
	% Error	0.6	0		0	0.6
	Time to Peak (hr)	2	2	2	2	2
Subcatchment	**Value**	**2.78**	**3.33**	**3.7**	**4.07**	**4.62**
Faria	Qpeak (m^3/s)	1.86	1.86	1.86	1.86	1.86
	% Error	0	0	0	0	0
	Time to Peak (hr)	2	2	2	2	2
Subcatchment	**Value**	**3.75**	**4.5**	**5**	**5.5**	**6.25**
Malaqi	Qpeak (m^3/s)	3.63	3.57	3.54	3.52	3.50
	% Error	2.5	0.8		0.6	1.1
	Time to Peak (hr)	3	3	3	4	4

Table 3-20 Peak discharge values and time to peak for for different values of number of ith-order channels N_i (order 1) for Faria subcatchments 1mm- IUHs (Dark column sells represent actual used values)

Subcatchment	**Value**	**37**	**41**	**45**
Badan	Qpeak (m^3/s)	1.56	1.64	1.71
	% Error	4.9		4.3
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**44**	**49**	**54**
Faria	Qpeak (m^3/s)	1.81	1.86	1.89
	% Error	2.7		1.6
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**56**	**62**	**68**
Malaqi	Qpeak (m^3/s)	3.41	3.54	3.67
	% Error	3.7		3.7
	Time to Peak (hr)	4	3	3

Table 3-21 Peak discharge values and time to peak for for different values of mean ith-order stream length $\overline{L}_{ci\,(m)}$ (order 1) for Faria subcatchments' 1mm- IUHs (Dark column sells represent actual used values)

Subcatchment	**Value**	**1241**	**1379**	**1517**
Badan	Qpeak (m^3/s)	1.57	1.64	1.70
	% Error	4.3		3.7
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**928**	**1031**	**1134**
Faria	Qpeak (m^3/s)	1.79	1.86	1.93
	% Error	3.8		3.8
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**1728**	**1920**	**2112**
Malaqi	Qpeak (m^3/s)	3.42	3.54	3.65
	% Error	3.4		3.0
	Time to Peak (hr)	3	3	3
	%Error			

Table 3-22 Peak discharge values and time to peak for different values of ith-order sub catchment contributing area $\overline{A}_{i}$ (km^2) (order 1) for Faria subcatchments' 1mm- IUHs (Dark column sells represent actual used values)

Subcatchment	**Value**	**1.23**	**1.37**	**1.51**
Badan	Qpeak (m^3/s)	1.65	1.64	1.58
	% Error	0.6		3.7
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**0.84**	**0.937**	**1.03**
Faria	Qpeak (m^3/s)	1.88	1.86	1.82
	% Error	1.1		2.2
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**1.63**	**1.81**	**1.99**
Malaqi	Qpeak (m^3/s)	3.51	3.54	3.55
	% Error	0.8		0.3
	Time to Peak (hr)	3	3	3

Table 3-23 Peak discharge values and and time to peak for different values of mean ith-order overland slope $\overline{S_{o\,i}}$ (*m/m*) (order 1) for Faria subcatchments' 1mm-IUHs (Dark column sells represent actual used values)

Subcatchment	**Value**	**0.153**	**0.17**	**0.187**
Badan	Qpeak (m^3/s)	1.60	1.64	1.67
	% Error	2.4		1.8
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**0.096**	**0.107**	**0.118**
Faria	Qpeak (m^3/s)	1.83	1.86	1.89
	% Error	1.6		1.6
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**0.131**	**0.146**	**0.161**
Malaqi	Qpeak (m^3/s)	3.48	3.54	3.60
	% Error	1.7		1.7
	Time to Peak (hr)	3	3	3

Table 3-24 peak discharge values and and time to peak for different values of mean ith-order channel slope $\overline{S}_{ci\,(m/m)}$ (order 1) for Faria subcatchments' 1mm- IUHs (Dark column sells represent actual used values)

Subcatchment	Value	0.126	0.14	0.154
Badan	Qpeak (m^3/s) % Error	1.64 0	1.64	1.64 0
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**0.105**	**0.117**	**0.129**
Faria	Qpeak (m^3/s) % Error	1.86 0	1.86	1.86 0
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**0.126**	**0.14**	**0.154**
Malaqi	Qpeak (m^3/s) % Error	3.54 0	3.54	3.54 0
	Time to Peak (hr)	3	3	3

Table 3-25 Peak discharge values and time to peak for different values of subcatchment area (km^2) for Faria subcatchments' 1mm- IUHs (Dark column cells represent actual used values)

Subcatchment	Value	76.8	85.3	93.8
Badan	Qpeak (m^3/s) % Error	1.56 4.9	1.64	1.77 7.9
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**57.6**	**64**	**70.4**
Faria	Qpeak (m^3/s) % Error	1.77 4.8	1.86	1.95 4.8
	Time to Peak (hr)	2	2	2
Subcatchment	**Value**	**166.5**	**185**	**203.5**
Malaqi	Qpeak (m^3/s) % Error	3.33 5.9	3.54	3.74 5.6
	Time to Peak (hr)	3	3	3

In applying the KW-GIUH model to a water resources design work for unguaged catchments, the design storm can be determined from the depth-duration-frequency curves obtained from widely available rainfall data. The time-varying rainfall excess (q_L) can be determined from the design rainfall temporal pattern hyetograph, after the abstractions are deducted from the design storm and then the resultant rainfall excess is applied to the Instantaneous Unit Hydrographs (IUHs) with different q_L's to generate the surface runoff hydrograph. In using the model for water resources planning and management, it is important to accurately estimate the excess rainfall. Several methods can be used to estimate the excess rainfall based on the available data on soil properties and infiltration characteristics.

3.5 MONTHLY RUNOFF FOR THE PLANNING MODEL

To plan the optimization of land and water to maximize the benefits there is a need to predict the amount of runoff that would be generated from rainfall, under different management alternatives. Since rainfall data is almost always available, even in ungauged catchments, the new approach using KW-GIUH is based on utilizing rainfall data together with the developed KW-GIUH model to predict the runoff volume that could be harvested from a storm. Furthermore the rainfall runoff model developed for the study area is essentially needed to estimate the peakflow from a storm for any water resources design work as part of proposed management options that would be applied to the study area and involve transport and storage of surface runoff or for recharge of groundwater using surface runoff.

The runoff will be estimated based on the available rainfall records from each rainfall event. On average, only very few rainfall events produce the runoff in arid atmosphere such as the Faria catchment. Runoff computed for rainfall events will be used for the planning model as part of the integrated land and water management framework.

The results of the excess rainfall estimated by the Horton method for each rainfall event in Badan, Faria and Malaqi subcatchments were presented in Section 3.4.2. The runoff was simulated using the KW-GIUH for each event and the total monthly runoff volumes were estimated. These monthly total runoff volumes were compared with the estimated surface runoff (from baseflow separation) as shown in Table 3-26, Table 3-27 and Table 3-28. In the arid Malaqi subcatchment, the low rainfall intensities and volumes did not produce any excess rainfall. Based on the personal communication from the local authorities and farmers there had not been any runoff from Malaqi subcatchment for many years. The results showed that the excess rainfall estimated by the Horton method resulted in total amounts of runoff that are very close to the actual measured runoff. The percent error between the total simulated and estimated surface flow was 3.9% and 10.9% for Badan and Faria subcatchments respectively.

As shown in Table 3-26 and Table 3-27 most of the runoff in the study area is generated during February when considerable rainfall had occurred and the soil moisture is saturated.

The ratio of simulated surface runoff to total rainfall volume ranged from 0 – 6.2% for different months with an average of 3% for the whole Faria catchment, which agrees with previous studies (Husary et al., 1995; Al-Nubani, 2000; Barakat, 2000; Rofe and Rafety, 1965) of runoff-rainfall relationship in the West Bank.

Table 3-26 Rainfall-Runoff simulation for Badan subcatchment

Month	Estimated Surface Runoff (ML)	KW-GIUH Simulated Runoff (ML)	Total Rainfall (ML)	Simulated Runoff/Rainfall (%)
November	10.8	0	11815	0
December	56.2	51.0	3400	1.5
January	175.4	195.4	9775	2.0
February	1373.7	1343.7	24480	5.5
March	213.0	167.0	3145	5.3
Total	1829.1	1756.9		

Table 3-27 Rainfall-Runoff simulation for Faria subcatchment

Month	Estimated Surface Runoff (ML)	KW-GIUH Simulated Runoff (ML)	Total Rainfall (ML)	Simulated Runoff/Rainfall (%)
November	3.6	0	9408	0
December	131.1	125.0	3136	4.0
January	93.4	92.4	8896	1.0
February	388.4	364.7	15360	2.4
March	171.8	120.0	1920	6.2
Total	788.3	702.0		

Table 3-28 Rainfall-Runoff simulation for Malaqi subcatchment

Month	KW-GIUH Simulated Runoff (ML)	Total Rainfall (ML)	Runoff/Rainfall (%)
November	0	12210	0
December	0	3515	0
January	0	10360	0
February	0	26455	0
March	0	3515	0
Total	0		

3.6 SUMMARY AND CONCLUSIONS

The nature of stream flow in a catchment is strongly related to the rainfall characteristics and catchment geomorphology. A GIS-based KW-GIUH approach has been developed and applied successfully for the estimation of flow hydrographs for the semiarid catchment of Faria, West Bank. In this method the excess rainfall is assumed to follow different paths on overland areas and in channels of different stream orders to reach the catchment outlet. All model parameters except overland flow roughness coefficient (n_o), channel flow roughness coefficient (n_c) and channel width at the subcatchment outlet (B_Ω) were obtained from GIS. The channel width at the subcatchment outlet (B_Ω) has been obtained from field investigation. Two parameters, overland flow roughness coefficient (n_o) and channel flow roughness coefficient (n_c) were obtained from published literature based on catchment and channel conditions.

The most important feature of the KW-GIUH model is a set of Instantaneous Unit Hydrographs (IUHs) for different amounts of rainfall excess for a unit of time (q_L). Application of the KW-GIUH model to generate IUHs for different q_L was investigated. The peak of the IUH for the three subcatchments increases with increasing rainfall excess intensity while the time to peak decreases. It is important to note that the increase in peak is not linearly proportional to the increase in rainfall excess intensity. In application of the KW-GIUH model to any rainwater storm, it is necessary to calculate IUH for different amounts of q_L prior to linear superposition is applied to combine the component hydrographs of hourly rainfall excess to produce the complete surface runoff hydrograph. In this study, the dimensionless relationships for the IUH peak and time to peak with q_L as generated by the KW-GIUH model were developed for Faria catchment. The nonlinearity of the IUH is shown as the exponents in the developed Equations.

Rainfall intensity was measured from three locations within the study area during the rainy season of 2004/2005. Runoff data were measured for two subcatchments namely Badan and Faria for the same rainy season. The spatial average rainfall over each sucatchment in the study area was estimated using the Thiessen polygon-GIS tools. Surface runoff was calculated from the observed flow data using the WHAT model. Results from the WHAT model were compared with the surface runoff obtained from the application of the AWBM model.

In order to generate the surface runoff hydrograph, hourly values of rainfall excess from the hyetograph should be applied to the developed IUH. Excess rainfall was estimated using the Horton method and the resulting excess rainfall was used for model application. During the rainy season of 2004/2005, a major storm recorded during February was used to verify the model. Hourly rainfall and runoff data were used to verify the developed KW-GIUH model by comparing the simulated and measured runoff during this event. The simulated and recorded hydrographs are in good agreement. The % error between observed and estimated runoff volume was less than 5%. The value of the coefficient of determination for both subcatchments was 0.81, and the coefficient of efficiency was 0.80. The % error of peak discharge was 4.6 and 6.2 for Badan and Faria subcatchments respectivelty. The error of time to peak discharge was limited to one hour.

Sensitivity analysis was conducted for all input parameters. The values of all parameters were changed to investigate the effect on the simulated hydrograph. The peak flow values increase as the overland flow roughness coefficient (n_o) decrease which reflects the land surface condition of the surface hydrologic system. However compared with the overland flow roughness coefficient (n_o), the channel flow roughness coefficient (n_c) had a smaller effect on both simulated peak flow and time to peak. A change in n_o value from 1.0 to 0.75 for Faria subcatchment, 2.0 to 1.5 for Badan subcatchment and 1.5 to 1.125 for Malaqi subcatchment can cause an error of 16 % to 17 % in the estimation of the peak flow. Results showed that increasing the channel flow roughness coefficient from 0.03 to 0.05 (or 67%) resulted in only a slight decrease of 1.8 % in the peak value from 1.64 m^3/s to 1.61 m^3/s. Results of the sensitivity test conducted on the stream order level of each subcatchment indicated that it is necessary to select all the stream orders given in the stream network map in developing the KW-GIUH model. The channel width at the catchment outlet (B_Ω) is the only geomorphic parameter that cannot be obtained from a topographic map or GIS maps. The results of the sensitivity analysis for B_Ω showed that the peak flow of IUH is not sensitive to B_Ω. For the test range of channel width 25 % from the width of the streams of Faria subcatchments, a change of 25 % in B_Ω resulted only in 0.6 % change of the peak discharge of Faria subcatchment. Changing the value of each of the independent geomorphological parameters by 10 % (number of ith-order channels N_i, mean ith-order stream length $\overline{L}_{ci}$, ith-order sub catchment contributing area $\overline{A}_{i}$, mean ith-order overland slope $\overline{S_{oi}}$, mean ith-order channel slope $\overline{S}_{ci}$ and subcatchment area) resulted in a change in the peak discharge value up to 8 %. However, with the GIS maps the geomorphologic properties of catchments could be measured accurately therefore minimizing the error in estimating runoff.

Excess rainfall was estimated by the Horton method. The surface runoff obtained from the Horton is 1757 ML and 702 ML for Badan and Faria subcatchments respectively. The percent error between the surface runoff obtained from the Horton with the surface runoff volume obtained from the baseflow separation using the WHAT model is 3.9 % and 10.9 % for Badan and Faria subcatchments respectively. Sensitivity of excess rainfall on Horton model parameters was investigated with ±5% change in Horton infiltration parameters. The results indicated that the excess rainfall was sensitive to all the parameters. In arid and semi arid catchments it is important to estimate the excess rainfall accurately for planning purposes.

Results of the rainfall runoff modelling developed in this chapter are needed for developing the planning model (AGSM) that requires the amounts of available runoff and consequently developing the integrated land and water management framework that involves different management alternatives. The amounts of runoff generated under rainfall are needed to proceed with optimal management of land and water resources for the catchment under study. Furthermore, it is important to estimate the peakflow from a storm for infrastructure development for transport and storage of surface runoff or for recharge of groundwater. In this study the basic approach involved the application of a simple rainfall-runoff model to determine the streamflow yield as well as the peakflow based on climatic data and catchment geomorphological characteristics.

CHAPTER 4

GROUNDWATER MODEL AND STATISTICAL ANALYSIS OF SPRING YIELD DATA

4.1 INTRODUCTION

As part of the integrated land and water management framework discussed in Chapter 2 there is a need to estimate the amounts of groundwater that could be safely extracted under different management alternatives and due to impact of climate variability. In this chapter, the approach to the estimation of the groundwater and the calculation of yield from springs to determine the sustainable-yield limits of groundwater resources within the catchment is presented.

The MODFLOW (Harbaugh and McDonald, 1996) software package is selected to estimate the amounts of groundwater that could be safely extracted under different management alternatives. Although the MODFLOW software package could account for the long term climate change conditions, in this case study it is not considered. However Chapter 4 will present the application of one scenario of long-term climate change conditions on the groundwater model. The two input parameters that directly influence groundwater flow model are discussed. These are groundwater recharge and pumping rates. Groundwater recharge reflects the climate conditions while groundwater pumping reflects the pumping rates to satisfy the safe yield conditions. These factors are important when evaluating the management alternatives.

Statistical analysis of historical spring discharge information is utilized to determine the average yield from springs in the study area. A statistical analysis of long-term data from springs in the catchment is essential to better understand the behavioral trends in the yield and assess the reliability of these springs. This is important to understand the uncertainty associated with springs' yield and the development of optimal management option alternatives to maximize economic revenue.

4.2 ROLE OF GROUNDWATER MODEL IN THE INTEGRATED LAND AND WATER MANAGEMENT FRAMEWORK

The flow chart shown in Figure 4-1 describes the overall conceptual functionality of the groundwater module of the management framework. The focal issue in this flowchart is the integration of the two possible driving scenarios that dictate the overall framework functionality; that is the climate change that entail possible change in precipitation intensity as well as the management alternatives that are independent from the climate change. The management alternatives will be developed to sustain the available water resources both in quantity and quality, optimize the use of low quality water including treated effluent and brackish water, and maximize both the irrigated areas and the income of the local farmers. As can be inferred from the flowchart which constitutes a major part of the integrated land and water management framework discussed in Chapter 2, the groundwater model MODFLOW (Harbaugh and McDonald, 1996) reads the recharge as computed from the precipitation distribution which is influenced by the climate variability. MODFLOW also processes the possible alteration/development of a new pumping management strategy that copes and addresses the needs necessitated by the planning model. MODFLOW reads these data as well as the fixed data that is independent from the management scenario and process it through the process-based mathematical modules that solve the general groundwater flow Equation 4.1 (Harbaugh and McDonald, 1996) to produce the general water table elevation distribution as well as the water mass balance for a specified zone. The general groundwater flow equation involves the mass conservation concept and Darcy's Law.

$$\frac{\partial}{\partial x}\left(K_x\xi\frac{\partial h}{\partial x}\right)+\frac{\partial}{\partial y}\left(K_y\xi\frac{\partial h}{\partial y}\right)+\frac{\partial}{\partial z}\left(K_z\xi\frac{\partial h}{\partial z}\right)+W=S_s\xi\frac{\partial h}{\partial t} \qquad (4\text{-}1)$$

where,

- K_x, K_y and K_z are the values of hydraulic conductivity along the x, y, and z coordinate axes (L/T),
- h is the potentiometric head (L),
- ξ is the thickness of the aquifer (L),
- W is the source or sink strength in the units of volume/time/area such as wells and recharge (L/T),
- S_s is the storage coefficient of the porous material (L^{-1}), and
- t is time (T)

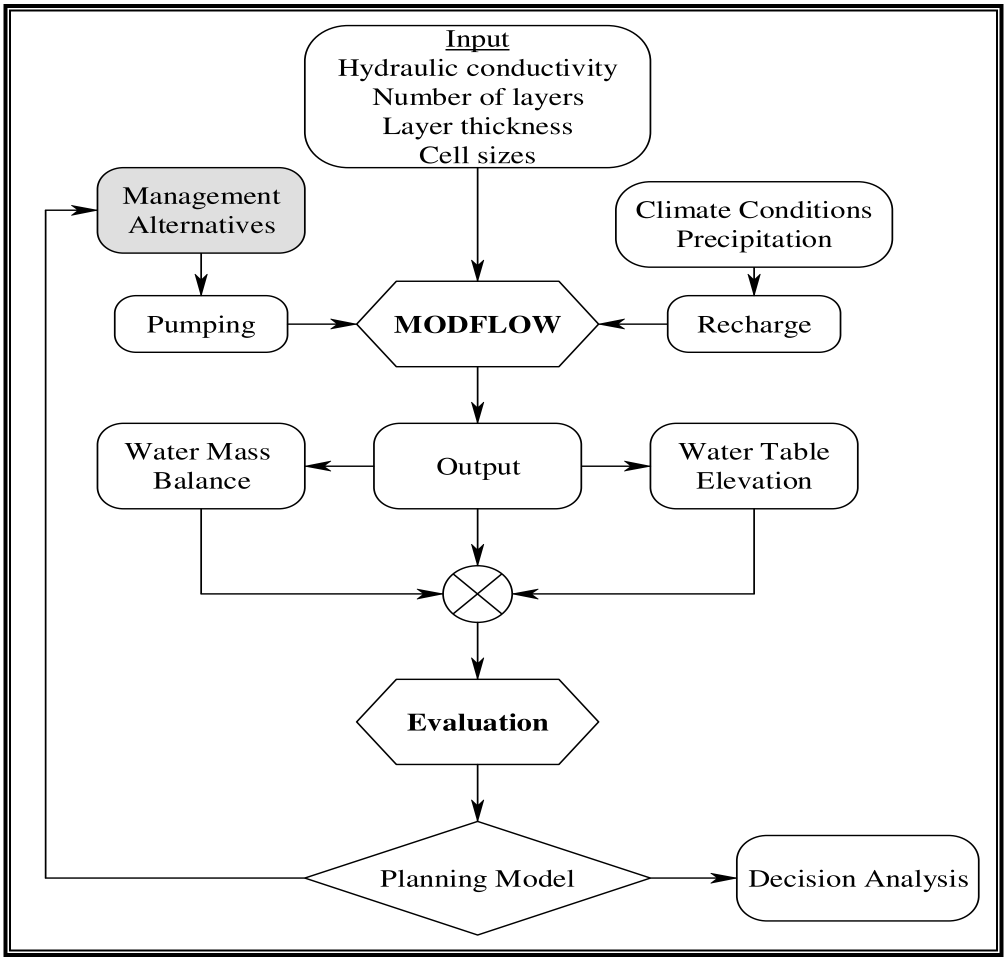

Figure 4-1 **A flow chart depicting the general conception for the utilization of the groundwater model in the general management framework**

Once the model outputs (water table elevations and pumpage distributions) are obtained an evaluation is made to compare the results with sustainable target drawdown values. An assessment will be made to formulate the management alternatives and proceed with the proposed sequence towards the decision analysis module.

4.3 GROUNDWATER MODEL

The MODFLOW (Harbaugh and McDonald, 1996) software package is used to estimate the annual amounts of groundwater that could be safely extracted under the selected management alternatives and climate conditions.

As detailed in Harbaugh and McDonald (1996), the MODFLOW model is a program that can be readily modified, is simple to use and maintain, can be executed on a variety of computers with minimal changes, and has the ability to manage the large data sets required when running large problems. The modular structure of MODFLOW consists of a Main Program and a series of highly-independent subroutines called modules. The modules are grouped in packages. Each package deals with a specific feature of the hydrologic system which is to be simulated such as flow from rivers or flow into drains. The division of MODFLOW into modules permits the user to examine specific hydrologic features of the model independently. This also facilitates development of additional capabilities because new modules or packages can be added to the program without modifying the existing ones. The input/output system of MODFLOW was designed for optimal flexibility. Ground-water flow within the aquifer is simulated in MODFLOW using a block-centered finite-difference approach. Layers can be simulated as confined, unconfined, or a combination of both. Flows from external stresses such as flow to wells, areal recharge, evapotranspiration, flow to drains, and flow through riverbeds can also be simulated. MODFLOW is most appropriate in those situations where a relatively precise understanding of the flow system is needed to make a decision. MODFLOW was developed using the finite-difference method. The finite-difference method permits a physical explanation of the concepts used in construction of the model. Therefore, MODFLOW is easily learned and modified to represent more complex features of the flow system.

4.3.1 Model Input Data

To use MODFLOW, the region to be simulated must be divided into cells with a rectilinear grid resulting in layers, rows and columns. Input data files contain hydraulic parameters (hydraulic conductivity, specific storage), boundary conditions (location of impermeable boundaries and constant heads) and stresses (locations of pumping wells, recharge rates, evapotranspiration). Above parameters are obtained from hydrological and hydrogeological investigations including observation wells and pumping tests.

Ghanem (1999) utilized MODFLOW for the development of a mathematical groundwater flow model for the study area. Details of the developed model for Faria catchment are provided by Ghanem (1999). The finite-difference grid of the model domain consists of 100 rows by 100 columns. The cell width along the column was 350m in both x and y directions. Hydraulic conductivities and layer thickness values for the three layers are provided for MODFLOW in a grid format. As described earlier in Chapter 1, the study area includes one unconfined upper and two confined lower aquifer systems. The model consists of three layers. The upper first layer is unconfined while the second and third are under confined conditions. Layer 1 represents the upper aquifer which consists of the sub-aquifers of Pleistocene, Neogene and Eocene. Layer 2 represents the middle aquitard which consists of Senonian sub-aquifer. Layer 3 represents the lower aquifer which consists of Cenomanian sub-aquifer. The first layer was defined by different zones of hydraulic conductivity (K) values as follows (Ghanem, 1999):

- Eocene western sub-aquifer of 11 m/day,
- Neogene sub-aquifer of 11 m/day,
- Pleistocene sub-aquifer of 0.06 m/day and
- Eocene eastern sub-aquifer of 0.06 m/day.

The hydraulic conductivity values for the second layer were 0.003 m/day and 0.03 m/day in the western and eastern parts respectively. For layer 3 the K values were found to be 1.5 m/day and 5 m/day in the western and eastern parts respectively. The storage coefficient was 0.002. The saturated thickness ranges from 3 to 82.9 m for the upper aquifer and from 131 to 440 m for the confined aquifers (Ghanem, 1999).

As mentioned earlier the groundwater recharge is a function of the rainfall intensity and rainfall distribution across the study area. Climate variability will influence the precipitation rate that could be reflected in long term trends observed in annual rainfall. The long-term average rainfall is used to estimate the recharge. The groundwater pumping rate changes with the selected management alternatives that will be discussed in Chapter 6.

4.3.2 Groundwater Pumping Rates

Water resources data were obtained from Palestinian Water Authority (PWA), and the Water and Environmental Studies Institute (WESI)/An-Najah National University. These include abstraction from groundwater wells and springs. Monthly and annual measurements for most of the existing resources were obtained from PWA over a relatively long period of time. As mentioned in Chapter 1, the average annual abstraction of Palestinian wells is 8300 ML of which 75% are utilized for agricultural purposes (WESI, 2005). This amount does not change much over the years and cannot be exceeded because of the restrictions imposed by the Israeli occupation authorities that still control the groundwater abstraction. Part of the groundwater that is abstracted from the study area is brackish water especially in the lower parts of the study area. Results of water salinity obtained for groundwater wells (WESI, 2005; PWA, 2005) indicated that a total of 5330 m^3/day (1950 ML per year) of brackish water were abstracted based on comparing the measured salinity with a threshold salinity of 2.2 mS/cm (as recommended by Ayers and Westcot, 1985). As shown in Table 4-1 the total groundwater abstractions is 22661 m^3/day (17330 m^3/day fresh water + 5330 m^3/day brackish water) which is equivalent to 8300 ML per year. Out of this amount 2075 ML per year are used for domestic purposes (WESI, 2005) resulting in a total amount of 6225 ML per year used for irrigation purposes.

Table 4-1 Total pumping rates from groundwater (fresh and brackish water) for the study area (WESI, 2005; PWA, 2005)

Zone	Pumping (m3/day)		
	Fresh	Brackish	Total
Zone1	11790.89	0.00	11790.89
Zone2	2048.97	0.00	2048.97
Zone3	3490.90	5330.89	8821.79
Total	17330.76	5330.89	22661.65

4.3.3 Groundwater Recharge

The following recharge equations (Equations 4-2 to 4-4) were utilized in estimation of recharge for the study area. These equations were developed for similar aquifers in the West Bank (SUSMAQ, 2004) and can best represent the local conditions for the study area. The same study recommends a 50% maximum allowable drawdown in order to sustain the groundwater aquifer.

$$R=0.6\,(P-285) \qquad P > 700\text{ mm} \tag{4-2}$$

$$R=0.46\,(P-159) \qquad 700\text{ mm} > P > 456\text{ mm} \tag{4-3}$$

$$R=0.3\,(P) \qquad P < 456\text{ mm} \tag{4-4}$$

where,

R = Recharge from rainfall in mm/yr

P = Annual rainfall in mm/yr.

Long-term average value of annual rainfall will be used to calculate the recharge as discussed in Section 4.4.1.

The recharge equations used in this study were developed for similar aquifers in the West Bank (SUSMAQ, 2004) and can best represent the local conditions for the study area. The same study recommends a 50% maximum allowable drawdown in order to sustain the groundwater aquifer. Many studies have defined the sustainable safe yield using the allowable drawdown limit considered to be half of the saturated aquifer thickness (AMEC Earth and Environmental, 2005; SUSMAQ, 2004; Almasri, 2003; Turney, 1997; South Florida Water Management District, 2000; USGS, 2005).

Alternatively, safe yield was traditionally defined as the amount of groundwater discharge which equates the annual groundwater recharge (Todd, 1959). Several groundwater studies have attempted to limit groundwater pumping based on the concept of safe yield, defined as the maintenance of a balance between the annual amount of groundwater pumping and the annual amount of recharge (Theis, 1940; Kazmann, 1956; SCS, 1967). This traditional definition of safe yield is too narrow and ignores the rights of groundwater-fed surface water, such as springs and baseflow, and groundwater-dependent ecosystems, such as wetlands and riparian vegetation. Consequently, if pumping equals recharge, streams and springs may eventually dry up. Additionally, continued pumping in excess of recharge may eventually deplete the aquifer and may have serious social and economic consequences (Sophocleous, 1997; Sophocleous, 2000). Alley and Leake (2004) recognized the dependence of yield on the amount of capture. Unlike natural recharge, which tends to be a constant for a given basin, capture is a function of the level of development; the greater the pumping, the greater the capture. Thus, capture could not be sustainable in all cases. Seward et al. (2006) found serious problems with the simplistic assumption that sustainable yield should equal recharge. In many cases, sustainable yield will be considerably less than average annual recharge; therefore, the general statement that sustainable or "safe" yield equals recharge is incorrect. Natural recharge does not determine sustainable yield; rather, the latter is determined by the amount of capture that it is permissible to abstract without causing undesirable or unacceptable consequences. Sustainable yield extends beyond the conventional boundaries of hydrogeology, to encompass surface water hydrology, ecology, and other related topics (Ponce, 2007). There is currently a lack of consensus as to what percentage of safe yield should constitute sustainable yield. The issue is complicated by the fact that knowledge of several related earth sciences is required for a correct assessment of sustainable yield. Additionally, there are social, economic, and legal implications which have a definite bearing on the analysis. Limited experience indicates that values of sustainable yield expressed as a percentage of recharge may range from 10% to more than 70%, and reasonable compromises may be established on a case-by-case basis (Ponce, 2007).

4.4 GROUNDWATER MODELLING PROCESSES

In order to use the MODFLOW model to calculate the pumping rates and evaluate different management alternatives, the following processes need to be implemented.

- Recharge preparation process
- Well preparation process
- Pre-pumping head determination process
- Post-pumping head determination process

Subroutines were developed to automate particular processes as detailed in the following sub-sections.

4.4.1 Recharge Preparation Process

This subroutine processes the rainfall distribution in the catchment and prepares the corresponding distribution of the groundwater recharge. In order to obtain the spatial distribution of the groundwater recharge, Thiessen polygons (Chow et al., 1988) were delineated for the catchment using ArcView GIS. The six rainfall stations present in the catchment were utilized in the demarcation process and the transpired polygons were codified into codes (Figure 4-2). Section 3.2.2 (Chapter 3) gives the data for each of the rainfall stations. Thereafter, using ArcView GIS a finite-difference grid was set up in concordant with the grid utilized in the groundwater flow model MODFLOW, such that the two grids are the same. Each cell in the recharge grid does carry the code of the polygon that falls within. Each code was spatially linked to the rainfall station and consequently to the long-term yearly average rainfall intensity. Figure 4-3 depicts the overall conceptual description of the groundwater recharge subroutine. The three input files are, **mask.asc** delineates the catchment boundary, **input_ recharge_data.asc** contains rainfall data and recharge equation parameters and **input_rain_ zones.asc** delineates the Thiessen polygons spatially. Once this was accomplished, the empirical recharge Equations 4-2 to 4-4 were utilized on cell-by-cell basis to compute the recharge given in Figure 4-4. As depicted in Figure 4-3, the main output from the recharge preparation subroutine is in **rech_faria.asc** file which is prepared in a grid format that is readable by MODFLOW. The file **recharge_gis.as**c contains recharge in a format that is readable by ArcView GIS for visual presentation of the recharge output (Figure 4-4).

4.4.2 Well Preparation Process

The main task of this subroutine is to prepare the data pertaining to groundwater pumping wells in a format that is readable by MODFLOW. The spatial location of each well is given in the coordinate system of *x* and *y*. The subroutine translates that into the grid based referencing system in terms of row and column of each cell that contains a well. Additionally, for each well the subroutine reads the salinity measure, the percentage of maximum drawdown allowed and the zone in which each well fall.

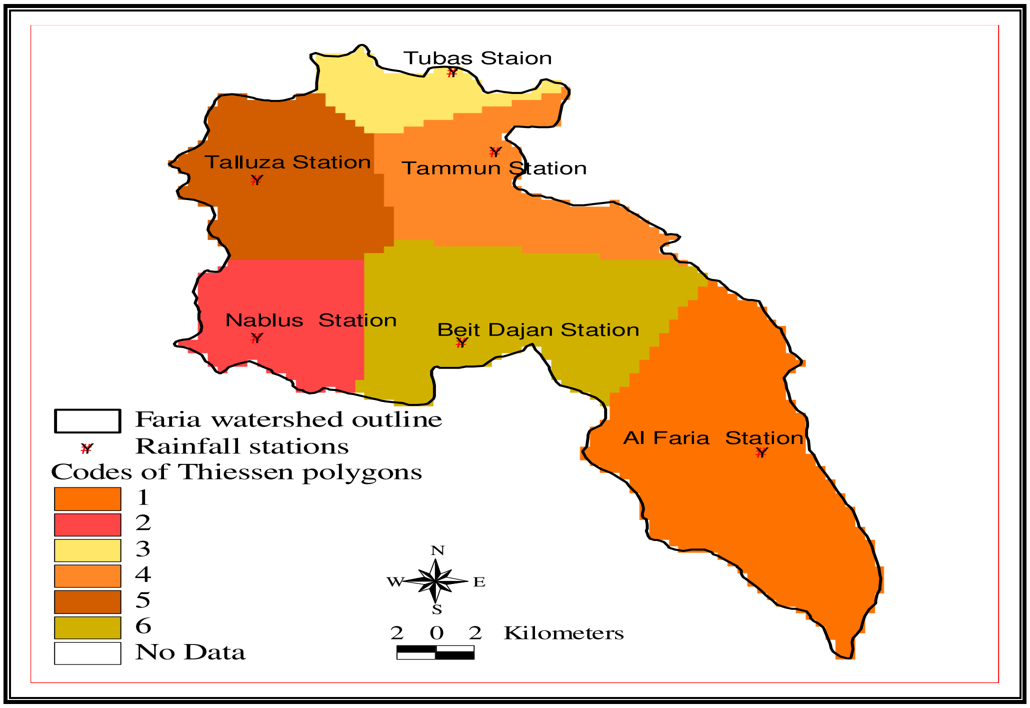

Figure 4-2 **The spatial distribution of rainfall stations and the corresponding Thiessen polygons as prepared using ArcView GIS capabilities**

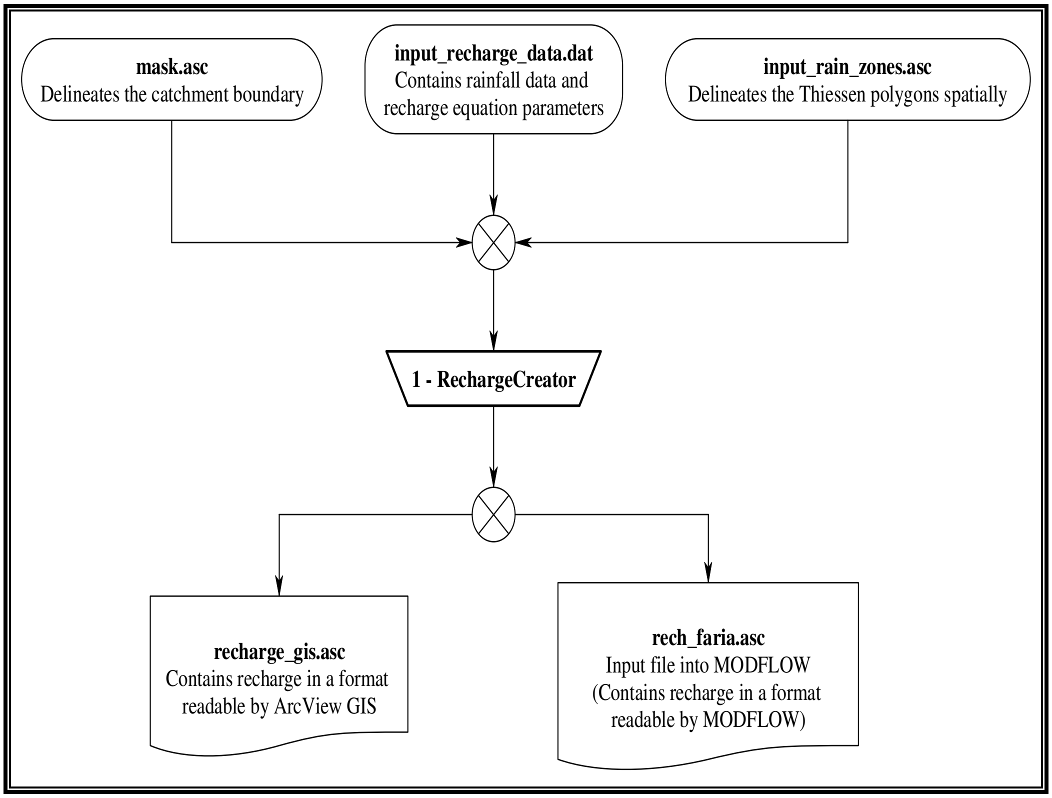

Figure 4-3 Conceptual representation of the functionality of the groundwater recharge preparation process

The subroutine named 2 – wellCreator reads for each well the *x* and *y* coordinates, the pumping rate, and the layer from which actual pumping is taking place. The subroutine follows a search technique to assign the row and column of each cell that does contain a well for the same finite-difference grid followed in the MODFLOW model. The output file is **faria.wel**. Figure 4-5 elucidates the conceptual representation of the functionality of the well preparation subroutine.

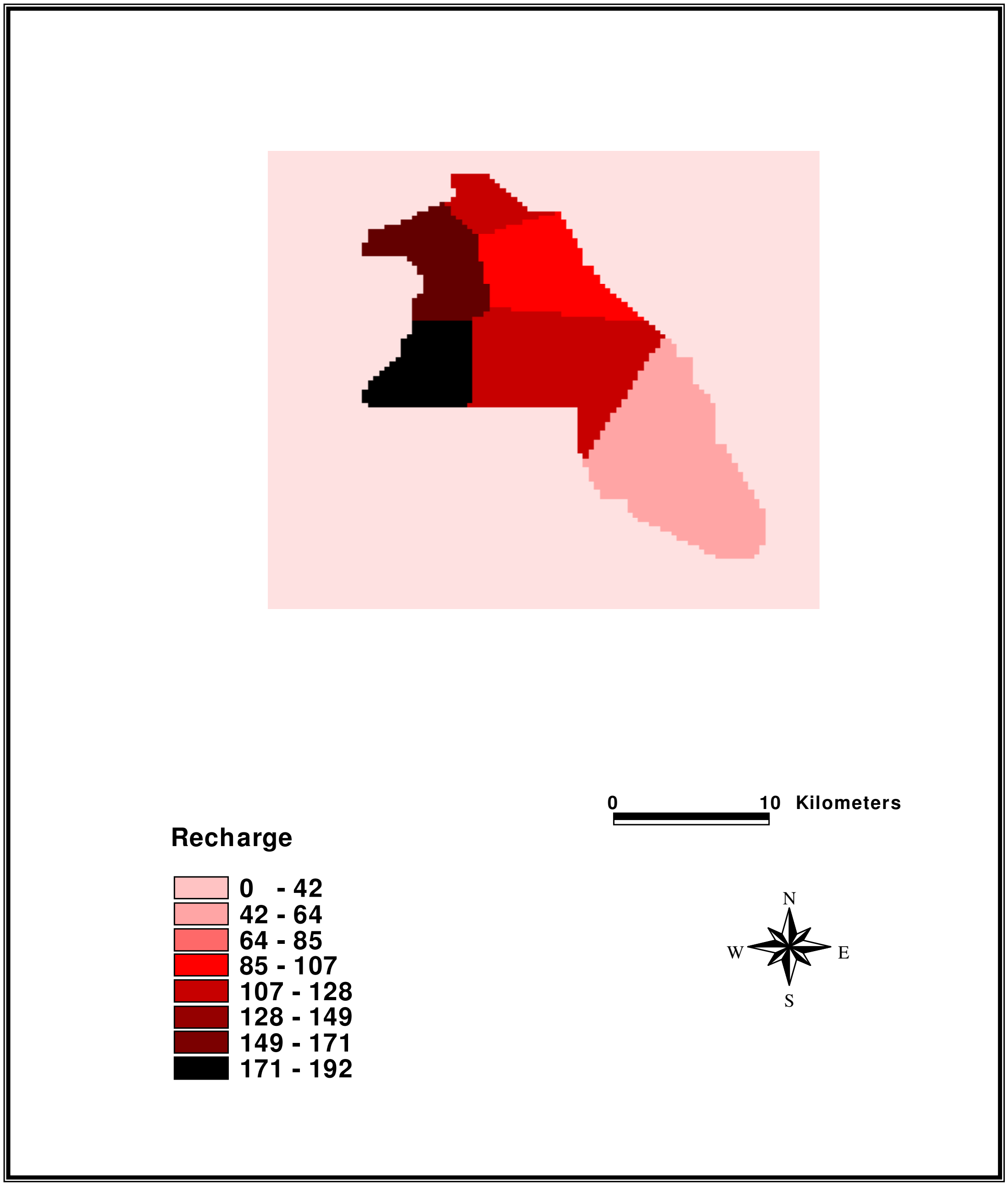

Figure 4-4 The spatial distribution of groundwater recharge (mm/yr) as prepared using ArcView GIS capabilities

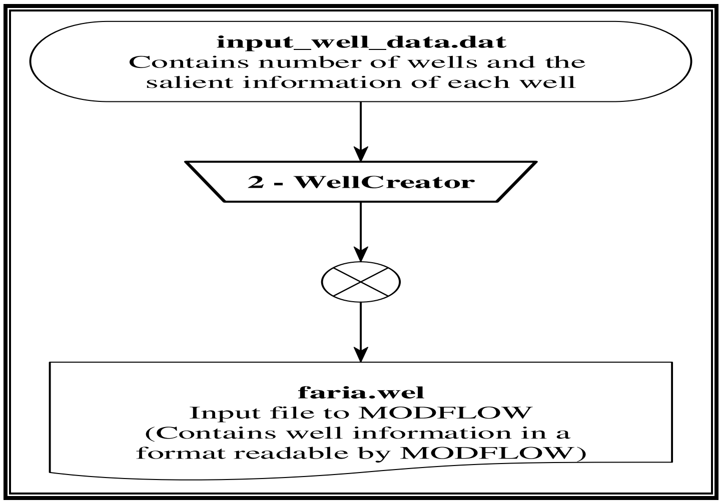

Figure 4-5 Conceptual representation of the functionality of the well preparation process

4.4.3 Pre-Pumping Head Determination Process

The main task of this subroutine is to obtain the water head corresponding to the pre-pumping conditions at the well locations. The main premise adopted in this subroutine is to read the grid that contains the spatial distribution of the head for the entire study area corresponding to the pre-pumping conditions that is generated by MODFLOW and give the pre-pumping head at each well location. The conceptual representation of the functionality of the pre-head preparation subroutine is depicted in Figure 4-6.

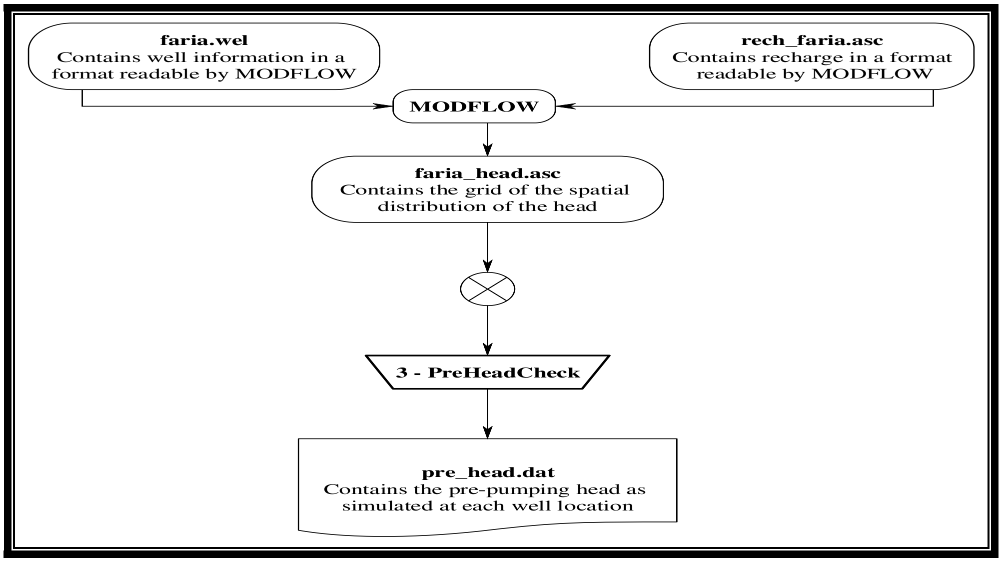

Figure 4-6 Conceptual representation of the functionality of the pre-pumping head determination process

The pre-pumping conditions are identified by the file named **faria.wel** where the pumping values are set to zero to signify the no-pumping conditions. The output file name signifying the study area pre-pumping head is **faria_head.asc**. Thereafter, the subroutine scan the grid of the head distribution (**faria_head.asc**), match the cell location of each well in the file **faria.wel** (row and column), and extract the corresponding head value. The output after the execution of the subroutine is the pre-pumping head reported at each well location as saved in the output file named **pre_head.dat**. Figure 4-7 elucidates the results of the pre-pumping head distribution for the study area as generated by MODFLOW and presented using ArcView GIS.

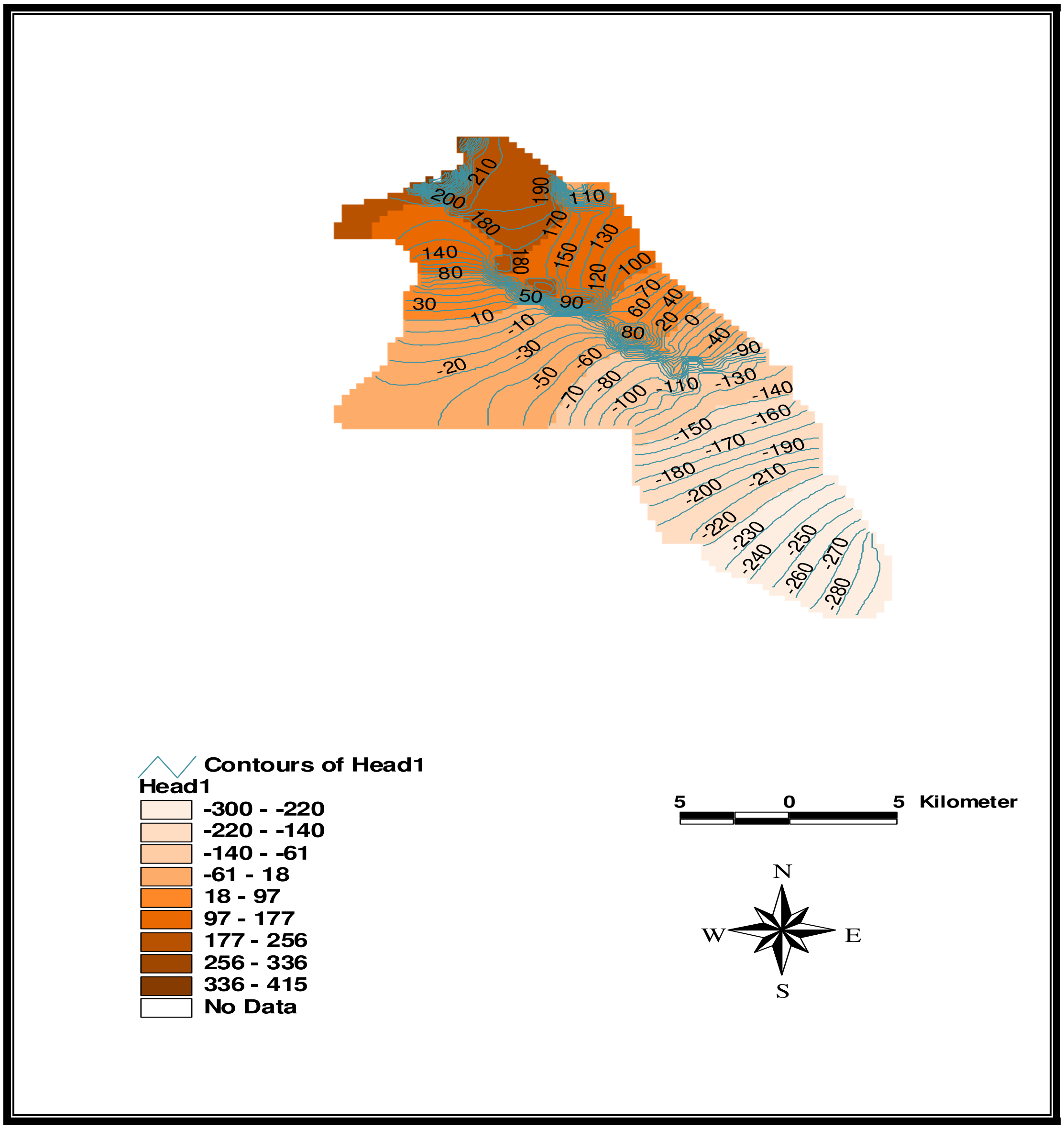

Figure 4-7 **The pre-pumping head distribution (m) for the study area as generated by MODFLOW and prepared using ArcView GIS capabilities**

4.4.4 Post-Pumping Head Determination Process

The main task of this subroutine named 4 – PostHeadCheck is to determine the head values at the well locations after considering the pumping scenario. The head determination is based on the output from MODFLOW. Figure 4-8 illustrates the entire conceptual functionality of this subroutine. The determination process is the same as in the previous except that the file that contains the grid of the spatial distribution of the head and named faria_head.asc is the file that corresponds to the pumping rates. Figure 4-9 elucidates the results of the post-pumping head distribution for the study area as generated by MODFLOW using Equation 4.1 and presented using ArcView GIS. The subroutine does further computation by calculating at each well location the percentage of drawdown in the saturated thickness corresponding to the two heads (before and after pumping) using Equation 4-5 and the depiction demonstrated in Figure 4-10.

$$\text{Drawdown percentage} = \frac{\text{saturated thickness difference}}{\text{pre-pumping saturated thickness}} = \frac{(\varepsilon + h_1) - (\varepsilon + h_2)}{(\varepsilon + h_1)} \times 100 \tag{4-5}$$

where,

- ε is the distance from the sea level to the bottom of the aquifer and is read by the subroutine from the file named **bottom3.asc** (Figure 4-10),
- h1 and h2 are the pre-pumping and after-pumping heads, respectively.

Based on the results of drawdown percentages the subroutine breaks down the wells into two groups based on the satisfaction of the percentage of drawdown constraint, wells with drawdown percentages within the allowable limits and wells exceeding the allowable limits. The most important output that is required for the decision analysis process is to calculate the total amounts of water that could be abstracted safely based on the given % of allowable drawdown. Accordingly and in order to provide useful outcome from the subroutine, total pumping rates from groundwater (fresh and brackish water) for each specified zone (zone 1 upper, zone 2 middle and zone 3 lower parts of the study area) are specified in the file named

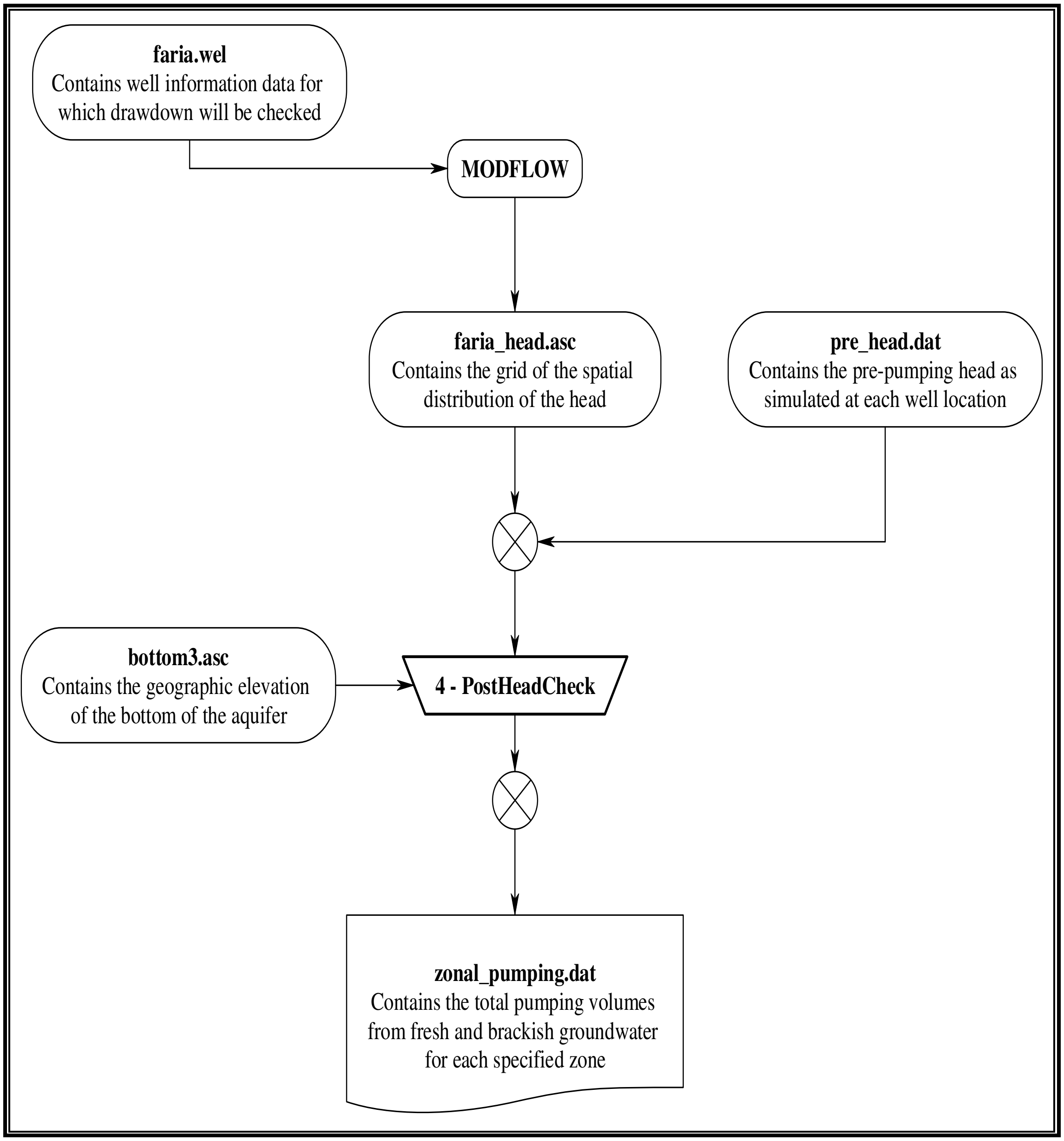

Figure 4-8 Conceptual representation of the functionality of the post-pumping head determination process

zonal_pumping.dat. Table 4-2 presents a summary of the total pumping rates (both with and without considering the head drawdown percentage of 50%) as obtained from the file **zonal_pumping.dat**.

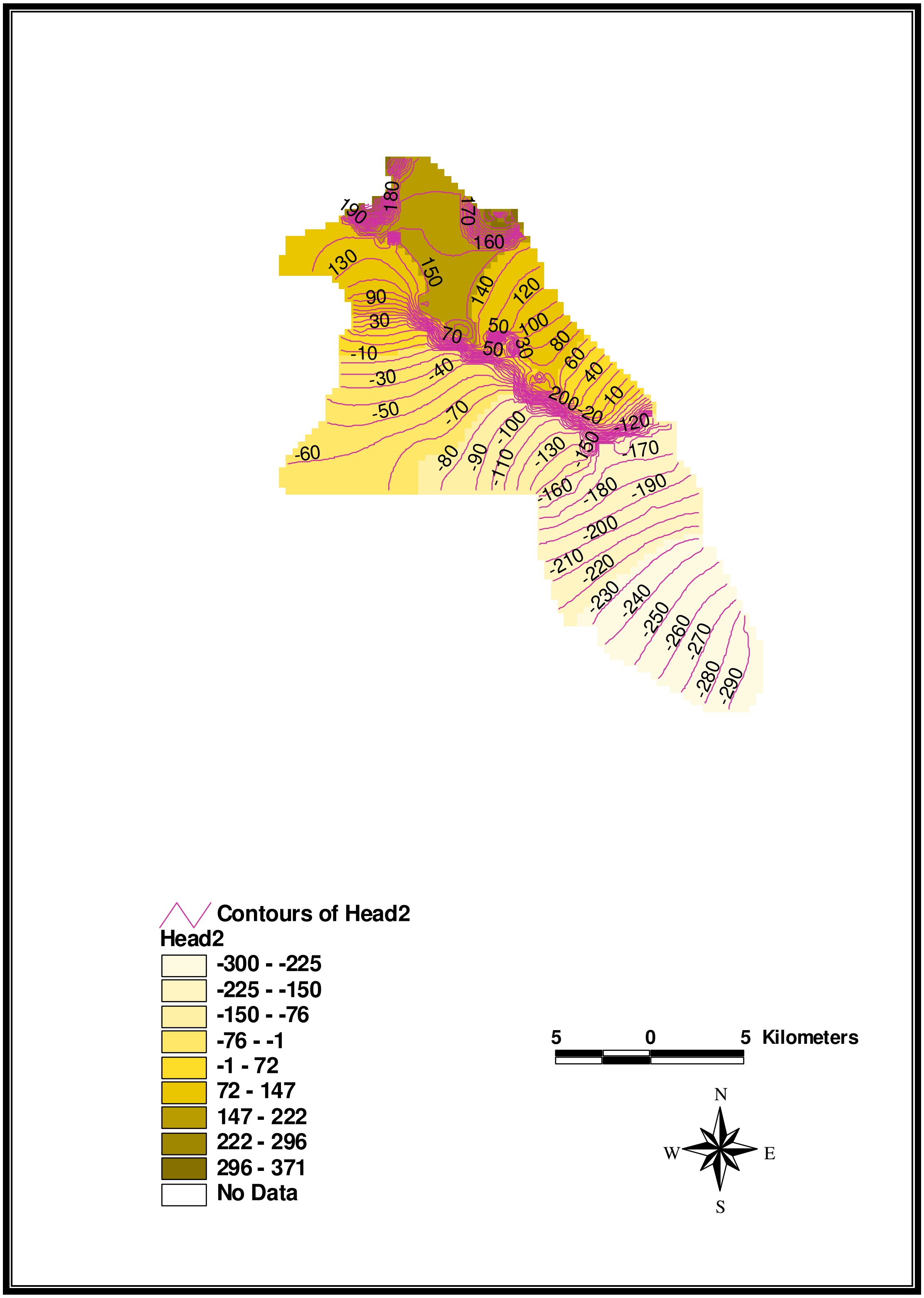

Figure 4-9 The post-pumping head distribution (m) for the study area as generated by MODFLOW and prepared using ArcView GIS capabilities

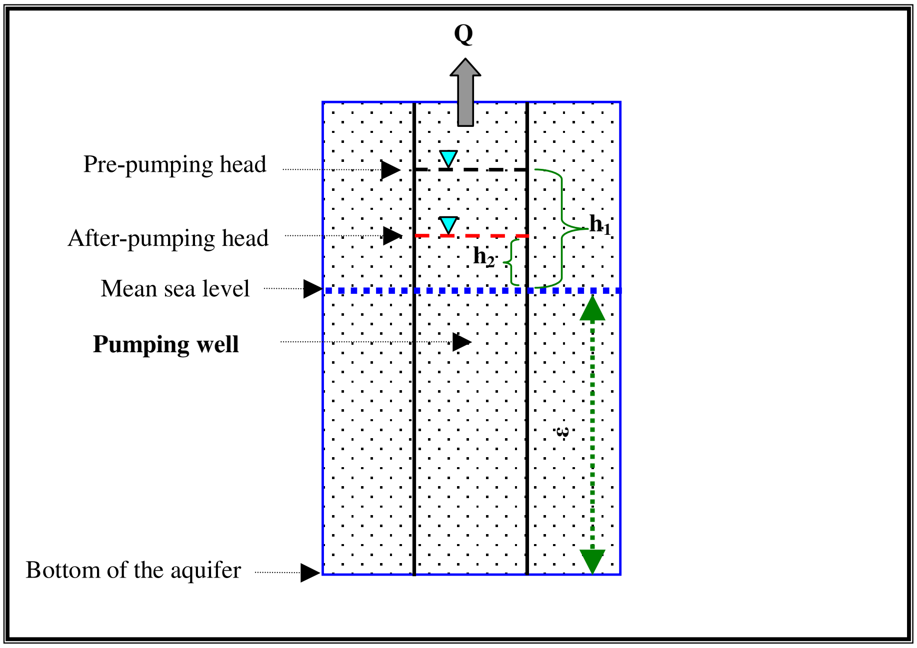

Figure 4-10 A simplified cross sectional view of a well pumping from an aquifer with two distinctive water table elevations corresponding to the pre and post pumping scenarios

As shown in Table 4-2 if there are no restrictions on groundwater abstractions the total groundwater withdrawals in the study area is equal to 22661m^3/day (17330 m^3/day fresh water + 5330 m^3/day brackish water). This is equivalent to 8300 ML per year. Based on an allowable limit of 50% drawdown from the pre-pumping head level, the safe yield is estimated at 14069 m^3/day (5100 ML/yr). This consists of 8738 m^3/day fresh water and 5330 m^3/day brackish water. If a drawdown of 50% is allowed the groundwater abstracted for irrigation purposes alone exceed the safe yield by 3200 ML. For sustainable irrigation practices the groundwater abstraction should not exceed the safe yield. The planning model AGSM in Chapter 5 as well as the selected management alternative in Chapter 6 will ensure the safe yield value calculated from the MODFLOW model is not exceeded when groundwater is abstracted for irrigation and domestic purposes. Results also indicated that based on the 50% allowable limit a total of 1950 ML/ year of brackish water (5330 m^3/day) is abstracted as shown in Table 4-2. This result will be used in evaluating management alternatives in Chapter 6.

Table 4-2 Total pumping rates from groundwater (fresh and brackish water) for the study area

Zone	Pumping (m3/day)		
	Fresh	Brackish	Total
Zone1			
Without drawdown restriction	11790.89	0.00	11790.89
With drawdown restriction	4369.80	0.00	4369.80
Zone2			
Without drawdown restriction	2048.97	0.00	2048.97
With drawdown restriction	878.29	0.00	878.290
Zone3			
Without drawdown restriction	3490.90	5330.89	8821.79
With drawdown restriction	3490.90	5330.89	8821.79
Total			
Without drawdown restriction	17330.76	5330.89	22661.65
With drawdown restriction	8738.99	5330.89	14069.88

The main assumption in estimating the safe yield is the percentage of allowable drawdown limit. For the study area and based on the available local groundwater studies (SUSMAQ, 2004; CPF, 1997), this percentage has been assumed at 50%. Accordingly it is important to investigate the impact of selecting a different value for the allowable drawdown on the safe yield and consequently on developing and evaluating management alternatives in Chapter 5 and Chapter 6.

A value of 80% allowable drawdown was tested following the same procedure described in Section 4.4. The resulting safe yield for agriculture is 5325 ML/yr as shown in Table 4-3. This value will be investigated using the planning and evaluation models that will be developed in Chapter 5 and Chapter 6.

Table 4-3 Total pumping rates from groundwater (fresh and brackish water) for the study area based on 80% allowable drawdown limit

Zone	Pumping (m3/day)		
	Fresh	Brackish	Total
Zone1			
Without drawdown restriction	11790.89	0.00	11790.89
With drawdown restriction	10604.34	0.00	10604.34
Zone2			
Without drawdown restriction	2048.97	0.00	2048.97
With drawdown restriction	889.43	0.00	889.43
Zone3			
Without drawdown restriction	3490.90	5330.89	8821.79
With drawdown restriction	3490.90	5330.89	8821.79
Total			
Without drawdown restriction	17330.76	5330.89	22661.65
With drawdown restriction	14984.67	5330.89	20315.56

4.5 CLIMATE CHANGE

As given in Section 2.9 time horizons are not available for the climate changes in the country, so the information is based on a qualitative evaluation of anticipated trends and effects. Nevertheless the effect of climate change has been addressed in developing the decision support system framework such that this system can integrate possible scenarios on the long term planning horizons (50 or 100 years) that are beyond the planning horizon of 10 years used for this study. As an example to show the capability of testing the future effect of climatic change when such a change can be quantified a scenario of 3% decrease in rainfall has been tested. Accordingly the input file **input_recharge_data.asc** which contains rainfall data and recharge equation parameters was amended to reflect a decrease in rainfall by 3%. The resulting recharge was obtained as detailed in Section 4.1.1 and similar processes were conducted as described in Sections 4.1.2 to 4.1.4 with an allowable limit of 50% drawdown. As shown in Table 4-4 if there are no restrictions on groundwater abstractions the total groundwater withdrawals in the study area is equal to 22661m^3/day (17330 m^3/day fresh water + 5330 m^3/day brackish water). This is equivalent to 8300 ML per year. Based on an allowable limit of 50% drawdown from the pre-pumping head level, the safe yield is estimated at 13858 m^3/day (5050 ML/yr). This consists of 8527 m^3/day fresh water and 5330 m^3/day brackish water. The results showed that the safe yield has slightly decreased from 5100 ML/yr to 5050 ML/yr under such a decrease in rainfall scenario. As detailed in Section

4.3.2, 2075 ML/yr are used for domestic purposes resulting in a safe yield for agriculture of 2975 ML/yr as shown in Table 4-4.

Table 4-4 Total pumping rates from groundwater (fresh and brackish water) for the study area based on climate change scenario of 3% decrease in rainfall

Zone	Pumping (m3/day)		
	Fresh	**Brackish**	**Total**
Zone1			
Without drawdown restriction	11790.89	0.00	11790.89
With drawdown restriction	4369.80	0.00	4369.80
Zone2			
Without drawdown restriction	2048.97	0.00	2048.97
With drawdown restriction	667.22	0.00	667.22
Zone3			
Without drawdown restriction	3490.90	5330.89	8821.79
With drawdown restriction	3490.90	5330.89	8821.79
Total			
Without drawdown restriction	17330.76	5330.89	22661.65
With drawdown restriction	8527.92	5330.89	13858.81

4.6 STATISTICAL ANALYSIS OF DATA FROM SPRINGS

As discussed in Chapter 2 the integrated land and water management framework utilizes statistical analysis of yield of springs in order to assess the reliability of this important source. Statistical analysis of data from springs was carried out in this study to obtain the following information:

- Average monthly and yearly statistics
- Time series analysis
- Seasonal variations

As shown in Figure 4-11, the data was first obtained from different sources and compiled in a composite database. GIS was used to signify the spatial distribution of the springs. In addition, the database was put up in a format that is accessible by MS Excel for ease of analysis. As part of the integrated land and water management framework, analysis of water yield from springs will capture any monthly or seasonal trends that will ultimately be incorporated in strategizing spring utilization such that maximum beneficial utilization is assured. In addition, analysis of yearly yields was carried out to provide an overview and generic insight to the potential yield of springs as well as the reliability of these springs. To comprehend the temporal variation of the yield, time series and seasonal analyses were conducted.

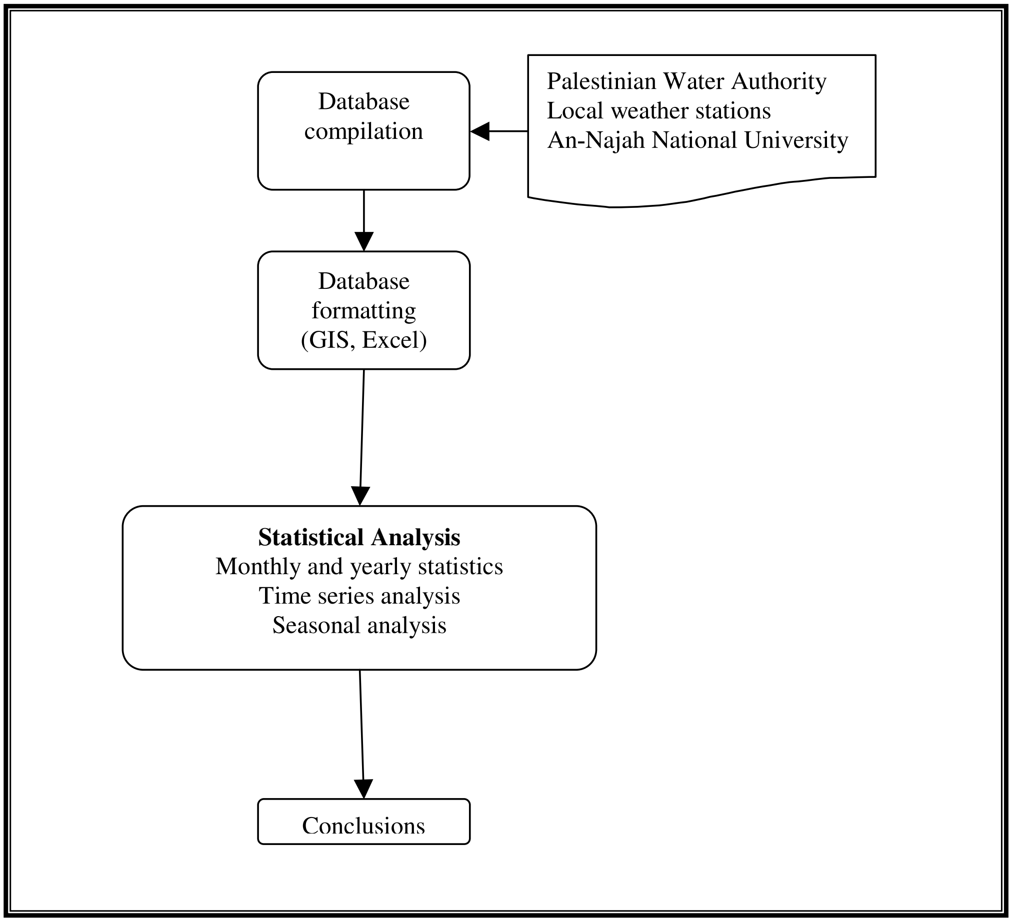

Figure 4-11 A flow chart depicting the general methodology followed in the statistical analysis of the springs' yield of Faria catchment

As described in Chapter 1 there are 11 fresh water springs in the study area that are fully utilized. Table 4-5 summarizes the ID and spatial location coordinates of the 11 springs located in the Faria catchment. These springs are divided into three groups; Faria, Bathan, and Miska (Table 4-5). In addition to these three groups there are two springs located within the upper part of the study area (Dafna and Balata springs) that are utilized directly for domestic use and do not discharge any water into the catchment main stream. As water from these two springs is utilized mainly for domestic purposes they will not be discussed further in this study.

Table 4-5 Springs of Faria catchment (PWA, 2005; WESI, 2005)

Spring ID	Name	Group	X coordinate	Y coordinate	Elevation (m)
AQ/030	Faria	Faria	182.40	188.40	160
AQ/032	Duleib		182.00	187.95	155
AQ/036	Sedreh	Badan	179.95	185.49	240
AQ/037A	Hamad & Baidah		180.12	185.32	215
AQ/037B	Qudaira		180.13	185.28	215
AQ/038	Jiser		180.37	185.10	170
AQ/039	Tabban		180.42	184.82	160
AQ/040	Subyan		180.44	184.42	130
AQ/022	Shibli	Miska	189.90	181.90	-80
AQ/024	Abu Saleh		186.26	183.57	-19
AQ/025	Ein Miska		187.03	182.90	-38

The coordinates of springs were utilized in developing a GIS shapefile of the spatial location of the springs for display and ease of visualization. The majority of the springs are located in the upper and middle parts of the catchment as depicted in Figure 4-12. Monthly springs' yield measurements for the years 1970-1998 were obtained from Palestinian Water Authority (PWA) for all springs with rates exceeding 0.1 L/s. The average annual yield of Abu Saleh spring is estimated at about 80 ML (WESI, 2005) but this spring does not have continuous record measurements and could not be statistically analyzed. The average annual yield for the other springs is reported in Section 4.5.2. In order to facilitate data analysis, the springs' yield data was arranged in MS Excel spreadsheet and later processed for further assessment and analysis.

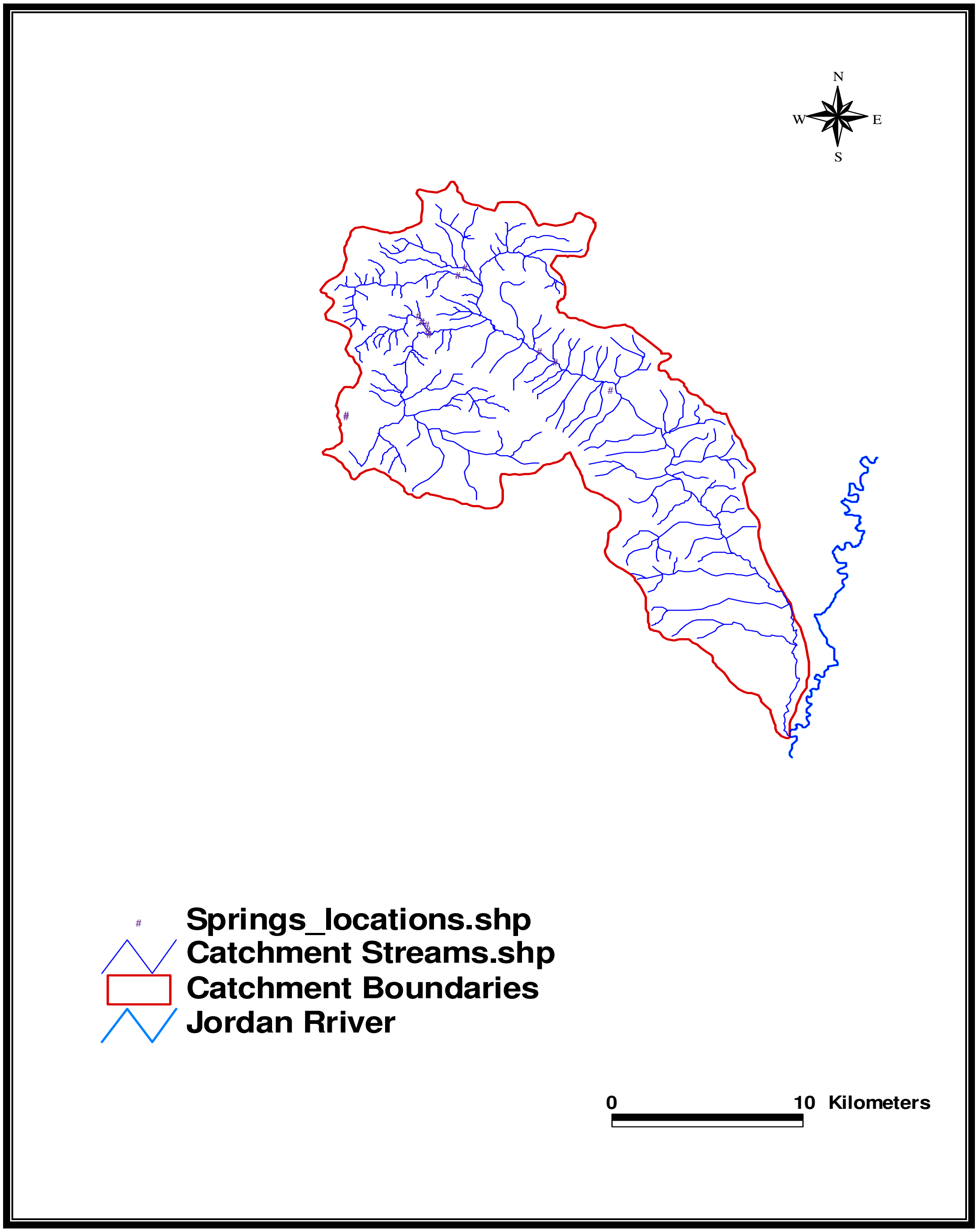

Figure 4-12 Spatial distribution of springs of Faria catchment as prepared using Arc View GIS

4.6.1 Descriptive Statistics of Monthly Spring Yield Data

Descriptive statistical measures of monthly spring yield data were computed per each spring in the Faria catchment. The main intent from carrying out these computations is to characterize the overall springs’ yield and to best draw up an understanding of the variations in the yield in order to design the proper management alternatives for the study area. Box plots were utilized to better visualize and comprehend the average monthly statistics of each spring (Figure 4-13). Illustration of box plot configuration is presented in Figure 4-14. First quartile represents the yield value that 25% of the values are below (25th percentile). Third quartile represents the yield value that 75% of the yields are below the value given in the box plot (75th percentile). The median is the 50% quartile.

For each spring, average monthly box plots were prepared (Figure 4-13). Considering the median of the springs’ yield, an obvious trend can be easily deduced in many springs where an increasing yield starts from November/December till May/June. To a great deal of extent, the first and third quartile values follow the median. Apparently, the springs show high variability in yield as characterized by the considerable difference between the minimum and maximum values. Accordingly it is very important to consider a storage facility as one of the proposed management alternatives that will be discussed in Chapter 6. Such a facility will enable the storage of water yield in winter to be used in other high demand seasons.

4.6.2 Descriptive Statistics of Yearly Spring Yield Data

The main intent of the yearly statistical analysis of springs’ yield is to provide a general outlined view regarding the yield of this important resource. Table 4-6 summarizes the main descriptive statistical measures for the annual yield of the springs in the Faria catchment. As can be inferred from Table 4-6, Faria spring has the highest mean annual yield amongst the springs of the study area followed by Sedreh, Tabban, Ein Miska and Duleib springs. The median also shows that Faria spring has the highest yield followed by Ein Miska and Tabban springs. Faria, Sedreh, and Dulieb springs have the highest standard deviation values which can be easily correlated with the high yield range. The yield range measures the difference between the maximum and minimum recorded yields. The standard deviation and the range signify the variability of springs’ yield and hence denote how reliable the spring is in terms of its persistence at a uniform yield and as a water resource.

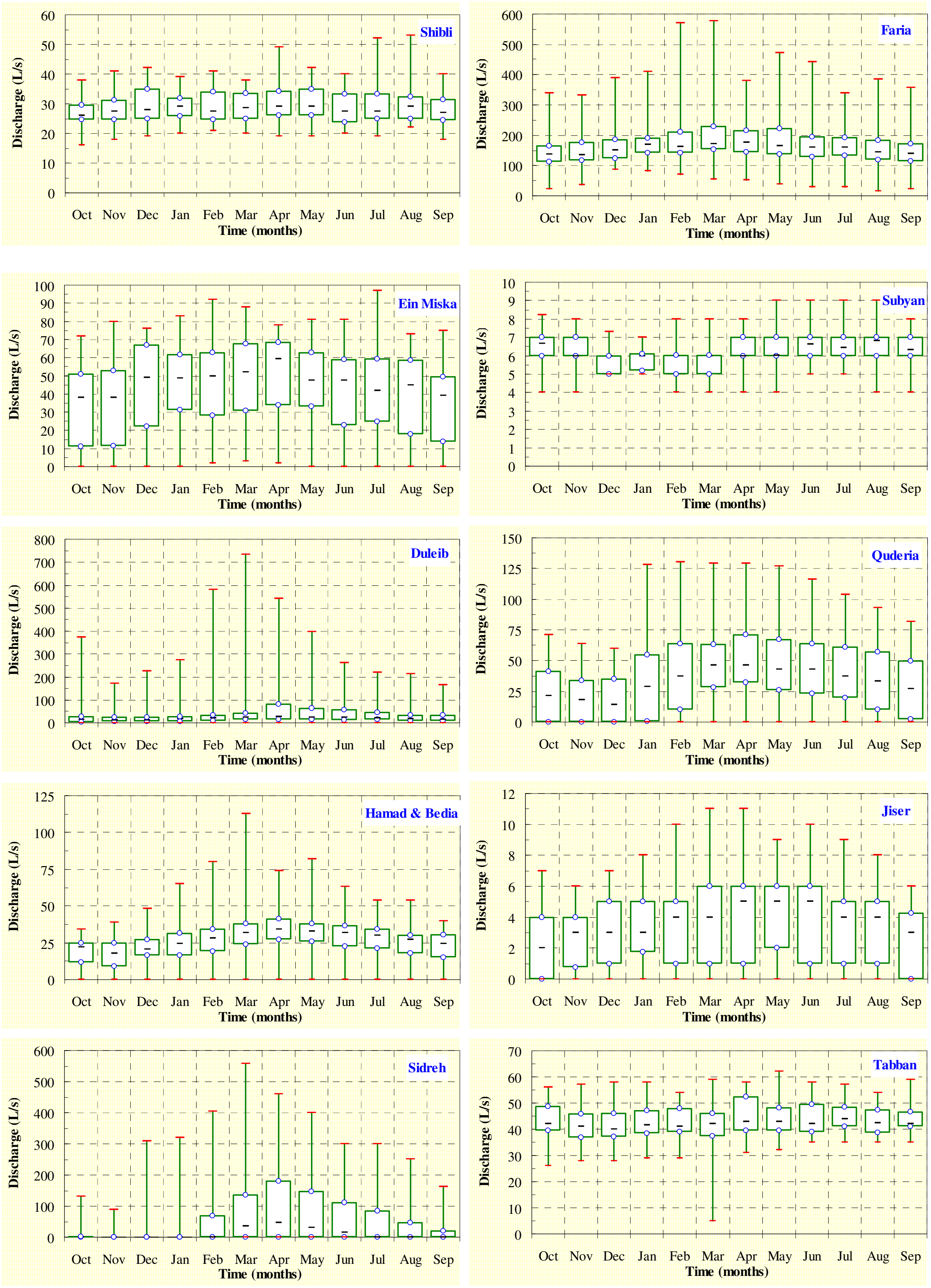

Figure 4-13 Box plots of monthly springs' yield for the years 1970 – 1998

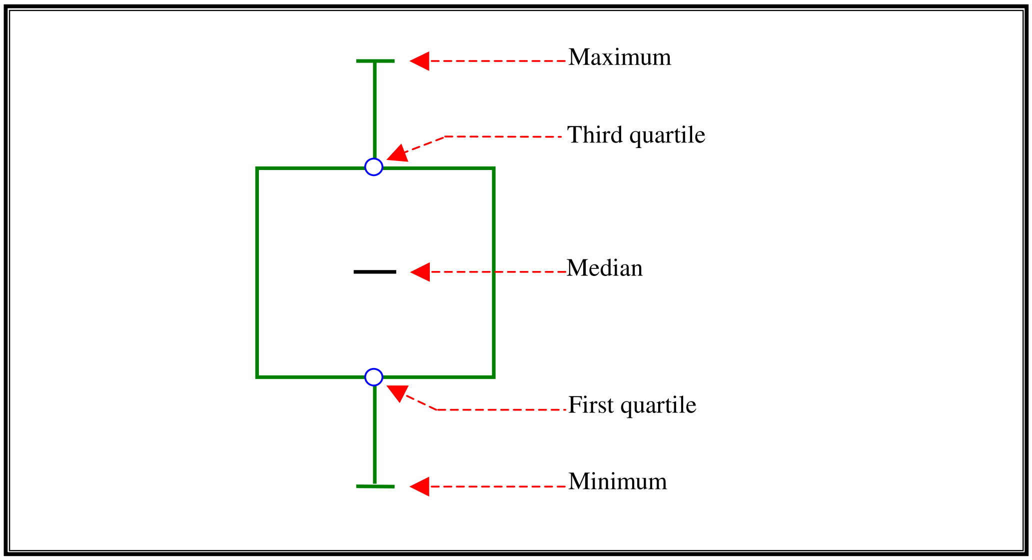

Figure 4-14 An explanation of the box plot configuration

Table 4-6 Descriptive statistics of the annual yield of springs in the study area (ML) for the period from 1970 to 1998

Spring	Mean	Median	Standard deviation	Variance	Range	Minimum	Maximum
Shibli	880	870	200	40	1010	170	1180
Faria	5000	4380	2190	4790	11270	1440	12720
Ein Miska	1250	1410	660	440	2190	20	2210
Subyan	190	190	40	0	200	40	240
Duleib	1200	620	1960	3820	10100	0	10100
Qudeira	1100	910	840	700	2960	0	2960
Hamad & Bedia	720	770	420	180	1960	0	1960
Jiser	110	130	70	10	270	0	270
Sedreh	1380	300	2110	4430	9520	0	9520
Tabban	1290	1300	280	80	1440	230	1660

Faria spring has the highest annual yield with median and maximum values of 4400 and 12700 ML. Total annual springs' yield for Faria catchment was also analyzed and results are depicted in Figure 4-15. There is a vast variability in the total annual springs' yield in the order of 39,000 ML with a mean of 13,000 ML/yr and a median value of 12,000 ML/yr.

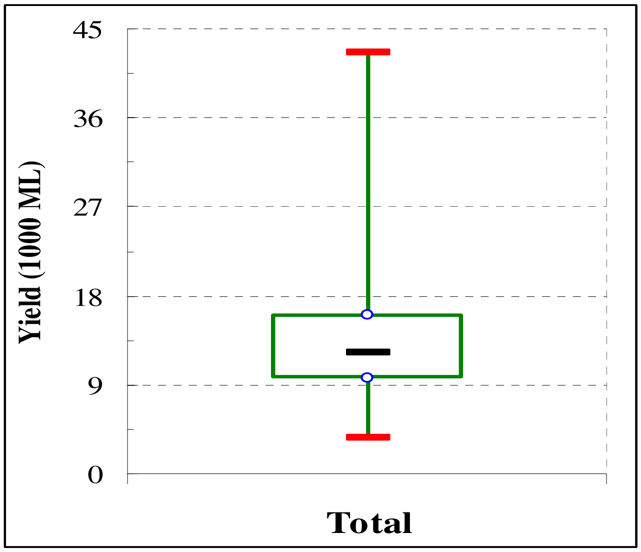

Figure 4-15 Box plot of the total annual springs' yield for Faria Catchment

The overall reliability of each spring was calculated using Equation 4-6 (Dracup et al., 2005):

$$R = 1 - \frac{\sum_{j=1}^{n} (Demand_j - Delivery_j)^+}{\sum_{j=1}^{n} Demand_j} x100\% \qquad (4\text{-}6)$$

where,

R = the overall reliability

j = the corresponding timestep taken as yearly

$Demand_j$ = the water demand for the timestep j

$Delivery_j$ = yield of the spring for the timestep j

The + sign denotes only the positive values are considered.

n = number of timesteps

The total water demand for the study area is calculated using the CROPWAT model and the areas of each crop irrigated in the study area and is estimated at 20117 ML/yr. Details on calculating the crop water requirements for different crops and the cropping pattern for the study area is given in details in Chapter 5. The water demand for each spring was calculated as a proportion of the total water demand using Equation 4-7:

$$D_s = (Y_s/Y_t)*D_t \tag{4-7}$$

where,

D_s = demand for each spring

Y_s = yield from a specific spring

Y_t = total yield from all springs

D_t = total water demand

Table 4-7 presents the results of the reliability for each spring calculated using Equations 4-6 and 4-7. It was assumed that a spring is reliable and is considered for planning purposes if its overall reliability exceeds 50%. The results in Table 4-7 show that the reliability for all the springs exceeded the target of 50% and ranged from 0.54 to 0.83. Accordingly it was decided to consider the mean value of the total yield of all the springs which is 13,000 ML. This value will be used as the available amounts of spring water when developing the planning model AGSM in Chapter 5.

Table 4-7 Reliability of the springs in the study area for the period from 1970 to 1998

Spring	**Reliability (%)**
Shibli	54
Faria	62
Ein Miska	63
Subyan	56
Duleib	79
Qudeira	67
Hamad & Bedia	64
Jiser	62
Sedreh	83
Tabban	55

4.6.3 Time Series of Springs' yield

Figure 4-16 depicts the time series of the yield from each spring. Upon comparing the different time series, the springs exhibit different time-based behaviors. Many springs show high oscillation in yield which indeed reflects the variability of the weather (e.g. precipitation). As shown in Figure 4-16 the majority of the springs have an escalated yield in the year 1991/1992 which was correlated with the high rainfall intensity in that specific time period. Many springs such as Subyan, Quderia, Hamad and Bedia show kind of monotonic or cyclic behavior as can be seen in Figure 4-16. On the other hand, specific springs like Shibli and Tabban have high unpredictability yield manner.

4.6.4 Seasonal Analysis of Springs' yield

As stated earlier Faria spring is the major spring in the study area. As such, Faria spring was considered for further analysis to investigate any seasonal variations in the yield. Faria spring yield data was arranged and processed using MS Excel spreadsheet to figure out the seasonal yield trends of this spring. Figure 4-17 depicts the seasonal time series of Faria springs' yield. In all four seasons a peak yield can be noticed in the year 1991/1992 which indeed correlates well with the extreme rainfall event at that year. The overall average seasonal yield is displayed on Figure 4-17. The Spring season has the highest average yield of 182 L/s followed by Winter, Summer and Autumn with average yield values of 171, 161, and 132 L/s respectively. This sequence is not valid for all the years as can be seen from Figure 4-18. The outcome of Figure 4-17 was utilized to rank the seasons (Figure 4-18) on yearly basis for Faria springs' yield. Winter and spring do have the maximum number of occurrences of high seasonal total yield of Faria spring. As expected, Autumn season does have the lowest yield due to the fact that it follows the hot summer season. Summer season has a relatively high number of occurrences of second and third ranking positions which indeed may denote a possible lag in spring response to the key factors such as precipitation and temperature. This analysis further emphasizes the need to store water during low demand seasons for later use in high demand seasons.

When developing the planning model AGSM the average seasonal yield of springs in the study area will be used on seasonal basis. The total average seasonal yield for all springs in Faria catchment are 3423 ML, 3817 ML, 3299 ML and 2489 ML for Winter, Spring, Summer and Autumn respectively as will be discussed in Chapter 5.

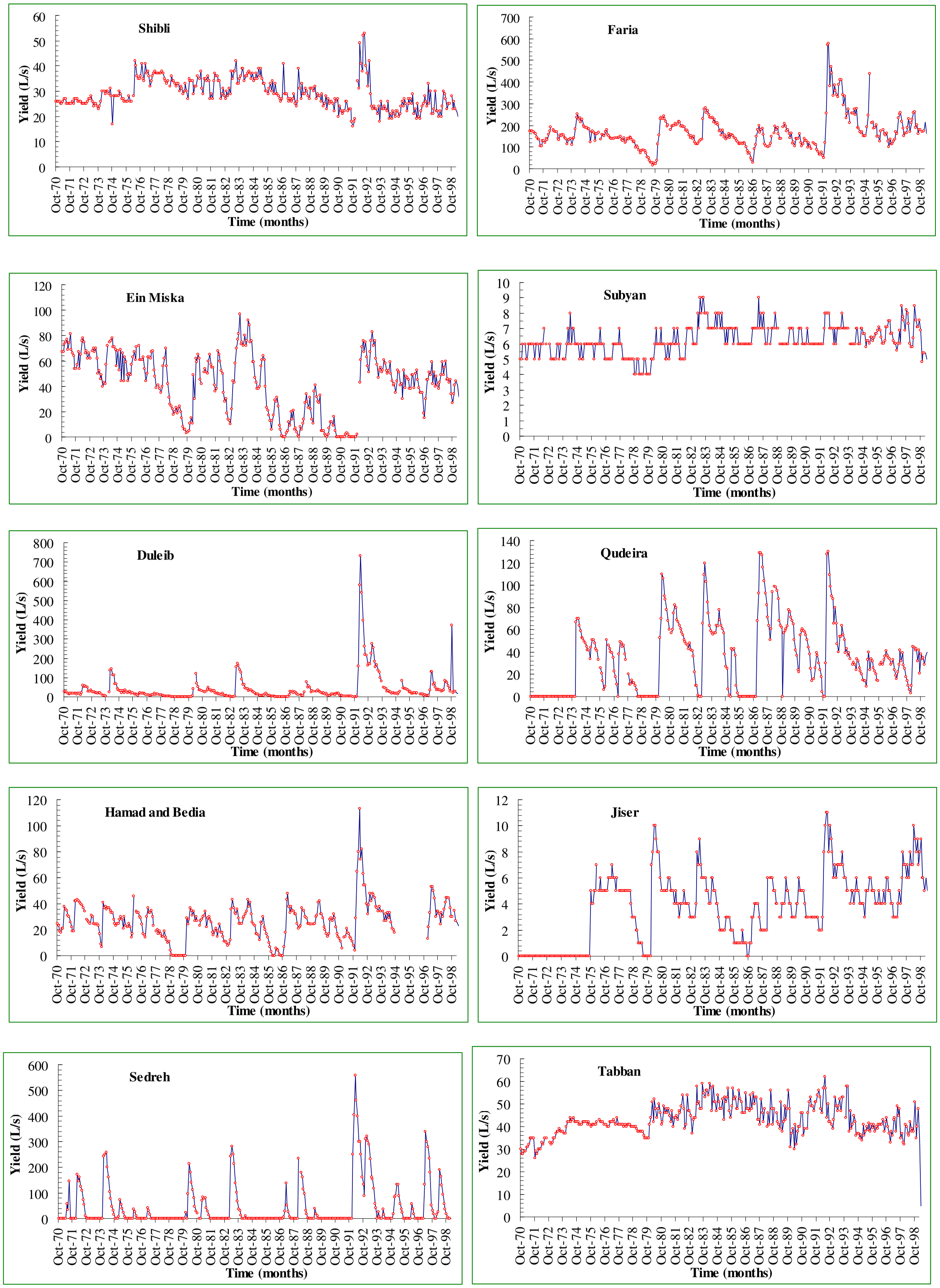

Figure 4-16 Time series of the yield of springs in the Faria catchment

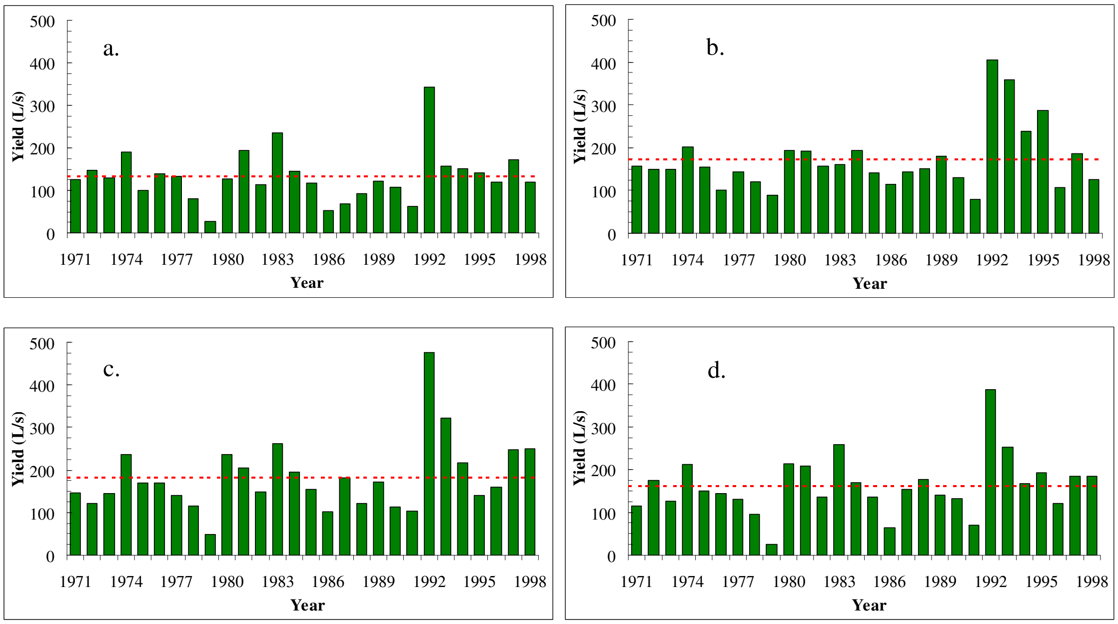

Figure 4-17 **Seasonal time series of average yield of Faria spring; (a) Autumn; (b) Winter; (c) Spring; (d) Summer. Dashed line marks the average seasonal springs' yield.**

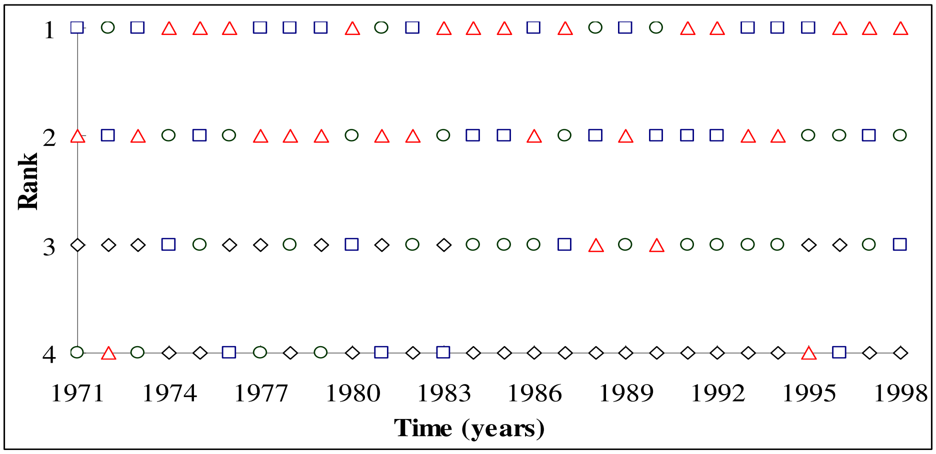

Figure 4-18 **Ranking of the four seasons in terms of Faria springs' yield for the period from 1971 to 1998 where 1 indicates the highest seasonal yield. Symbols are as follows: ◊ Autumn Δ Spring ○ Summer and □ Winter**

4.7 SUMMARY AND CONCLUSIONS

The MODFLOW software package has been utilized to estimate the amounts of groundwater that could be safely extracted under rainfall variation and different management alternatives. The main intent for the utilization of the groundwater flow model is for the development of management alternatives for the study area. There are two input parameters that influence MODFLOW directly. These are groundwater recharge and pumping rates. Groundwater recharge is a function of the rainfall intensity and rainfall distribution across the study area. Pumping rates reflect the pumping rates to satisfy the safe yield conditions.

Results showed that, based on an allowable limit of drawdown percentage of 50%, the safe yield for agriculture is 3000 ML/yr. Accordingly, the current irrigation groundwater wells are abstracting water above the safe yield with a total amount of 3200 ML which negatively affects the sustainability of the aquifer and necessitates the formulation of management alternatives to address such a problem. For sustainable irrigation practices the groundwater abstraction should not exceed the safe yield. The safe yield value calculated from the MODFLOW model will be considered when developing the planning model AGSM in Chapter 5 as well as the selected management alternative in Chapter 6. The safe yield value calculated from the MODFLOW model should not be exceeded when groundwater is abstracted for irrigation and domestic purposes. Results also indicated that an annual total of 1950 ML of brackish water were abstracted. These results are needed to formulate the management alternatives for the catchment.

To investigate the impact of selecting a value for the allowable drawdown on the safe yield and consequently on developed alternatives in Chapter 6, a different value of 80% allowable drawdown was tested. Results showed that the safe yield for agriculture increased to 5325 ML/yr.

As an example to show the capability of testing the future effect of climatic change when such a change can be quantified a scenario of 3% decrease in rainfall has been tested. The results showed that the safe yield for agriculture has slightly decreased to 2975 ML/yr.

A statistical analysis was conducted on data from springs' yield. The main objective of carrying out this analysis was to assess the amounts of springs' yield of the Faria catchment. The springs show high variability in yield as characterized by the considerable difference between the minimum and maximum values. An obvious trend can be easily deduced in many springs where an increasing yield starts from November/December until May/June. Faria spring has the highest mean annual yield amongst the springs of the study area followed by Sedreh, Tabban, Ein Miska and Duleib springs. There is a considerable variation between the minimum and maximum annual springs' yield in the order of 39000 ML with a total mean of 13000 ML. Time series show that yield is dependent on precipitation. The majority of the springs have an escalated yield in the year 1991/1992 which was correlated with the high rainfall intensity in that specific time period. The reliability of each spring was tested and the results showed that the reliability of all the springs exceeded 50% and it was concluded that all of the springs would be used for planning purposes. The total average seasonal yield for Faria springs are 3423 ML, 3817 ML, 3299 ML and 2489 ML for Winter, Spring, Summer and Autumn respectively. It is very important to consider a storage facility as one of the proposed management alternatives that will be discussed in Chapter 6. Such a facility will enable the storage of water yield in winter to be used in other high demand seasons. Results from this analysis will be used to supply the spring yield data needed for the planning model that will be developed in Chapter 5.

CHAPTER 5

DEVELOPMENT OF THE AGRICULTURAL PLANNING MODEL FOR FARIA CATCHMENT

5.1 INTRODUCTION

Water scarcity and low per capita water allocation is the major characteristic of arid and semi-arid regions. In such environment water becomes an important and precious resource for domestic and agricultural supplies and social and economic development. This situation is further exacerbated when such areas are agriculturally dominated and encounter a high population growth rate, compelling the motivation for an optimal allocation of scarce water resources.

Irrigated agriculture is the most important economic in the study area. The study area is part of the Jordan Valley which is considered as the only potential area for agricultural development during the coming years, in addition to its current importance as the largest agricultural production area in the West Bank (WSSPS, 2000). The predominantly rural population in the catchment faces a series of environmental threats and poor economic conditions. The rapid population growth has resulted in increased demand for natural resources, mainly land and water. Lack of proper management of land and water resources has caused over abstraction of the scarce water resources and ineffective use of land. Surface runoff in the catchment is not utilized in winter as there are no dams in the catchment to store the excess water. Existing cropping patterns are rigid in response to changing conditions including the ongoing shifts in the market demand and supply sides, available water quality, price and quantity.

As stated in Chapter 2, AGricultural Sub-Model (AGSM) reported in Fisher et al. (2005) will be used as the planning model to simulate the behaviour of farmers and optimally determine the cropping pattern that maximizes the farmer economic return under various management policies and scenarios. The model can provide catchment, regional and district level planners and decision-makers with a management tool for planning agricultural production under various water amounts, qualities, timing and prices, as well as testing "what-if" scenarios of water pricing policy. As explained earlier (Figure 2-1) the AGSM needs information on available surface and groundwater yield as well as costs on building or upgrading infrastructure to make decisions on different management alternatives. The surface water yield and peak flow for infrastructure design will be obtained from rainfall-runoff modelling as explained in Chapter 3. The yield from groundwater and from springs will be calculated as explained in Chapter 4 using results from the MODFLOW model and total annual mean of the yield from all springs which have a reliability of 50% or more. This chapter elucidates the development of the planning model to optimize use of land and water. A description of the AGSM and its input-output parameters is discussed. An elucidation of the AGSM building is also given utilizing the input data that was collected from different sources. Verification of the AGSM against existing data is discussed. After successful model building and verification process the AGSM will be ready to be used to evaluate different management alternatives through optimizing the relationship between the available natural resources and the corresponding cropping patterns. This will be discussed in Chapter 6.

5.2 MODEL DESCRIPTION

In the current study AGSM is operated at the catchment level. Its final outcome is the net agricultural income of the catchment, which is maximized by selecting the optimal mix of water-consuming activities (optimal cropping pattern). Maximization is conducted through a built-in optimization tool. The optimized variables are the land areas of each of the crop types. Planting of each crop type (agricultural activity) is called an activity. Each activity is characterized by its water requirements per unit area and the net income it produces per land area which do not include water payments. This is called water-related contribution (WRC). The water-related contribution of activity j, WRCj is defined as the gross income generated by activity j per unit area minus all direct expenses (machinery, labor, materials, fertilizers) associated with the activity except for direct payments for water.

Each activity in principle can use one or more types of water in all or any of the four seasons. The water quality types are fresh water, surface water (including spring water), brackish water and recycled wastewater. The four seasons and four water quality types will make sixteen season-quality combinations. AGSM can support up to 16 different water prices based on season and type of water quality as shown in Equation 5-1. However, in the current study only two types of water have been considered. That is freshwater and surface water. The brackish water is mixed with fresh water and distributed as one source of water. Therfore fresh water refers to groundwater abstractions for irrigation purposes including the brackish water. Surface water refers to spring water, storm water runoff and untreated wastewater flowing through the same Faria stream. The amount of water used will depend on the activities carried out in the catchment.

As shown in Equation 5-1, the final outcome is the annual net income (Z) of a district.

$$Z = \sum_{j=1}^{n} X_j \left[WRC_j - \sum_{i=1}^{m} (P_i W_{ij}) \right] \qquad (5\text{-}1)$$

where,

WRC_j - water-related contribution of activity j in US$ per unit area (US$ is the most common local currency used in the study area and the data and reports collected);

P_i - price of one cubic meter of the ith water type, in US$ per cubic meter;

W_{ij} - demand of water of type i per unit area of activity j in cubic meter per unit area;

X_j - optimal area assigned to activity j;

n - number of activities; and

m - number of types of water qualities at different seasons.

Net income is calculated in two parts. The first of these is “water-related contribution” (WRC). As mentioned earlier the WRC does not account for direct cost of water. The cost of water is subtracted from WRC to obtain the net income (Z). The second component of net income consists of direct payments for water (P_iW_{ij}).

The optimization parameters are the land areas of the activities. There will be constraints on the activities based on land and water demands and their availabilities. The constraints in AGSM are given as in Equations 5-2 and 5-3.

Water constraint

For each water type the water constraints are given by the general form as in Equation 5-2.

$$\sum_{j=1}^{n} W_{ij} X_j \leq W_i \qquad (5\text{-}2)$$

where,

W_i is the total amount of type i water available for all activities,

W_{ij} and X_j are as defined in Equation 5-1.

Land area constraint

The land area available for activities is constraint by the maximum land available for each category. Categories are groups of land areas that accounts for similar activities (field crops, orchards, nonirrigated, etc). Such categories address the suitability of land parcels for different activities. The land area constraint could be written in the general form as given in Equation 5-3.

$$\sum_{j=1}^{n} X_{jk} \leq A_k \tag{5-3}$$

where,

k is the category ;

X_{jk} is the area of activity j in category k; and

A_k is the total area available for category k.

Based on the type of crops grown in each catchment the user can define different categories that include one or more of crop types. In the current study categories included the following:

- crops of the same group (e.g. greenhouses, covered vegetables, field crops, open field vegetables, trees, citrus, rainfed);
- total area of activities, total area of irrigated activities and total area of non irrigated activities;
- crops of the same group (e.g. greenhouses, covered vegetables, field crops, open field vegetables, trees, citrus, rainfed) grown within each of the three catchment zones (upper, middle, lower).

Land availabilities within each category reflect soil-crop compatibility and other limitations that may be imposed by institutional directives through imposed cropping patterns. As explained in Chapter 1, the study area constitutes three agroclimatic zones including the upper, middle and lower zones. Accordingly, in developing the model, land area was also divided into three parts, Upper, Middle and Lower.

5.3 MODEL INPUT-OUTPUT PARAMETERS

The model input data are:

- maximum land area available for each activity;
- maximum quantity of each type of water available and price of water;

- total irrigation water requirements by each activity; and
- water related contribution WRC for each activity.

To calculate the above parameters information is needed on all agricultural activities existing in the catchment or that can be introduced there; water demands for each activity using the allowed types of water and total availability and price of all types of water. To calculate the WRC information is also needed on yields of crops; product prices and input costs (sum of input costs including labor, materials, machinery and fertilizers). The following subsections describe the methodology to calculate the above parameters. Detailed calculation of the above parameters for the study area is given in Section 5.4.1.

The model output data are:

- total net income from the catchment;
- income by each agricultural activity;
- optimal land area for each activity;
- total quantity of water used for each quality;
- comparison of water and land allocations versus the associated constraints with relevant shadow prices.

Shadow price of water is the increase in the net benefit (US$) per unit increase of water allocation (m^3). Shadow price of water increases as water becomes scarce. The shadow value of water at a particular location shows the increase in system wide benefits that would occur if an additional cubic meter of water were available at that location (Fisher et al., 2005). The resulted cropping pattern should at least satisfy the domestic demand and no enormous change in the actual cropping pattern is allowed by use of constraints on areas.

5.3.1 Water Availability

Results of the rainfall runoff and groundwater models as well as the analysis of spring data given in Chapter 3 and Chapter 4 were utilized to determine the available quantities of different types of water. Prices of water were collected from the local authorities in the catchment. Available water amounts and prices for the study area are given in Section 5.4.1.

5.3.2 Total Irrigation Water Requirements

The total irrigation water requirement for each of the activities in the catchment is an input parameter in the AGSM. Total irrigation requirements include Crop Irrigation requirements (Crop water requirements (CWR) minus effective rainfall) plus leaching requirements and the losses in the distribution system.

Crop water requirements (Actual Evapotranspiration) for common crops in the catchment were estimated using the CROPWAT model based on crop coefficients recommended by FAO. Crop coefficient varies according to the type of crop and the stage of growth. Values of Kc were based on the revised FAO methodology for crop water requirements (FAO, 1998b), where,

$$ET_a=ET_o*K_c \tag{5-4}$$

where,

ET_a = Actual evapotranspitration

ET_o = Reference evapotranspitration

K_c = Crop coefficient

Reference Evapotranspiration (ET_o) was estimated using the CROPWAT Version 4.2 model (FAO, 1998a) based on the average climatic data for the upper and lower parts of the catchment (MoT, 1998; WESI, 2005; ARIJ, 1998).

The FAO model takes into consideration the effective rainfall. Effective rainfall had been calculated based on the Dependable Rain Method developed by FAO (FAO, 1998a) for arid and semi-arid region climates (Equations 5-5 and 5-6).

$$Rn= 0.6 * Rt-10 \quad \text{(for Rt < 70mm/month)} \tag{5-5}$$

$$Rn= 0.8 * Rt-24 \quad \text{(for Rt > 70mm/month)} \tag{5-6}$$

where,

Rn = Effective rainfall (mm) per month

Rt = Total rainfall (mm) per month

To determine the total water requirements of activities planted in the study area, the efficiency of the distribution system should also be considered and the crop irrigation requirements would be adjusted for distribution losses. In addition, leaching requirements should be taken into account resulting in an increase in the irrigation water demand.

5.3.3 Maximum Land Areas For Each Crop Type

Data regarding the actual areas grown with different crop types should be collected from the agricultural departments and local authorities in the catchment. The distribution of these quantities among different locations within the catchment should also be recognized. In addition the grouping of different crops into certain categories that are familiar to the farmers and planners in the catchment would also be available. The actual data of these activities would be used to verify the building of AGSM for the study area. Information on the above data for the study area is given in Section 5.4.

5.3.4 Water Related Contribution (WRC)

The water related contribution for each crop should be calculated (where WRC in US$/ha= Yield Price per kg (US$) * Yield (kg/ha) – Input Costs (US$/ha)) and the data to be entered to the AGSM. The water related contribution of different crops grown in the study area is given in Section 5.4.1.

5.4 MODEL BUILDING

Information related to the most common crops cultivated in Faria catchment such as, yield, variable costs, fixed costs, gross margin, sale price and net income is required for AGSM application. Crop water requirements will be estimated using the model developed by the FAO (FAO, 1998a). After building the model, the model will be verified against existing conditions. Furthermore, analysis of the effect of different water pricing policies on the utilization of water for irrigation is discussed. The following sections details the methodology followed in building the AGSM for the study area.

Data were collected from official sources, research institutions as well as from field visits and personal interviews. Official sources included:

- Ministry of Agriculture (MoA),

- Palestinian Water Authority (PWA),
- Environmental Quality Authority (EQA),
- Palestinian Central Bureau of Statistics (PCBS).

Relevant agricultural data was obtained from Ministry of Agriculture (MoA) and Palestinian Central Bureau of Statistics (PCBS). These data describe agricultural activities in the area, including:

- existing agricultural activities and cropping patterns;
- statistical data about the cultivated areas;
- existing irrigation systems;
- irrigation schedules;
- planting dates; and
- inputs and outputs of farm data for different crops.

Available water resources data were obtained from the results of the rainfall runoff and groundwater models presented in Chapter 3 and Chapter 4 respectively.

5.4.1 Model Input Data

Water availability

Agricultural water in the study is supplied through a number of small pipe networks systems from irrigation wells or through water supplied from Faria stream. The Faria stream includes spring water and winter stormwater runoff mixed with wastewater flowing from the upper urban area of the catchment. As shown earlier in Chapter 3 and Chapter 4, the estimated total water available for irrigation is 22800 ML per year. Approximately 70% of this irrigation water is supplied from the Faria stream which is equivalent to 16600 ML. As stated in Chapters 3 and 4, 13000 ML is available from springs and 2600 ML from winter surface runoff. The remaining 1000 ML is supplied from wastewater flowing from the upper urban area of the catchment (PWA, 2005; WESI, 2005). The estimated water supply for agriculture from irrigation wells is 6225 ML per year. The estimated efficiency of the spring distribution system and the irrigation wells distribution system is 70% and 90%, respectively (WESI, 2005). Accordingly, the average distribution efficiency is estimated at 77 %.

In the model application the water supply is grouped into two categories: fresh water refers to groundwater abstractions for irrigation purposes (including brackish water) and surface water refers to spring water, storm water runoff, and untreated wastewater flowing through the Faria stream. However, other water quality categories including treated wastewater can be used separately, especially when formulating future management plans for the catchment.

As given earlier in Chapter 4 the total available water supply for agriculture was distributed among the four seasons (WESI, 2005; PWA, 2005) and is depicted in Table 5-1. The average water price per cubic meter for all seasons was 0.20 US$ for fresh water (WESI, 2005), whereas, surface water was distributed free of charge.

Table 5-1 Available water per season and average water price for Faria Catchment

Quality Water	Season	ML,Water supply	Water Price,SU$/$m^3$
Fresh	Winter	275	0.20
	Spring	1630	0.20
	Summer	1820	0.20
	Autumn	2500	0.20
Surface	Winter	5923	0
	Spring	4417	0
	Summer	3549	0
	Autumn	2739	0

Total irrigation water requirements

In order to get the optimal cropping pattern, the model needs the total irrigation water requirements for each of the activities in the catchment. Total irrigation requirements were calculated as detailed in Section 5.4.1. Reference Evapotranspiration (ETo) was estimated using the CROPWAT Version 4.2 model (FAO, 1998a) based on the average climatic data for the upper and lower parts of the catchment (MoT, 1998, WESI, 2005, ARIJ, 1998). Results of the ETo values for different months for the upper and lower parts of the catchment are shown in Chapter 3 (Table 3-2 and Table 3-3 respectively).

The name of each crop and its planting date were entered into the CROPWAT model which calculates crop water requirements, and irrigation water requirements for each crop after considering effective rainfall. Monthly crop water requirements for all the crops were estimated using CROPWAT model. Table 5-2 and Table 5-3 show an example of the monthly crop water requirements for Onion crop (a crop that belongs to the field crops) grown in the upper and lower areas, respectively. Crop coefficients (Kc) for each month of the growing season are also given in Table 5-2 and Table 5-3.

Table 5-2 Example of Monthly crop water requirements for Onion crop grown in the upper areas.

Date	ETo (mm/period)	Planted Area(%)	Crop Kc	CWR (ETm)	Total Rain (mm/period)	Effect Rain	Irr. Req.
1/11	72.32	100.00	0.72	51.77	78.33	38.66	13.10
1/12	50.80	100.00	0.95	47.92	133.69	83.62	0.00
31/12	45.70	100.00	1.05	47.99	144.11	87.03	0.00
30/1	62.52	100.00	1.05	65.65	147.43	97.59	0.00
1/3	91.19	100.00	1.05	95.75	102.86	53.95	41.80
31/3	124.77	100.00	1.02	127.39	23.25	1.21	126.19
30/4	154.99	100.00	0.85	130.79	0.82	0.00	130.79
Total	602.31			567.26	630.49	362.06	311.88

* ETo data is distributed using polynomial curve fitting.
* Rainfall data is distributed using polynomial curve fitting.

Table 5-3 Example of Monthly crop water requirements for Onion crop grown in the lower areas.

Date	ETo (mm/period)	Planted Area(%)	Crop Kc	CWR (ETm)	Total Rain (mm/period)	Effect. Rain	Irr. Req.
1/11	72.31	100.00	0.72	51.72	23.82	1.25	50.47
1/12	46.84	100.00	0.95	44.13	43.45	16.06	28.06
31/12	44.53	100.00	1.05	46.76	45.86	20.96	25.80
30/1	62.48	100.00	1.05	65.60	42.61	5.36	60.25
1/3	96.64	100.00	1.05	101.47	37.27	14.19	87.29
31/3	138.58	100.00	1.02	141.44	18.75	0.56	140.89
30/4	177.31	100.00	0.85	149.56	0.37	0.00	149.56
Total	638.69			600.68	212.13	58.37	542.30

* ETo data is distributed using polynomial curve fitting.
* Rainfall data is distributed using polynomial curve fitting.

The calculation for Onion crop in the upper area is summarized below:

Total reference evapotranspiration = 602 mm

Total crop water requirements (CWR=ETo*Kc) = 567 mm

Total effective rainfall ($R_{effective}$) = 362 mm

Total crop irrigation requirements (IR) = 311 mm

Irrigation requirements are calculated on monthly basis where:

Irrigation requirement (IR) = CWR-$R_{effective}$ ($R_{effective} < CWR$)

If $R_{effective} > CWR$ then IR=0

To determine the total gross irrigation water requirements of activities planted in the study area, the efficiency of the distribution system was considered and the crop irrigation requirements were adjusted for distribution losses. In addition, leaching requirements were considered that will increase the irrigation water demand, accordingly. In the upper areas, precipitation is sufficient to leach all the salts added by irrigation. In the lower areas a 20% leaching fraction was assumed, while the central areas, a 10% leaching fraction was assumed. Accordingly, the calculation for the gross irrigation requirements of Onion crop in the upper area is as follows:

Gross irrigation requirements after adjusting for losses=IR/distribution efficiency

= 311/0.77

= 403 mm

Since this crop is in the upper area, no further adjustment for leaching requirements is done.

Similar analysis is conducted for onion in the lower area giving a final crop irrigation requirement of about 542 mm. This amount will be adjusted for distribution losses to about 703 mm. Finally, this amount will be increased to account for leaching requirements and the gross irrigation requirements for onion in the lower areas is 843 mm as detailed below:

Total reference evapotranspiration = 638 mm

Total crop water requirements (CWR=ETo*Kc) = 600 mm

Total effective rainfall ($R_{effective}$) = 58 mm

Total crop irrigation requirements (IR) = 542 mm

Gross irrigation requirements after adjusting for losses = 542/0.77 = 703 mm

Total Gross irrigation requirements after adjusting for leaching = 703*1.2=843 mm

Most of the irrigation systems in the study area are drip systems with more than 90% efficiency (MoA, 2005a). As such no adjustment was done for water losses due to application and it was assumed that the total water requirements are equal to the gross crop irrigation water requirements.

Maximum land areas for each crop type

Irrigated agriculture includes three main groups: vegetables, field crops and trees. Irrigated vegetables are grown either as open field vegetables or as protected vegetables under plastic. The protected vegetables can be either covered vegetables or greenhouses. Open field vegetables cover about 1900 ha. Protected agriculture under plastic occupies about 200 ha including about 450 Greenhouses (a total area of about 45 ha). Greenhouses usually have more than 6 times returns than open field vegetables as the productivity under greenhouses is much more than that for open field crops. However, greenhouses require more investments. For irrigated trees, the most common irrigated trees in the Catchment are citrus trees which cover about 300 ha. However, due to the high prices of water and the salinity of water especially in the lower areas, farmers are uprooting citrus trees to replace them by vegetables, grapes or palm dates. Table 5-4 presents a summary of agricultural land used for different crop types in the Catchment (MoA, 2005a). Actual data for each crop type were collected from the Ministry of agriculture for the 2003 growing season, this data were used as land constraints when building up and verifying the AGSM for the study area as will be discussed in details in Section 5.4.2.

Table 5-4 Total land area used by each crop type in the Faria Catchment (MoA, 2005a).

Activity	Agricultural Area, ha			
	Catchment	Upper	Middle	Lower
1. Irrigated vegetables	2112.80	380.60	375.50	1356.70
1.1 Open field vegetables	1893.10	277.10	319.50	1296.50
1.2 Covered vegetables	174.20	100.00	53.00	21.20
1.3 Greenhouses	45.50	3.50	3.00	39.00
2. Irrigated trees	392.10	111.10	113.90	167.10
3. Irrigated field crops	852.10	197.00	118.50	536.60
Total Irrigated	3357.00	688.70	607.90	2060.40
4. Rainfed vegetables	50.00	22.00	28.00	-
5. Rainfed trees	1123.70	1123.70	-	-
6. Rainfed field crops	521.00	86.50	405.00	29.50
Total Rainfed	1694.70	1232.20	433.00	29.50
Total cultivated	5051.70	1920.90	1040.90	2089.90

Water related contribution (WRC)

As explained in Section 5.3.4, detailed analysis of the WRC data for each crop had been conducted based on data available from Ministry of Agriculture, published reports, and field visits. Table 5-5 shows an example of the cost and profit data collected for all crops in the study area in order to calculate the WRC. Data in Table 5-5 is for onion crop as an example. According to MoA (2005a), the total gross income will be US$ 8340/ha. Income from cultivating Onions is 278 US$/1000kg and the yield is about 30000 kg/ha (MoA, 2005a). To obtain the WRC the input cost was calculated as follows:

Input cost = Total variable cost- cost of water = 5820 US$/ha - 1550 US$/ha = 4270 US$/ha

The water related contribution (WRC) is calculated as follows:

WRC (US$/ha) = Yield Price (US$/kg) * Yield (kg/ha) – Input Costs (US$/ha)

= (0.278 US$/kg *30000 kg/ha)- 4270 US$/ha

= 4070 US$/ha

The water related contribution per unit land area for each crop was calculated as described above and the data was entered to the AGSM. Table 5-6 shows a list of different crops and their WRC as well as irrigation water requirements for each crop in the study area.

Table 5-5 Input-Output data, gross margin and profit per ha for onions (MoA, 2005a)

Item	Total US$/ha
Product [0.278 (US$/kg)*30000 (kg/ha)]	8340
Total gross income	8340
Seedling/Seed	379
Water Requirements	1550
Mulch	0
Total Fertilizers	870
Total-Chemicals	812
Total-Hired Machinery	373
Total-d LabourHire	1836
Total Variable Cost	5820
Gross Margin (Gross income-variable cost)	2520
Fixed Costs	
-Depreciation	209
-Interest on capital	116
-Land rent	475
Total Fixed Costs	800
Total Costs (variable cost+fixed cost)	6620
Profit	1720

Data from Ministry of Agriculture (MoA, 2005a) show that the traditional irrigated cropping systems include vegetables and trees. Vegetables in the area are grown in two systems; one of them is open field irrigated vegetables and the other one is greenhouses irrigated vegetables. Open field irrigated vegetables in the area represent more than 75% of the total irrigated vegetables in the area. These vegetables are distributed in a large scale in the upper, central and lower parts. Irrigated vegetables in greenhouses are found mainly in the lower part areas and few others scattered in the rest of the study area. Cost of production for this type of farming is much more than that for open irrigated farming because of the high initial cost of the greenhouses. The running cost for this type varies according to the inputs and the quantities as well as the maintenance of the greenhouse itself. The most common crops in this type are: tomatoes, cucumbers and beans. Production under this type of farming produces more than 8 times that of the open irrigated farming for many crops, and the labor force costs are twice as much as those required for open field vegetables. Irrigated trees include citrus, grapes and dates. Citrus orchards are distributed in the upper parts as well as in the central parts. Many farmers uprooted their citrus orchards and planted their lands with vegetables to achieve more economic returns. A move towards grape trees has been successful in the middle parts where grapes are being planted in open areas as well as in greenhouses. In the lower parts, date palm and grapes are considered one of the most feasible trees in the area as they have high returns as well as high tolerance to salinity and drought.

The economic impacts of irrigated cropping patterns are evaluated through estimating costs and benefits from each type of irrigated cropping pattern and its role in employing the labor force. According to MoA (2005a), the net profits from irrigated open field vegetables range from 1700 to 3300 US$/ha with a water return ranging from 0.30 to 1.0 US$/m3. Water return for covered vegetables ranges from 0.50 to 2.0 US$/m3, while a cubic meter of water utilized for greenhouses production offers a return of 1.8 to 2.6 US$/m3. However, this profit is highly dependable on market prices. Family profits significantly increase (50% or more) when family members provide labor. Many families in the area work in agriculture on the basis of sharecropping through providing labor required for agriculture. Labor required for irrigated vegetables depends on the type of crop planted. Cucumbers require a lot of labor. It is a common practice that labor cost to pick up cucumbers is considered about 30% of the value of the crop. Labor required for tomatoes, potatoes and onions is estimated at about 17% of the value of the crop. Water return for irrigated trees ranges from 0.30 to 1.6 US$/m3. The highest inputs in the running costs are water costs and labor force. Labor force for these crops represents 30% of the total inputs of the production in the area. Profit from irrigated trees is highly dependable on market prices especially for citrus as prices of oranges vary a lot from one year to another. Family profits increase significantly when family members provide labor. However, variable prices of agricultural products will highly impact family incomes. Irrigated field crops water return ranges from 0.20 to 1.0 US$/m3 (MoA, 2005a).

5.4.2 Results and Discussion

Equation 5-1 gives the optimization function that maximizes the annual net income from agriculture in an area. The input data used for the model building process was given in Section 5.4.1. The WRC (Table 5.6), the total available amount of each water type (Table 5.1), price of each type of water per unit volume (Table 5.1) and the amount of water required per unit area per each activity (Table 5.6) are the input data for Equation 5-1. As given by Equations 5-2 and 5-3 total available amount of each type of water and the total land area available for each crop type are limited and given as a constraint.

The AGSM will optimize the Equation 5-1 by changing the land area of each activity (onions, lemon, tomatoes etc) depending on the available amounts of water and their prices. The resulting total land areas under each crop type and total water used for each water type will be compared with the actual values given in Table 5-1 and Table 5-4. As mentioned earlier during the model building process only two types of water were considered namely freshwater and surface water. Treated wastewater and brackish water are not considered separately at this stage but rather as part of the spring system in case of wastewater or as part of the groundwater in case of brackish water. This is due to lack of wastewater treatment facilities and the mixed utilization of brackish water within the groundwater abstractions.

The model was run in two stages. The first stage was the verification of the developed AGSM. This is aimed at examining the ability of the model to reflect the existing conditions. The verification stage was done by comparing the actual data of the year 2003 with the model results per unit area and unit water as well as for total water and land use. The second stage was to simulate a series of systematic runs aimed at analyzing the trends of the response of agricultural production to a wide range of water prices. The following subsections give a detailed description of the model verification and systematic runs.

Table 5-6 A list of different crops and their WRC as well as average water requirements for each crop in the study area.

Crop	Irrigation Requirements (mm/ha)	WRC(US $/ha)	Total land available (ha)
1. Irrigated vegetables			
1.1 Open field vegetables			
Tomatoes	6880	4430	209.8
Cauliflower	3150	3910	35.9
Cucumber	6370	3900	478.8
Squash	3080	3050	407.4
Snake cucumber	4370	3670	16.6
Jewsmellow	3220	3980	7.8
Beans	2890	3020	138
Maize	6870	3190	393.5
Okra	5630	2820	4.8
Eggplant	6900	4060	183.9
Paprica	4760	4100	16.6
1.2 Covered vegetables			
Eggplant(cov)	8470	12980	3.00
Cucumber(cov)	8980	12680	140.00
Beans(cov)	5280	11960	3
Squash(cov)	4690	10300	3
Strawberry(cov)	12210	9030	3
Tomatoes(cov)	11620	11350	19.2
Okra(cov)	4210	9040	3
1.3 Greenhouses			
Cucumber(gh)	8980	24880	20
Tomatoes(gh)	11980	27160	10
Beans(gh)	5990	15180	13.4
Paprica(gh)	7530	14640	0.7
Jewsmallow(gh)	3000	10650	1.4
2. Irrigated trees			
Lemon	11580	11930	67.7
Clement	11580	6640	83.0
Oranges	11580	6870	159.7
Dates	21420	22560	18.5
Grapes	12220	13270	9.1
Figs	12020	5990	2.0
Banana	24380	14370	4.0
Olives	1600	2820	48.1
3. Irrigated field crops			
Onions	6130	4070	100.0
Broad bean	2890	3830	100.1
Sern	1500	830	10.0
Potatoes	3900	3960	200.0
Barley	3000	1620	22.0
Wheat	4000	2100	350.0
Clover	16910	6700	70.0

Model verification

To verify the developed model actual data for the year 2003 were used to check the consistency of the results drawn from AGSM compared with the actual observation in the agricultural sector. Actual amounts of water available for each season (Table 5-1) were used as the values of the water constraints. The actual total cultivated land areas for different crop categories in Table 5-4 were used as land constraint. The actual total land areas of perennial crops (trees, greenhouse) were used as constraints because in the short-term these land areas are fixed. The land areas for annual crops such as field crops and vegetables have more flexibility in the short term. Therefore, a difference of up to 20% between the actual data and the constraint were permitted. The price per cubic meter of fresh water was 0.2 US$ (WESI, 2005) whereas, surface water was distributed free of charge (Fresh water refers to groundwater abstractions for irrigation purposes; Surface water refers to spring water, storm water runoff, and untreated wastewater flowing through the Faria stream). Table 5-7 presents the outputs of the verification process for the optimal land areas, water use and mix of activities as compared with the corresponding actual values. The results in Table 5-7 showed a good agreement with the actual mix of activities and the use of land and water.

Table 5-7 Actual and model calculated land areas and water requirements for Faria

Activity	Unit	Actual data	Model results	Difference %
Total irrigated Upper	ha	688.7	710.7	+3.2
Total irrigated Middle	ha	607.9	621.7	+2.3
Total irrigated Lower	ha	2060.4	2024.6	-1.7
Total irrigated area	ha	3357.0	3357.0	0
Total rainfed area	ha	1695.1	1695.1	0
Total area	ha	5052.1	5052.1	0
Total irrigated trees	ha	392.1	392.1	0
Total irrigated vegetables (including greenhouses)	ha	2112.8	2112.8	0
Total irrigated field crops	ha	852.1	852.1	0
Total irrigated greenhouses	ha	45.5	45.5	0
Total fresh water	ML	6225	6213	-0.2
Total surface water	ML	16628	13576	-18.4
Total water	ML	22853	19789	-13.4
Total net income	US$	17114000	17389000	+1.6

The total irrigated area in the Faria catchment is about 3357 ha, of which 688.7 ha is in the upper areas, 607.9 ha is in the middle and 2060.4 ha is in the lower area. Results of the model indicated that the optimum allocation of areas showed differences of 3.2%, 2.3% and 1.7% in the cultivated areas in the upper, middle and lower parts of the catchment. This is not a large divergence from the actual situation.

The model estimated optimal total water use was 13.4% less than actual supply values (Table 5-7). This difference could be due to the amount of surface runoff, totalling about 2600 ML, which are available in winter where the demand of irrigation water is very low compared to other seasons. The lack of a storage dam or reservoir causes such an amount to be lost and not utilized in the spring, summer and autumn seasons when the demand of water is very high. According to Table 5-8 the shadow prices of surface water for the catchment were 0 US$/m^3, 0.33 US$/m^3, 0.21 US$/m^3, and 0.29 US$/m^3 for winter, spring, summer and autumn respectively (i.e. as an example when the surface water supply increases by one cubic meter in spring the net income will increase by 0.33 US$). Higher shadow price of water is noticed in spring and autumn compared to summer. This could be due to the fact that farmers in the lower area (which is the largest consumer of water) plant during winter months and they avoid planting in the summer.

Table 5-8 Model calculated versus actual water supply (water constraints) and shadow prices

rWate	Season	Model Calculated	Water supply	Shadow prices
Fresh	Winter	262	275	0
	Spring	1630	1630	0.11
	Summer	1820	1820	0.03
	Autumn	2500	2500	0.05
Surface	Winter	2870	5923	0
	Spring	4417	4417	0.33
	Summer	3549	3549	0.21
	Autumn	2739	2739	0.29

Higher shadow price of water indicates more water scarcity which encourages farmers to save water by adopting modern irrigation technology and at the same time helps the decision maker to evaluate the price of water that the consumer is willing to pay for an additional unit of water.

Model results showed that an amount of 13573 ML and 6213 ML of surface water and fresh water respectively were necessary to maximize the net income. This accounts to 66.1% of the total irrigated area consuming surface water and 33.9% using fresh water. Model results also showed that the total amounts of water used in the optimal solution for the upper, lower and middle areas were 4252 ML, 4401 ML and 11132 ML (or 21.5%, 22.2%, 56.3%) respectively as shown in Table 5-9. The existing water rights system does not clearly specify water rights between the different areas of the catchment. The existing conditions show that farmers in upper parts use water to satisfy their irrigation demands. Extra water, which is not used by those farmers flows from upper springs to the middle parts where lands located below the elevation of the main two irrigation ditches are registered as irrigated lands (WESI, 2005). These lands can get water free of charge from the irrigation ditches (spring water). Other lands can get water from irrigation wells at the expense of the farmers. Only extra not needed water is allowed to leave to lands in lower localities (WESI, 2005).

Total net income from the optimum irrigated cropping pattern was 15 million US$, of which 21.8%, 21% and 57.2% is generated from irrigated areas in the upper, middle and lower parts respectively as shown in Table 5-9. It is noticed that the net income is directly related to the total amount of water used in each area. As shown in Table 5-9 out of the total income from cultivated areas in the catchment, irrigated agriculture constituted about 87.3% (15.2 M US$/17.4 M US$) while 12.7% originated from rainfed activities mostly in the upper areas and to a less extent in the middle areas. This is due to the fact that rainfed agriculture is mainly in the upper areas as it is not feasible in the lower areas because of the small amounts of rainfall.

Table 5-9 Model calculated water requirements, land areas and income for different areas of Faria

Model Results	Upper	Middle	Lower	Total
Total water use(ML)	4252	4401	11132	19786
Percentage of total water use	21.5	22.2	56.3	100
Total irrigated area (ha)	710.7	621.7	2024.6	3357.0
Percentage of total irrigated area	21.2	18.5	60.3	100
Total income irrigated (M US$)	3.314	3.186	8.674	15.2
Percentage of total income from irrigated	21.8	21	57.2	100
Total income of rainfed (M US$)	1.9	0.28	0.02	2.2
Percentage of total income from rainfed	86	13	1	100
Total income (M US$)	5.2	3.5	8.7	17.4
Percentage of total income	29.9	20.1	50	100

At this stage AGSM is ready to be used as a tool to optimize the relationship between the available natural resources necessisated by the management alternatives and the corresponding cropping patterns as will be discussed in Chapter 6.

After successful model building and verification process, further investigations were conducted to evaluate the response of agricultural production to increasing water prices as detailed in Section 5.5. Furthermore AGSM generates water demand curves that evaluate the response of water quantity to a wide range of possible water prices. Section 5.6 elucidates the generated water demand curves for the study area as well as an investigation of the price elasticity. Such an investigation provides the decision makers with systematic information on the optimal water price to be charged to the farmers as well as an outlook on the possible effects associated with different water for irrigation pricing policies.

5.5 SIMULATION OF AGRICULTURAL PRODUCTION

AGSM was run systematically to evaluate the response of agricultural production to a wide range of water prices. This will give the demand for water and net income from agricultural products with varying prices of water per unit volume. The prices were increased gradually from 0.05 US$/m^3 up to the price where all irrigated activities left the optimal solution (i.e no irrigated activities, no water used and zero net income). In these runs the same water price was used for all water quality types and for all seasons in order to investigate the relation between water demand and water price. The social impact of water pricing policy on farmers is an issue that should be also considered when deciding on the price of water fpr irrigation. This will be further discussed in the next section on water demand curves.

Table 5-10 presents the ouput results of the systematic increase of the water prices for Faria catchment. Table 5-10 presents the total irrigated area, total amount of water as well as the breakdown of fresh water and surface water used under varying water prices. A description of each column in Table 5-10 is summarized below for the third row as an example.

Price of water (Column 1) = 0.10 US$/m^3
Total irrigated area (Column 2) = 3472.6 ha
Total water demand (Column 3) = 18513 ML

Fresh water demand (Column 4) = 6225 ML

% of fresh water used (Column 5) = 6225/18513 = 33.6 %

Surface water demand (Column 6) = 12288 ML

% of surface water used (Column 7) = 12288/18513 = 66.4 %

Total cost of water (Column 8) = (Column 1) * (Column 3) = 1.85 Million US$

The net income (Equation 5-1) (Column 9) = 15.26 Million US$

As explained earlier WRC is defined as gross income minus all direct expenses except for direct payments for water or in other words it is the sum of the net benefit and the total cost of water. Accordingly WRC for the catchment is calculated as follows.

WRC (Column 10) = Cost of water (Column 8) + Net income (Column 9) = 17.11 M US$

The values in Column 11 give the gross margin of water (US$/m^3) which was calculated by dividing the WRC (gross income) by the total water used.

Column 11 = (Column 10) / (Column 3) = 0.92 US$/m^3

The gross margin value indicates the affordability of different crops to pay for water. As noticed in Table 5-10 the gross margin of water showed an increasing trend as the water price increases, and this is due to the fact that only the more profitable crops stay in the optimal solution as water prices increases and less profitable crops cannot afford such a high water price.

The last column shows the activities that leave the optimal basis at different prices as water price increases. The last column indicates that maize crop cannot pay the price of 0.10 US$/m3 and leaves the optimal basis when prices rise to 0.10 US$/m3.

Table 5-10 Optimal selected values for different average water prices-Faria Catchment

Water price (US\$/m^3)	Irrigated area (ha)	Total water use (ML)	Fresh water (ML)	Fresh water (%)	Surface water (ML)	Surface water (%)	Total Cost for Water M US\$	Net income M US\$	WRC M US\$	Gross water margin (US\$/m^3)	Activities leaving the optimal solution
0.05	3489.7	18598	6225	33.5	12373	66.5	0.93	16.217	17.15	0.92	Part of field crops
0.10	3472.6	18513	6225	33.6	12288	66.4	1.8513	15.264	17.12	0.92	Maize
0.20	3508.6	18418	6225	33.8	12193	66.2	3.683	13.479	17.16	0.93	
0.30	3497.0	18383	6225	33.9	12158	66.1	5.5149	11.62	17.13	0.93	Part of citrus
0.40	3061.0	14516	3806	26.2	10710	73.8	5.806	9.334	15.14	1.04	Part of open vegetables
0.50	2495.6	10928	4463	40.8	6465	59.2	5.464	7.199	12.66	1.16	
0.60	1106.5	5478	3720	67.9	1758	32.1	3.285	4.135	7.42	1.35	Bananas
0.70	783.4	3965	3589	90.5	376	9.5	2.7755	3.145	5.92	1.49	
0.80	474.5	3038	2516	82.8	522	17.2	2.429	2.321	4.75	1.56	
1.00	230.5	1781	1781	100	0	0	1.781	1.383	3.16	1.78	Field crops,Open vegetables,Date,Grape
1.20	119.7	686	686	100	0	0	0.823	0.944	1.77	2.58	Trees, part of covered vegetables
1.40	119.7	686	686	100	0	0	0.9604	0.807	1.77	2.58	
1.60	119.7	686	686	100	0	0	1.0976	0.671	1.77	2.58	
1.80	101.5	594	594	100	0	0	1.069	0.509	1.58	2.66	
2.00	100.8	589	589	100	0	0	1.178	0.609	1.79	3.03	usesPart of greenho
2.20	45.4	331	331	100	0	0	0.7282	0.229	0.96	2.89	
2.40	33.4	215	215	100	0	0	0.516	0.164	0.68	3.16	Covered vegetables
2.60	31.4	197	197	100	0	0	0.5122	0.118	0.63	3.20	
3.00	1.4	4.2	4.2	100	0	0	0.0126	0.002	0.015	3.48	
3.10	0	0	0	0	0	0	0	0	0	0	All irrigated activities

WRC is the water related contribution

5.6 WATER DEMAND CURVES FOR THE STUDY AREA

Water prices in an optimizing model play an important and very similar role to that they play in a system of competitive markets. In competitive markets, prices measure both what buyers are just willing to spend for additional units of the good in question (marginal value) and the cost of producing such additional units (marginal cost). A price higher than marginal cost signals that the value placed by buyers in an additional unit is greater than the cost of production. Similarly, a price less than marginal cost is a signal to cut back on production. Prices serve as guides to efficient (optimal) resource allocation (Fisher et al., 2005). When maximization of net income involves one or more constraints (e.g land ,water) there is a system of prices involved in the solution. These prices, called shadow values are associated with the constraints. Each shadow value shows the rate at which the quantity being maximized (here, net benefits from water) would increase if the associated constraint were relaxed by one unit. In effect, the shadow value is the amount the user should be just willing to pay (in terms of the quantity being maximized) to obtain a unit relaxation of the associated constraint (Fisher et al., 2005). Particularly for agriculture, specification of water demand means specifying a water demand curve that shows water demand as a function of price. Fisher et al. (2005) derived the water demand curve as given in Equation 5-7:

$$Q = AP^{-\eta} \qquad (5\text{-}7)$$

where,

Q is the quantity of water demanded by a given user;
P is the price per cubic meter charged to the user;
A is an empirical constant in the demand curve ($A > 0$);
η is price elasticity.

Nicholson (1992) derived the price elasticity of demand for water as the percentage change in demand for water with respect to a one percent change in the price of the water supply.

The above equation could be re-written as:

$$\text{Log } Q = \text{Log } A - \eta \text{ Log } P \qquad (5\text{-}8)$$

A graph of Log Q versus Log P will give a straight line with the gradient equal to the constant price elasticity η.

Thus price elasticity measures responsiveness of demand of water to changes in price. Nicholson (1992) presented a linear demand curve as given in Equation 5-9. The above author derived the elasticity at a certain price as given in Equation 5-10 which is a function of both the independent variable (P) and dependent variable (Q). The optimum value of price is obtained by optimizing the Equation 5-9 until the absolute value of the elasticity (Equation 5-10) is equal to 1.

$$Q = a\,P + b \qquad (5\text{-}9)$$

where,
Q is the quantity of water demanded by a given user;
P is the price per cubic meter charged to the user;
a is the slope of the line of P versus Q;
b is the intercept of the line.

$$\eta = a\,P/Q \qquad (5\text{-}10)$$

where,
η is price elasticity for the above linear demand curve at a certain water price.

Price elasticity greater than one is called price elastic and price elasticity less than one is called price inelastic. A given percentage increase in the price of an elastic good will reduce the quantity demanded for the good by a higher percentage than for an inelastic good. In general, a necessary good is less elastic than a luxury good. Developing the demand curve and determining the price elasticity for agricultural catchments helps planners and decision makers in formulating policies and management alternatives that can reflect the farmer's response to these policies. This will help the policy planners and decision makers in their decisions on water pricing and management scenarios.

AGSM generates quantity of water demand at different water prices. Figure 5-1 and Figure 5-2 depict the demand for total water with the increase in price for Faria catchment both as linear (Equation 5-9) and power (Equation 5-7) forms respectively. Figure 5-3 depicts the relationaship of the demand for total water with the increase in price in the Log scale given by Equation 5-8. Following is a detailed discussion to investigate these three approaches (linear, power and logarithmic) in terms of obtaining the elasticity values and the optimum price of water.

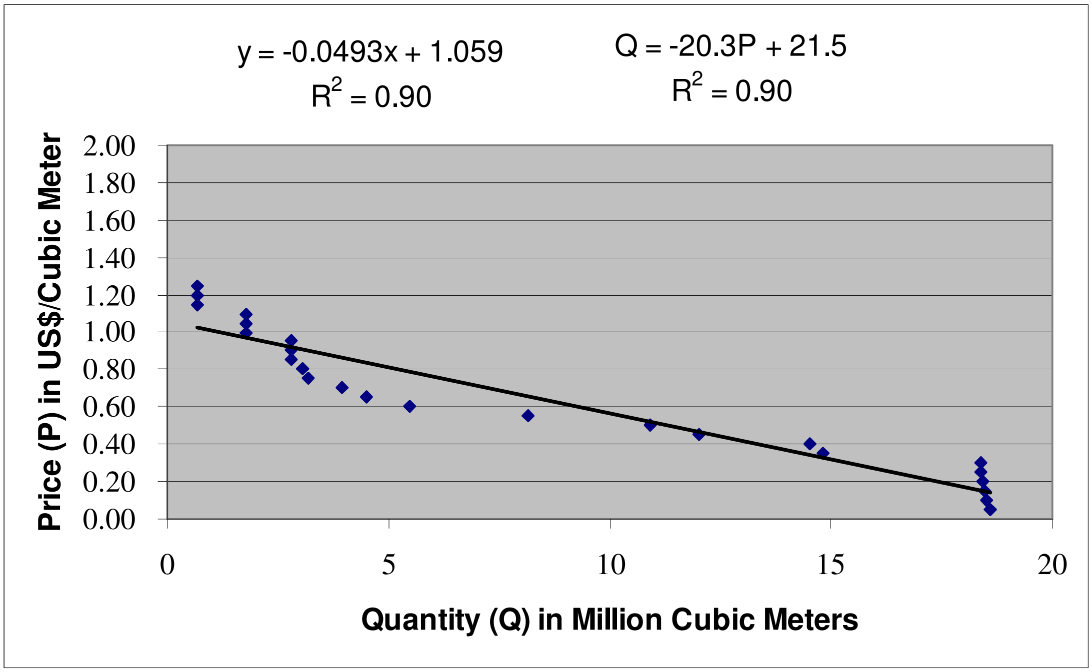

Figure 5-1 **Optimal linear water demand curve for Faria catchment**

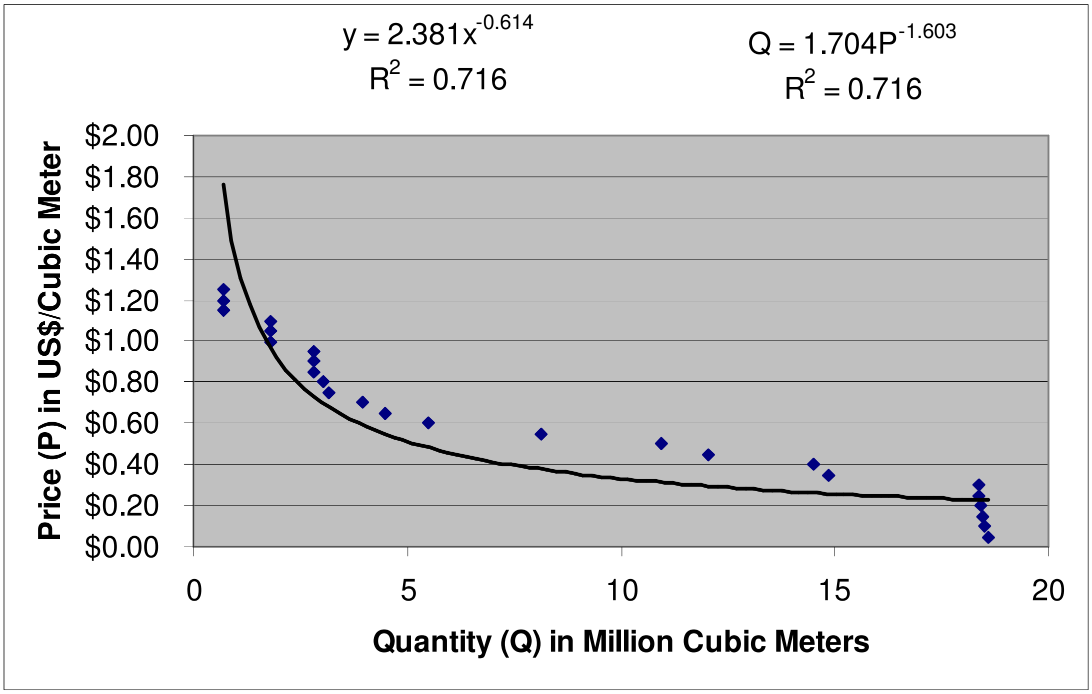

Figure 5-2 **Optimal power water demand curve for Faria catchment**

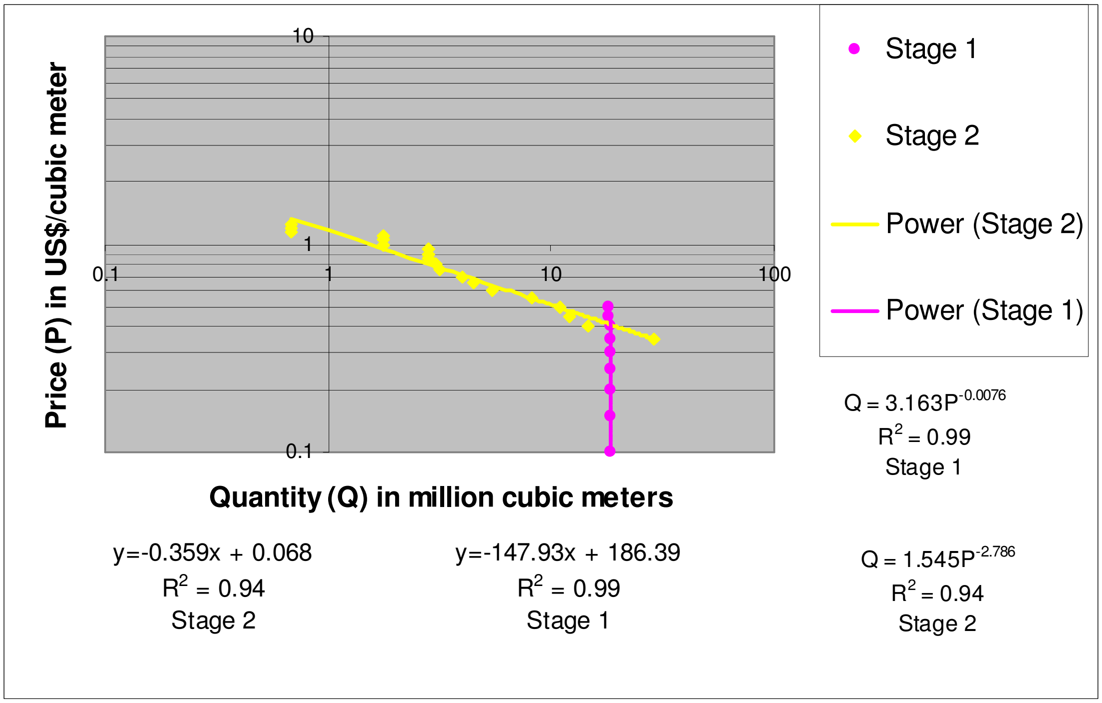

Figure 5-3 Logarithmic optimal water demand curves for Faria catchment

For Faria catchment, the weighted average price of water calculated based on the actual prices of fresh water and surface water is 0.06 US$/m^3 (6225 ML of fresh water at 0.20 US$/ m^3 and 16600 ML surface water at 0 US$/m^3).

Figure 5-1 and Table 5-11 presents the linear regression equation obtained by application of the demand curve derived by Nicholson (1992) for data from Faria catchment. The elasticity of total demand for water for the linear demand curve given in Equation 5-9 is shown at the weighted average water price of 0.06US$/m^3 and at the price of 0.6 US$. Results showed low elasticity value of 0.068 for Faria Catchment when evaluated at the average low price of 0.06US$/m^3. At the price of 0.60 US$/m^3 the elasticity is high and reached 1.31 for Faria Catchment. This means that, increasing the price of water from 0.60 US$/m^3 to 0.72 US$/m^3 (20%) will decrease the quantity demanded by 26.2%. Accordingly, the demand is price elastic, indicating that the quantity of water that will be demanded is highly sensitive to water price. These results agree with previous results found by Amir and Fisher (1999 and 2000) and Salman et al. (2001) using linear demand curves approach and solving of linear equations to find the elasticity values of water demand for catchments in Israel and Jordan. Amir and Fisher (1999 and 2000) reported elasticities of total demand for water at an average price of 0.20 US$/m^3 in the range of 0.186 to 0.488 for different Israeli catchments. This is comparable with an elasticity value of 0.23 for Faria catchment as obtained from Figure 5-1 and Table 5-11. In contrast Salman et al. (2001) obtained a value of 0.12 for elasticity of total demand for water at an average price of 0.20 US$/m^3 for the Jordan Valley in Jordan.

The demand equation in Table 5-11 was solved graphically to find out the value of elasticity (Equation 5-10) at different water prices in order to determine the price level at which the absolute value of the elasticity will be equal to one. Figure 5-4 depicts the elasticity values in logarithmic scale at different values of water price. The water prices levels at which the price elasticity of water is unitary elastic was 0.53 US$/m^3 for Faria Catchment. Above this price, water demand is price elastic and at lower prices, water demand is price inelastic. These water prices serve the decision makers in determining the price of water to be charged to farmers. Such prices are needed in the decision making process to evaluate different management alternatives for optimal management of land and water resources.

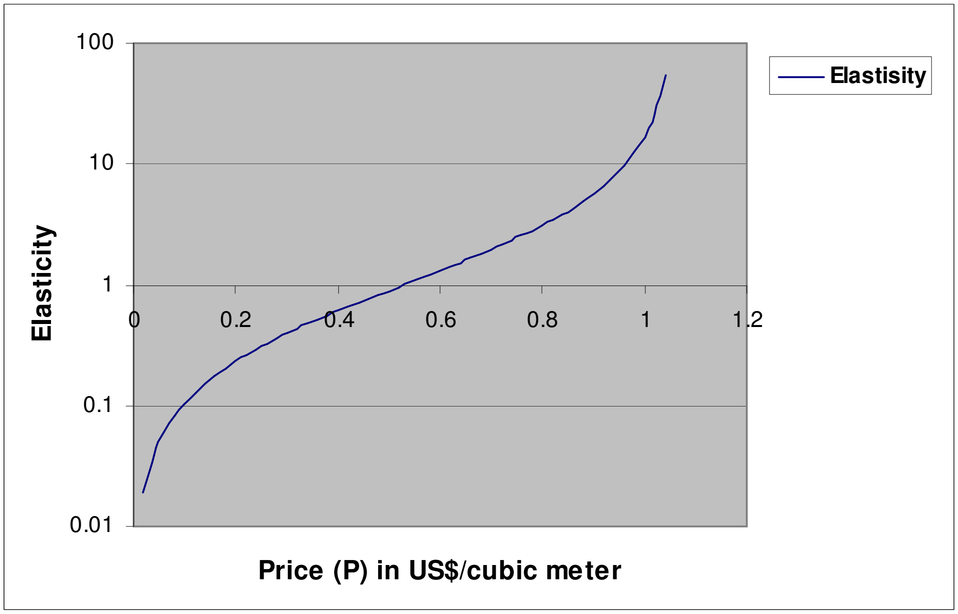

Figure 5-4 Elasticity values at different water prices in Faria catchment

According to Equation 5-7 and Figure 5-2 the demand curve gives a fixed price elasticity of 1.6028 with a regression coefficient of 0.71. However, from Figure 5-3 it is clear that there are two distinguished curves which give two price elasticity values. Initially increasing the water price by 1% will decrease the demand for water only by 0.008%. However when the price of water is increased beyond 0.44 US$/m^3 the price elasticity increases from 0.008% to

Table 5-11 Linear demand functions and price elasticities at different water prices for Faria catchment

Equation	Catchment	Demand Function[1]	R^2	Price Elasticity at 0.06US$/$m^3$	Price Elasticity at 0.6US$/$m^3$
1	Faria	Q = -20.3 P + 21.5 (-14.52)*	90.2	-0.068	-1.31

[1] Where Q denotes water quantity demanded (Million Cubic Meters), P denotes price of water ($/$m^3$).

***Statistical Test (t-Stat) Significant at 1% level**

2.8% indicating that price is very elastic. That is a 1% increase in price of water decrease the demand by 2.8% indicating that part of the crops in the cropping pattern are not capable to pay for the water. These crops are leaving the optimal solution leading to a drop in the water demand. The point where these two curves meet is the optimal price that decision makers have to consider when formulating their management alternatives. The % difference between the two optimal prices (0.53 and 0.44) obtained from solving the linear demand equation and the logarithmic approach respectively is 17%. The advantage of the logarithmic approach method is that it is easier to visualize and hence the planner will be very clearly able to determine the actual value of elasticities that represent the low and high range of water prices as well as critical point where the optimal water price at elasticity equal one with no need to solving or optimizing the equations. The first method will give the actual price elasticity at each price. The main benefit of this exercise is to get a value of the water price that reflects the economic value of water use. Currently the price of water that is being charged to farmers is 0.2 US$/m^3 for groundwater and 0 US$/m^3 for surface water. When developing the management plans for a catchment it will be very useful for decision makers to have an estimate of the optimal water price that could be charged to farmers. However, the social impacts of water prices should be also considered when deciding on the price of water to be charged to farmers which will also affect the water usage. Hijawi (2003) designed and applied three family optimization models to investigate the socio-economic impact of water scarcity including water pricing policy on Palestinian agriculture in the West Bank. Using data based on a field survey carried out in Faria catchment the above author concluded that doubling the fresh water price up to 0.4 US$/m^3 resulted in a 22.2% decrease in family farm income while introducing treated wastewater to surface water users at a price of 0.15 US$/m^3 resulted in a 15.4 % lower family farm income. The Palestinian agriculture has a specificity of being under the control of the Israeli Occupation, highly dependent on limited and fluctuating local markets, with limited access to external markets and exposed to Israeli products. Adding to these factors agricultural water prices charged to farmers in the neighboring countries, namely Jordan and Israel are much lower than those charged to Palestinian farmers (Fisher et al., 2005). The competition in both local and external markets and the price of water for irrigation that is paid by the Palestinian resulted in higher costs and prevented farmers on the West Bank from competing with Israeli products. These factors should be considered in feasibility analysis of future water infrastructural projects anticipated through international aid and grants. Therefore in this study it is assumed that for management alternatives that involve high investment costs to generate additional sources of irrigation the price of water that would be charged to the farmers is that price currently paid for fresh water which is 0.20 US$ per cubic meter.

The second approach of using the logarithmic curves provides the planners and decision makers with a simple and efficient method to derive the optimal price that could be charged to farmers.

Developing water demand curves for different water qualities

The AGSM was run with different water prices to produce demand curves for surface water and fresh water separately. In these runs the prices of one of the water qualities was held constant while changing the price of the remaining one. Thus, the surface water price were allowed to range from 0.1 US$/m^3 to 1.2 US$/m^3 holding the price of fresh water constant at their actual price of 0.20 US$/m^3. The fresh water prices were allowed to range from 0.1 US$/m^3 to 0.70 US$/m^{3}per m^3 while holding the price of surface water constant at their actual price of 0 US$/m^3. In these runs the water prices for all four seasons were kept constant.

Table 5-12 and Table 5-13 present relationships between water prices for surface and fresh water and optimized irrigated area, water quantities and income produced from Faria catchment. As an example, from Table 5-12 when the price of surface water rises from 0.30 US$/m^3 to 0.40 US$/m^3, there is a 12.2% reduction in the irrigated area (from 3497.0 ha to 3070.8 ha). This is due to the fact that crops such as field crops and citrus leave the optimal solution because they cannot compete with the other crops. The demand for surface water is reduced by 3687 ML (30.3%). Similarly, from Table 5-13 when the price of fresh water rises from 0.30 US$/m^3 to 0.40 US$/m^3 irrigated area reduces by 10.5% (i.e from 3482.2 ha to 3116.7 ha). This is due to the fact that high water demanding such as field crops and citrus leave the optimal solution. The demand for fresh water is reduced by 2787 ML or about 48.6%. The gross margin per cubic meter of water showed an increasing trend as the water prices increases, and this is due to the fact that only the more profitable crops stay in the optimal solution. As water prices increase less profitable crops cannot afford such a high water price.

The demand curves were developed for surface and fresh water for the Faria catchment using linear as well as logarithmic curves shown in Figure 5-5 to Figure 5-8. Table 5-14 shows the elasticities of water demand as derived from each of the presented equations at the actual water price (0 US$/m^3 for surface water and 0.20 US$/m^3 for fresh water) and at the midpoints of the range of water prices studied (0.60 US$/m^3 for surface water and 0.35 US$/m^3 for fresh water). For surface water, results showed that elasticity is equal to zero since water was free, whereas a value of 0.423 was obtained for fresh water at the actual low price of 0.20 US$/m^3.

The linear demand equation was optimized to determine the price level at which the absolute value of the elasticity will be equal to one. The water prices levels at which the price elasticity of water is unitary elastic and were at 0.41 US$/m^3, 0.34 US$/m^3 for surface and fresh water, respectively. Above this price, water demand is price elastic and at lower prices, water demand is price inelastic.

The logarithmic demand curves resulted in optimum water prices of 0.40 US$/m^3 and 0.32 US$/m^3 for surface water and fresh water respectively. These values match with the previous values obtained from solving the linear water demand functions. Therfore it is recommended to use the logarithmic approach to determine the optimum price of water as well as the representative elasticity values at different ranges of irrigation water prices.

Table 5-12 Responsiveness to incremental increase in surface water price -Faria Catchment

Water price ($/m^3)	Irrigated area (ha)	Total water use (ML)	Fresh water (ML)	Fresh water (%)	Surface water (ML)	Surface water (%)	Water expenses M US$	Net income M US$	Gross water margin ($/m^3)
0.1	3473.6	18486	6210	33.59	12276	66.41	2.47	14.645	0.93
0.2	3508.6	18418	6225	33.80	12193	66.20	3.68	13.478	0.93
0.3	3497.0	18383	6225	33.86	12158	66.14	4.89	12.242	0.93
0.4	3070.8	14696	6225	42.36	8471	57.64	4.63	10.553	1.03
0.5	2560.5	11861	6225	52.48	5636	47.52	4.06	9.047	1.11
0.6	1352.6	7334	6225	84.88	1094	14.92	1.90	6.756	1.18
0.7	1083.8	6542	6225	95.15	317	4.85	1.47	6.26	1.18
0.8	1062.5	6436	6225	96.72	208	3.23	1.41	6.202	1.18
0.9	1046.3	6286	6225	99.03	61	0.97	1.30	6.166	1.19
1	1046.3	6286	6225	99.03	61	0.97	1.31	6.16	1.19
1.2	1040.3	6225	6225	100.00	0	0.00	1.25	6.146	1.19

Table 5-13 Responsiveness to incremental increase in fresh water price -Faria Catchment

Water price ($/m^3)	Irrigated area (ha)	Total water use (ML)	Fresh water (ML)	Fresh water (%)	Surface water (ML)	Surface water (%)	Water expenses M US$	Net income M US$	Gross water margin ($/m^3)
0.1	3538.4	18654	6225	33.37	12429	66.63	2.49	16.608	1.02
0.2	3538.4	18654	6225	33.37	12429	66.63	3.73	15.986	1.06
0.3	3482.2	18255	5729	31.38	12526	68.62	4.90	15.274	1.11
0.4	3116.7	15143	2942	19.43	12201	80.57	5.47	14.273	1.30
0.5	3082.4	14778	2608	17.65	12170	82.35	6.61	13.946	1.39
0.6	2689.4	13029	867	6.65	12.62	93.35	7.47	13.188	1.59
0.7	2440.8	12086	0	0.00	12086	100.00	8.46	12.759	1.76

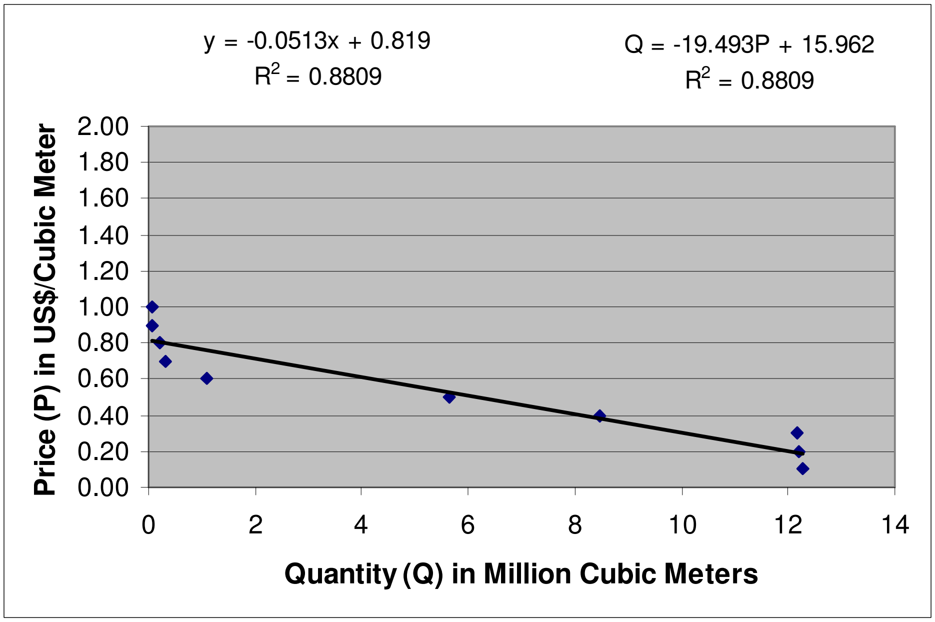

Figure 5-5 Surface water linear demand curve for Faria catchment

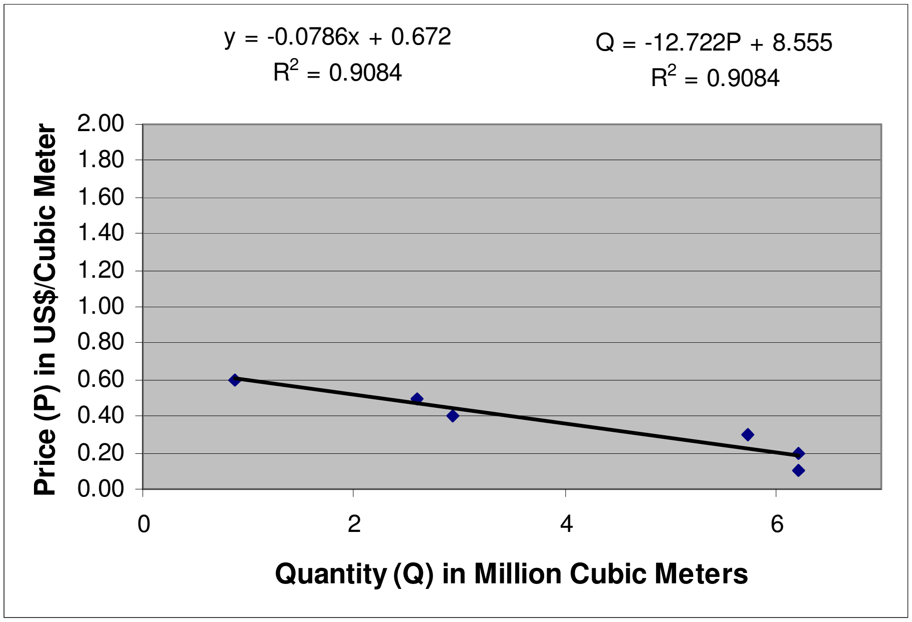

Figure 5-6 Fresh water linear demand curve for Faria catchment

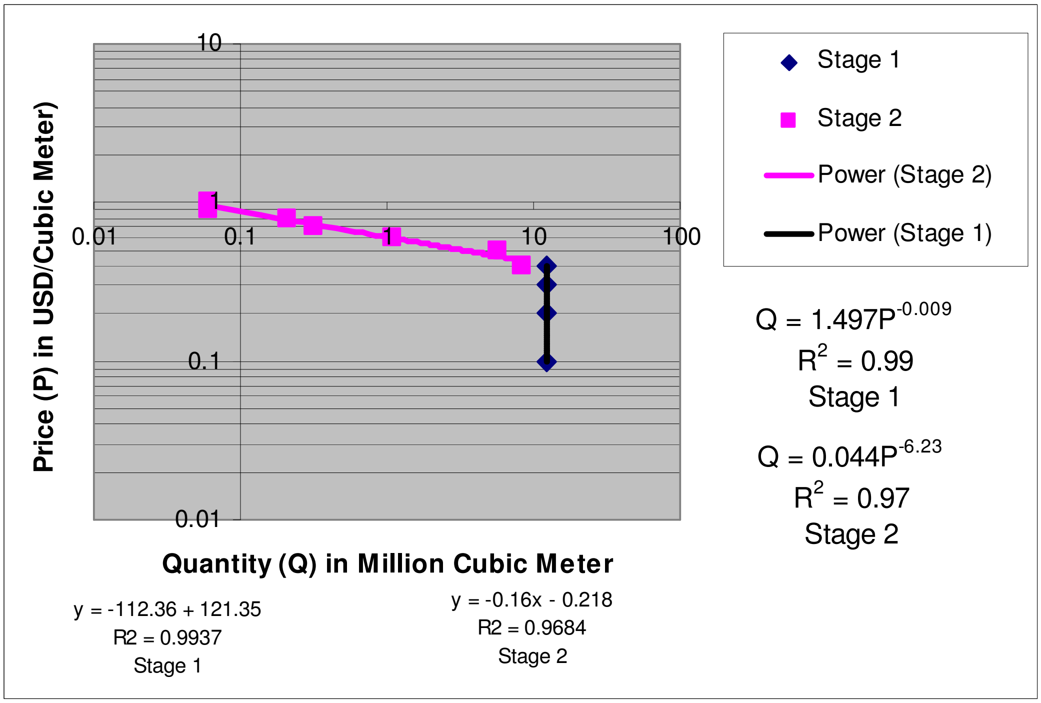

Figure 5-7 Surface water logarithmic demand curves for Faria catchment

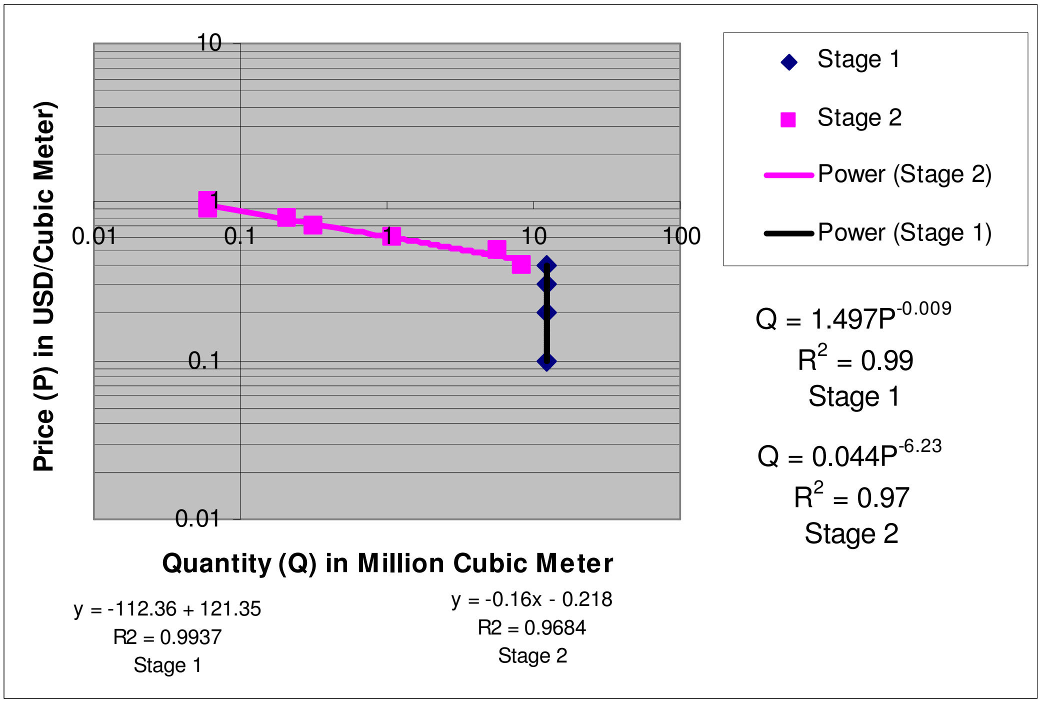

Figure 5-8 Fresh water logarithmic demand curves for Faria catchment

Table 5-14 Linear demand functions and price elasticities for different qualities of irrigation water

Equation	Area & Conditions	Demand Function[1]	R^2	Price Elasticity at Actual Price
1	Faria-Surface water	$Q = -19.493\,P + 15.962$ (-7.693)*	88.09	0 (at 0 US$/$m^3$)
2	Faria- Fresh water	$Q = -12.722\,P + 8.555$ (-6.297)*	90.84	-0.423 (at 0.2 US$/$m^3$)

[1] **Where Q denotes water quantity demanded (Million Cubic Meters), P denotes price of water ($/$m^3$))**

[*] **Statistical Test (t-Stat) Significant at 1% level**

Even if a water trading market with some financial opportunities to farmers, or a policy of increasing the water prices to farmers, may result in reducing water consumption in agriculture. Nevertheless the more important factor is whether the price of water delivered to farmers is so highly subsidized that there is no significant demand response to modest price changes. Furthermore representative estimates of the price elasticity of agricultural water demand are essentially needed to properly address such a major concern, this is an aspect that had not been adequately covered in previous research. The logarithmic approach to finding the optimum water price and the elasticities could be a promising answer to such a problem.

5.7 SUMMARY AND CONCLUSIONS

AGSM is used as a planning model to simulate the behaviour of farmers and optimally determine the cropping pattern that maximizes the farmer economic return under various management policies and scenarios. The AGSM computes the optimal land area for each crop type and total amounts of surface and fresh water used for different agricultural crops to maximize the net income generated by different scenarios of land and water use.

Data from Faria catchment was used to build the AGSM. The model uses the outputs of the rainfall-runoff model as given in Chapter 3 as well as the groundwater model and yield from springs as elucidated in Chapter 4 to determine the total amounts of water available for irrigation. Verification of the AGSM was conducted against actual data from the study area. Results from the AGSM showed a good agreement for the optimal land areas, water uses and mix of activities as compared with the corresponding actual values. The total irrigated area in the Faria catchment is about 3357 ha, of which 688.7 ha is in the upper areas, 607.9 ha is in the middle and 2060.4 ha is in the lower area. Results of the model indicated that the optimum allocation of areas showed differences between actual cultivated and model allocated areas of 3.2%, 2.3% and 1.7% in the cultivated areas in the upper, middle and lower parts of the catchment. The lack of a storage dam or reservoir causes a considerable amount of 2600 ML of winter stormwater runoff to be lost and not utilized in the spring, summer and autumn seasons where the demand of water is very high. This is indicated by the model output of high shadow prices of surface water for spring (0.33), summer (0.21) and autumn (0.29), and zero shadow price for water in winter. This shows the need for a management plan for the catchment that considers such problems.

The results from the AGSM gave a total net income from the optimum irrigated cropping pattern of 15 million US$, of which 21.8%, 21% and 57.2% is generated from irrigated areas in the upper, middle and lower parts respectively. It is noticed that the net income is directly related to the total amount of water used in each area. Results of the AGSM simulations on varying water prices showed that increasing the price of water had a significant effect on the optimal cropping pattern. Low profitable crops could not afford high water prices.

The application of AGSM to data for Faria Catchment suggests that the model can be used as a planning and management model in the proposed decision support framework. The model closely approximates the actual response of farmers to water prices. The optimal allocation of agricultural water permits the efficient use of water, which in turn plays an important role in increasing the total agricultural income in the region. The model can produce insights for agricultural planners who must allocate scarce water resources among agricultural activities by time, space, and different water qualities. It also generates estimates of the effects of different water prices and can be an appropriate and efficient means of controlling agricultural water consumption.

After successful model building and verification process the AGSM was ready to be used to evaluate different management alternatives through optimizing the relationship between the available natural resources and the corresponding cropping patterns as will be discussed in Chapter 6.

Water demands as a function of water price were calculated and curves were developed for Faria catchment. Water demand functions as well as price elasticities for different water qualities were developed. A logarithmic approach was considered to obtain the optimal water price that could be charged to farmers as well as price elasticity values. The logarithmic approach was compared with an optimization approach that resulted in an optimal water price of 0.53 US$/m^3. The logarithmic approach clearly identified two distinguished curves which gave two price elasticity values. The first stage showed a low elasticity value indicating that the price was inelastic and the demand of water will not change with increasing the price of water within this stage. The second stage indicated a high value of price elasticity which indicated an elastic demand that would be affected by increasing the water price. The point where these two curves meet is the optimal price that decision makers have to consider when formulating their management alternatives. However other factors such as the social impacts on the farmers should be considered when deciding on the water pricing policy. The result of this optimal price from the above procedure was 0.44 US$/m^3 which is comparable to the result obtained from solving the linear demand equation.

The logarithmic demand curves resulted in optimum water prices of 0.40 US$/m^3 and 0.32 US$/m^3 for surface water and fresh water respectively. These values match with the values obtained from solving the linear water demand functions. Results from Faria catchment supported the recommendation to use the logarithmic approach to determine the optimum price of water as well as the representative elasticity values at different ranges of irrigation water prices. The logarithmic approach provides the planners and decision makers with a simple and efficient method to derive the optimal price that could be charged to farmers. The advantage of the logarithmic approach method is that it is easier to visualize and hence the planner will be very clearly able to determine the actual value of elasticities that represent the low and high range of water prices as well as critical point where the optimal water price at elasticity equal one with no need to solving or optimizing the equations.

The main benefit of this exercise is to get a value of the water price that reflects the economic value of water use and obtain representative values of price elasticity that will facilitate developing the management plans for a catchment. Having an estimate of the optimal water price that could be charged to farmers and the possible response of farmers to changes in water prices will not only allow a realistic and beneficial water trading, but may also result in optimizing water consumption in agriculture under extremely drought conditions.

CHAPTER 6

OPTIMAL MANAGEMENT OF LAND AND WATER

6.1 INTRODUCTION

The surface runoff (KW-GIUH) model and groundwater (MODFLOW) model input data files developed for Faria catchment in Chapters 3 and 4 were utilized in building the planning model, the AGSM. The preceding chapter reported allocation of land and water resources due to different water prices and the impact on the net income from the catchment. However, in selecting a management alternative, it is important to consider the environmental and socio-economic factors. In this chapter, the optimization approach is developed to determine the optimal management alternative for the study area. This is carried out by defining a set of decision criteria and ranking the decision criteria against each management alternative. The Chapter also explaines management alternatives and the decision criteria selected for the Faria catchment. The development of the Multi Criteria Decision Support System (MCDSS) for evaluation of the best management alternative is presented in this Chapter. The developed MCDSS is applied to Faria catchment data and results are also presented in this Chapter.

The management alternatives and related decision criteria were developed to reflect the conditions predominant in the study area based on local studies and reports, personal communications, simulations of the rainfall-runoff and groundwater models, statistical analysis of springs' yield and the literature-based information. The management alternatives include changing the cropping pattern by introducing high income crops, artificial recharge of groundwater with surface runoff and treated wastewater, surface water and treated wastewater storage (dam), desalination of brackish water, groundwater pumping strategy, utilization of surface water from the Jordan River and combinations between selective management alternatives. Each management alternative will be evaluated in terms of different decision criteria. These management alternatives aimed at sustaining water and land resources and maximizing net income in the Faria catchment. Figure 6-1 depicts the developed framework for the decision analysis process followed in the study for integrated management of water and land resources in the Faria catchment. As can be seen from Figure 6-1, decision analysis depends mainly on the availability of surface water, groundwater and springs' yield to design possible management alternatives through the planning model.

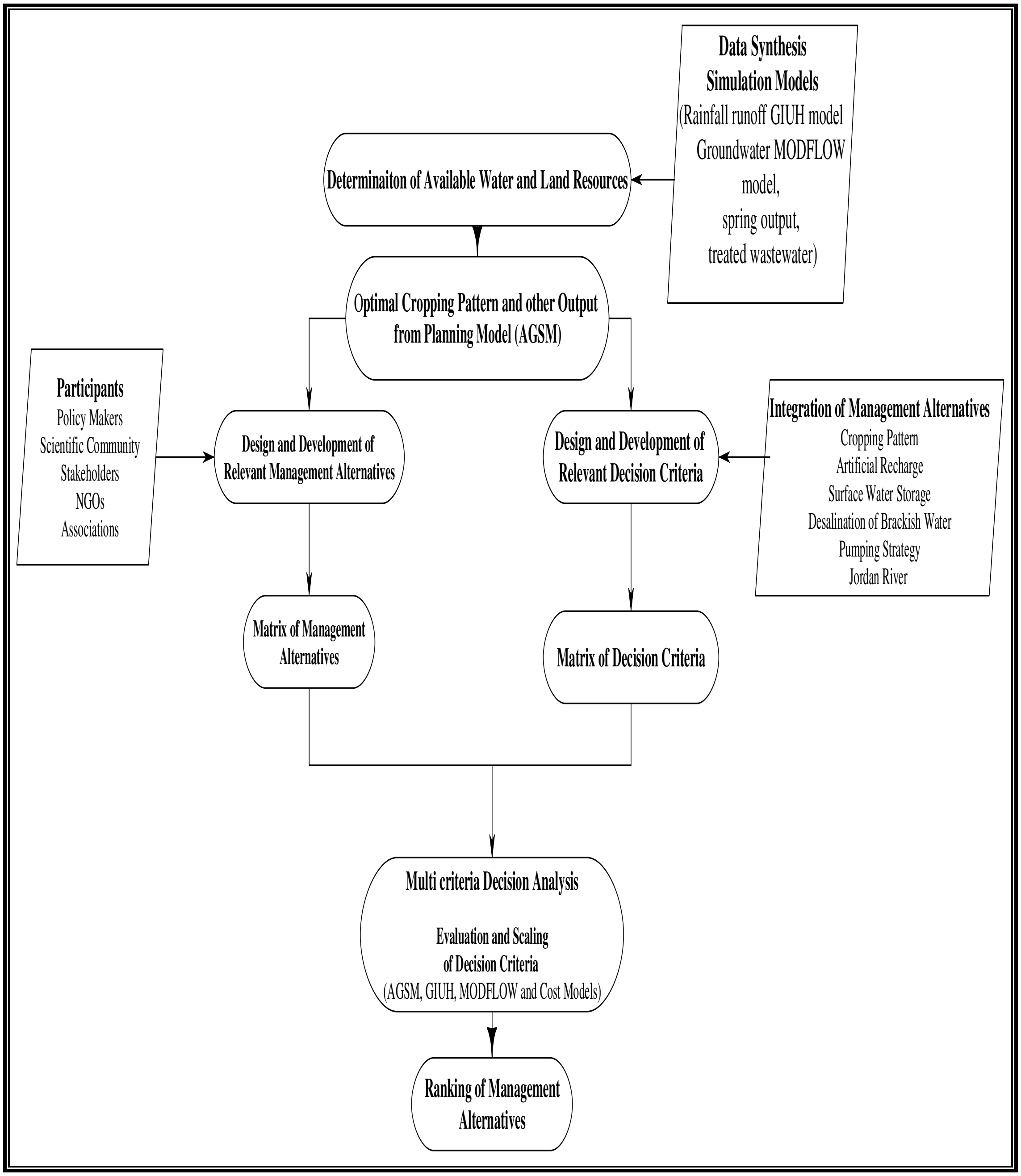

Figure 6-1 Decision analysis framework for the selection of the best management alternatives for integrated land and water management in agriculture-dominated catchments

Decision-making to select the best management alternative requires the identification of the decision objective, which is decisive to the outcome. The direct objectives sustain the available water resources and maximize the use of low quality water as well as the net income for each management alternative. Similarly it is also important to prioritize the management alternatives based on social, economic and environmental factors. The environmental criteria address safe yield and sustainability of the aquifer system. The socio-economic criteria include net income and different cost implications as well as social impacts. Net cost of different alternatives is evaluated using spreadsheets. Planning model (AGSM) can be utilized to evaluate the decision criteria for each proposed management alternative as can be deduced from Figure 6-1. Since the management goals entail conflicting objectives which in turn yield different economic consequences and diverse prioritization schemes based on practicality and applicability of each proposed alternative, a multi-criteria decision analysis is needed to prioritize the proposed management alternatives and to balance between competing economic and environmental goals.

As highlighted in Chapter 2, the Importance Order of Criteria (IOC) method will be used for the multi-criteria decision analysis. The IOC method developed by Yakowitz et al. (1993) is conceptually simple and provides the decision-maker with clear evidence if one management alternative is strongly dominant over another. According to Almasri (2003); and Almasri and Kaluarachchi (2005) the IOC method is easy to program and provide rational results.

Once the management alternatives and their corresponding criteria are defined quantitatively, a multi-criteria decision analysis can be conducted and the most feasible alternative is selected. Yet, it is important to bear in mind that the transpired ranking does reflect the preference of the decision maker in terms of the order of the decision criteria. Each alternative will be evaluated in terms of different decision criteria. These criteria can be classified into five main categories:

(i) the cost of implementing the alternatives;
(ii) the degradation of water resources;
(iii) the governmental policies;
(iv) the social impacts; and
(v) the political uncertainity.

In the following sections these management alternatives and decision criteria will be discussed in detail and assessed for the different alternatives developed for the study area.

The developed framework depicted in Figure 6-1 for the decision making process is used to develop a Multi Criteria Decision Support System (MCDSS) using the C programming language and can be operated easily under the Visual C++ environment. The programming codes used in developing the MCDSS are shown in Appendix III. The user should be able to decide on the management alternatives and the decision criteria appropriate for the area under study. The values of the decision criteria for different management alternatives could be determined from relevant modeling approaches and available studies. The MCDSS compiles the input data systematically as will be explained in this chapter and the final output is a ranking of the management alternatives.

6.2 DEVELOPMENT OF MANAGEMENT ALTERNATIVES FOR OPTIMAL UTILIZATION OF LAND AND WATER RESOURCES

Due to the fact that the study area is located in a triangle where three districts (governorates) share the jurisdiction of the area and the fact that the populated centers in the area are limited, the catchment was underrepresented in regional plans of relevant authorities. The restriction of many developmental activities in the area by the occupation authorities as well as the closure that has been imposed on the area for the past four years led also to the lack of proper planning on the regional and even local levels.

Within the context of the National Water Policy and the draft Water Management Strategy (Palestinian Water Authority, 2002; Ministry of Agriculture, 2005b; PCBS, 2003; Ministry of Environmental Affairs, 2000), the overall objective and guiding vision is the Equitable and Sustainable Management and Development of Palestine's Water Resources. The ultimate goal of the water strategy was defined as "to find the optimal way to manage, protect and conserve the limited water resources". The general objectives of any future management plans as outlined under the National Water Policy include:

1. The utilization of agricultural resources especially land and water should be optimal, efficient, economical and sustainable. The utilization of agricultural resources should assist in environmental conservation and assist in reaching food security.
2. Integrated rural development to be achieved through public participation, integration and coordination with public organizations.
3. Increase and improve the comparative advantage of the agricultural production in local and foreign markets.
4. Reuse of treated wastewater in irrigation.
5. Increase water harvesting and water storage through constructing simple dams, which achieve social and economic themes.

Different alternatives that were considered are given in Table 6-1. These combine land use, governmental policies and water resources protection and development to optimize the use of land and water resources to meet the triple bottom line objectives. Alternatives 2 to 6 were developed to reflect the conditions predominant in the study area based on local studies and reports, personal communications, simulations of the rainfall-runoff and groundwater models, statistical analysis of springs' yield and the literature-based information.

Table 6-1 Summary description of the management alternatives

Alternative Number	Description
Alt. 1	Do nothing (maintain current conditions)
Alt. 2	Changing the cropping pattern by introducing high income crops
Alt. 3	Artificial recharge of groundwater with surface runoff and treated wastewater
Alt. 4	Surface water and treated wastewater storage (dam)
Alt. 5	Desalination of brackish water
Alt. 6	Groundwater pumping strategy
Alt. 7	Utilization of Jordan River
Alt. 8	Changing the cropping pattern by introducing high income crops+ Surface water and treated wastewater storage (dam)
Alt.9	Changing the cropping pattern by introducing high income crops+ Surface water storage (dam)+ Groundwater pumping strategy
Alt. 10	Artificial recharge of surface runoff and treated wastewater+ Desalination of brackish water+ Groundwater pumping strategy
Alt. 11	Utilization of Jordan River+ Changing the cropping pattern by introducing high income crops+ Surface water and treated wastewater storage (dam)
Alt. 12	Utilization of Jordan River+ Changing the cropping pattern by introducing high income crops+ Surface water storage (dam)+ Groundwater pumping strategy
Alt. 13	Utilization of Jordan River+ Artificial recharge of surface runoff and treated wastewater+ Desalination of brackish water+ Groundwater pumping strategy

These management alternatives aimed at sustaining water and land resources and maximizing net income in the Faria catchment. Furthermore combinations between these management alternatives can produce more effective alternatives that can maximize the net income to the catchment and optimize natural resources utilization (Sharifi, 2003). Alternative 8 combines changing the cropping pattern by introducing high income crops (Alternative 2) and surface water and treated wastewater storage (Alternative 4). In addition to combining Alternatives 2 and 4, Alternative 9 looks at cost optimization by adding Alternative 6 (groundwater pumping strategy) to Alternative 8. The Alternative 10 combines three water development alternatives of artificial recharge of surface runoff and treated wastewater (Alternative 3), desalination of brackish water (Alternative 5) and groundwater pumping strategy (Alternative 6). Each Alternative 8, 9 and 10 were combined with Alternative 7 (utilization of Jordan River) to form Alternatives 11, 12 and 13 respectively. Results from each alternative were benchmarked against Alternative 1 (do nothing) to obtain the efficiency level of the other management alternatives.

Table 6-2 summarizes the effectiveness of the alternatives in meeting the direct objectives of sustaining available water resources including the groundwater and surface water as well as optimized cropping pattern in the catchment.

Table 6-2 Effectiveness of the different alternatives in meeting the water availability constraints as well as optimized cropping pattern

ID	Groundwater sustainability	Surface water utilization	Optimizing cropping pattern
Alt. 1	✗	✗	✗
Alt. 2	✗	✗	✔
Alt. 3	✔	✔	✗
Alt. 4	✗	✔	✗
Alt. 5	✔	✗	✗
Alt. 6	✔	✗	✗
Alt. 7	✗	✔	✔
Alt. 8	✗	✔	✔
Alt. 9	✔	✔	✔
Alt.10	✔	✔	✗
Alt. 11	✗	✔	✔
Alt. 12	✔	✔	✔
Alt. 13	✔	✔	✔

✔ meets the objective ✗ does not meet the objective

The above management alternatives pertaining to optimal utilization of water and land resources in the study area are described as follows.

Alternative 1 - Do Nothing (Reference Alternative)

For the *do nothing* alternative in the study area, the following were assumed.

- no actions were taken to sustain the groundwater safe yield ,
- no change in cropping pattern,
- no additional water sources were searched ,
- no additional land areas to be irrigated, and
- no economic ramifications.

Alternative 2 - Change the cropping pattern by introducing high income crops

Existing cropping patterns in the study area should respond to changing conditions including the ongoing shifts on the supply market and demand, available water quality, price of water and quantity. Many farmers in the study area have uprooted their citrus trees and planted their lands with vegetables to achieve more economic return where the area of citrus trees had dropped from about 400 ha in the year 1995 to less than 150 ha during 2004 (Quteishat, 2004; MoA, 2005a).

Changing the cropping pattern management alternative involves the provision and supply of date palm as proposed by the Ministry of Agriculture for the lower part of the study area where the estimated target reaches up to 4000 ha to be planted with date palm in the Jordan Valley within the next 20 years (Alquds newspaper, 2006). As an immediate plan, the Ministry of Agriculture estimated that twenty thousand palm trees will be needed to plant a further area of 150 ha with an estimated cost of 1.2 million US$ (Qteishat, 2004). This imposed change in the cropping pattern will be reflected in evaluating the cost of this alternative together with the AGSM that will provide the net income and other ramifications resulting from this management alternative.

Alternative 3 - Artificial recharge of groundwater with surface runoff and treated wastewater

Discharges of untreated wastewater into water causes pollute scarce water resources, both groundwater as well as surface water. One of the options to use treated wastewater is to recharge it into the aquifer. Infiltration systems may be very effective in the study area where recharge of stormwater could be used to augment local water resources for irrigation purposes. The stormwater would have to be diverted from the stream and routed to infiltration basins. The alluvial aquifer of the study area in effect become temporary storage systems for excess water in the winter months and the water would be extracted and piped to farmers during high demand seasons (Moe et al., 1998). The Palestinian Water Authority (PWA) in the West Bank and UNESCO-IHE in cooperation with An-Najah National University have agreed to carry out a five-year pilot study on artificial recharge with surface water in the study area (WESI, 2005). Nevertheless, such an alternative implies serious economic and maintenance ramifications that may prohibit the adoption of this alternative. Cost analysis and other design issues of artificial recharge in similar areas of the Jordan Valley is available in published reports (Moe et al., 1998) and will be used to evaluate such an alternative. As detailed in (Moe et al., 1998) the total capital costs for the proposed recharge facilities is 34 million US$ (that is distributed as 5 million US$, 12 million US$, 12 million US$ and 5 million US$ in the second, third, fourth and fifth years respectively) and the annual operation and maintenance costs were estimated at 1 million US$ starting from the sixth year. It is assumed for alternatives that involve high investment costs to generate additional sources of irrigation that the price of water that would be charged to the farmers is that price currently paid for fresh water which is 0.20 US$ per cubic meter. This price is higher than agricultural water prices charged to farmers in the neighboring countries, namely Jordan and Israel (Fisher et al., 2005).

It is assumed 35% of the treated wastewater will be used for recharge of the aquifer (WSSPS, 2000) resulting in a total amount of 1400 ML. The cost of wastewater treatment was not included since planning and funding for treatment facilities had been finalized to protect the environment (WESI, 2005). In addition 2600 ML of stormwater runoff will be available for recharge. Detailed technical studies are still needed to be conducted but for the purposes of this study it has been assumed that this additional total amount of 4000 ML/yr will be made available for reuse from this alternative. Current available water supplies will continue to be available including fresh water (6225 ML/yr), spring water (13000 ML/yr) and wastewater from parts of the upper urban areas (1000 ML/yr).

Alternative 4 - Surface water and treated wastewater storage (dam)

Surface runoff in the catchment is not stored for use in summer and spring as there are no dams in the catchment to store the excess water (AbuSafat, 1990). A management alternative to address this problem is through dams and detention basins that offer the opportunity to store significant quantities of stormwater in winter season to be used later in spring and summer. The size of dams and detention basins makes it more desirable in areas where stormwater resources can be augmented with baseflow from springs which is the case with Faria stream. A further advantage is the capture of wastewater that is also discharged to Faria stream making this alternative more viable. Moe et al. (1998) have proposed a storage dam in Faria catchment. Cost analysis and other design issues of the proposed dam are available in Moe et al. (1998) and Abu Safat (1990). According to the above authors the total capital costs for the proposed dam in Faria catchment is 70 million US$ (that is distributed as 5 million US$, 30 million US$, 25 million US$ and 10 million US$ in the second, third, fourth and fifth years respectively) and the annual operational and maintenance costs were estimated at 2.30 million US$ starting from the sixth year. These values will be used in estimating the associated costs in this alternative. It is assumed that the price of water that would be charged to the farmers is 0.20 US$ per cubic meter.

The storage of winter stormwater together with treated wastewater alternative involves constructing a dam that offer the opportunity to store significant quantities of stormwater in winter season to be used later in high demand seasons. The additional amount of wastewater that will be available within the ten years planning horizon is 4000 ML (WSSPS, 2000). A total of 10000 ML that consists of stormwater runoff (2600 ML/year), wastewater (4000 ML/year) and winter spring water (3400 ML/year) were estimated to be stored behind the dam. As a result an additional amount of 6600 ML/year will be made available from this alternative.

Alternative 5 - Desalination of brackish water

When salinity level is 5 mS/cm the groundwater is classified as brackish water (Ayers and Westcot, 1985). Brackish water had been monitored in some groundwater wells in the lower part of the study area (PWA, 2005). Desalination of brackish water is introduced as a strategy to improve the quality of water to an acceptable standard. For the study area 1950 ML/year of brackish water were estimated to be abstracted as detailed in Chapter 4.

The WSSPS (2000) reported that the total capital costs involved in a desalination facility of 9000 ML in the Jordan Valley to be 13.5 million US$ and the annual operation and maintenance costs were estimated to be 0.58 million US$. Based on the above figures it was assumed that the total capital costs for a desalination facility of 1950 ML is 2.9 million US$ and the annual operational and maintenance costs are 0.1 million US$ (WSSPS, 2000). Furthermore, similar to Alternative 3 for this alternative it is assumed that the price of water that would be charged to the farmers is 0.20 US$ per cubic meter. Since brackish water is currently being pumped as part of the groundwater abstraction this alternative will not add extra amounts of water but the quality of water would be improved.

Alternative 6 -Groundwater pumping strategy

Records for wells in upper Cenomanian, Alluvium and Eocene aquifers in the study area showed 10 to 20 meters of reductions in water table elevations (PWA, 2005) for the last 30 years. Another alternative to address the groundwater quality problem in the study area is through testing new pumping strategy that could help in optimizing the utilization of water resources in the study area. As metioned earlier the groundwater model MODFLOW is capable to process the possible alteration and development of a new pumping management strategy.

Groundwater pumping strategy includes closing the wells that are abstracting water above the safe yield based on 50 % allowable limit of drawdown. As reported in Chapter 5 a total of 3200 ML/year is estimated to be abstracted above the safe yield. No additional cost is associated with this alternative. The benefit from this alternative would protect the aquifer. However this alternative would not be valid without providing an equivalent water to compensate the wells' owners with similar amounts of water which could otherwise cause a negative social impact.

Alternative 7 - Utilization of surface water from Jordan River

Currently the Jordan River is not available to the Palestinians for use. However as a result of "OSLO II" agreement between the Palestinians and Israelies in 1994, it was agreed that Israel recognizes the Palestinian water rights. It was also agreed to postpone the water file to the final status negotiations between the two parties (WSSPS, 2000).

Utilization of surface water from Jordan River alternative would be available once the final peace settlement is reached with Israel. Accordingly this alternative has been based on a future plan of utilizing the Jordan River for the study area as proposed by the Palestinian Water

Authority (WSSPS, 2000). It has been estimated that a total annual quantity of 18000 ML would be available for utilization and the total capital cost for this proposed scheme would be 43 million US$ (that is distributed as 3 million US$, 20 million US$, 15 million US$ and 5 million US$ in the second, third, fourth and fifth years respectively) and the annual operation and maintenance costs were estimated at 1.1 million US$ (WSSPS, 2000) starting from the sixth year.

6.3 DECISION CRITERIA

The management alternatives described in the preceding section will be evaluated for the study area based on decision criteria. Decision criteria for the multi-criteria decision analysis could be related to the following classifications:

- Economic;
- sustainability of water resources;
- governmental policies;
- social; and
- political criteria.

The above decision criteria covers all aspects related to social, economic and environmental principles.

Decision analysis conducted for the purpose of ranking of the management alternatives requires an evaluation of a set of decision criteria. For a management alternative to be dominant, it should be the best in terms of all the decision criteria. Since the decision criteria developed in this work reflect both socioeconomic as well as environmental-related criteria, it is anticipated that no alternative will be dominant in terms of all set of criteria. This ranking of the management alternatives necessitates the use of a multi-criteria decision analysis.

Proposed decision criteria in the multi-criteria decision analysis are summarized in Table 6-3. Each management alternative is appraised for these decision criteria by using the AGSM planning model, MODFLOW groundwater model, KW-GIUH rainfall-runoff model and spreadsheets as depicted in Figure 6-1 and summarized in Table 6-3.

The groundwater (MODFLOW) model computes the total quantities of groundwater that can be abstracted within the safe yield. The rainfall-runoff (KW-GIUH) model gives the amounts of surface runoff that can be utilized for irrigation. As stated earlier in Chapter 5, the planning model (AGSM) integrates the outputs of the rainfall-runoff and groundwater model to determine the total amounts of water available for irrigation. As explained earlier different management alternatives result in different amounts and qualities of available water which in turn are entered to the AGSM. The AGSM computes the net income and the total amounts and qualities of water used to produce the optimal cropping pattern with associated land areas. In addition spreadsheets were developed to compute the net cost incurred from implementing each management alternative. Net cost is the difference between total alternative cost and benefit. The benefit from an alternative includes the value of the total amount of water generated by this alternative. The following subsections provide an explanation on Table 6-3 in regards to the methodology for evaluation of the decision criteria pertinent to the economic cost analysis, social impacts, governmental policy, political impacts and water constraint satisfaction for the different management alternatives proposed for the study area. Values of the decision criteria for each management alternative are given in Sections 6.3.1 to 6.3.5.

Table 6-3 Summary of the decision criteria, the corresponding abbreviations, and the evaluation methodology

No.	Description of decision criteria	Abbreviation	Evaluation
1	Total net benefit ($)	NB	AGSM
2	Net return of water ($/m^3)	NR	AGSM
3	Net costs associated with each alternative ($)	COST	Spreadsheets
4	Total quantities of abstracted groundwater above the safe yield (ML)	SY	AGSM & MODFLOW
5	Total quantities of winter surface water and wastewater available (ML/Year)	SW	AGSM & KW-GIUH
6	Total quantities of brackish water available (ML/Year)	BW	AGSM
7	Total irrigated land (ha/Year)	TIL	AGSM
8	Percent utilization of available water resources (%)	UAWR	AGSM
9	Percent utilization of available land resources (%)	UALR	AGSM
10	Degree of political uncertainty	PU	Literature and personal
11	Degree of social conflict due to historical rights utilization	SCWR	Literature and personal

6.3.1 Evaluation of Economic Criteria

Evaluation of the net benefit

Each management alternative uses a certain amount of available water. Results from the application of the AGSM for each management alternative gives the total net benefit (NB in \$) as well as the net return on water (NR in \$/m^3) which is equal to the net benefit divided by the amount of water used under each alternative. For example results from the AGSM for Alternative 3 shows that the net benefit for this alternative is 19.1 million US\$ (Table 6-4). The total amounts of water utilized under this alternative is 24225 ML including 4000 ML recharge water (as discussed in Section 6.2 Alternative 3) and 20225 ML from Spring water, groundwater and wastewater of parts of the urban areas (as discussed in Chapter 5). This results in a net return of 0.79 US\$/m^3.

Evaluation of the cost criteria

Conducting an economic cost analysis for the different proposed management alternatives is a very important step. Yet, this is the most difficult decision criterion to evaluate since it requires a close examination of the prevailing conditions in the study area. The cost criterion considers the net costs incurred from adopting the management alternative (COST in \$). Net cost is defined as the difference between total cost associated with this alternative to produce additional amounts of water and the benefit gained from the value of that additional water (Net cost = total cost of alternative - value of water generated by this alternative). An optimal value of additional water generated by different alternatives would be the price at which the absolute value of elasticity will be equal to one (0.44 US\$/m^3) that was developed in Chapter 5. However further social investigation should be conducted to avoid any negative impacts on the farmers. Accordingly for the purposes of this study sale price of any additional water resources made available through management alternatives would be 0.20 US\$/m^3.

The basic premise in calculating the cost of an alternative for the entire planning period is to compute the cost incurred from an alternative and then to calculate the present value of this cost as a cost incurred at the end of the planning period. Present value is defined as given in Equations 6-1 and 6-2 (Kleinfeld, 1992). The final present value is the summation of the annual costs for the planning horizon of ten years.

Table 6-4 The decision criteria values for each proposed management alternative (Abbreviations and units are as defined in Table 6-1 and Table 6-3)

Criteria	Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5	Alt. 6	Alt. 7	Alt. 8	Alt. 9	Alt. 10	Alt. 11	Alt. 12	Alt. 13
NB($)	1.52E+07	1.74E+07	1.91E+07	1.94E+07	1.70E+07	1.45E+07	2.88E+07	2.11E+07	2.04E+07	1.77E+07	3.30E+07	3.28E+07	3.27E+07
NR($/m^3)	0.77	0.94	0.79	0.77	0.91	0.96	0.81	0.83	0.88	0.88	0.87	0.89	0.90
COST($)	0.00E+00	1.20E+06	2.74E+07	5.66E+07	3.12E+06	0.00E+00	2.75E+07	5.78E+07	5.78E+07	3.05E+07	8.53E+07	8.53E+07	5.80E+07
SY(ML)	3.213E+03	3.23E+03	3.23E+03	2.36E+03	3.23E+03	0.00E+00	2.42E+03	2.37E+03	0.00E+00	0.00E+00	9.73E+02	0.00E+00	0.00E+00
SW(ML/yr)	0.00E+00	0.00E+00	4.00E+03	6.60E+03	0.00E+00	0.00E+00	0.00E+00	6.60E+03	6.60E+03	4.00E+03	6.60E+03	6.60E+03	4.00E+03
BW(ML/yr)	1.95E+03	1.95E+03	1.95E+03	1.95E+03	0.00E+00	1.95E+03	1.95E+03	1.95E+03	1.95E+03	0.00E+00	1.95E+03	1.95E+03	0.00E+00
TIL(ha/yr)	3.36E+03	3.33E+03	4.32E+03	4.41E+03	3.47E+03	3.03E+03	5.05E+03	4.36E+03	4.15E+03	3.73E+03	5.05E+03	5.05E+03	5.03E+03
UALR(%)	66.40	65.90	85.50	87.30	68.70	59.90	100.00	86.40	82.10	73.90	100.00	100.00	99.60
UAWR(%)	86.60	81.00	100.00	94.50	81.40	77.10	86.70	94.50	97.80	95.00	84.40	88.40	92.70
PU	0.00	0.00	0.75	0.75	0.00	0.00	1.00	0.75	0.75	0.75	1.00	1.00	1.00
SCWR	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00	1.00	1.00	0.00	1.00	1.00

$$PV = C * PWF \tag{6-1}$$

$$PWF = \left[\frac{1}{(1+n)^t} \right] \tag{6-2}$$

where,

- PV is the present value of the cost of a management alternative for a ten year simulation period;
- C is the Net Cost which is equal to the annual cost incurred from adopting a specific management alternative and considers both the costs and the benefits;
- PWF is the present worth factor;
- n is the market interest rate; and
- t is the length of the planning period.

A 10% interest rate is assumed when pricing the costs of management alternatives (CPF, 1997, WSSPS, 2000).

Equations 6-1 and 6-2 were utilized in calculating the present value of alternative costs considering a planning period of 10 years. For Alternative 3, the net cost was estimated to be 27.4 Million US$. Appendix IV presents a sample spreadsheet used to calculate the net cost for Alternative 3. The annual breakdown of the capital costs, the operation and maintinace cost and the year of start of operation were based on Moe et al. (1998) and detailed under the description of the alternative given in Section 6.2. Net cost of water was calculated on present value basis for a discount rate of 10% and for a planning period of 10 years resulting in a total cost of 29.5 Million US$. The water quantity that would be available was also calculated based on the same discount rate resulting in a total amount of 10360 ML with a market value of 2.1 Million US$, which in turn gives a net cost of 27.4 Million US$ (29.5 Million US$ - 2.1 Million US$).

Similarly for Alternative 4, the total cost was 60.9 Million US$. The water quantity that would be available was also calculated based on the same discount rate resulting in a total amount of 21600 ML with a market value of 4.3 Million US$, which in turn gives a net cost of 56.6 Million US$ (60.9 Million US$ - 4.3 Million US$) as shown in Table 6-4.

For Alternative 5, the total cost was 3.12 Million US$. No extra water quantity would be available through this alternative hence the net cost is 3.12 Million US$.

Similarly for Alternative 7 the total cost was 36.8 Million US$. The water quantity that would be available was 46600 ML with a market value of 9.3 Million US$, which in turn gives a net cost of 27.5 Million US$ (36.8 Million US$ - 9.3 Million US$).

Regarding combined management alternatives (Alternatives 8, 9, 10, 11, 12 and 13) the total costs of these combined alternatives were computed by summing up the costs of the corresponding individual alternatives. The values of the cost criteria for each management alternative are given in Section 6.4.

6.3.2 Evaluation of the Sustainable Yield Limit Criteria

The criteria related to the sustainability of water resources include three decision criteria to be calculated. The first is the total quantities of groundwater abstracted above the safe yield (SY in ML/yr). This is estimated by substracting the amounts of groundwater used under each alternative from the safe yield. The amounts of available groundwater and the amount of the safe yield are estimated using MODFLOW model as shown in Chapter 4. It is important to mention that for the management alternatives involving a groundwater pumping strategy (Alternatives 6, 9, 10, 12 and 13) the amounts of groundwater available would be only the safe yield amounts. The AGSM gives the amounts of groundwater that are utilized under each alternative and the SY criteria can be calculated accordingly. For example for Alternative 3 the available groundwater amount is 6225 ML. The results of the AGSM showed that this entire amount has been utilized. Accordingly the SY value is 3225 ML (Table 6-4) which is the difference between the 6225 ML and the safe yield which is 3000 ML/Year.

The second is the total quantities of surface water and wastewater available for irrigation (SW in ML/Year). As detailed in Section 6.2 describing each alternative the values of SW are 0 ML, 0 ML, 4000 ML, 6600 ML, 0 ML, 0 ML, 0 ML, 6600 ML, 6600 ML, 4000 ML, 6600 ML, 6600 ML and 4000 ML for Alternatives 1, 2, 3, 4, 5, 6, 7, 8, 9, 10, 11, 12 and 13 respectively as shown in Table 6-4.

The third decision criterion is the total quantities of brackish water (BW in ML/Year) that is based on the results of the MODFLOW model as detailed earlier in Chapter 4 and the consequences of each alternative is mentioned earlier in Section 6.2. The value of BW is 1950

ML/Year for all the alternatives except those that entail desalination alternatives (which are Alternatives 5, 10, 13) where the BW value is 0 ML/Year.

The values of the above criteria for each management alternative are given in Section 6.4.

6.3.3 Evaluation of the Governmental Policy Criteria

The decision criteria related to governmental policies consider the total irrigated area (TIL in ha/Year), the percent utilization of land (UALR) that relates the irrigated area to the total available land and the percent utilization of water (UAWR) resources that relate the amont of water utilized in relation to the total available water. These will be estimated using the AGSM and compared for each alternative with respect to the available water resources under each alternative and the potential irrigable area in the Faria catchment is 5052 ha (WESI, 2005). Higher percentage values will be ranked as better alternatives under these criteria.

As an example the results of the AGSM for Alternative 3 show that the total irrigated land (TIL) is 4320 ha/yr (Table 6-4) which constitutes 85.5 % of the total available land as the UALR value. The total available water is 24250 ML and the total utilized water is 24250 which gives a 100% value for UAWR. The values of the above criteria for each management alternative are given in Section 6.4.

6.3.4 Evaluation of the Political Uncertainty Criteria

Political related criteria (PU) address the uncertainty associated with implementing the alternative where political negotiation between Israel and Palestine would determine the applicability of such an alternative. An alternative that will not imply such a political effect would be ranked higher than an alternative that requires political negotiation. Grades between zero and one will be allocated to different alternatives based on their vulnerability to political situations and taking into account the national policies and strategies of different decision-making authorities. Accordingly a value of one was given to Alternatives 7, 11, 12 and 13 while a value of 0.75 was given to Alternatives 3, 4, 8 and 10. Zero value was allocated to Alternatives 1, 2, 5, 6 and 9.

6.3.5 Evaluation of the Social Criteria

The social decision criteria (SCWR) consider the historical water rights of the locals who own the water shares for decades. This criterion is very important when examining the groundwater pumping strategy especially in the case of groundwater wells exceeding the safe yield and needs to be shut off. This criterion will be evaluated against each alternative whether it is having this effect or not. An alternative that will not imply such a social effect would be ranked higher than that does imply it. Grades between zero and one will be allocated to different alternatives based on their associated social impact and taking into account the national policies and strategies of different decision-making authorities. As a result a value of one was given to Alternatives 6, 9, 10, 12 and 13 while a value of zero was given to other alternatives.

6.4 EVALUATION OF THE MANAGEMENT ALTERNATIVES

Each management alternative was evaluated in terms of different decision criteria which cover socio-economic and environmental conditions described in Section 6.3. The quantitative values given to each decision criteria (Table 6-3) were calculated for all management alternatives and were reported in Table 6-4. Figure 6-2 depicts the graphical presentation of the decision criteria values for each alternative. As given in Table 6-3 the quantitative values were obtained using KW-GIUH model, MODFLOW model and AGSM. Table 6-5 depicts the ranking of management alternatives based on the value obtained for each decision criteria (Table 6-4). Ranking of 1 indicates the most desirable alternative and Rank 13 is the worst management alternative for a single decision criteria. A best alternative would be the most desirable one (Rank 1) in terms of the decision criteria.

Alternatives 11, 12 and 13 are the most economic alternatives in terms of net benefits (NB). Alternative 6 is obviously the most efficient alternative in sustaining the aquifer and fulfilling the safe yield (SY). As expected Alternatives 9, 10, 12 and 13 are similarly efficient in sustaining the aquifer safe yield since they combine Alternative 6.

In regards to the total quantities of brackish water utilized (BW), Alternative 5 and Alternatives 10 and 13 were less desirable than the other alternatives in utilizing brackish water since they involve brackish water desalination. Therefore Alternatives 5, 10 and 13 equally have the last ranking among other alternatives.

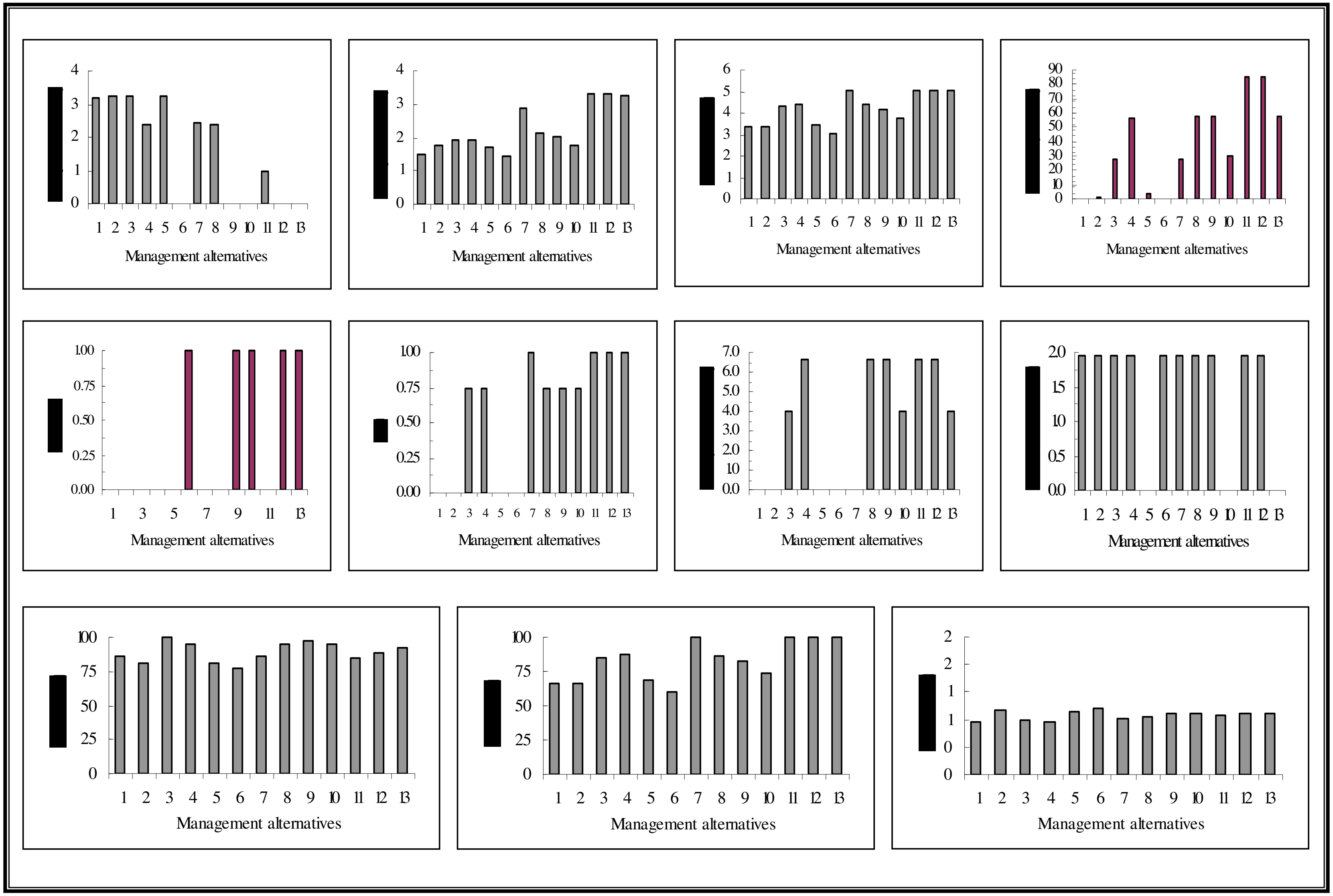

Figure 6-2 A pictorial description of the decision criteria as computed for the proposed management alternatives. Abbreviations are as defined in Table 6-1 and Table 6- 3

The total net benefit (NB in $) as well as the total irrigated area (TIL in ha/Year) are the highest for Alternatives 11, 12, and 13 since each of these three alternatives combine Alternative 7 which itself provides water from the Jordan River allowing for the highest total quantities of water available under these alternatives.

The Alternatives 11 and 12 are combinations of one or more of Alternatives 3, 4 and 7. The cost of Alternatives 3, 4 and 7 constituted the major part of the cost for these combined Alternatives. Alternatives 11 and 12 resulted in the highest cost of 85.3 Million US$. However, the net benefit resulting from these two alternatives were the highest reaching about 33 Million US$ since both of them included the Jordan River Alternative 7, and the storage dam Alternative 4 that resulted in the highest available amounts of water and highest total irrigated lands which reached the maximum irrigable areas (5052 ha).

Table 6-5 Ranking of the management alternatives based on the decision criteria. Alternatives in bold (red color and blue color) signify a possible switch in location.

Rank	NB	NR	COST	SY	SW	BW	TIL	UALR	UAWR	PU	SCWR
1	11	6	**1**	**6**	**4**	**12**	**7**	**12**	3	**6**	**11**
2	12	2	**6**	**9**	**8**	**11**	**11**	**11**	9	**5**	**8**
3	13	5	2	**10**	**9**	**9**	**12**	**7**	10	**2**	**7**
4	7	13	5	**12**	**11**	**8**	13	13	**4**	**1**	**5**
5	8	12	3	**13**	**12**	**7**	4	4	**8**	**10**	**4**
6	9	10	7	11	**13**	**6**	8	8	13	**9**	**3**
7	4	9	10	4	**10**	**4**	3	3	12	**8**	**2**
8	3	11	4	8	**3**	**3**	9	9	7	**4**	**1**
9	10	8	**8**	7	**7**	**2**	10	10	1	**3**	**13**
10	2	7	**9**	1	**6**	**1**	5	5	11	**7**	**12**
11	5	3	13	**2**	**5**	**13**	1	1	5	**11**	**10**
12	1	**4**	**11**	**3**	**2**	**10**	2	2	2	**12**	**9**
13	6	**1**	**12**	**5**	**1**	**5**	6	6	6	**13**	**6**

Since Alternative 7 (utilization of Jordan River) involves the highest political uncertainty (PU rated 1), Alternatives 11, 12 and 13 resulted in similar high political uncertainty (rated 1) due to the combination of Alternative 7 (utilization of Jordan River) in each of them.

As could be noticed from the above and since the decision criteria developed in this work reflect both socioeconomic as well as environmental-related criteria no alternative is dominant in terms of all the set of criteria. Therfore, the ranking of the management alternatives necessitates the use of a multi-criteria decision analysis as detailed in Section 6.5.

6.5 MULTI-CRITERIA DECISION ANALYSIS

Multi-criteria decision analysis evaluates a "utility" which is a numerical value for an alternative based on a single criteria or number of decisions. The number of decision criteria in a utility is selected by the decision maker based on the importance of the criteria for the study. This study will carry out the multi-criteria decision analysis proposed by Yakowitz et al. (1993). Above authors use the Importance Order of Criteria (IOC) of the decision variables to select the best management alternative.

The utility value of a single criterion for each alternative was calculated in Section 6.3 and presented in Table 6-4 for each criterion and for each alternative. The dimensions for different criteria are different. The criteria values are standardized to remove the dimensions. The standardized dimensionless criteria will have values between zero and one. The standardized value is calculated using Equation 6-3 (Hope, 1996; Yoon and Hwang, 1995).

$$\left| v_{ij}^{N} \right| = \left| \frac{v_{ij} - \min\{v_{ij}\}}{\max\{v_{ij}\} - \min\{v_{ij}\}} \right| \tag{6-3}$$

where,

i - criterion;

j - alternative;

v_{ij} - the value of the ith criterion for each jth alternative;

v_{ij}^{N} - the standarized value of v_{ij};

$\min\{v_{ij}\}$ - the minimum value of the ith decision criterion of all the alternatives; and

$\max\{v_{ij}\}$ - the maximum values of the ith decision criterion of all the alternatives.

The standarized values for each alternative and criteria are given in Table 6-6.

Table 6-6 The standarized decision criteria values for each management alternative and the rank of the importance of the criteria

IOC		Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5	Alt. 6	Alt. 7	Alt. 8	Alt 9	Alt 10	Alt11	Alt 12	Alt. 13
Rank	**Criteria**													
1	**SY**	0.004	0.000	0.000	0.268	0.000	1.000	0.248	0.265	1.000	1.000	0.698	1.000	1.000
2	**NB**	0.037	0.157	0.251	0.267	0.137	0.000	0.776	0.359	0.323	0.173	1.000	0.993	0.988
3	**TIL**	0.162	0.149	0.638	0.682	0.219	0.000	1.000	0.660	0.552	0.347	1.000	1.000	0.989
4	**COST**	1.000	0.986	0.679	0.336	0.963	1.000	0.678	0.322	0.322	0.642	0.000	0.000	0.320
5	**PU**	1.000	1.000	0.250	0.250	1.000	1.000	0.000	0.250	0.250	0.250	0.000	0.000	0.000
6	**SCWR**	1.000	1.000	1.000	1.000	1.000	0.000	1.000	1.000	0.000	0.000	1.000	0.000	0.000
7	**SW**	0.000	0.000	0.606	1.000	0.000	0.000	0.000	1.000	1.000	0.606	1.000	1.000	0.606
8	**BW**	1.000	1.000	1.000	1.000	0.000	1.000	1.000	1.000	1.000	0.000	1.000	1.000	0.000
9	**UAWR**	0.415	0.170	1.000	0.760	0.188	0.000	0.419	0.760	0.904	0.782	0.319	0.493	0.681
10	**UALR**	0.162	0.150	0.638	0.683	0.219	0.000	1.000	0.661	0.554	0.349	1.000	1.000	0.990
11	**NR**	0.000	0.895	0.105	0.000	0.737	1.000	0.211	0.316	0.579	0.579	0.526	0.632	0.684

The IOC method is conceptually simple and provides the decision maker with clear graphical evidence if one alternative is strongly dominant over another. It relies on defining the best and worst total utilities of an alternative through the ranking of the decision criteria of each alternative.

Initially Yakowitz et al. (1993) identified the best utility out of a set of alternatives through assigning weights to different criteria. Through changing the value of weight assigned to each criteria the best total utility of an alternative is computed via a linear program that maximizes the expected utility U_j, as given in Equation 6-4.

$$U_j = \sum_{i=1}^{m} w_i v_{ij}^N \qquad (6\text{-}4)$$

where ,

U_j - total utility function

m – maximum number of criteria of alternative j;

v_{ij}^N - normalized value of j^{th} alternative with respect to the i^{th} criterion; and

w_i - weight assigned to criterion i.

The maximization of the utility in Equation 6-4 is subject to the following constraints

$$w_1 \geq w_2 \geq \ldots\ldots\ldots \geq w_m \qquad (6\text{-}5)$$

$$\sum_{i=1}^{m} w_i = 1 \qquad (6\text{-}6)$$

$$w_m \geq 0 \qquad (6\text{-}7)$$

The first constraint (Equation 6-5) defines the ranking of the Importance Order of the Criteria (IOC) based on the weighting. Similar to calculating the best total utility the lowest total utility is also found via a linear program that minimizes the utility in Equation 6-4 instead of maximizing it.

However, this requires solving two linear programs for each alternative under consideration to obtain the solutions of the minimum and maximum utilities. Consequently, Yakowitz, et al.

(1993) identified the alternatives that dominate with respect to an additive function for a given IOC as given in Equation 6-8. This eliminated the necessity for determining a specific weight associated with each decision criteria for the purpose of ranking the alternatives. In this method the decision criteria is ranked based on the importance to meet the triple bottom line requirements (socio-economic and environmental requirements) for the study area. In the current study the criteria pertinent to the constraints of the safe yield pumping of the aquifer were given the utmost preference over the remaining criteria. The economical aspect of the management alternatives was given the subsequent position. Utility scores for each alternative were calculated by adding one criterion at a time to the highest ranked criteria. The utility value in the IOC criteria is given by Equation 6-8.

$$S_{kj} = \frac{1}{k}\sum_{i=1}^{k} v_{ij}^{N} \tag{6-8}$$

where,

S_{kj} – utility value of the jth alternative for k number of criteria;

k – rank of the criteria varies from 1 to m which is the maximum number of criteria selected for the study

j - alternative;

i - criteria;

v_{ij}^{N} - normalized value of jth alternative with respect to the ith criterion;

For each alternative there will be *m* number of utility values.

If BU_j and WU_j indicate the best and worst utility values for jth alternative as given in Equations 6-9 and 6-10.

$$BU_j = \max\{S_{kj}\} \tag{6-9}$$

and

$$WU_j = \min\{S_{kj}\} \tag{6-10}$$

If $WU_A \geq BU_B$, Alternative A dominates Alternative B. However, if the computation of the best and worst utilities did not yield a complete ranking of the alternatives, then the best and worst total utilities for each alternative are averaged out and the alternatives are ranked in descending order of these averages. That is if $\frac{BU_A + WU_A}{2} \geq \frac{BU_B + WU_B}{2}$ Alternative A is ranked higher than Alternative B.

The following steps summarize the approach to determine the best alternative for the study area:

- Identify management alternatives and decision criteria for the study area.
- Calculate the utility value for each alternative with respect to each criterion (v_{ij}) using the AGSM and spreadsheets as detailed in Section 6.3.
- Standarize the v_{ij} using Equation 6-3.
- Rank the decision criteria based on the importance to the study (IOC).
- Calculate the utility scores using Equation 6-8. The number of criteria used for each score will follow according to the defined ranking.
- Identify the best and worst utility scores (maximum and minimum for each alternative) using Equations 6-9 and 6-10.
- Rank the management alternatives using the average of maximum and minimum utility scores.

6.6 THE APPLICATION OF METHOD OF THE IMPORTANCE ORDER OF CRITERIA (IOC)

The alternatives and the decision criteria selected for the analysis are listed in Tables 6-1 and 6-3. The values of the decision criteria for each management alternative were given in Table 6-4. These values were standarized using Equation 6-3. The standarized decision criteria values obtained for each alternative are given in Table 6-6. The standarized values are ranked according to the importance order of the decision criteria and also depicted in Table 6-6. The decision criteria were ranked based on the order of importance to meet the triple bottom line requirements (socio-economic and environmental requirements) for the study area. The criteria pertinent to the constraints of the safe yield pumping of the aquifer were given the utmost preference over the remaining criteria. The economical aspect of the management alternatives was given the subsequent position. Utility scores for each alternative were calculated using Equation 6-8. That is by adding one criterion at a time to the highest ranked criteria and dividing by the rank of the criteria as shown in Table 6-7. For example in Table 6-7 the first utility score for Alternative 1 is equal to the value (0.004) of the first decision criteria (SY) for this alternative divided by the first rank (1) which gives a utility score of

Table 6-7 Utility values of the jth alternative for k number of criteria

Alt.	K	Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5	Alt. 6	Alt. 7	Alt. 8	Alt. 9	Alt. 10	Alt. 11	Alt. 12	Alt.13
[1]	1	0.004	0.000	0.000	0.268	0.000	1.000	0.248	0.265	1.000	1.000	0.698	1.000	1.000
[1, 2]	2	0.021	0.078	0.126	0.267	0.068	0.500	0.512	0.312	0.661	0.586	0.849	0.997	0.994
[1, …, 3]	3	0.068	0.102	0.296	0.405	0.118	0.333	0.675	0.428	0.625	0.507	0.899	0.998	0.992
[1, …, 4]	4	0.301	0.323	0.392	0.388	0.330	0.500	0.676	0.402	0.549	0.540	0.675	0.748	0.824
[1, …, 5]	5	0.441	0.458	0.364	0.360	0.464	0.600	0.540	0.371	0.489	0.482	0.540	0.599	0.659
[1, …, 6]	6	0.534	0.549	0.470	0.467	0.553	0.500	0.617	0.476	0.408	0.402	0.616	0.499	0.550
[1, …, 7]	7	0.458	0.470	0.489	0.543	0.474	0.429	0.529	0.551	0.492	0.431	0.671	0.570	0.558
[1, …, 8]	8	0.525	0.537	0.553	0.600	0.415	0.500	0.588	0.607	0.556	0.377	0.712	0.624	0.488
[1, …, 9]	9	0.513	0.496	0.603	0.618	0.390	0.444	0.569	0.624	0.595	0.422	0.669	0.610	0.509
[1, …, 10]	10	0.478	0.461	0.606	0.625	0.373	0.400	0.612	0.628	0.590	0.415	0.702	0.649	0.557
[1, …, 11]	11	0.435	0.501	0.561	0.568	0.406	0.455	0.576	0.599	0.589	0.430	0.686	0.647	0.569

0.004. The second utility score for Alternative 1 is equal to the sum of the two values (0.004+0.037) of the first and second decision criteria (SY and NB) for this alternative divided by the second rank (2) which gives a utility score of 0.021 and so on. The best and worst utility scores and their average values are presented in Figure 6-3.

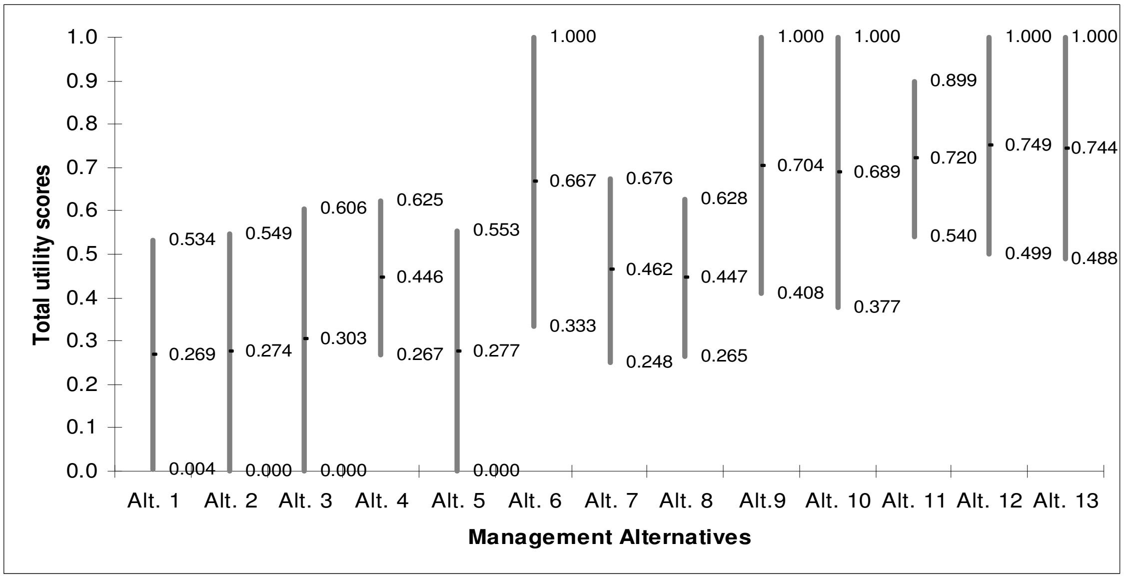

Figure 6-3 **Best, average, and worst utility scores for the different management alternatives**

Ranking of the management alternatives was accomplished by the use of the average utility scores shown in Figure 6-3. The results of ranking the management alternatives considering the average, best, and worst utility scores are summarized in Table 6-8. As anticipated, Table 6-8 shows that the combined management Alternative 12 that includes utilization of Jordan River, changing the cropping pattern by introducing high income crops, building a surface water storage (dam) and implementing a groundwater pumping strategy was found to be the best alternative. Results also show that Alternative 13 which incorporates Utilization of Jordan River, Artificial recharge of groundwater with surface runoff and treated wastewater, Desalination of brackish water and Groundwater pumping strategy is the second best alternative.

Comparing the values given in Table 6-4, the do nothing alternative with the best alternative indicates that the net benefit for Alternative 12 is more than two times that of Alternative 1. This is due to the larger amount of irrigated land (with higher income than rainfed land) under

Table 6-8 Rankings of the management alternatives for the best, average, and worst utility scores

Ranking	Utility score		
	Average	Best	Worst
1	Alt. 12	Alt. 12	Alt. 11
2	Alt. 13	Alt. 13	Alt. 12
3	Alt. 11	Alt. 9	Alt. 13
4	Alt. 9	Alt. 10	Alt. 9
5	Alt. 10	Alt. 6	Alt. 10
6	Alt. 6	Alt. 11	Alt. 6
7	Alt. 7	Alt. 7	Alt. 4
8	Alt. 8	Alt. 8	Alt. 8
9	Alt. 4	Alt. 4	Alt. 7
10	Alt. 3	Alt. 3	Alt. 1
11	Alt. 5	Alt. 5	Alt. 5
12	Alt. 2	Alt. 2	Alt. 2
13	Alt. 1	Alt. 1	Alt. 3

Alternative 12 where the entire available land of 5052 ha (percent utilization of available land resources of 100%) was irrigated compared to only 3357 ha (percent utilization of available land resources of 66.4%) irrigated under Alternative 1. Under Alternative 12 there is no groundwater abstracted above the safe yield compared to Alternative 1 that utilizes 3213 ML/yr above the safe yield. This criterion has the most importance since it is the first criteria in the ranking scheme. The amounts of surface water and wastewater that are available under Alternative 12 are 6600 ML compared to none for Alternative 1. Although the other criteria like the cost criteria, political uncertainity and social impacts were in favour of the do nothing scenario but the overall assessment that considers all the criteria and the order of those criteria resulted in Alternative 12 as the best alternative. Alternative 12 meets the triple bottom line of land use, governmental policies and water resources protection and development to optimize use of land and water resources.

Alternatives 11, 9 and 10 reserved the third, fourth and fifth places, respectively as signified by the average utility scores while Alternatives 1, 2, 3, 4 and 5 exchanged the last five positions suggesting a low preference for the current order of decision criteria.

Nevertheless, satisfying the safe yield criterion expressed in Alternative 6 and combining it with other alternatives as in Alternatives 9 and 10 have resulted in two efficient alternatives, namely Alternative 9 and Alternative 10 that occupied the forth and fifth places.

Although the groundwater pumping strategy (Alternative 6) does satisfy the safe yield limits, yet it occupied the sixth place when considering the average utility scores due to its inefficiency in satisfying the cost and benefit criteria. It is also worth mentioning that if the allowable limit of drawdown percentage (50% as detailed in Chapter 4) were assigned a different value, then different safe yield and drawdown percentage values might transpire and a different alternative ranking scheme could be anticipated. A detailed analysis is given in the next section to investigate the impact of such a change. Furthermore the results consider the importance order of the decision criteria that was adopted for the current study. A change in the order of the decision criteria might impact the ranking of the alternatives. An investigation of such a change is also discussed in the next section.

As a conclusion, rankings in Table 6-8 do not indicate an absolute efficiency or deficiency of a management alternative but rather reflects the outcome of the simulation and planning models (MODFLOW, KW-GIUH and AGSM) as well as the order of the decision criteria. Therefore a sensitivity analysis should be conducted to study the effect of the order of decision criteria on the decision process which is given in Section 6.7.

The developed MCDSS was tested for the case study using the values of the decision criteria for each management alternative given in Table 6-4. The input-output data files are presented in Appendix V (Tables AV.1 to AV.6).

6.7 SENSITIVITY ANALYSIS OF THE RANKING SCHEME

Sensitivity analysis was conducted to study the effect of the ranking scheme of different decision criteria on the ranking of different management alternatives and the selection of the best management alternative. In addition to the ranking scheme used for the current study Table 6-9 shows eleven different ranking scenarios. The best and worst utility scores and their average values for each ranking scenario are presented in Figures 6-4 to 6-14. The resulting ranking of different management alternatives based on these ranking scenarios is shown in Table 6-10. Results indicated that changing the order of the criteria resulted in different ranking of the management alternatives. Scenario 1 gives the order of criteria used for the

Table 6-9 Different ranking scenarions of the decision criteria. Abbreviations and units are as defined in Table 6-3

Order of Criteria	**Scenario 1**	**Scenario 2**	**Scenario 3**	**Scenario 4**	**Scenario 5**	**Scenario 6**	**Scenario 7**	**Scenario 8**	**Scenario 9**	**Scenario 10**	**Scenario 11**	**Scenario 12**
1	SY	PU	COST	COST	SY	COST	SCWR	TIL	SCWR	SCWR	BW	SY
2	NB	NB	PU	SCWR	COST	SCWR	NB	SCWR	PU	PU	SW	COST
3	TIL	TIL	TIL	PU	SCWR	PU	TIL	SW	NR	SW	SCWR	PU
4	COST	COST	SY	SW	PU	SW	COST	NR	TIL	UAWR	PU	SW
5	PU	SY	NB	NR	SW	NR	PU	UALR	UAWR	UALR	NR	SCWR
6	SCWR	SCWR	SCWR	UAWR	NR	UALR	SY	UAWR	BW	BW	UALR	NR
7	SW	SW	SW	UALR	UALR	UAWR	SW	BW	SY	SY	UAWR	UALR
8	BW	BW	BW	BW	UAWR	BW	BW	SY	SW	TIL	SY	UAWR
9	UAWR	UAWR	UAWR	SY	BW	SY	UAWR	PU	UALR	NB	TIL	BW
10	UALR	UALR	UALR	TIL	TIL	TIL	UALR	COST	COST	NR	COST	TIL
11	NR	NR	NR	NB	NB	NB	NR	NB	NB	COST	NB	NB

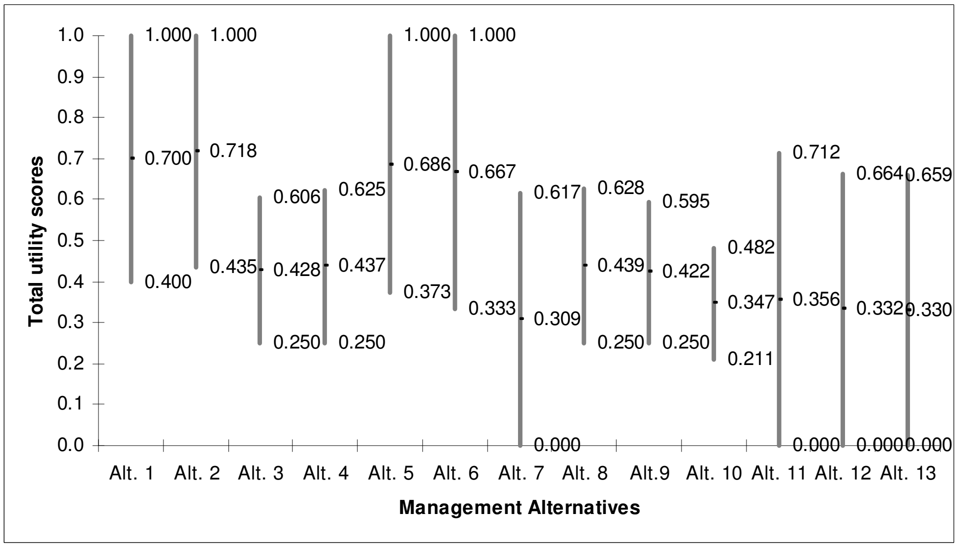

Figure 6-4 Best, average, and worst utility scores for Scenario 2

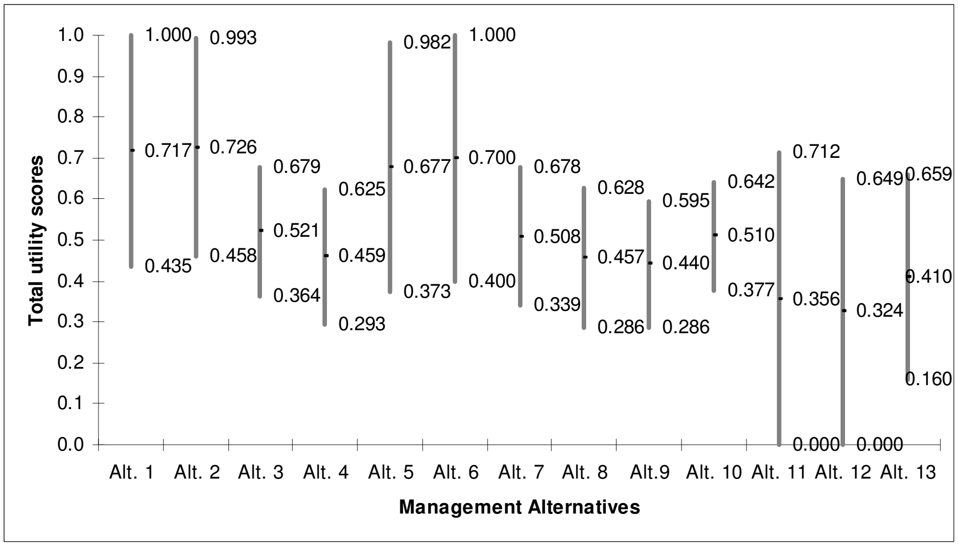

Figure 6-5 Best, average, and worst utility scores for Scenario 3

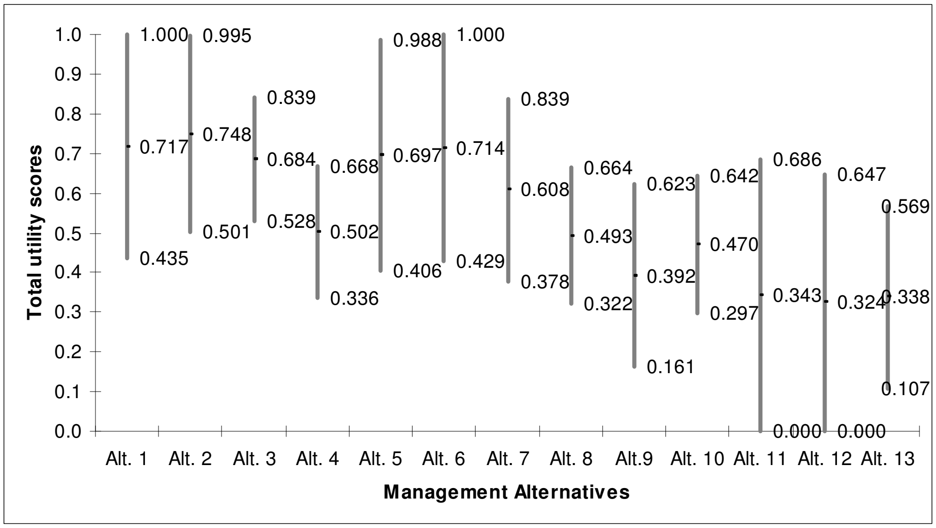

Figure 6-6 Best, average, and worst utility scores for Scenario 4

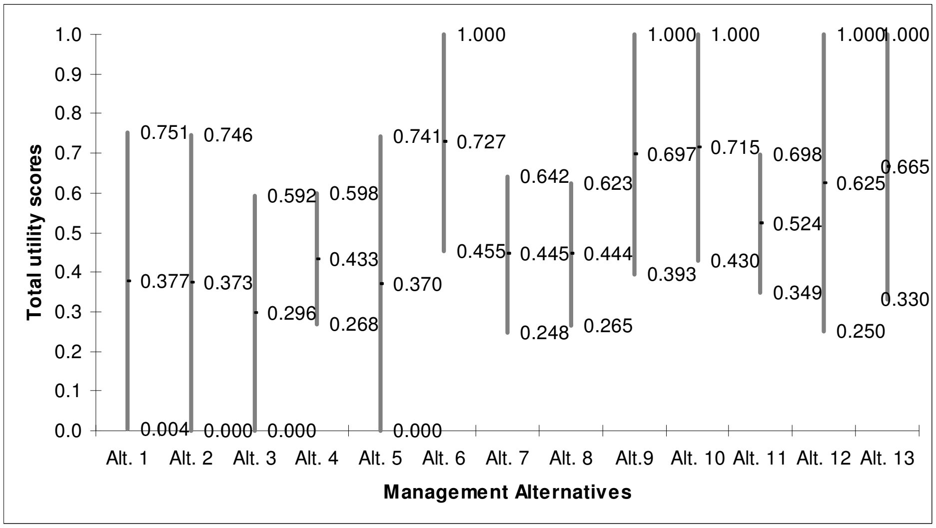

Figure 6-7 Best, average, and worst utility scores for Scenario 5

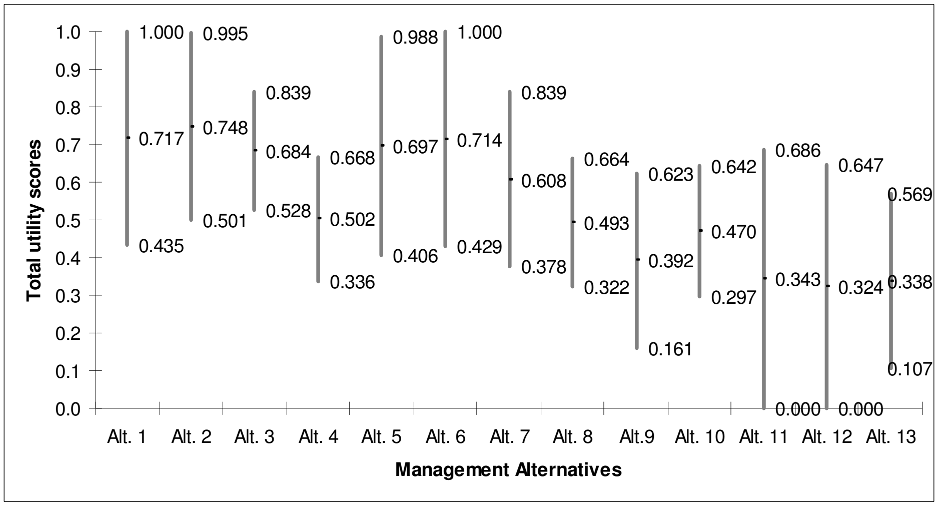

Figure 6-8 Best, average, and worst utility scores for Scenario 6

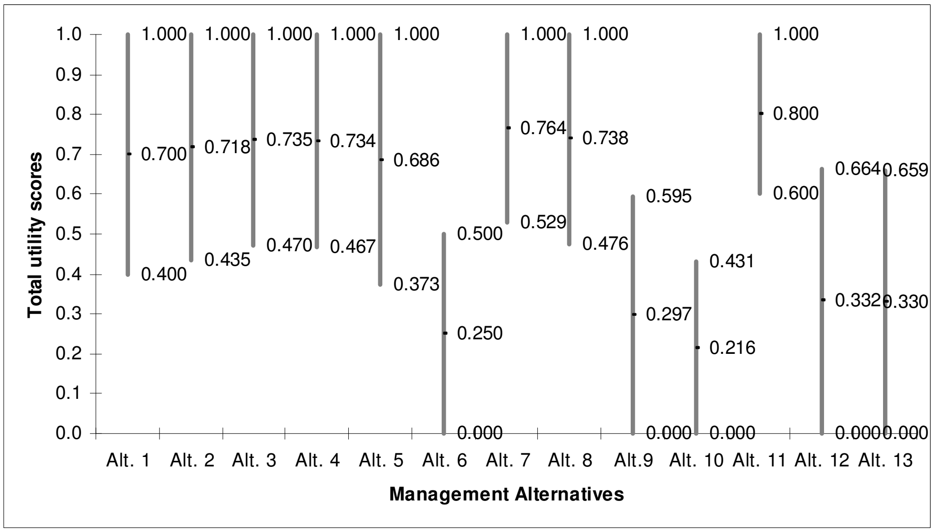

Figure 6-9 Best, average, and worst utility scores for Scenario 7

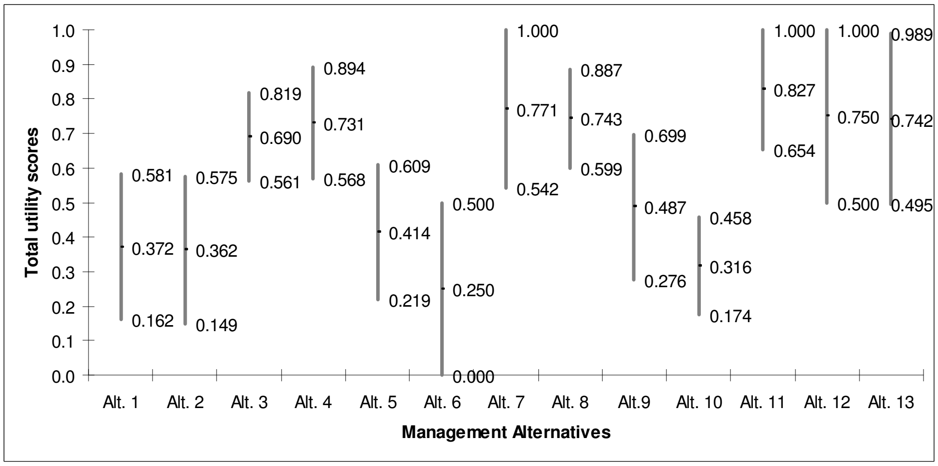

Figure 6-10 Best, average, and worst utility scores for Scenario 8

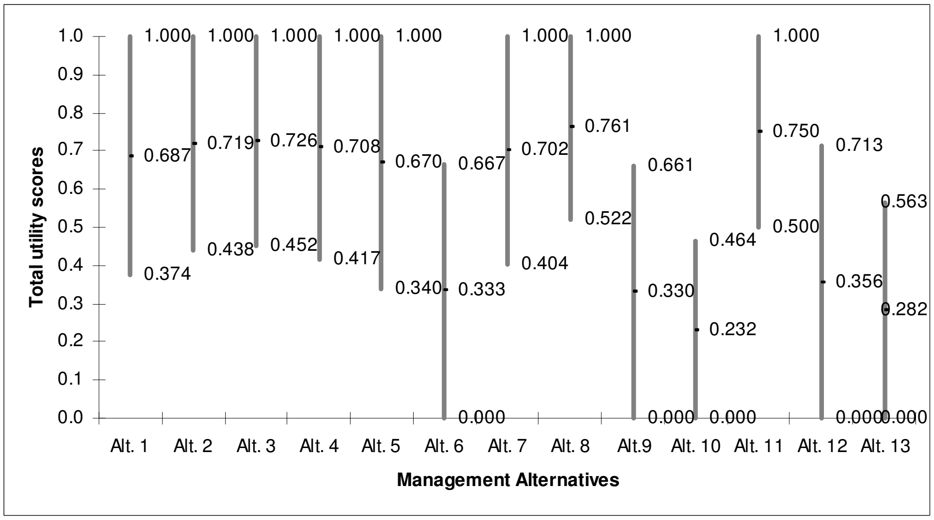

Figure 6-11 Best, average, and worst utility scores for Scenario 9

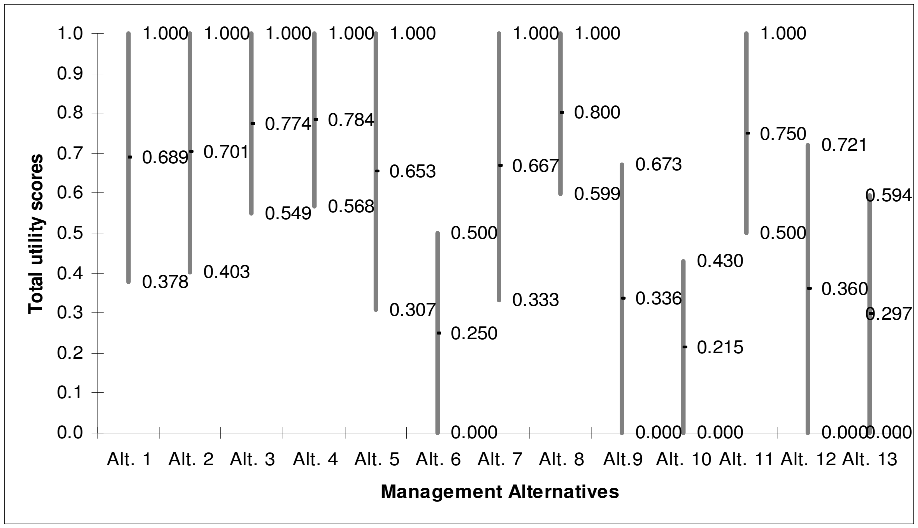

Figure 6-12 Best, average, and worst utility scores for Scenario 10

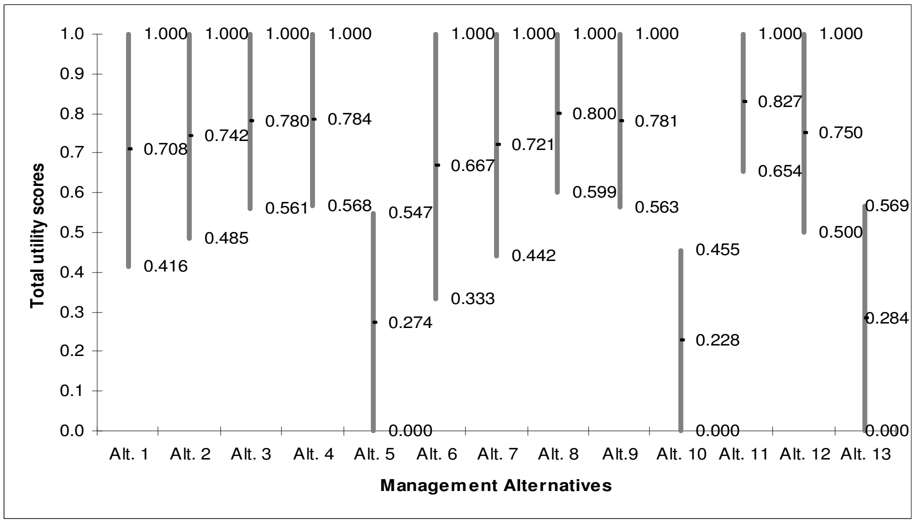

Figure 6-13 Best, average, and worst utility scores for Scenario 11

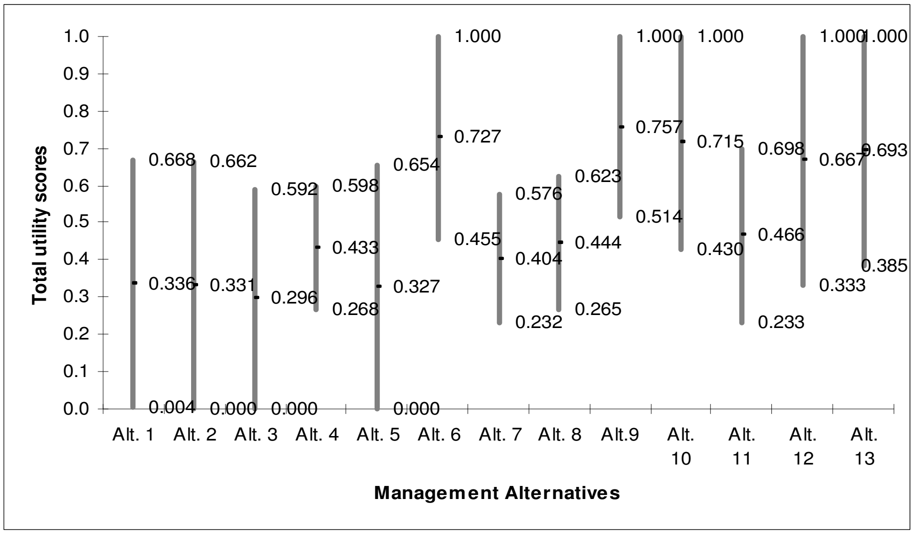

Figure 6-14 Best, average, and worst utility scores for Scenario 12

Table 6-10 Results of the ranking of different management alternatives under different scenarios of decision criteria ranking

Ranking	Scenario 1	Scenario 2	Scenario 3	Scenario 4	Scenario 5	Scenario 6	Scenario 7	Scenario 8	Scenario 9	Scenario 10	Scenario 11	Scenario 12
1	Alt. 12	Alt. 2	Alt. 2	Alt.2	Alt.6	Alt.2	Alt.11	Alt.11	Alt.8	Alt.8	Alt.11	Alt.9
2	Alt. 13	Alt. 1	Alt. 1	Alt.1	Alt.10	Alt.1	Alt.7	Alt.7	Alt.11	Alt.4	Alt.8	Alt.6
3	Alt. 11	Alt. 5	Alt. 6	Alt.6	Alt.9	Alt.6	Alt.8	Alt.12	Alt.3	Alt.3	Alt.4	Alt.10
4	Alt. 9	Alt. 6	Alt. 5	Alt.5	Alt.13	Alt.5	Alt.3	Alt.8	Alt.2	Alt.11	Alt.9	Alt.13
5	Alt. 10	Alt. 8	Alt. 3	Alt.3	Alt.12	Alt.3	Alt.4	Alt.13	Alt.4	Alt.2	Alt.3	Alt.12
6	Alt. 6	Alt. 4	Alt.10	Alt.7	Alt.11	Alt.7	Alt.2	Alt.4	Alt.7	Alt.1	Alt.12	Alt.11
7	Alt. 7	Alt. 3	Alt.7	Alt.4	Alt.7	Alt.4	Alt.1	Alt.3	Alt.1	Alt.7	Alt.2	Alt.8
8	Alt. 8	Alt. 9	Alt.4	Alt.8	Alt.8	Alt.8	Alt.5	Alt.9	Alt.5	Alt.5	Alt.7	Alt.4
9	Alt. 4	Alt.11	Alt.8	Alt.10	Alt.4	Alt.10	Alt.12	Alt.5	Alt.12	Alt.12	Alt.1	Alt.7
10	Alt. 3	Alt.10	Alt.9	Alt.9	Alt.1	Alt.9	Alt.13	Alt.1	Alt.6	Alt.9	Alt.6	Alt.1
11	Alt. 5	Alt. 12	Alt.13	Alt.11	Alt.2	Alt.11	Alt.9	Alt.2	Alt.9	Alt.13	Alt.13	Alt.2
12	Alt. 2	Alt.13	Alt.11	Alt.13	Alt.5	Alt.13	Alt.6	Alt.10	Alt.13	Alt.6	Alt.5	Alt.5
13	Alt. 1	Alt. 7	Alt.12	Alt.12	Alt.3	Alt.12	Alt.10	Alt.6	Alt.10	Alt.10	Alt.10	Alt.3

current study. For example in Scenario 2 the criteria pertinent to the political uncertainity was given the utmost preference over the remaining criteria and the economical aspect of the management alternatives was given the subsequent position. The resulting ranking of the management alternatives based on this scenario shows that the best alternative is Alternative 2 followed by Alternative 1 which reflects a situation where the political constraints prohibits the implementation of management alternatives that involve water infrastructural projects in the study area which necessitates a political implication as given for each alternative. Therefore Alternative 2 came first as it implies only a change in the cropping pattern through introducing the dates palm as a high income crop with no political restrictions. Alternative 1 that reflects the no change scenario came second since it also implies no political restrictions but with less net profit compared to Alternative 2.

Similar analysis could be noticed with Scenarios 3 and 4 where the cost criterion was given the utmost preference over the remaining criteria and the political uncertainity was given the second and third positions. The two scenarios resulted in similar ranking of the management alternatives for the positions 1 to 5. The sixth position was allocated to Alternatives 10 and 7 in the case of Scenarios 3 and 4 respectively. This could be due to the fact that the social criterion was given the second preference in the case of Scenario 4. This drives Alternative 7 to the sixth position since it involves less social impact compared to Alternative 10 and consequently this ranking scheme drove Alternative 10 to the ninth position.

Further analysis was conducted to study the effect of the allowable limit of drawdown percentage (50% as detailed in Chapter 4) on the selection of best management alternative. The % of allowable limit of drawdown was assigned a different value of 80% to investigate the effect on the ranking of different management alternatives. Based on an allowable limit of 80% drawdown and as detailed in Chapter 4, the safe yield for agriculture is estimated at 5325 ML per year. The decision process described earlier in Section 6.6 was similarly applied to the management alternatives considering the consequences of the 80% drawdown limit. The main difference with the 80% drawdown limit lies in the larger safe yield offered through this limit that will have an effect on the results of the decision criteria for Alternatives 6, 9, 10, 12 and 13 in terms of the following criteria: total net benefit, net return per cubic meter, total irrigated land and the percent utilization of both the available land and water resources. In general the larger amount of available water through the 80% limit will implicate an increase in the irrigated land and net benefit as compared to the 50% limit. The rest of the decision criteria (net costs, groundwater abstractions above the safe yield, quantities of winter surface runoff and wastewater available, quantities of brackish water available, political uncertainity and degree of social conflict) will not change for both the 50% and 80% limits. For the other management alternatives the only decision criterion that will change is the groundwater abstractions above the safe yield that will change according to the new safe yield limit resulted from the 80% limit.

The decision criteria values for all management alternatives are presented in Table 6-11 . The alternatives and the decision criteria selected for the analysis were listed in Tables 6-1 and 6-3. The standarized values obtained for each alternative based on each decision criteria are given in Table 6-12. The standarized values depicted in Table 6-12 are ranked according to the same importance order of the decision criteria used for the current study. Utility scores for each alternative were calculated as shown in Table 6-13. The best and worst utility scores and their average values are presented in Figure 6-15. The results of ranking the management alternatives considering the average, best, and worst utility scores are summarized in Table 6-14. Table 6-14 shows that the combined management Alternative 12 that includes utilization of Jordan River, changing the cropping pattern by introducing high income crops, building a surface water storage (dam) and implementing a groundwater pumping strategy was moved from the first position to the third position under the 80% drawdown limit as compared to the 50% drawdown limit. Instead Alternative 11 which is identical to Alternative 12 except that it does not involve implementing a groundwater pumping strategy was found to be the best alternative. This result is anticipated since the 80% drawdown limit increased the amount of the safe yield compared to the 50% drawdown limit (from 3000 ML to 5325 ML) resulting in minor amounts of groundwater abstracted above the safe yield for the management alternatives as shown in Table 6-11 and consequently decreasing the differences among management alternatives in terms of this criterion. Similarly satisfying the safe yield criterion expressed in Alternative 6 and combining it with other alternatives as in Alternatives 9 and 10 have resulted in moving these two alternatives further down in the ranking where Alternative 9 and Alternative 10 occupied the seventh and eighth places.

Table 6-11 Summary evaluation of the decision criteria as computed for the proposed management alternatives for the 80% drawdown scenario. Abbreviations and units are as defined in Table 6-1 and Table 6-3

Criteria	**Alt. 1**	**Alt. 2**	**Alt. 3**	**Alt. 4**	**Alt. 5**	**Alt. 6**	**Alt. 7**	**Alt. 8**	**Alt. 9**	**Alt. 10**	**Alt. 11**	**Alt. 12**	**Alt. 13**
NB($)	1.52E+07	1.74E+07	1.91E+07	1.94E+07	1.70E+07	1.56E+07	2.88E+07	2.11E+07	2.10E+07	1.87E+07	3.30E+07	3.29E+07	3.29E+07
NR($/m^3)	0.77	0.94	0.79	0.77	0.91	0.89	0.81	0.83	0.85	0.82	0.87	0.87	0.88
COST($)	0.00E+00	1.20E+06	2.74E+07	5.66E+07	3.12E+06	0.00E+00	2.75E+07	5.78E+07	5.78E+07	3.05E+07	8.53E+07	8.53E+07	5.80E+07
SY(ML)	8.88E+02	9.00E+02	9.00E+02	3.70E+01	9.00E+02	0.00E+00	9.90E+01	4.40E+01	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
SW(ML/yr)	0.00E+00	0.00E+00	4.00E+03	6.60E+03	0.00E+00	0.00E+00	0.00E+00	6.60E+03	6.60E+03	4.00E+03	6.60E+03	6.60E+03	4.00E+03
BW(ML/yr)	1.95E+03	1.95E+03	1.95E+03	1.95E+03	0.00E+00	1.95E+03	1.95E+03	1.95E+03	1.95E+03	0.00E+00	1.95E+03	1.95E+03	0.00E+00
TIL(ha/yr)	3.36E+03	3.33E+03	4.32E+03	4.41E+03	3.47E+03	3.37E+03	5.05E+03	4.36E+03	4.32E+03	4.14E+03	5.05E+03	5.04E+03	5.05E+03
UALR(%)	66.40	65.90	85.50	87.30	68.70	66.80	100.00	86.40	85.50	82.00	100.00	99.80	100.00
UAWR(%)	86.60	81.00	100.00	94.50	81.40	80.00	86.70	94.50	95.20	98.10	84.40	85.80	90.20
PU	0.00	0.00	0.75	0.75	0.00	0.00	1.00	0.75	0.75	0.75	1.00	1.00	1.00
SCWR	0.00	0.00	0.00	0.00	0.00	1.00	0.00	0.00	1.00	1.00	0.00	1.00	1.00

Table 6-12 Summary of the standarized management alternatives for the different decision criteria for the 80% drawdown scenario

IOC		Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5	Alt. 6	Alt. 7	Alt. 8	Alt 9	Alt 10	Alt11	Alt 12	Alt. 13
Rank	Criteria													
1	SY	0.013	0.000	0.000	0.959	0.000	1.000	0.890	0.951	1.000	1.000	1.000	1.000	1.000
2	NB	0.000	0.124	0.222	0.238	0.103	0.022	0.767	0.334	0.326	0.200	1.000	0.999	0.997
3	TIL	0.015	0.000	0.574	0.626	0.082	0.024	1.000	0.600	0.574	0.472	1.000	0.995	1.000
4	COST	1.000	0.986	0.679	0.336	0.963	1.000	0.678	0.322	0.322	0.642	0.000	0.000	0.320
5	PU	1.000	1.000	0.250	0.250	1.000	1.000	0.000	0.250	0.250	0.250	0.000	0.000	0.000
6	SCWR	1.000	1.000	1.000	1.000	1.000	0.000	1.000	1.000	0.000	0.000	1.000	0.000	0.000
7	SW	0.000	0.000	0.606	1.000	0.000	0.000	0.000	1.000	1.000	0.606	1.000	1.000	0.606
8	BW	1.000	1.000	1.000	1.000	0.000	1.000	1.000	1.000	1.000	0.000	1.000	1.000	0.000
9	UAWR	0.330	0.050	1.000	0.725	0.070	0.000	0.335	0.725	0.760	0.905	0.220	0.290	0.510
10	UALR	0.015	0.000	0.575	0.628	0.082	0.026	1.000	0.601	0.575	0.472	1.000	0.994	1.000
11	NR	0.000	1.000	0.118	0.000	0.824	0.706	0.235	0.353	0.471	0.294	0.588	0.588	0.647

Table 6-13 Summary of the utility scores for the decision criteria for the different alternatives for the 80% drawdown scenario

Alt.	K	Alt. 1	Alt. 2	Alt. 3	Alt. 4	Alt. 5	Alt. 6	Alt. 7	Alt. 8	Alt. 9	Alt. 10	Alt. 11	Alt. 12	Alt.13
[1]	1	0.013	0.000	0.000	0.959	0.000	1.000	0.890	0.951	1.000	1.000	1.000	1.000	1.000
[1, 2]	2	0.007	0.062	0.111	0.599	0.052	0.511	0.829	0.643	0.663	0.600	1.000	1.000	0.999
[1, …, 3]	3	0.010	0.041	0.266	0.608	0.062	0.349	0.886	0.628	0.633	0.557	1.000	0.998	0.999
[1, …, 4]	4	0.257	0.278	0.369	0.540	0.287	0.511	0.834	0.552	0.556	0.578	0.750	0.749	0.829
[1, …, 5]	5	0.406	0.422	0.345	0.482	0.430	0.609	0.667	0.491	0.494	0.513	0.600	0.599	0.663
[1, …, 6]	6	0.505	0.518	0.454	0.568	0.525	0.508	0.723	0.576	0.412	0.427	0.667	0.499	0.553
[1, …, 7]	7	0.433	0.444	0.476	0.630	0.450	0.435	0.619	0.637	0.496	0.453	0.714	0.571	0.560
[1, …, 8]	8	0.504	0.514	0.541	0.676	0.394	0.506	0.667	0.682	0.559	0.396	0.750	0.624	0.490
[1, …, 9]	9	0.484	0.462	0.592	0.682	0.358	0.450	0.630	0.687	0.581	0.453	0.691	0.587	0.493
[1, …, 10]	10	0.437	0.416	0.591	0.676	0.330	0.407	0.667	0.678	0.581	0.455	0.722	0.628	0.543
[1, …, 11]	11	0.398	0.469	0.548	0.615	0.375	0.434	0.628	0.649	0.571	0.440	0.710	0.624	0.553

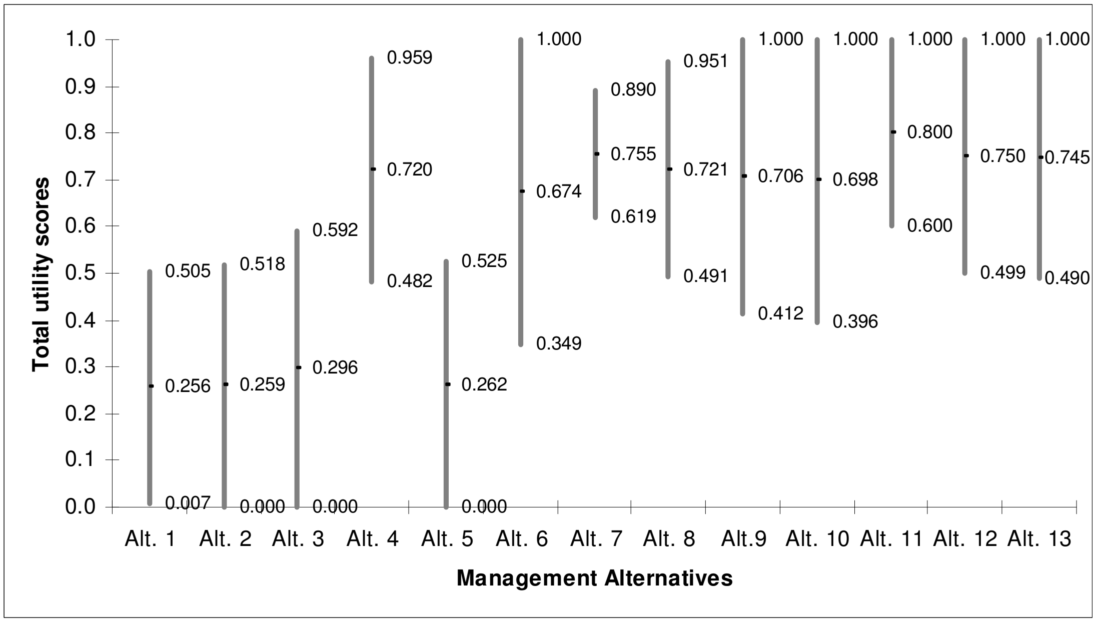

Figure 6-15 Best, average, and worst utility scores for different management alternatives for the 80% drawdown scenario

Table 6-14 Rankings of the management alternatives for the best, average, and worst utility scores for the 80% drawdown scenario

Ranking	Utility score		
	Average	Best	Worst
1	Alt. 11	Alt. 11	Alt. 7
2	Alt. 7	Alt. 12	Alt. 11
3	Alt. 12	Alt. 13	Alt. 12
4	Alt. 13	Alt. 9	Alt. 8
5	Alt. 8	Alt. 10	Alt. 13
6	Alt. 4	Alt. 6	Alt. 4
7	Alt. 9	Alt. 4	Alt. 9
8	Alt. 10	Alt. 8	Alt. 10
9	Alt. 6	Alt. 7	Alt. 6
10	Alt. 3	Alt. 3	Alt. 1
11	Alt. 5	Alt. 5	Alt. 5
12	Alt. 2	Alt. 2	Alt. 3
13	Alt. 1	Alt. 1	Alt. 2

Although the groundwater pumping strategy (Alternative 6) does satisfy the safe yield limits, yet it occupied the ninth place when considering the average utility scores not only due to its inefficiency in satisfying the cost and benefit criteria but also because of the increased safe yield effect caused by the 80% drawdown limit. Such an effect has moved this alternative down in the ranking from the sixth position to the ninth position. Finally it is worth mentioning that when deciding on the % of the drawdown limit, the decision maker should consider the sustainability of the groundwater aquifer that will provide water for the next generations.

A second application of the developed MCDSS was conducted for the case study with 80% drawdown limit using the values of the decision criteria for each management alternative given in Table 6-11. The input-output data files are presented in Appendix V (Tables AV.7 to AV.12).

The analysis presented in Section 6.6 and Section 6.7 indicates the importance in selecting the percentage of allowable drawdown limit. Such a percentage should address the actual existing conditions and sustain the groundwater resources of any catchment. It is worth mentioning that adopting the drawdown percentage for the current study was based on the analysis of data from the study area as well as a review of available literature. Finally it is worth mentioning that when deciding on the % of the drawdown limit, the decision maker should consider the sustainability of the groundwater aquifer that will provide water for the next generations.

The analysis presented in Section 6.7 indicates the importance in ranking the decision criteria. Such ranking should address the actual existing conditions and potential needs of any catchment. It should also address a comprehensive assessment of different stakeholders. It is worth mentioning that adopting the ranking scheme for the current study was based on the actual needs and potential of the study area, in-depth analysis of data from the study area, policies and strategies of the governmental bodies, feedback of decision makers and stakeholders in the study area and local community needs. Furthermore, the above analysis concludes importance of the developed MCDSS tool as a user friendly tool for decision making that will provide planners and decision makers with a useful tool to evaluate different management alternatives based on the previously developed framework.

6.8 MULTI CRITERIA DECISION SUPPORT SYSTEM (MCDSS)

As stated above the developed framework depicted in for the decision making process was used to develop a Multi Criteria Decision Support System (MCDSS). The user should be able to decide on the management alternatives and the decision criteria appropriate for the area under study. The values of the decision criteria for different management alternatives could be determined from relevant modeling approaches and available studies. The MCDSS compiles the input data systematically as explained earlier in this chapter and the final output is a ranking of the management alternatives.

Two examples were given to test the developed MCDSS the first is the case where input data relevant to the current study was applied to the developed MCDSS and the second is the case where input data relevant to the current study but with the 80% drawdown limit was applied to the developed MCDSS. The MCDSS can be used by different decision makers and planners in various areas to investigate the efficiency of different management alternatives in satisfying the decision criteria that ultimately aim at achieving the policies and strategies envisaged for the area under study.

6.9 SUMMARY AND CONCLUSIONS

Management alternatives were developed such that the optimal water and land utilization are met. The management alternatives include changing the cropping pattern by introducing high income crops, artificial recharge of surface runoff and treated wastewater, surface water and treated wastewater storage (dam), desalination of brackish water, groundwater pumping strategy, utilization of surface water from the Jordan River and combinations between selective management alternatives. Decision criteria were developed to cover socio-economic and environmental aspects. Therefore, single criteria evaluation is not capable of identifying the best alternative in terms of all the set of criteria. Accordingly, the ranking of the management alternatives requires a multi-criteria decision analysis. A Multi Criteria Decision Support System (MCDSS) software package was developed to calculate the utility values and rank the management alternatives.

Each management alternative was evaluated using more than one criterion. The decision criteria were ranked based on the order of importance to meet the triple bottom line requirements (socio-economic and environmental requirements) for the study area. The criteria pertinent to the constraints of the safe yield pumping of the aquifer were given the utmost preference over the remaining criteria. The economical aspect of the management alternatives was given the subsequent position. The importance order of criteria (IOC) method was employed to rank the alternatives. The IOC method relies on the preference of the decision maker in stipulating of the decision criteria. Such an order reflects the importance of these criteria to the decision maker.

A combined management alternative that includes utilization of Jordan River, changing the cropping pattern by introducing high income crops, building a surface water storage (dam) and implementing a groundwater pumping strategy proved to be the best alternative to maximize net benefit and satisfy the yield limits. Managing groundwater pumping strategy meets the safe yield as compared to other alternatives but failes to satisfy the economic criteria.
Sensitivity analysis conducted to study the effect of the ranking scheme of different decision criteria on the selection of the best management alternative indicates the successfulness of the alternatives for a specific order of the criteria. However the ranking scheme for the decision criteria should be based on the data synthesis and available information for any study area.

The effect of the allowable limit of drawdown percentage on the ranking of management alternatives was also investigated. Increasing the drawdown limit from 50% to 80% increased the amount of the safe yield. Such an increase in safe yield reduced the differences among management alternatives which ultimately resulted in a different ranking scheme.

In the decision making process the decision maker is often faced with the problem of selecting alternatives that are associated with conflicting criteria. Accordingly the MCDSS software package developed under this study is a very useful tool that can be used by different decision makers and planners in various areas to investigate the efficiency of different management alternatives in satisfying the decision criteria that ultimately aim at achieving the policies and strategies envisaged for the area under study.

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 SUMMARY

Simulation and optimization models are essential in developing management alternatives that sustain the available water resources and maximize both the irrigated areas and the income of the local farmers. Rainfall-runoff and groundwater models together with a planning model provide the sustainable yield limits of different water resources. Through these models, response to the current practices and proposed management alternatives can be easily evaluated based on different socio-economic and environmental criteria. The integrated land and water management framework developed in Chapter 2 provides planners and decision makers with a multi criteria decision support system. The most important output from this integrated modelling approach of combining rainfall-runoff model, groundwater model, statistical analysis of spring discharge and the planning model is the optimal land and water management alternative that maximizes net income of the catchment and sustains its natural resources. The developed framework considers different water sources including surface, groundwater and low quality water to irrigate all possible agricultural activities of different water requirements and cost benefit outputs.

The KW-GIUH rainfall-runoff model was selected to apply for the study area. The catchment geomorphological information was obtained using GIS to apply the KW-GIUH model to study area to determine runoff volume that could be harvested from a storm for irrigation purposes as well as to estimate the peakflow from a storm for infrastructure development. Excess rainfall was estimated using the Horton method. Sensitivity of excess rainfall on Horton model parameters was also investigated. The KW-GIUH model is verified against actual rainfall and streamflow data. Sensitivity analysis was conducted for each model input parameter to determine the effect on the peakflow and time to peak as the input parameters are obtained from GIS maps and published information.

The MODFLOW groundwater model was also applied to the study area. Possible alteration and development of different pumping strategies were investigated to addresses the needs necessitated by the management alternatives. The groundwater model reads the recharge as computed from the precipitation distribution which is influenced by climate variability.

Statistical analysis of the long-term yield from Faria springs was conducted for the purpose of comprehending the general trends in the yield of the springs, to analyse the reliability of extracting the water from springs and estimate the average springs yield in the study area.

The planning model (AGSM) was applied for the study area. The model simulates the behaviour of farmers and optimally determines the cropping pattern that maximizes the farmer economic return under various management scenarios. The model provides insights for planners who must allocate scarce water resources among agricultural activities by time, space and different water qualities. It also generates estimates of the effect of different water prices on the optimal cropping patterns and agricultural water demand. A new approach was developed to derive the optimal price of water from demand curves.

Management alternatives were developed such that the optimal water and land utilization are met. The management alternatives include changing the cropping pattern by introducing high income crops, artificial recharge of groundwater with surface runoff and treated wastewater, surface water and treated wastewater storage (dam), desalination of brackish water, groundwater pumping strategy, utilization of surface water from the Jordan River and combinations between selective management alternatives. Decision criteria were developed and utilized in a multi-criteria decision analysis. Each management alternative will be evaluated in terms of different decision criteria. These criteria can be classified into five main categories: (i) criteria pertaining to the cost of implementing the alternatives; (ii) criteria pertaining to the degradation of water resources; (iii) criteria pertaining to the governmental policies; (iv) criteria pertaining to political uncertainty and (v) criteria pertaining to the social impacts.

The importance order of criteria (IOC) method, described in Chapter 6, was employed in the multi-criteria decision analysis to rank the alternatives. The IOC method relies on the preference of the decision maker in stipulating of the decision criteria. Such order reflects the importance of these criteria to the decision maker. A Multi Criteria Decision Support System (MCDSS) was developed to aid decision makers and planners in various areas to investigate the efficiency of different management alternatives in satisfying the decision criteria that ultimately aim at achieving the policies and strategies envisaged for the area under study. Strengths and limitations of the methodology are thoroughly demonstrated and discussed.

7.2 CONCLUSIONS

The framework for the integrated management of land and water suggested in this study and the resulting Multi Criteria Decision Support System serve as a tool for planners and decision makers to evaluate management alternatives and select an optimal alternative. The following conclusions were drawn from the study.

7.2.1 Developing the Framework for Integrated Land and Water Management

- Lack of proper water allocation and optimal cropping systems accompanied with prolonged drought periods negatively affect the obtainable surface water and groundwater resources compelling the need for developing optimal water allocation policies that consider the available water resources in the catchments such that the socioeconomic revenue is maximized.
- There is a need for a framework for integrated management of agriculture-dominated catchments in arid and semi-arid regions that involves diverse modules of surface water and groundwater models, yield from natural springs, a planning model for economic evaluation, a multi-criteria decision analysis model, and a GIS technology to facilitate processing and visualization.
- It is important to develop management alternatives that sustain the available water resources both in quantity and quality, optimize the use of low quality water including treated effluent and brackish water, and maximize both the irrigated areas and the income of the local farmers.

7.2.2 Rainfall Runoff Model

- The applicability of the KW-GIUH model to predict runoff was successfully investigated for Faria catchment as an example of ungauged semi-arid catchments;
- The applicability of two baseflow separation models, namely Web-based Hydrograph Analysis Tool (WHAT) system and the Australian Water Balance Model (AWBM) model, were tested to separate baseflow from observed flow from Faria catchment. The two models showed almost equal amounts of baseflow and surface runoff;
- Application of the Horton method to estimate excess rainfall for Faria catchment indicated the importance of accurate estimation of excess rainfall for planning

purposes in arid and semi-arid areas. The percent error between the excess rainfall volume obtained from the Horton with the surface runoff volume obtained from the baseflow separation using the WHAT model is 3.9% and 10.9% for Badan and Faria subcatchments respectively. The sensitivity analysis of Horton model parameters to the estimation of the excess rainfall indicated that all parameters are sensitive;

- The effect of KW-GIUH model input parameters on the model output was investigated through a sensitivity analysis. The peak flow values increased by 16% as the overland flow roughness coefficient (n_o) decreased by 25% which reflects the land surface condition of the surface hydrologic system. However compared with the overland flow roughness coefficient (n_o), the channel flow roughness coefficient (n_c) had a smaller effect on both simulated peak flow and time to peak;
- It is necessary to follow the stream order level obtained from the stream network map in rainfall runoff modelling;
- The peak flow of Faria IUH is not sensitive to channel width at the catchment outlet (B_Ω). For the test range of channel width 25 % from the width of the streams of Faria subcatchments, a change of 25 % in B_Ω resulted only in 0.6 % change of the peak discharge of Faria subcatchment;
- Changing the value of each of the independent Faria geomorphological parameters by 10 % (number of ith-order channels N_i, mean ith-order stream length $\overline{L}_{ci}$, ith-order sub catchment contributing area $\overline{A}_i$, mean ith-order overland slope $\overline{S_{oi}}$, mean ith-order channel slope $\overline{S}_{ci}$ and subcatchment area) resulted in a change in the peak discharge value up to 8 %. With the GIS maps the geomorphologic properties of catchments could be measured accurately minimizing the error in estimating runoff;
- The average ratio of runoff to rainfall in Faria catchment is 3 %;
- The rainfall runoff model is capable of testing the future effect of climate variability on the obtainable amounts of runoff. A change in the precipitation pattern will be captured through the resulting amounts of excess rainfall and ultimately the amounts of runoff and peak flows.

7.2.3 Groundwater Model and Statistical Analysis of Spring Data

- Application of MODFLOW groundwater model to Faria catchment indicated, based on an allowable limit of drawdown percentage of 50%, that the current groundwater wells are abstracting water above the safe yield with a total amount of 3200 ML. This negatively affects the sustainability of the aquifer and necessitates the formulation of management alternatives to address such a problem;
- The groundwater model is capable of testing the future effect of climate variability on the obtainable amounts of groundwater;
- A statistical analysis conducted on long term yield data from springs show high variability in yield between each year. An obvious trend can be easily deduced in many springs where an increasing yield starts from November/December until May/June;
- Faria spring has the highest mean annual yield amongst the springs of Faria catchment followed by Sedreh, Tabban and Duleib springs. There is a considerable variability in the total annual yield of springs in the order of 39000 ML with a total mean of 13000 ML. Time series analysis of data show that yield is dependent on precipitation.
- The use of reliability test of springs was investigated for Faria catchment. The annual reliability of Faria springs exceeded 50%.

7.2.4 The Planning Model

- A planning model (AGSM) was successfully developed and applied on Faria catchment to simulate the behaviour of farmers and optimally determine the cropping pattern that maximizes the farmer economic return under various management policies and scenarios.
- For Faria catchment, application of the planning model showed a good agreement for the optimal land areas, water use and mix of activities as compared with the corresponding actual values. The differences between actual cultivated and model allocated areas of 3.2%, 2.3% and 1.7% in the cultivated areas in the upper, middle and lower parts of the catchment.
- The lack of a storage dam or reservoir in Faria catchment causes 2600 ML of winter stormwater runoff to be lost and not utilized in the spring, summer and autumn seasons where the demand of water is very high. This is indicated by the model output of high shadow prices of surface water for spring, summer, and autumn, and zero

shadow price for water in winter. This shows the need for a management plan for the catchment that considers such problems.

- Water demand curves that show water demand as a function of water price were developed for Faria catchment. Log of water demand is directly proportional to the log of water price and the gradient of the curve gives the price elasticity.
- A logarithmic approach was considered to obtain the optimal water price that could be charged to farmers as well as price elasticity values. The logarithmic approach resulted in an optimal water price of 0.44 US$/m^3.
- Results from Faria catchment supported the recommendation to use the logarithmic approach to determine the optimum price of water as well as the representative elasticity values at different ranges of irrigation water prices. The logarithmic approach provides the planners and decision makers with a simple and efficient method to derive the optimal price that could be charged to farmers. The advantage of the logarithmic approach method is that it is easier to visualize and hence the planner will be very clearly able to determine the actual value of elasticities that represent the low and high range of water prices as well as critical point where the optimal water price at elasticity equal one with no need to solving or optimizing the equations.
- Obtaining the optimal water price that reflects the economic value of water use and the representative values of price elasticity will facilitate developing the management plans for a catchment. Having an estimate of the optimal water price that could be charged to farmers and the possible response of farmers to changes in water prices will result in optimizing water consumption in agriculture.

7.2.5 Optimal Management of Land and Water

- The optimal allocation of water permits the efficient use of water which in turn maximizes total agricultural income from the catchment.
- The integrated land and water management framework provided a powerful tool for determining the optimal water and land use such that groundwater abstractions are below the safe yield limits.
- The proposed framework integrated successfully the rainfall-runoff and the groundwater models and aided the AGSM effectively in the search process for the optimal management plan that included utilization of Jordan River, changing the cropping pattern by introducing high income crops, building surface water storage (dam) and implementing a groundwater pumping strategy.

- The proposed framework utilized a multi-criteria decision analysis model that is based on the Importance Order of Criteria (IOC) in finding the dominating alternative out of a set of defined alternatives for the study area. The IOC methodology is conceptually simple and provides the decision maker with clear graphical evidence if one alternative is strongly dominant over another. The IOC method provides a logical ranking of the alternatives. The decision criteria cover socio-economic and environmental conditions and the user can prioritize decision criteria;
- The Multi Criteria Decision Support System (MCDSS) tool developed under this study is a straightforward and efficient method for decision analysis and allows for ranking the management alternatives based on the importance order of the decision criteria. The MCDSS provides planners and decision makers with a decision support system to find out optimal management plans for any catchment.
- Detailed and site-specific cost estimates mainly for infrastructural projects affiliated with proposed management plans should be investigated.
- Combining different management alternatives proved to be an efficient approach for maximizing net benefit and satisfying the yield limits. For the study area, a combined management alternative (optimal plan) met the triple bottom line of land use, governmental policies and water resources protection and development to optimize use of land and water resources.
- Managing groundwater pumping strategy satisfied the safe yield as compared to other alternatives. However, it is not efficient to assume this alternative to be effective due to its inefficiency in satisfying the cost and benefit criteria.
- The ranking of the management alternatives indicates the successfulness of the alternatives for a specific order of the criteria. The optimal water and land management obtained from the developed approach are only preliminary and should provide insight for a more exhaustive and comprehensive approaches for site exploration.
- The economic module of the proposed framework assesses the overall utilization management of land and water resources by estimating the net benefit of an alternative;
- Although the management alternatives could not be tested under different long term climatic scenarios within the planning horizon. However the developed system and its components do have the capability of testing the future effect of climatic change when

such a change can be measured or quantified. This opens the door for an area for future research.

- The cost evaluation spreadsheets assumed that the sale price of any additional water resources made available through management alternatives would be 0.20 US$ which might not be an optimal price from an economic point of view. A proposed economic price for such additional water could be the price at which the absolute value of elasticity will be equal to one (0.44 US$/m^3) that was developed in Chapter 5. However, the inability of a major part of agricultural activities to pay high water prices may have undesirable social impacts. For example, shrinking agriculture and economic losses to farmers may lead farmers to leave their district for more industrialized, more populated centers. Such a phenomenon would impose difficulties in many countries, but may be particularly severe for the area studied where a decline of agriculture would cause much unemployment and negative social impact. Whether to avoid such difficulties through subsidizing water for agriculture or other means is of course a matter for national policy the effects and costs of which should be investigated.

7.3 RECOMMENDATIONS

While the framework developed in this work proved to be efficient in optimal water and land utilization the application of the methodology for the study area has some limitations that should be addressed. The following are the major limitations to the current state of this work:

1. The ranking of the management alternatives indicates the successfulness of the alternatives for a specific order of the criteria that is drawn for the specific needs of study area. Accordingly rankings given for the management alternatives for the study area do not indicate an absolute efficiency or deficiency of a management alternative but rather reflects the outcome of the simulation and planning models (MODFLOW, KW-GIUH and AGSM) as well as the order of the decision criteria;
2. The alternatives considered in the analysis did not address the possible rehabilitation of irrigation systems and use of improved irrigation technologies. These were excluded because they contribute minimum amounts of additional water due to the fact that most of the irrigation systems in the study area are drip systems with more than 90% efficiency as given in Chapter 5;

3. The methodology did not account for the stochastic nature of the parameters pertaining to the surface and groundwater models. The selection of the optimal management alternative was based on the best estimates of the parameters of the surface and groundwater models. It is necessary to account for the parameter uncertainty in determining the optimal yield limits of both surface water and groundwater.

Based on the concepts developed, the results demonstrated throughout this research, the discussions presented and the limitations of the current state of the work, the following recommendations might be considered for future research:

1. Management alternatives should be tested and evaluated under possible quantitative climatic change scenarios when developed for the study area;
2. Detailed and site-specific technical studies as well as cost estimates mainly for infrastructural projects affiliated with proposed management plans should be investigated;
3. A more user friendly interfaces should be developed to integrate different modules of the management framework in a fully automated mode;
4. Decision analysis should consider the stochastic variability and uncertainty of the input parameters of the rainfall-runoff and groundwater models.

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APPENDIX I

RAINFALL-RUNOFF MODELLING USING SCS CURVE NUMBER MODEL

The US Department of Agriculture, Soil Conservation Services (SCS) runoff curve number (CN) (SCS, 1972) method was used to calculate the excess rainfall for each event according to the empirical equations given in Equation AI-1 and AI-2

$$P_e = (P - 0.2\ S)^2 / (P + 0.8\ S) \quad \text{for } P > 0.2\ S\text{, otherwise } P=0 \qquad \text{(AI- 1)}$$

$$S = (1000 / CN) - 10 \qquad \text{(AI- 2)}$$

where,

P_e = depth of excess precipitation of the storm (inches)

P = depth of precipitation of the storm (inches)

S = maximum potential retention (inches)

CN= curve number that relates the parameter S to the soil and cover conditions of the catchment.

The major factors that determine CN are the hydrologic soil group, cover type, hydrologic condition and antecedent runoff condition (Maidment, 1993). The values of CN for various land uses on different soil types are given in Tables AI-1 to AI-4 to developed by the SCS (1986) for urban, cultivated agriculture, other agriculture, and arid and semi-arid range land uses. According to SCS (1986) soils are classified into four hydrologic soil groups (A, B, C, and D) based on their infiltration rates and are as follows:

Group A - soils have low runoff potential and high infiltration rates even when thoroughly wetted. They consist chiefly of deep, well to excessively drained sands or gravels.

Group B soils have moderate infiltration rates when thoroughly wetted and consist chiefly of moderately deep to deep, moderately well to well drained soils with moderately fine to moderately coarse textures.

Group C - soils have low infiltration rates when thoroughly wetted and consist chiefly of soils with a layer that impedes downward movement of water and soils with moderately fine to fine texture.

Group D - soils have high runoff potential. They have low infiltration rates when thoroughly wetted and consist mainly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a claypan or clay layer at or near the surface, and shallow soils over a nearly impervious material.

Figure AI-1 shows the soil classification of the Faria catchment (WESI, 2005) that includes six soil types. They are Grumosols (Group C), Terrarosa, Brown Rendzina and Pale Rendzina (Group C), Brown Rendzina and Pale Rendzina (Group B), Loessal Serozems (Group B), Brown Litholsols and loessal arid brown soils (Group C), Regosols (Group D). For the purposes of developing the CN numbers for each subcatchment, ArcView GIS was used to calculate the percentages of each soil type in each of the three subcatchments as shown in Figure AI-2 to AI-4.

Table AI-2 depicts the CNs for cultivated agricultural land. These curve numbers describe the management practices of cultivated agricultural lands. It includes mechanical practices such as contouring and terracing and management practices such as crop rotations and reduced or no tillage. Hydrologic condition indicates the effect of cover type and treatment on infiltration and runoff. Good hydrologic condition indicates that the soil usually has a low runoff potential for that specific hydrologic soil group, cover type and treatment. Some factors to consider in estimating the effect of cover on infiltration and runoff as stated in SCS (1986) are:

(a) canopy or density of lawns, crops, or other vegetative areas;

(b) amount of year-round cover;

(c) amount of grass or close-seeded legumes in rotations;

(d) percent of residue cover; and

(e) degree of surface roughness.

Table AI- 1 Runoff curve numbers for urban areas [1/] (SCS, 1986)

Cover description		Curve numbers for --hydrologic soil group--			
Cover type and hydrologic condition	Average percent impervious area [2/]	A	B	C	D
Fully developed urban areas (vegetation established)					
Open space (lawns, parks, golf courses, cemeteries, etc.) [3/]:					
Poor condition (grass cover < 50%)		68	79	86	89
Fair condition (grass cover 50% to 75%)		49	69	79	84
Good condition (grass cover > 75%)		39	61	74	80
Impervious areas:					
Paved parking lots, roofs, driveways, etc. (excluding right-of-way)		98	98	98	98
Streets and roads:					
Paved; curbs and storm sewers (excluding right-of-way)		98	98	98	98
Paved; open ditches (including right-of-way)		83	89	92	93
Gravel (including right-of-way)		76	85	89	91
Dirt (including right-of-way)		72	82	87	89
Western desert urban areas:					
Natural desert landscaping (pervious areas only) [4/]		63	77	85	88
Artificial desert landscaping (impervious weed barrier, desert shrub with 1- to 2-inch sand or gravel mulch and basin borders)		96	96	96	96
Urban districts:					
Commercial and business	85	89	92	94	95
Industrial	72	81	88	91	93
Residential districts by average lot size:					
1/8 acre or less (town houses)	65	77	85	90	92
1/4 acre	38	61	75	83	87
1/3 acre	30	57	72	81	86
1/2 acre	25	54	70	80	85
1 acre	20	51	68	79	84
2 acres	12	46	65	77	82
Developing urban areas					
Newly graded areas (pervious areas only, no vegetation) [5/]		77	86	91	94

1 Average runoff condition, and Ia = 0.2S.

2 The average percent impervious area shown was used to develop the composite CN's. Other assumptions are as follows: impervious areas are directly connected to the drainage system, impervious areas have a CN of 98, and pervious areas are considered equivalent to open space in good hydrologic condition.

3 CN's shown are equivalent to those of pasture. Composite CN's may be computed for other combinations of open space cover type.

4 The pervious area CN's are assumed equivalent to desert shrub in poor hydrologic condition.

5 Composite CN's to use for the design of temporary measures during grading and construction should be computed based on the degree of development (impervious area percentage) and the CN's for the newly graded pervious areas.

Table AI- 2 Runoff curve numbers for cultivated agricultural lands [1] (SCS, 1986)

Cover description			Curve numbers for --hydrologic soil group--			
Cover type	Treatment [2]	Hydrologic condition [3]	A	B	C	D
Fallow	Bare soil	—	77	86	91	94
	Crop residue cover (CR)	Poor	76	85	90	93
		Good	74	83	88	90
Row crops	Straight row (SR)	Poor	72	81	88	91
		Good	67	78	85	89
	SR + CR	Poor	71	80	87	90
		Good	64	75	82	85
	Contoured (C)	Poor	70	79	84	88
		Good	65	75	82	86
	C + CR	Poor	69	78	83	87
		Good	64	74	81	85
	Contoured & terraced (C&T)	Poor	66	74	80	82
		Good	62	71	78	81
	C&T+ CR	Poor	65	73	79	81
		Good	61	70	77	80
Small grain	SR	Poor	65	76	84	88
		Good	63	75	83	87
	SR + CR	Poor	64	75	83	86
		Good	60	72	80	84
	C	Poor	63	74	82	85
		Good	61	73	81	84
	C + CR	Poor	62	73	81	84
		Good	60	72	80	83
	C&T	Poor	61	72	79	82
		Good	59	70	78	81
	C&T+ CR	Poor	60	71	78	81
		Good	58	69	77	80
Close-seeded or broadcast legumes or rotation meadow	SR	Poor	66	77	85	89
		Good	58	72	81	85
	C	Poor	64	75	83	85
		Good	55	69	78	83
	C&T	Poor	63	73	80	83
		Good	51	67	76	80

1 Average runoff condition, and Ia=0.2S

2 Crop residue cover applies only if residue is on at least 5% of the surface throughout the year.

3 Hydraulic condition is based on combination factors that affect infiltration and runoff, including (a) density and canopy of vegetative areas, (b) amount of year-round cover, (c) amount of grass or close-seeded legumes, (d) percent of residue cover on the land surface (good ≥20%), and (e) degree of surface roughness.

Poor: Factors impair infiltration and tend to increase runoff.

Good: Factors encourage average and better than average infiltration and tend to decrease runoff.

Table AI- 3 Runoff curve numbers for agricultural lands [1] (SCS, 1986)

------------------------------Cover description-----------------------------		Curve numbers for --hydrologic soil group--			
Cover type	Hydrologic condition	A	B	C	D
Pasture, grassland, or range—continuous forage for grazing. [2]	Poor	68	79	86	89
	Fair	49	69	79	84
	Good	39	61	74	80
Meadow—continuous grass, protected from grazing And generally mowed for hay.	___	30	58	71	78
Brush—brush-weed-grass mixture with brush the major element. [3]	Poor	48	67	77	83
	Fair	35	56	70	77
	Good	30 [4]	48	65	73
Woods—grass combination (orchard or tree farm). [5]	Poor	57	73	82	86
	Fair	43	65	76	82
	Good	32	58	72	79
Woods. [6]	Poor	45	66	77	83
	Fair	36	60	73	79
	Good	30 4/	55	70	77
Farmsteads—buildings, lanes, driveways, and surrounding lots.	__	59	74	82	86

1 Average runoff condition, and Ia = 0.2S.

2 Poor: <50%) ground cover or heavily grazed with no mulch.
Fair: 50 to 75% ground cover and not heavily grazed.
Good: > 75% ground cover and lightly or only occasionally grazed.

3 Poor: <50% ground cover.
Fair: 50 to 75% ground cover.
Good: >75% ground cover.

4 Actual curve number is less than 30; use CN = 30 for runoff computations.

5 CN's shown were computed for areas with 50% woods and 50% grass (pasture) cover. Other combinations of conditions may be computed from the CN's for woods and pasture.

6 Poor: Forest litter, small trees, and brush are destroyed by heavy grazing or regular burning.

Fair: Woods are grazed but not burned, and some forest litter covers the soil.
Good: Woods are protected from grazing, and litter and brush adequately cover the soil.

Table AI- 4 Runoff curve numbers for arid and semiarid rangelands [1] (SCS, 1986)

------------------------------Cover description-----------------------------		Curve numbers for --hydrologic soil group--			
Cover type	Hydrologic condition [2]	A [3]	B	C	D
Herbaceous—mixture of grass, weeds, and low-growing brush, with brush the minor element.	Poor		80	87	93
	Fair		71	81	89
	Good		62	74	85
Oak-aspen—mountain brush mixture of oak brush, aspen, mountain mahogany, bitter brush, maple, and other brush.	Poor		66	74	79
	Fair		48	57	63
	Good		30	41	48
Pinyon-juniper—pinyon, juniper, or both; grass understory.	Poor		75	85	89
	Fair		58	73	80
	Good		41	61	71
Sagebrush with grass understory.	Poor		67	80	85
	Fair		51	63	70
	Good		35	47	55
Desert shrub—major plants include saltbush, greasewood, creosotebush, blackbrush, bursage, palo verde, mesquite, and cactus.	Poor	63	77	85	88
	Fair	55	72	81	86
	Good	49	68	79	84

1 Average runoff condition, and Ia, = 0.2S.
2 Poor: <30% ground cover (litter, grass, and brush overstory).
Fair: 30 to 70% ground cover.
Good: > 70% ground cover.
3 Curve numbers for group A have been developed only for desert shrub.

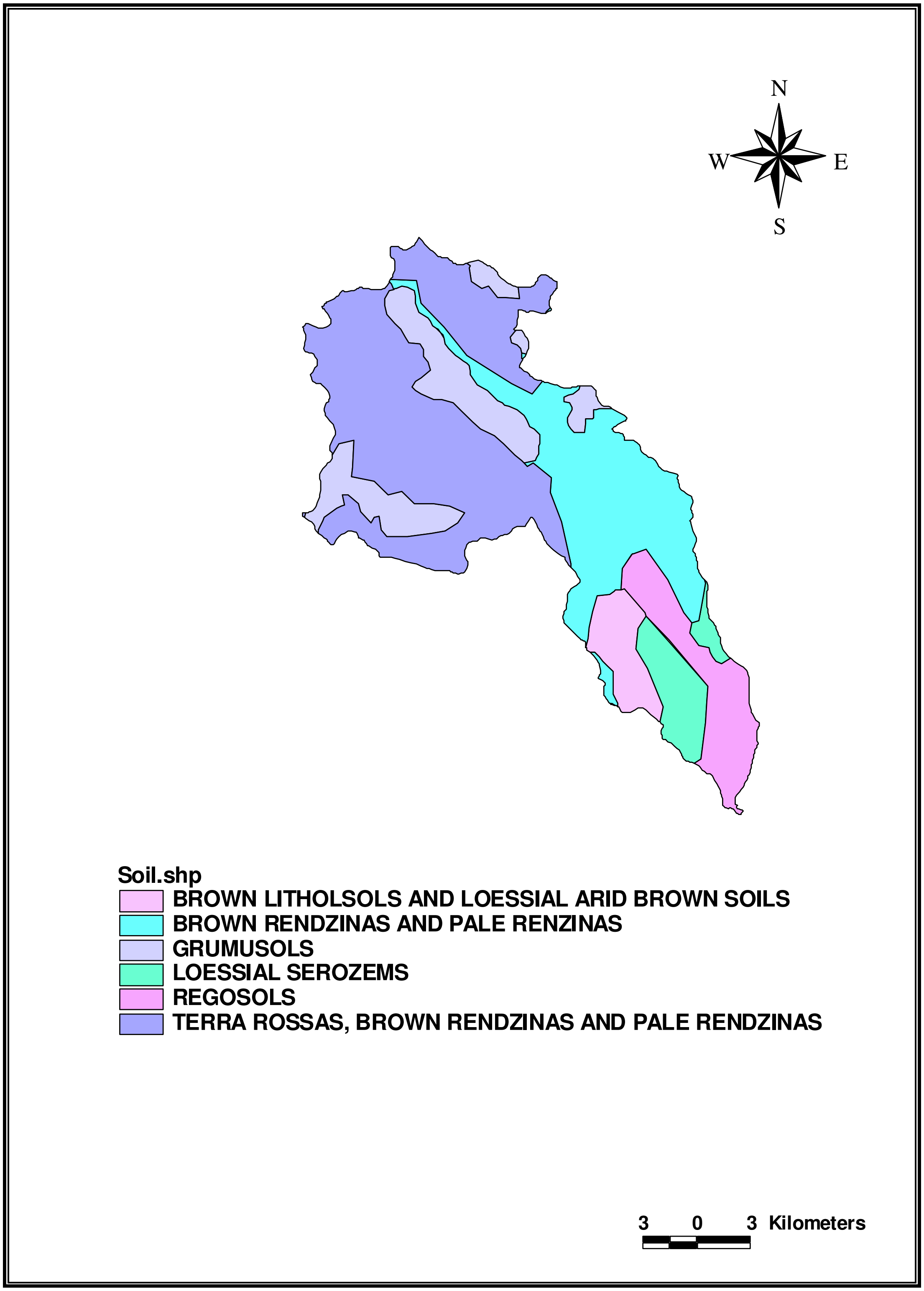

Figure AI- 1 Soil classification of the Faria catchment

Figure AI- 2 Soil classification of Badan Subcatchment as prepared using ArcView GIS capabilities

Figure AI- 3 Soil classification of Faria Subcatchment as prepared using ArcView GIS capabilities

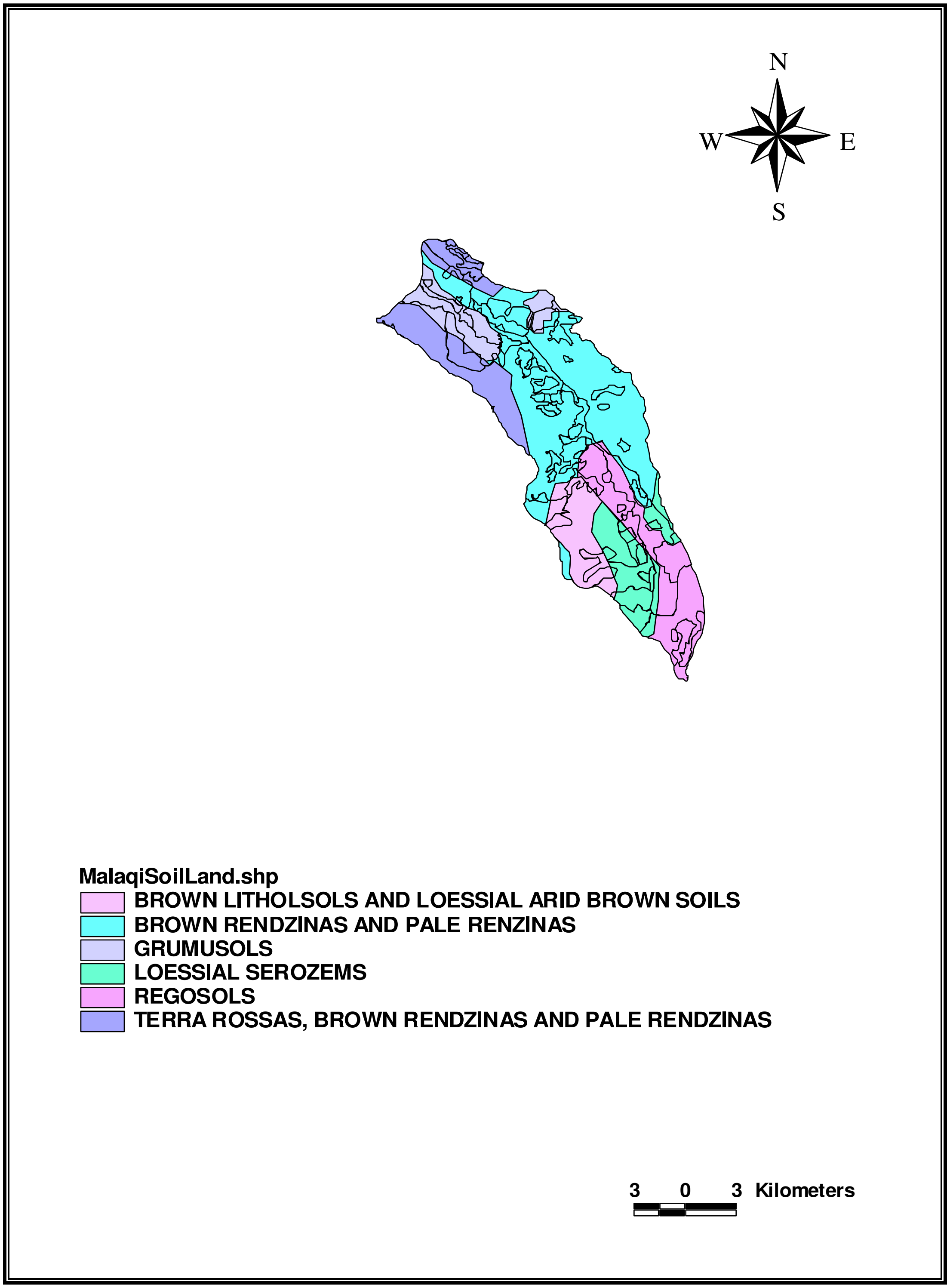

Figure AI- 4 Soil classification of Malaqi Subcatchment as prepared using ArcView GIS capabilities

Landuse maps are also needed to develop the CN numbers for each subcatchment. Figure AI-5 shows the landuse map of the Faria catchment (WESI, 2005). Ground truthing was conducted to verify the landuse map. ArcView GIS was used to calculate the percentages of each type of landuse on each soil type in each of the three subcatchments as shown in Figures AI-6 to AI-8. The land use map identifies seventeen categories (coded as numbers). They are: Non-irrigated arable land (1), Land principally occupied by agriculture (2), Citrus plantations (3), Coniferous forests (4), Natural grassland (5), Olive groves (6), Refugee camps (7), Israeli colonies (8), Urban Fabrics (9), Military camps (10), Bare rock (11), Drip-irrigated arable land (12), Irrigated and non-irrigated complex cultivated pattern (13), Sparsely vegetated area (14), Palm groves (15), Halophytes (16), Water bodies/Artificial surfaces (17).

The composite CNs for each of the three subcatchments were developed based on the soil and landuse maps and information provided in Tables AI-1 to AI-4. Tables AI-5 to AI-7 show the weighted CN for each soil category and landuse in each subcatchment calculated based on Equation AI-3. The weighted CNs were 75.21, 74.40 and 71.88 for Badan, Faria and Malaqi subcatchments respectively.

$$CN_{weighted} = \sum_{i=1}^{n} (CN_i * A_i)/A_n \qquad \text{(AI-3)}$$

where,

$CN_{weighted}$ = Weighted average CN

$CN_{i\,=}$ CN for area i

A_i = Area i representing the landuse-soil category (du)

A_n = Total catchment area (du)

The index of runoff potential before a storm event is the antecedent runoff condition (ARC). The ARC is an attempt to account for the variation in CN at a site depending on the soil dryness. The CNs derived from SCS Tables AI-1 to AI-4, represent the average ARC (CNII where the 5-day antecedent rainfall is between 12.7mm to 27.9mm), which is used primarily for design applications. For dry conditions (CNI where the 5-day antecedent rainfall is between less than 12.7mm) or wet conditions (CNIII where the 5-day antecedent rainfall is between more than 27.9mm) equivalent curve numbers can be computed (SCS, 2004, Chow et al., 1988) using Equations AI-3 and AI-4,

$$CNI = 4.2\ CNII / (10 - 0.058\ CNII) \qquad \text{(AI- 3)}$$

$$CNIII = 23\ CNII / (10 + 00.13\ CNII) \qquad \text{(AI- 4)}$$

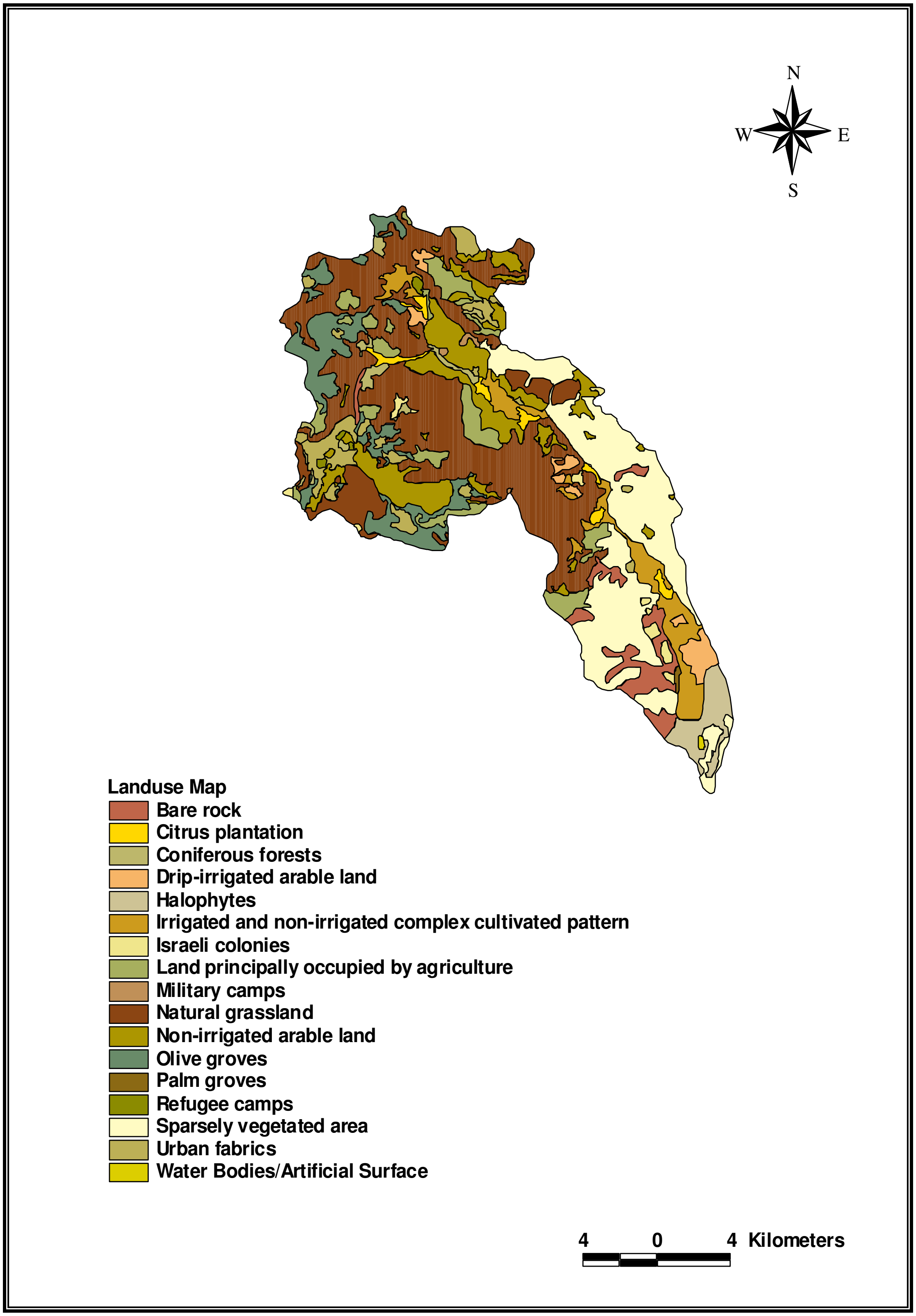

Figure AI- 5 Landuse map of Faria catchment

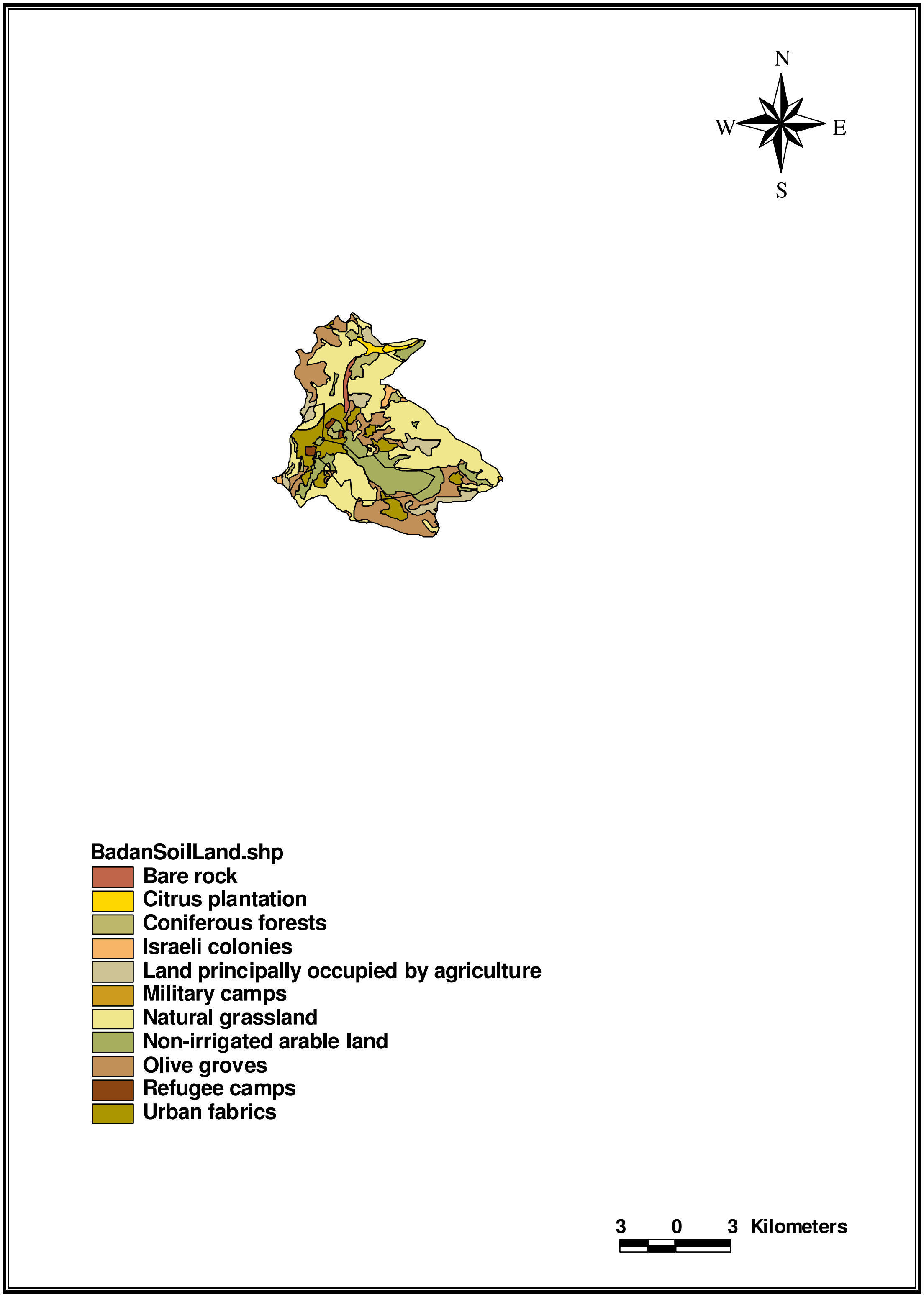

Figure AI- 6 Landuse map of Badan Subcatchment as prepared using ArcView GIS capabilities

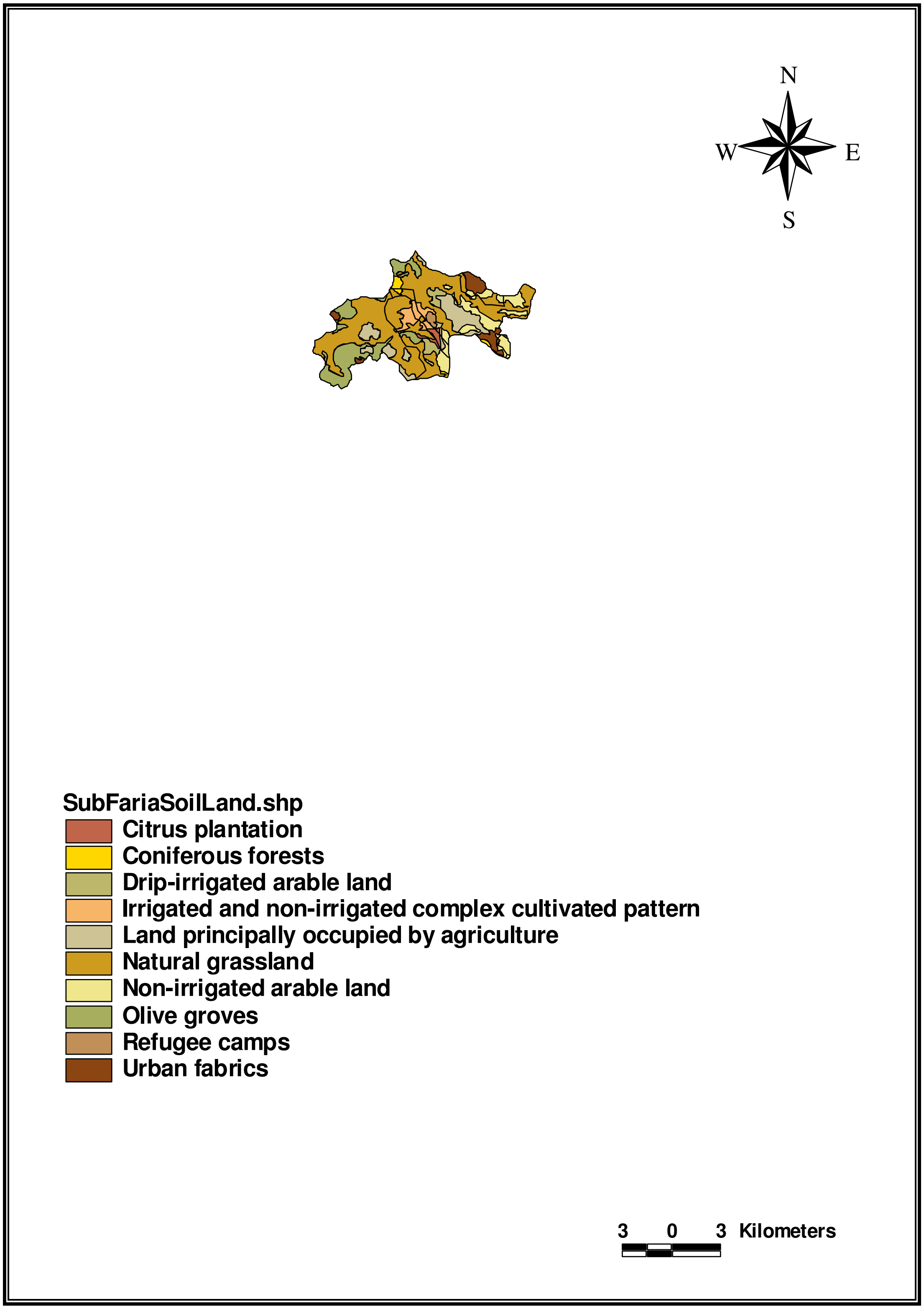

Figure AI- 7 **Landuse map of Faria Subcatchment as prepared using ArcView GIS capabilities**

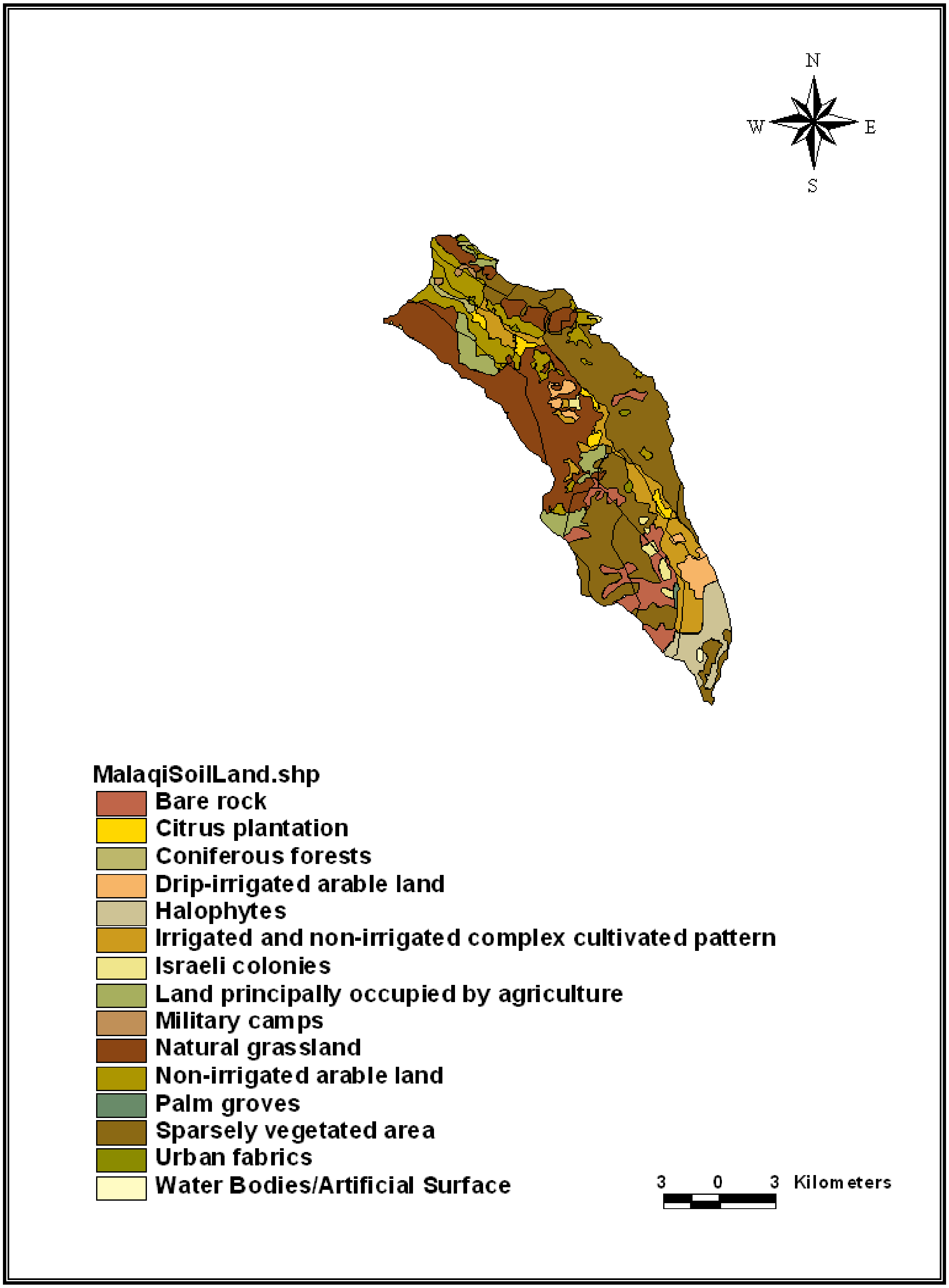

Figure AI- 8 Landuse map of Malaqi Subcatchment as prepared using ArcView GIS capabilities

Table AI- 5 Composite CN for Badan Subcatchment

Grumosols, C		
Landuse	**Area**	**CN**
1	6880	78
2	620	78
3	454	72
4	371	70
5	6240	74
6	1219	72
7	248	98
8	186	79
9	3532	79
Terrarosa,Brown Rendzina and Pale Rendzina, C		
Landuse	**Area**	**CN**
1	5890	78
2	4841	78
3	624	72
4	2118	70
5	29693	74
6	15027	72
7	262	98
8	624	79
9	5345	79
10	39	79
11	786	91
Weighted CN Badan		**75.21**

Table AI- 6 Composite CN Faria Subcatchment

Brown Rendzina and Pale Rendzina, B		
Landuse	**Area**	**CN**
1	561	71
2	331	71
3	528	58
4	198	55
5	1224	61
13	1058	67
Grumosols C		
Landuse	**Area**	**CN**
1	2494	78
2	22	78
3	295	72
4	113	70
5	6985	74
6	612	72
9	1428	79
12	838	76
13	1315	76
Terrarosa,Brown Rendzina and Pale Rendzina C		
Landuse	**Area**	**CN**
1	4308	78
2	6462	78
3	41	72
4	580	70
5	23551	74
6	7933	72
7	414	98
9	1429	79
12	787	76
13	497	76
Weighted CN Faria		**74.40**

Table AI- 7 Composite CN Malaqi Subcatchment

Grumosols, C		
Landuse	**Area**	**CN**
1	6398	78
2	914	78
3	548	72
4	853	70
5	2803	74
9	81	79
10	143	79
13	2132	76
14	1889	74
Loessal Seozems, B		
Landuse	**Area**	**CN**
8	1195	69
11	5816	86
12	567	67
13	2756	67
14	8430	62
15	365	58
16	771	75
Brown Litholsols and loessal arid brown soils, C		
Landuse	**Area**	**CN**
2	1099	78
5	936	74
11	3743	91
14	10883	74
Regosols, D		
Landuse	**Area**	**CN**
2	40	81
3	101	79
5	201	80
8	464	84
9	262	84
11	1733	94
12	3206	80
13	7500	80
14	8265	85
16	8104	89
17	302	81
Brown Rendzina and Pale Rendzina, B		
Landuse	**Area**	**CN**
1	9071	71
2	3064	71
3	2142	58
5	21746	61
8	500	69
9	401	69
10	141	69
11	841	86
12	1542	67
13	4605	67
14	31678	62
Terrarosa.Brown Rendzina and Pale Rendzina. C		
Landuse	Area	CN
1	1736	78
2	3452	78
4	202	70
5	19159	74
8	61	79
9	343	79
10	323	79
14	1494	74
Weighted CN Malaqi		**71.88**

Table AI- 8 The CNs and the maximum retention S (inches) values for each subcatchment under different moisture conditions

Subatchment	CNI		CNII		CNIII	
	CNI	S(in)	CNII	S(in)	CNIII	S(in)
Badan	56.03	7.848	75.21	3.30	87.46	1.434
Faria	54.97	8.19	74.40	3.441	86.99	1.496
Malaqi	51.77	9.316	71.88	3.912	85.46	1.70

In application of SCS CN method, the CN for each subcatchment under different moisture conditions need to be calculated. Based on the CN numbers given in Table AI-8 the maximum retention S values (Equation AI-2) were estimated. For each rainfall event the 5-day antecedent rainfall was determined to decide on the antecedent moisture conditions. Based on this the corresponding CN and S values were used to calculate the effective rainfall for each event (Equation AI-1) and ultimately the total volume of runoff.

Table AI-9 shows the results of the excess rainfall estimated by the SCS method for each rainfall event in Badan and Faria subcatchments. The amounts of runoff for each event and the total monthly volumes were estimated. Table AI-10 compares the monthly total volumes with the estimated surface runoff (after separating the baseflow as detailed in Section 3.4.1). Based on SCS method for Malaqi Subcatchment excess rainfall was generated only from one event. There was no flow measurement data for Malaqi Subcatchment to verify the SCS simulations. However observations from the field did not indicate any runoff from this subcatchment for many years.

The results showed that the SCS method overestimated the runoff amounts and is not suitable to be used for the estimation of excess rainfall for the study area. These results agree with other published research related to the use of SCS method (Mishra and Singh, 2004; SCS, 2004; Maidment, 1993). The SCS relationship generally did reasonably well where the runoff was a substantial fraction of the rainfall, but poorly in cases where the runoff was a small fraction of the rainfall; i.e., the CNs are low or rainfall values are small (SCS, 2004; Maidment, 1993). Curve numbers were originally developed from annual flood flows from experimental catchments in the United States, and their application to low flow conditions or for small peak flows is not recommended (SCS, 2004). Mishra and Singh (2004) applied the SCS CN method to a large set of event data in the Amicolala Creek catchment, the SCS CN method generally overestimates the runoff (Mishra and Singh, 2004).

Table AI- 9 Excess rainfall estimated by the SCS method for each rainfall event in Badan and Faria subcatchments

Subcatchment	Date	Event Rainfall (mm)	Excess Rainfall-SCS CN (mm)
Badan	17/11/04	21.8	0.00
	21/11	40.62	5.29
	26/11	46.85	20.60
	7/12	12.21	0.00
	24/12	19.28	0.00
	2/01/05	27.46	0.00
	15/01	15.16	0.00
	19/01	21.51	0.25
	23/01	27.08	6.97
	1/02	42.85	0.04
	4/02	185.9	148.36
	11/02	28.73	7.94
	10/03	11.24	0.00
Faria	17/11/04	24.37	0.00
	21/11	55.05	11.30
	26/11	48.19	20.98
	7/12	15.08	0.00
	24/12	26.19	0.00
	2/01/05	36.48	0.00
	15/01	20.31	0.00
	19/01	28.9	1.32
	23/01	21.8	3.87
	1/02	37.08	0.00
	4/02	123.4	87.21
	11/02	24.91	5.42
	10/03	17.38	0.00
Malaqi	17/11/04	10.46	0
	21/11	18.5	0
	26/11	20.48	0
	7/12	5.2	0
	24/12	9.6	0
	2/01/05	14.44	0
	15/01	7.88	0
	19/01	11.08	0
	23/01	18.41	0
	1/02	25.85	0
	4/02	88.98	28.35
	11/02	9.78	0
	10/03	8.01	0

Table AI- 10 The monthly total surface runoff volumes as compared with observed runoff

Subcatchment	Month	SCS-CN		Estimated Surface Runoff (m3)	% Error
		Excess Rainfall (mm)	Simulated Runoff (m3)		
Badan	November	25.89	2200650	10800	99.5
	December	0	0	56200	100
	January	7.22	613700	175400	71.4
	February	156.3	13285500	1373700	89.7
	March	0	0	213000	100
Total		189.46	16099850	1829100	88.6
Faria	November	32.28	2065920	3600	99.8
	December	0	0	131100	100
	January	5.19	332160	93400	71.9
	February	92.63	5928320	388400	93.4
	March	0	0	171800	100
Total		130.1	8326400	788300	90.5

APPENDIX II

KINEMATIC-WAVE GIUH MODEL

AII.1 TRAVEL-TIME ESTIMATION (Lee and Yen, 1997)

In the kinematic-wave simulation of the surface-runoff process resulting from rainfall excess, an ith-order subbasin of the catchment is conceptually simplified as consisting of two identical rectangular overland-flow planes. Each plane contributes a lateral discharge into a channel of constant cross section and slope (Figure AII- 1). The mean length of the ith-order V-shape overland- flow planes is

$$\bar{L}_{oi} = \frac{AP_{OAi}}{2N_i \bar{L}_{ci}} \qquad \text{(AII- 1)}$$

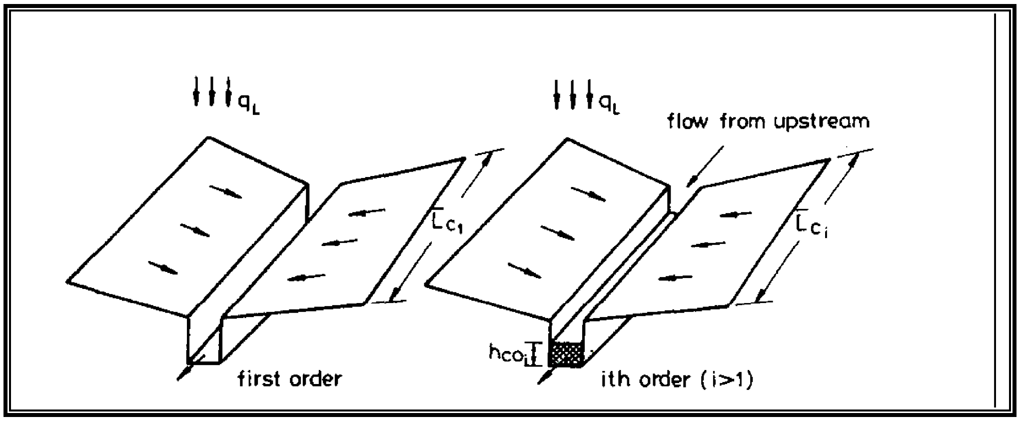

Figure AII- 1 Schematic Diagram of the V-Shape Subbasins (Lee and Yen, 1997)

The travel time for the ith-order overland plane is

$$Tx_{oi} = \left(\frac{n_o \bar{L}_{oi}}{\bar{S}_{Oi}^{1/2} \, q_L^{m-1}} \right)^{1/m} \qquad \text{(AII- 2)}$$

The depth of water at the entrance of the ith-order channel is

$$h_{coi} = \left[\frac{q_L n_c (N_i \bar{A}_i - AP_{OAi}}{N_i B_i \bar{S}_{ci}^{1/2}} \right]^{1/m} \qquad \text{(AII- 3)}$$

The travel time for the ith-order channel is

$$T_{x_i} = \frac{B_i}{2 q_L \bar{L}_{oi}} \left[\left(h_{coi}^{m} + \frac{2 q_L n_c \bar{L}_{oi} \bar{L}_{ci}}{B_i \bar{S}_{ci}^{1/2}} \right)^{1/m} - h_{coi} \right] \qquad \text{(AII- 4)}$$

The ith-order channel width is

$$B_i = \frac{B_\Omega \sum_{l=1}^{i} \bar{L}_{c_l}}{\sum_{l=1}^{\Omega} \bar{L}_{c_l}} \qquad \text{(AII- 5)}$$

where,

- T_{x_i} is the rainwater travel time for the ith order channel
- $T_{x_{oi}}$ is the travel time through the ith order overland plane
- x_{oi} denote the *i*th-order overland flow regions
- x_i denote the *i*th-order channels
- $i=1, 2, \ldots, \Omega$
- i is the channel order
- Ω is the maximum number of channel order
- h_{co_i} is the depth of water at the entrance of the ith-order channel
- $\bar{L}_{oi}$ is the mean length of the *i*th-order overland flow planes
- N_i is the number of *i*th-order channels
- $\bar{L}_{ci}$ is the mean *i*th-order stream length
- $\bar{A}_i$ is the *i*th-order sub watershed contributing area
- P_{OA_i} is the ratio of *i*th-order overland area to the watershed area
- $\bar{S}_{ci}$ is the mean *i*th-order channel slope
- $\bar{S}_{oi}$ is the mean *i*th-order overland slope
- n_o is the overland flow roughness
- n_c is the channel flow roughness
- B_i is the *i*th-order channel width
- q_L is the intensity of rainfall excess
- B_Ω is the channel width at watershed outlet
- m is an exponent
- A is the catchment area

AII.2 GEOMORPHIC INSTANTANEOUS UNIT HYDROGRAPH STRUCTURE (Lee and Yen, 2000)

When a unit depth of excess rain falls uniformly and instantaneously onto a catchment, the unit rainfall excess is assumed to consist of a large number of independent, noninteraction raindrops. Thus, the whole rainfall-runoff process can be represented by tracing the rainfall excess moving along different paths towards the catchment outlet to produce the outflow hydrograph. The amount of rain that falls directly onto the channels is small compared to that falling on overland areas and can therefore be neglected.

Based on the Strahler (Strahler, 1957) stream-ordering scheme, a catchment of order Ω can be divided into different states. The ith-order overland region and channel is denoted by x_{oi}, and x_i in which i = 1, 2,…, Ω. Each raindrop falling on the overland region will move successively from lower to higher order channels until it reaches the outlet. Figure AII- 2 depicts schematically the network ordering scheme according to the Strahler method. It also shows the possible travel paths of the raindrops for a third-order catchment. The following presentation of the GIUH is based on previous results of Rodriguez-Iturbe and Valdes (1979) and Gupta et al. (1980), in which the catchment geomorphology is represented probabilistically based on the stream order, instead of simulating the overland surfaces and channels by their individually actual geometry as in a deterministic modeling. If a specified flow path is from $x_{oi} \rightarrow x_i \rightarrow x_j \rightarrow \ldots x_\Omega$, the probability of a drop of rainfall excess adopting this path can be expressed as given in Equation AII-6.

$$P(w) = P_{OA_i}.P_{x_{oi}x_i}...P_{x_ix_j}...P_{x_kx_\Omega} \qquad \text{(AII- 6)}$$

where

- w = specified flow path
- P(w) = probability of a drop of rainfall excess adopting this path
- i, j, … k, Ω are stream order numbers
- $Px_{oi}x_j$ = transitional probability of the raindrop moving from the ith-order overland region to the ith-order channel; and
- Px_ix_j is the transitional probability of the raindrop moving from an ith-order channel to a jth-order channel and is computed as given in Equation AII-7.

$$P_{x_i xj} = \frac{N_{i,j}}{N_i} \qquad \text{(AII- 7)}$$

where $N_{i,j}$ is the number of the ith-order channels contributing to jth-order channels and other variables are as defined earlier.

Using the travel time equations above, the Laplace Series is used to estimate the instantaneous unit hydrograph of the catchment which can be expressed (Rodriguez-Iturbe and Valdes, 1979; Gupta et al., 1980) as given in Equation AII-8.

$$u(t) = \sum_{w \in W} [f_{x_{oi}}(t) * f_{x_i}(t) * f_{x_j}(t) * \ldots * f_{x_\Omega}(t)]_w . P(w) \qquad \text{(AII- 8)}$$

where,

- $w \in W$, W is the path space given as $\langle x_{oi},\ x_i, x_j, \ldots, x_\Omega \rangle$
- * denotes a convolution integral
- $f_{x_j}(t)$ is the travel-time probability-density function in state x_j with a mean value of T_{xj}

The mean of the drainage area of order i $(\overline{A}_i)$ and the ratio of *i*th-order overland area to the catchment area (P_{OA_i}) are estimated using the two Equations AII-9 and AII-10 respectively. P_{OA_i} is used to calculate P(w).

$$\overline{A}_i = \frac{1}{N_i} \sum_{j=1}^{N_i} A_{ij} \qquad \text{(AII- 9)}$$

$$P_{OA_i} = \frac{1}{A}\left(N_i \overline{A}_i - \sum_{l=1}^{i-1} N_l \overline{A}_l P_{x_l x_i} \right) \qquad \text{(AII- 10)}$$

where,

- $\overline{A}_i$ is the ith-order sub catchment contributing area
- P_{OA_i} is the ratio of ith-order overland area to the catchment area

- $P_{x_i x_j}$ is the stream network transitional probability
- N_i is the number of ith-order channels
- A_{ji} is the area of the overland flow regions that drains directly into the jth channel of order i, and also includes overland areas draining into the lower order channels tributary to this jth channel of order i
- A is the catchment area

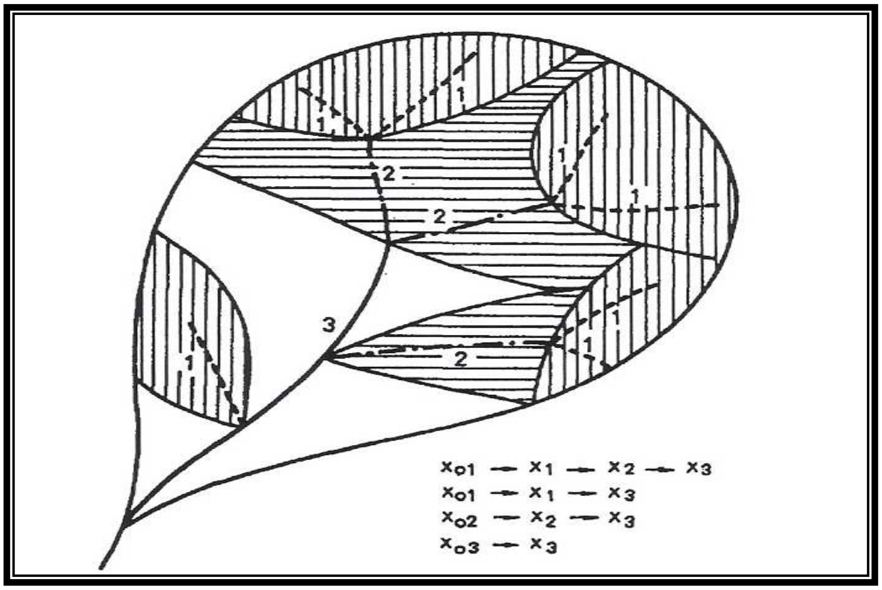

Figure AII- 2 Flow paths of third-order catchment wth Strahler stream-ordering system (Lee and Yen, 1997)

The determination of travel time in each state is the most difficult problem in the GIUH approach (Rodriguez-Iturbe and Valdes, 1979; Gupta et al., 1980). The kinematic wave approach is used to estimate the travel time as explained earlier. As applied in a linear response system, the system output generated by using the KW-GIUH model can be determined using the convolution integral of the rainfall input and the IUH (Equation AII-8), which can be expressed as in Equation AII-11 (Lee and Yen, 2000). In traditional hydrology the IUH of a catchment is unique. However, the IUH u(t) in Equation AII-11 temporally varies with the rainfall excess intensity. The dynamic nature of the catchment hydrologic response function is regarded as the major merit of the KW-GIUH model (Lee and Yen, 2000).

$$Q(t) = \int_0^t R_e(\tau)u(t-\tau)d\tau \qquad \text{(AII- 11)}$$

where,

Q=the direct runoff at the catchment outlet

R_e=the input (rainfall excess) of the catchment

$u(t)$=the IUH generated by KW-GIUH model, and

τ = a dummy variable

APPENDIX III

MULTI CRITERIA DECISION SUPPORT SYSTEM (MCDSS) CODES

```
/************************************************************

Program:  MCDSS.c
Author: Ammar Jarrar
Date:2/1/07

*************************************************************/

/*-------------------- PREPROCESSING DIRECTIVE --------------------*/

#include"stdafx.h"
#include<stdio.h>
#include<stdlib.h>


/*-------------------- FUNCTION PROTOTYPES ------------------------*/

  int LoadArray(int crtNum, int altNum);            /*load data into
arrays*/

/*----------------------- PROGRAM SETEP---------------------------*/

    /*> P R O G R A M   V A R I A B L E S   <*/
```

```
    char crtName[100][20];     /* array for 100 criteria and 20 charater */
    float altArray[100][100];     /* array for 100 X 100 alternative */
    float altArray2[100][100];
    float altArray3[100][100];
    int crtType[100];
    int crtNum ;    /* number of criteria */
    int altNum;     /* number of alternatives */
    float minArray[100];
    float maxArray[100];
    float minArray2[100];
    float maxArray2[100];
    float avg[100];
    int rank[100];
    float min;
    float max;
    float sum;


/*
-----------------------------------------------------------------------------
                     LOAD DATA INTO ARRAYS

-----------------------------------------------------------------------------*/

int LoadArray(int crtNum, int altNum)
{
    int iRow;       /* row subscript */
    int iCol;       /* colum subscript */
```

```
    int Row;
    int Col;
    int swap;
    float temp;
    int temp1;
    printf("**** Enter Criteria in order of importance ****\n");
    for(iRow=0; iRow<crtNum; iRow++)
    {
        printf("Enter Criteria[%d]: ",iRow+1);
        scanf("  %20[^\n]", crtName[iRow]);
        fflush(stdin);

        printf("Enter Criteria type [1] for BENEFIT  [0] for COST: ");
        scanf("%d",&crtType[iRow]);

        for (iCol=0; iCol<altNum; iCol++)
        {
           printf("Enter Alternative[%d]: ",iCol+1);
           scanf("%f",&altArray[iRow][iCol]);
           fflush(stdin);
        }
    printf("\n");
    }
/*
-----------------------------------------------------------------------------
                     print DATA INTO ARRAYS

-----------------------------------------------------------------------------*/
```

```
        printf("\n\n>>> Decision Criteria Values for Different Alternatives <<<<\n");

        printf("===========================================================================\n");
        printf("CRITERIA                TYPE        ALTERNATIVES\n");
        printf("                                    ");
        for (iCol=0; iCol<altNum; iCol++)
        {
                printf("%-8d",iCol+1);
        }
        printf("\n");

        printf("===========================================================================\n");
        for(iRow=0; iRow<crtNum; iRow++)
        {
            printf("%-20s",crtName[iRow]);
            printf("%5d           ",crtType[iRow]);
            for (iCol=0; iCol<altNum; iCol++)
            {
                printf("%-6.3f  ",altArray[iRow][iCol]);
            }
            printf("\n");
        }

/*
-----------------------------------------------------------------------------
                    first calculation

-----------------------------------------------------------------------------*/
```

```
    for(Row=0; Row<crtNum; Row++)
    {
        min=altArray[Row][0];
        max=altArray[Row][0];
        for (Col=1; Col<altNum; Col++)
        {
            if(min>altArray[Row][Col])
                min=altArray[Row][Col];

            if(max<altArray[Row][Col])
                max=altArray[Row][Col];
        }
        minArray[Row]=min;
        maxArray[Row]=max;
    }

    /*   printf("\nthe min Array -- max Array \n");
       printf("------------------------------\n");
         for(Row=0; Row<crtNum; Row++)
           printf("%4.3f    %4.3f\n",minArray[Row],maxArray[Row]);*/




/*
-------------------------------------------------------------------------------------*/
    for(iRow=0; iRow<crtNum; iRow++)
```

```
    {
        for (iCol=0; iCol<altNum; iCol++)
        {
            if  (crtType[iRow]==1 )
                {
                altArray2[iRow][iCol]=(altArray[iRow][iCol]- minArray[iRow])/(maxArray[iRow]-minArray[iRow]);
            }

            if (crtType[iRow]==0 )
            {
                altArray2[iRow][iCol]=(altArray[iRow][iCol]- minArray[iRow])/(minArray[iRow]-maxArray[iRow])+1;
            }

        }
    }
/*
-----------------------------------------------------------------------------
                     print  ARRAYS 2

-----------------------------------------------------------------------------*/

    printf("\n\n>>>> Standarized Decision Criteria Values for Different Alternatives <<<<\n");
    printf("===========================================================================\n");
    printf("CRITERIA                TYPE        ALTERNATIVES\n");
    printf("                                    ");
    for (iCol=0; iCol<altNum; iCol++)
    {
        printf("%-8d",iCol+1);
```

```
    }
    printf("\n");
    printf("---------------------------------------------------------------------------\n");
    for(iRow=0; iRow<crtNum; iRow++)
    {
        printf("%-20s",crtName[iRow]);
        printf("%5d           ",crtType[iRow]);
        for (iCol=0; iCol<altNum; iCol++)
        {
            printf("%-6.3f  ",altArray2[iRow][iCol]);
        }
        printf("\n");
    }

/*
-----------------------------------------------------------------------------
                    second calculation

-----------------------------------------------------------------------------*/
    for(Row=0; Row<altNum; Row++)
    {
        sum=0;
        for (Col=0; Col<crtNum; Col++)
        {
            sum= sum + altArray2[Col][Row];
            altArray3[Col][Row]= sum/(Col+1);
        }
    }
```

```
/*
---------------------------------------------------------------------------
                     print  ARRAYS 3

-----------------------------------------------------------------------------*/


    printf("\n\n>>>> Utility Scores for Decision Criteria for Different Alternatives <<<<\n");

    printf("===========================================================================\n");
    printf("CRITERIA                ALTERNATIVES\n");
    printf("                        ");
    for (iCol=0; iCol<altNum; iCol++)
    {
        printf("%-8d",iCol+1);
    }
    printf("\n");
    printf("***************************************************************************\n");
    for(iRow=0; iRow<crtNum; iRow++)
    {
    if (iRow == 0)
        printf("A[1]                    ");
    else
        printf("A[1 - %3d]              ",iRow+1);

    for (iCol=0; iCol<altNum; iCol++)
    {
        printf("%-6.3f  ",altArray3[iRow][iCol]);
```

```
    }
    printf("\n\n");
    }

/*----------------------------------------------------------------------------------
                              last result
-----------------------------------------------------------------------------------*/


    for(Row=0; Row<altNum; Row++)
    {
        min=altArray3[0][Row];
        max=altArray3[0][Row];
        for (Col=1; Col<crtNum; Col++)
        {
        if(min>altArray3[Col][Row])
            min=altArray3[Col][Row];

        if(max<altArray3[Col][Row])
            max=altArray3[Col][Row];
        }
        rank[Row]=Row+1;
        minArray2[Row]=min;
        maxArray2[Row]=max;
        avg[Row]=(min+max)/2;
    }
      printf("\n\n>>>> Minimum, Maximum & Avarege Utility Score Values for  Different Alternatives <<<<\n");
      printf("===========================================================================\n");
```

```
  printf("ALTERNATIVES       MIN      MAX      AVERAGE\n");
  printf("-----------------------------------------------\n");
for(Row=0; Row<altNum; Row++)
{
    printf("Alternative[%d]     %4.3f     %4.3f    %4.3f\n",rank[Row], minArray2[Row],maxArray2[Row],avg[Row]);
}

swap=1;
while (swap == 1)
{
    swap=0;
    for (Row=0; Row<altNum; Row++)
    {
        if (avg[Row] < avg[Row+1])
        {
            temp=avg[Row];
            temp1=rank[Row];
            avg[Row]=avg[Row+1];
            rank[Row]=rank[Row+1];
            avg[Row+1]=temp;
            rank[Row+1]=temp1;
            swap=1;
        }
    }
}
printf("\n\n >>>> RANKING OF ALTERNATIVES <<<<\n");
printf("**************************************\n");
for (Row=0; Row<altNum; Row++)
```

```
        printf( "Alternative [%d]     %4.3f\n", rank[Row], avg[Row]);
    return (0);
}


/*-----------------------------------------------------------------------------
                      MAINLINE CONTROL

------------------------------------------------------------------------------*/

int main(int argc, char* argv[])
{
    printf("\n");
    printf("*-------------------------------------------------* \n");
    printf(" Enter number of Criteria: ");
    scanf("%d",&crtNum);
    printf(" Enter number of Alternatives: ");
    scanf("%d",&altNum);
    printf("*-------------------------------------------------* \n\n");

    LoadArray(crtNum,altNum);


    getchar();
      return 0;
}
```

APPENDIX IV

COST ANALYSIS SPREADSHEET FOR ALTERNATIVE 3 (ARTIFICIAL RECHARGE OF SURFACE RUNOFF AND TREATED WASTEWATER)

year	flow mcm/yr	capital cost M$	O&M cost M$	PWF(i=10%)	Present worth value			
					capital(M$)	O&M(M$)	Subtotal(M$)	Flow(mcm/yr)
1								
2		5		0.909090909	4.545454545		4.545454545	
3		12		0.826446281	9.917355372		9.917355372	
4		12		0.751314801	9.015777611		9.015777611	
5		5		0.683013455	3.415067277		3.415067277	
6	4		1	0.620921323	0	0.62092132	0.620921323	2.483685292
7	4		1	0.56447393	0	0.56447393	0.56447393	2.25789572
8	4		1	0.513158118	0	0.51315812	0.513158118	2.052632473
9	4		1	0.46650738	0	0.46650738	0.46650738	1.866029521
10	4		1	0.424097618	0	0.42409762	0.424097618	1.696390473
							29.48281317	10.35663348
							COST	27.41148648
							UCW	2.646756452

APPENDIX V

DATA INPUT-OUTPUT FILES FOR THE MCDSS

Table AV- 1 Data input file for the MCDSS for the case study

```
file:mcds1.c1
*--------------------------------------------------*
Enter number of Criteria: 11
Enter number of Alternatives: 13
*--------------------------------------------------*

**** Enter Criteria in order of importance ****

Enter Criteria[1]: SY
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]:  3213
Enter Alternative[2]:  3225
Enter Alternative[3]:  3225
Enter Alternative[4]:  2362
Enter Alternative[5]:  3225
Enter Alternative[6]:  0
Enter Alternative[7]:  2424
Enter Alternative[8]:  2369
Enter Alternative[9]:  0
Enter Alternative[10]: 0
Enter Alternative[11]: 973
Enter Alternative[12]: 0
Enter Alternative[13]: 0

Enter Criteria[2]: NB
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  15175000
Enter Alternative[2]:  17385000
Enter Alternative[3]:  19126000
Enter Alternative[4]:  19415000
Enter Alternative[5]:  17012000
Enter Alternative[6]:  15561000
Enter Alternative[7]:  28823000
Enter Alternative[8]:  21119000
Enter Alternative[9]:  20977000
Enter Alternative[10]: 18724000
Enter Alternative[11]: 32963000
Enter Alternative[12]: 32946000
Enter Alternative[13]: 32910000

Enter Criteria[3]: TIL
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  3357
Enter Alternative[2]:  3330.7
Enter Alternative[3]:  4319.6
Enter Alternative[4]:  4408
Enter Alternative[5]:  3471.1
Enter Alternative[6]:  3028.4
Enter Alternative[7]:  5052.1
Enter Alternative[8]:  4363.1
Enter Alternative[9]:  4145.9
Enter Alternative[10]: 3731.2
Enter Alternative[11]: 5052.1
Enter Alternative[12]: 5052.1
Enter Alternative[13]: 5030.8
```

Data input file for the MCDSS for the case study…Continued

```
Enter Criteria[4]: COST
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]:  0
Enter Alternative[2]:  1.20E+6
Enter Alternative[3]:  2.74E+7
Enter Alternative[4]:  5.66E+7
Enter Alternative[5]:  3.12E+6
Enter Alternative[6]:  0
Enter Alternative[7]:  27460000
Enter Alternative[8]:  5.78E+7
Enter Alternative[9]:  5.78E+7
Enter Alternative[10]: 30530000
Enter Alternative[11]: 85260000
Enter Alternative[12]: 85260000
Enter Alternative[13]: 57990000

Enter Criteria[5]: PU
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]: 0
Enter Alternative[2]: 0
Enter Alternative[3]: 0.75
Enter Alternative[4]: 0.75
Enter Alternative[5]: 0
Enter Alternative[6]: 0
Enter Alternative[7]: 1
Enter Alternative[8]: 0.75
Enter Alternative[9]: 0.75
Enter Alternative[10]: 0.75
Enter Alternative[11]: 1
Enter Alternative[12]: 1
Enter Alternative[13]: 1

Enter Criteria[6]: SCWR
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]: 0
Enter Alternative[2]: 0
Enter Alternative[3]: 0
Enter Alternative[4]: 0
Enter Alternative[5]: 0
Enter Alternative[6]: 1
Enter Alternative[7]: 0
Enter Alternative[8]: 0
Enter Alternative[9]: 1
Enter Alternative[10]: 1
Enter Alternative[11]: 0
Enter Alternative[12]: 1
Enter Alternative[13]: 1

Enter Criteria[7]: SW
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]: 0
Enter Alternative[2]: 0
Enter Alternative[3]: 4.00E+3
Enter Alternative[4]: 6.60E+3
Enter Alternative[5]: 0
Enter Alternative[6]: 0
Enter Alternative[7]: 0
Enter Alternative[8]: 6.60E+3
Enter Alternative[9]: 6.60E+3
Enter Alternative[10]: 4.00E+3
Enter Alternative[11]: 6.60E+3
Enter Alternative[12]: 6.60E+3
Enter Alternative[13]: 4.00E+3
Enter Criteria[8]: BW
```

Data input file for the MCDSS for the case study…Continued

```
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]: 1.95E+3
Enter Alternative[2]: 1.95E+3
Enter Alternative[3]: 1.95E+3
Enter Alternative[4]: 1.95E+3
Enter Alternative[5]: 0
Enter Alternative[6]: 1.95E+3
Enter Alternative[7]: 1.95E+3
Enter Alternative[8]: 1.95E+3
Enter Alternative[9]: 1.95E+3
Enter Alternative[10]: 0
Enter Alternative[11]: 1.95E+3
Enter Alternative[12]: 1.95E+3
Enter Alternative[13]: 0

Enter Criteria[9]: UAWR
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]: 86.60
Enter Alternative[2]: 81
Enter Alternative[3]: 100
Enter Alternative[4]: 94.5
Enter Alternative[5]: 81.4
Enter Alternative[6]: 77.1
Enter Alternative[7]: 86.7
Enter Alternative[8]: 94.5
Enter Alternative[9]: 97.8
Enter Alternative[10]: 95
Enter Alternative[11]: 84.4
Enter Alternative[12]: 88.4
Enter Alternative[13]: 92.7

Enter Criteria[10]: UALR
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]: 66.4
Enter Alternative[2]: 65.9
Enter Alternative[3]: 85.5
Enter Alternative[4]: 87.3
Enter Alternative[5]: 68.7
Enter Alternative[6]: 59.9
Enter Alternative[7]: 100
Enter Alternative[8]: 86.4
Enter Alternative[9]: 82.1
Enter Alternative[10]: 73.9
Enter Alternative[11]: 100
Enter Alternative[12]: 100
Enter Alternative[13]: 99.6

Enter Criteria[11]: NR
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]: 0.77
Enter Alternative[2]: 0.94
Enter Alternative[3]: 0.79
Enter Alternative[4]: 0.77
Enter Alternative[5]: 0.91
Enter Alternative[6]: 0.96
Enter Alternative[7]: 0.81
Enter Alternative[8]: 0.83
Enter Alternative[9]: 0.88
Enter Alternative[10]: 0.88
Enter Alternative[11]: 0.87
Enter Alternative[12]: 0.89
Enter Alternative[13]: 0.9
```

Table AV- 2 MCDSS output file showing a summary of the decision criteria values for different alternatives for the case study

```
>>> Decision Criteria Values for Different Alternatives <<<<
======================================================================================================================================
CRITERIA   TYPE  ALTERNATIVES
                 1        2        3        4        5        6        7        8        9       10       11       12       13
======================================================================================================================================
SY          0    3.21E+03 3.23E+03 3.23E+03 2.36E+03 3.23E+03 0.00E+00 2.42E+03 2.37E+03 0.00E+00 0.00E+00 9.73E+02 0.00E+00 0.00E+00
NB          1    1.52E+07 1.74E+07 1.91E+07 1.94E+07 1.70E+07 1.45E+07 2.88E+07 2.11E+07 2.04E+07 1.77E+07 3.30E+07 3.28E+07 3.27E+07
TIL         1    3.36E+03 3.33E+03 4.32E+03 4.41E+03 3.47E+03 3.03E+03 5.05E+03 4.36E+03 4.15E+03 3.73E+03 5.05E+03 5.05E+03 5.03E+03
COST        0    0.00E+00 1.20E+06 2.74E+07 5.66E+07 3.12E+06 0.00E+00 2.75E+07 5.78E+07 5.78E+07 3.05E+07 8.53E+07 8.53E+07 5.80E+07
PU          0    0.00E+00 0.00E+00 7.50E-01 7.50E-01 0.00E+00 0.00E+00 1.00E+00 7.50E-01 7.50E-01 7.50E-01 1.00E+00 1.00E+00 1.00E+00
SCWR        0    0.00E+00 0.00E+00 0.00E+00 0.00E+00 0.00E+00 1.00E+00 0.00E+00 0.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00
SW          1    0.00E+00 0.00E+00 4.00E+03 6.60E+03 0.00E+00 0.00E+00 0.00E+00 6.60E+03 6.60E+03 4.00E+03 6.60E+03 6.60E+03 4.00E+03
BW          1    1.95E+03 1.95E+03 1.95E+03 1.95E+03 0.00E+00 1.95E+03 1.95E+03 1.95E+03 1.95E+03 0.00E+00 1.95E+03 1.95E+03 0.00E+00
UAWR        1    8.66E+01 8.10E+01 1.00E+02 9.45E+01 8.14E+01 7.71E+01 8.67E+01 9.45E+01 9.78E+01 9.50E+01 8.44E+01 8.84E+01 9.27E+01
UALR        1    6.64E+01 6.59E+01 8.55E+01 8.73E+01 6.87E+01 5.99E+01 1.00E+02 8.64E+01 8.21E+01 7.39E+01 1.00E+02 1.00E+02 9.96E+01
NR          1    7.70E-01 9.40E-01 7.90E-01 7.70E-01 9.10E-01 9.60E-01 8.10E-01 8.30E-01 8.80E-01 8.80E-01 8.70E-01 8.90E-01 9.00E-01
```

Table AV- 3 MCDSS output file showing the standarized decision criteria values for different alternatives for the case study

```
>>>> Standarized Decision Criteria Values for Different Alternatives <<<<
======================================================================================================================================
CRITERIA   TYPE  ALTERNATIVES
                 1        2        3        4        5        6        7        8        9       10       11       12       13
======================================================================================================================================
SY          0    4.00E-03 0.00E+00 0.00E+00 2.68E-01 0.00E+00 1.00E+00 2.48E-01 2.65E-01 1.00E+00 1.00E+00 6.98E-01 1.00E+00 1.00E+00
NB          1    3.70E-02 1.57E-01 2.51E-01 2.67E-01 1.37E-01 0.00E+00 7.76E-01 3.59E-01 3.23E-01 1.73E-01 1.00E+00 9.93E-01 9.88E-01
TIL         1    1.62E-01 1.49E-01 6.38E-01 6.82E-01 2.19E-01 0.00E+00 1.00E+00 6.60E-01 5.52E-01 3.47E-01 1.00E+00 1.00E+00 9.89E-01
COST        0    1.00E+00 9.86E-01 6.79E-01 3.36E-01 9.63E-01 1.00E+00 6.78E-01 3.22E-01 3.22E-01 6.42E-01 0.00E+00 0.00E+00 3.20E-01
PU          0    1.00E+00 1.00E+00 2.50E-01 2.50E-01 1.00E+00 1.00E+00 0.00E+00 2.50E-01 2.50E-01 2.50E-01 0.00E+00 0.00E+00 0.00E+00
SCWR        0    1.00E+00 1.00E+00 1.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00 0.00E+00 0.00E+00 1.00E+00 0.00E+00 0.00E+00
SW          1    0.00E+00 0.00E+00 6.06E-01 1.00E+00 0.00E+00 0.00E+00 0.00E+00 1.00E+00 1.00E+00 6.06E-01 1.00E+00 1.00E+00 6.06E-01
BW          1    1.00E+00 1.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00 0.00E+00
UAWR        1    4.15E-01 1.70E-01 1.00E+00 7.60E-01 1.88E-01 0.00E+00 4.19E-01 7.60E-01 9.04E-01 7.82E-01 3.19E-01 4.93E-01 6.81E-01
UALR        1    1.62E-01 1.50E-01 6.38E-01 6.83E-01 2.19E-01 0.00E+00 1.00E+00 6.61E-01 5.54E-01 3.49E-01 1.00E+00 1.00E+00 9.90E-01
NR          1    0.00E+00 8.95E-01 1.05E-01 0.00E+00 7.37E-01 1.00E+00 2.11E-01 3.16E-01 5.79E-01 5.79E-01 5.26E-01 6.32E-01 6.84E-01
```

Table AV- 4 MCDSS output file showing the utility scores for different alternatives for the case study

```
>>>> Utility Scores for Decision Criteria for Different Alternatives <<<<
****************************************************************************************************************************
CRITERIA       ALTERNATIVES
              1       2       3       4       5       6       7       8       9       10      11      12      13
****************************************************************************************************************************
A[1]          0.004   0.000   0.000   0.268   0.000   1.000   0.248   0.265   1.000   1.000   0.698   1.000   1.000

A[1 -   2]    0.021   0.078   0.126   0.267   0.068   0.500   0.512   0.312   0.661   0.586   0.849   0.997   0.994

A[1 -   3]    0.068   0.102   0.296   0.405   0.118   0.333   0.675   0.428   0.625   0.507   0.899   0.998   0.992

A[1 -   4]    0.301   0.323   0.392   0.388   0.330   0.500   0.676   0.402   0.549   0.540   0.675   0.748   0.824

A[1 -   5]    0.441   0.458   0.364   0.360   0.464   0.600   0.540   0.371   0.489   0.482   0.540   0.599   0.659

A[1 -   6]    0.534   0.549   0.470   0.467   0.553   0.500   0.617   0.476   0.408   0.402   0.616   0.499   0.550

A[1 -   7]    0.458   0.470   0.489   0.543   0.474   0.429   0.529   0.551   0.492   0.431   0.671   0.570   0.558

A[1 -   8]    0.525   0.537   0.553   0.600   0.415   0.500   0.588   0.607   0.556   0.377   0.712   0.624   0.488

A[1 -   9]    0.513   0.496   0.603   0.618   0.390   0.444   0.569   0.624   0.595   0.422   0.669   0.610   0.509

A[1 -  10]    0.478   0.461   0.606   0.625   0.373   0.400   0.612   0.628   0.590   0.415   0.702   0.649   0.557

A[1 -  11]    0.435   0.501   0.561   0.568   0.406   0.455   0.576   0.599   0.589   0.430   0.686   0.647   0.569
```

Table AV- 5 MCDSS output file showing the Minimum, Maximum & Avarege Utility Score Values for different alternatives for the case study

```
>>>> Minimum, Maximum & Avarege Utility Score Values for Different Alternatives <<<<
====================================================================================
ALTERNATIVES        MIN       MAX       AVERAGE
-----------------------------------------------
Alternative[ 1]     0.004     0.534     0.269
Alternative[ 2]     0.000     0.549     0.274
Alternative[ 3]     0.000     0.606     0.303
Alternative[ 4]     0.267     0.625     0.446
Alternative[ 5]     0.000     0.553     0.277
Alternative[ 6]     0.333     1.000     0.667
Alternative[ 7]     0.248     0.676     0.462
Alternative[ 8]     0.265     0.628     0.447
Alternative[ 9]     0.408     1.000     0.704
Alternative[10]     0.377     1.000     0.689
Alternative[11]     0.540     0.899     0.720
Alternative[12]     0.499     1.000     0.749
Alternative[13]     0.488     1.000     0.744
```

Table AV- 6 MCDSS output file showing the ranking of different alternatives for the case study

```
 >>>> RANKING OF ALTERNATIVES <<<<
**************************************
Alternative [12]     0.749
Alternative [13]     0.744
Alternative [11]     0.720
Alternative [ 9]     0.704
Alternative [10]     0.689
Alternative [ 6]     0.667
Alternative [ 7]     0.462
Alternative [ 8]     0.447
Alternative [ 4]     0.446
Alternative [ 3]     0.303
Alternative [ 5]     0.277
Alternative [ 2]     0.274
Alternative [ 1]     0.269
yallara.cs.rmit.edu.au%
```

Table AV- 7 Data input file for the MCDSS for the 80% drawdown

```
file2:mcds2.c
*-------------------------------------------------*
 Enter number of Criteria: 11
 Enter number of Alternatives: 13
*-------------------------------------------------*
**** Enter Criteria in order of importance ****
Enter Criteria[1]: SY
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]:  888
Enter Alternative[2]:  900
Enter Alternative[3]:  900
Enter Alternative[4]:  37
Enter Alternative[5]:  900
Enter Alternative[6]:  0
Enter Alternative[7]:  99
Enter Alternative[8]:  44
Enter Alternative[9]:  0
Enter Alternative[10]: 0
Enter Alternative[11]: 0
Enter Alternative[12]: 0
Enter Alternative[13]: 0

Enter Criteria[2]: NB
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  15175000
Enter Alternative[2]:  17385000
Enter Alternative[3]:  19126000
Enter Alternative[4]:  19415000
Enter Alternative[5]:  17012000
Enter Alternative[6]:  15561000
Enter Alternative[7]:  28823000
Enter Alternative[8]:  21119000
Enter Alternative[9]:  20977000
Enter Alternative[10]: 18724000
Enter Alternative[11]: 32963000
Enter Alternative[12]: 32946000
Enter Alternative[13]: 32910000
```

Data input file for the MCDSS for the 80% drawdown…Continued

```
Enter Criteria[3]: TIL
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  3357
Enter Alternative[2]:  3330.7
Enter Alternative[3]:  4319.6
Enter Alternative[4]:  4408
Enter Alternative[5]:  3471.1
Enter Alternative[6]:  3372.4
Enter Alternative[7]:  5052.1
Enter Alternative[8]:  4363.1
Enter Alternative[9]:  4318.6
Enter Alternative[10]: 4142.6
Enter Alternative[11]: 5052.1
Enter Alternative[12]: 5043.8
Enter Alternative[13]: 5052.1

Enter Criteria[4]: COST
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]:  0
Enter Alternative[2]:  1200000
Enter Alternative[3]:  27400000
Enter Alternative[4]:  56600000
Enter Alternative[5]:  3120000
Enter Alternative[6]:  0
Enter Alternative[7]:  27460000
Enter Alternative[8]:  57800000
Enter Alternative[9]:  57800000
Enter Alternative[10]: 30530000
Enter Alternative[11]: 85260000
Enter Alternative[12]: 85260000
Enter Alternative[13]: 57990000

Enter Criteria[5]: PU
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]:  0
Enter Alternative[2]:  0
Enter Alternative[3]:  0.75
Enter Alternative[4]:  0.75
Enter Alternative[5]:  0
Enter Alternative[6]:  0
Enter Alternative[7]:  1
Enter Alternative[8]:  0.75
Enter Alternative[9]:  0.75
Enter Alternative[10]: 0.75
Enter Alternative[11]: 1
Enter Alternative[12]: 1
Enter Alternative[13]: 1

Enter Criteria[6]: SCWR
Enter Criteria type [1] for BENEFIT  [0] for COST: 0
Enter Alternative[1]:  0
Enter Alternative[2]:  0
Enter Alternative[3]:  0
Enter Alternative[4]:  0
Enter Alternative[5]:  0
Enter Alternative[6]:  1
Enter Alternative[7]:  0
Enter Alternative[8]:  0
Enter Alternative[9]:  1
Enter Alternative[10]: 1
Enter Alternative[11]: 0
Enter Alternative[12]: 1
Enter Alternative[13]: 1
```

Data input file for the MCDSS for the 80% drawdown…Continued

```
Enter Criteria[7]: SW
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  0
Enter Alternative[2]:  0
Enter Alternative[3]:  4000
Enter Alternative[4]:  6600
Enter Alternative[5]:  0
Enter Alternative[6]:  0
Enter Alternative[7]:  0
Enter Alternative[8]:  6600
Enter Alternative[9]:  6600
Enter Alternative[10]: 4000
Enter Alternative[11]: 6600
Enter Alternative[12]: 6600
Enter Alternative[13]: 4000

Enter Criteria[8]: BW
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  1947
Enter Alternative[2]:  1947
Enter Alternative[3]:  1947
Enter Alternative[4]:  1947
Enter Alternative[5]:  0
Enter Alternative[6]:  1947
Enter Alternative[7]:  1947
Enter Alternative[8]:  1947
Enter Alternative[9]:  1947
Enter Alternative[10]: 0
Enter Alternative[11]: 1947
Enter Alternative[12]: 1947
Enter Alternative[13]: 0

Enter Criteria[9]: UAWR
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  86.60
Enter Alternative[2]:  81.00
Enter Alternative[3]:  100.00
Enter Alternative[4]:  94.50
Enter Alternative[5]:  81.40
Enter Alternative[6]:  80.00
Enter Alternative[7]:  86.70
Enter Alternative[8]:  94.50
Enter Alternative[9]:  95.20
Enter Alternative[10]: 98.10
Enter Alternative[11]: 84.40
Enter Alternative[12]: 85.80
Enter Alternative[13]: 90.20

Enter Criteria[10]: UALR
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  66.40
Enter Alternative[2]:  65.90
Enter Alternative[3]:  85.50
Enter Alternative[4]:  87.30
Enter Alternative[5]:  68.70
Enter Alternative[6]:  66.80
Enter Alternative[7]:  100.00
Enter Alternative[8]:  86.40
Enter Alternative[9]:  85.50
Enter Alternative[10]: 82.00
Enter Alternative[11]: 100.00
Enter Alternative[12]: 99.80
Enter Alternative[13]: 100.00
```

Data input file for the MCDSS for the 80% drawdown…Continued

```
Enter Criteria[11]: NR
Enter Criteria type [1] for BENEFIT  [0] for COST: 1
Enter Alternative[1]:  0.77
Enter Alternative[2]:  0.94
Enter Alternative[3]:  0.79
Enter Alternative[4]:  0.77
Enter Alternative[5]:  0.91
Enter Alternative[6]:  0.89
Enter Alternative[7]:  0.81
Enter Alternative[8]:  0.83
Enter Alternative[9]:  0.85
Enter Alternative[10]: 0.82
Enter Alternative[11]: 0.87
Enter Alternative[12]: 0.87
Enter Alternative[13]: 0.88
```

Table AV- 8 MCDSS output file showing a summary of the decision criteria values for different alternatives for the 80% drawdown

```
>>> Decision Criteria Values for Different Alternatives <<<<
====================================================================================================================================================
CRITERIA    TYPE   ALTERNATIVES
                   1         2         3         4         5         6         7         8         9         10        11        12        13
====================================================================================================================================================
SY           0     8.88E+02 9.00E+02 9.00E+02 3.70E+01 9.00E+02 0.00E+00 9.90E+01 4.40E+01 0.00E+00 0.00E+00 0.00E+00 0.00E+00 0.00E+00
NB           1     1.52E+07 1.74E+07 1.91E+07 1.94E+07 1.70E+07 1.56E+07 2.88E+07 2.11E+07 2.10E+07 1.87E+07 3.30E+07 3.29E+07 3.29E+07
TIL          1     3.36E+03 3.33E+03 4.32E+03 4.41E+03 3.47E+03 3.37E+03 5.05E+03 4.36E+03 4.32E+03 4.14E+03 5.05E+03 5.04E+03 5.05E+03
COST         0     0.00E+00 1.20E+06 2.74E+07 5.66E+07 3.12E+06 0.00E+00 2.75E+07 5.78E+07 5.78E+07 3.05E+07 8.53E+07 8.53E+07 5.80E+07
PU           0     0.00E+00 0.00E+00 7.50E-01 7.50E-01 0.00E+00 0.00E+00 1.00E+00 7.50E-01 7.50E-01 7.50E-01 1.00E+00 1.00E+00 1.00E+00
SCWR         0     0.00E+00 0.00E+00 0.00E+00 0.00E+00 0.00E+00 1.00E+00 0.00E+00 0.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00
SW           1     0.00E+00 0.00E+00 4.00E+03 6.60E+03 0.00E+00 0.00E+00 0.00E+00 6.60E+03 6.60E+03 4.00E+03 6.60E+03 6.60E+03 4.00E+03
BW           1     1.95E+03 1.95E+03 1.95E+03 1.95E+03 0.00E+00 1.95E+03 1.95E+03 1.95E+03 1.95E+03 0.00E+00 1.95E+03 1.95E+03 0.00E+00
UAWR         1     8.66E+01 8.10E+01 1.00E+02 9.45E+01 8.14E+01 8.00E+01 8.67E+01 9.45E+01 9.52E+01 9.81E+01 8.44E+01 8.58E+01 9.02E+01
UALR         1     6.64E+01 6.59E+01 8.55E+01 8.73E+01 6.87E+01 6.68E+01 1.00E+02 8.64E+01 8.55E+01 8.20E+01 1.00E+02 9.98E+01 1.00E+02
NR           1     7.70E-01 9.40E-01 7.90E-01 7.70E-01 9.10E-01 8.90E-01 8.10E-01 8.30E-01 8.50E-01 8.20E-01 8.70E-01 8.70E-01 8.80E-01
```

Table AV- 9 MCDSS output file showing the standarized decision criteria values for different alternatives for the 80% drawdown

```
>>>> Standarized Decision Criteria Values for Different Alternatives <<<<
====================================================================================================================================================
CRITERIA    TYPE   ALTERNATIVES
                   1         2         3         4         5         6         7         8         9         10        11        12        13
====================================================================================================================================================
SY           0     1.33E-02 0.00E+00 0.00E+00 9.59E-01 0.00E+00 1.00E+00 8.90E-01 9.51E-01 1.00E+00 1.00E+00 1.00E+00 1.00E+00 1.00E+00
NB           1     0.00E+00 1.24E-01 2.22E-01 2.38E-01 1.03E-01 2.17E-02 7.67E-01 3.34E-01 3.26E-01 2.00E-01 1.00E+00 9.99E-01 9.97E-01
TIL          1     1.53E-02 0.00E+00 5.74E-01 6.26E-01 8.16E-02 2.42E-02 1.00E+00 6.00E-01 5.74E-01 4.72E-01 1.00E+00 9.95E-01 1.00E+00
COST         0     1.00E+00 9.86E-01 6.79E-01 3.36E-01 9.63E-01 1.00E+00 6.78E-01 3.22E-01 3.22E-01 6.42E-01 0.00E+00 0.00E+00 3.20E-01
PU           0     1.00E+00 1.00E+00 2.50E-01 2.50E-01 1.00E+00 1.00E+00 0.00E+00 2.50E-01 2.50E-01 2.50E-01 0.00E+00 0.00E+00 0.00E+00
SCWR         0     1.00E+00 1.00E+00 1.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00 0.00E+00 0.00E+00 1.00E+00 0.00E+00 0.00E+00
SW           1     0.00E+00 0.00E+00 6.06E-01 1.00E+00 0.00E+00 0.00E+00 0.00E+00 1.00E+00 1.00E+00 6.06E-01 1.00E+00 1.00E+00 6.06E-01
BW           1     1.00E+00 1.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00 1.00E+00 1.00E+00 0.00E+00 1.00E+00 1.00E+00 0.00E+00
UAWR         1     3.30E-01 5.00E-02 1.00E+00 7.25E-01 7.00E-02 0.00E+00 3.35E-01 7.25E-01 7.60E-01 9.05E-01 2.20E-01 2.90E-01 5.10E-01
UALR         1     1.47E-02 0.00E+00 5.75E-01 6.28E-01 8.21E-02 2.64E-02 1.00E+00 6.01E-01 5.75E-01 4.72E-01 1.00E+00 9.94E-01 1.00E+00
NR           1     0.00E+00 1.00E+00 1.18E-01 0.00E+00 8.24E-01 7.06E-01 2.35E-01 3.53E-01 4.71E-01 2.94E-01 5.88E-01 5.88E-01 6.47E-01
```

Table AV- 10 MCDSS output file showing the utility scores for different alternatives for the 80% drawdown

```
>>>> Utility Scores for Decision Criteria for Different Alternatives <<<<
**************************************************************************************************************************************
CRITERIA      ALTERNATIVES
              1       2       3       4       5       6       7       8       9       10      11      12      13
**************************************************************************************************************************************
A[1]          0.013   0.000   0.000   0.959   0.000   1.000   0.890   0.951   1.000   1.000   1.000   1.000   1.000

A[1 -   2]    0.007   0.062   0.111   0.599   0.052   0.511   0.829   0.643   0.663   0.600   1.000   1.000   0.999

A[1 -   3]    0.010   0.041   0.266   0.608   0.062   0.349   0.886   0.628   0.633   0.557   1.000   0.998   0.999

A[1 -   4]    0.257   0.278   0.369   0.540   0.287   0.511   0.834   0.552   0.556   0.578   0.750   0.749   0.829

A[1 -   5]    0.406   0.422   0.345   0.482   0.430   0.609   0.667   0.491   0.494   0.513   0.600   0.599   0.663

A[1 -   6]    0.505   0.518   0.454   0.568   0.525   0.508   0.723   0.576   0.412   0.427   0.667   0.499   0.553

A[1 -   7]    0.433   0.444   0.476   0.630   0.450   0.435   0.619   0.637   0.496   0.453   0.714   0.571   0.560

A[1 -   8]    0.504   0.514   0.541   0.676   0.394   0.506   0.667   0.682   0.559   0.396   0.750   0.624   0.490

A[1 -   9]    0.484   0.462   0.592   0.682   0.358   0.450   0.630   0.687   0.581   0.453   0.691   0.587   0.493

A[1 -  10]    0.437   0.416   0.591   0.676   0.330   0.407   0.667   0.678   0.581   0.455   0.722   0.628   0.543

A[1 -  11]    0.398   0.469   0.548   0.615   0.375   0.434   0.628   0.649   0.571   0.440   0.710   0.624   0.553
```

Table AV- 11 MCDSS output file showing the Minimum, Maximum & Avarege Utility Score Values for different alternatives for the 80% drawdown

```
>>>> Minimum, Maximum & Avarege Utility Score Values for Different
Alternatives <<<<
============================================================================
ALTERNATIVES        MIN       MAX       AVERAGE
-----------------------------------------------
Alternative[ 1]     0.007     0.505     0.256
Alternative[ 2]     0.000     0.518     0.259
Alternative[ 3]     0.000     0.592     0.296
Alternative[ 4]     0.482     0.959     0.720
Alternative[ 5]     0.000     0.525     0.262
Alternative[ 6]     0.349     1.000     0.674
Alternative[ 7]     0.619     0.890     0.755
Alternative[ 8]     0.491     0.951     0.721
Alternative[ 9]     0.412     1.000     0.706
Alternative[10]     0.396     1.000     0.698
Alternative[11]     0.600     1.000     0.800
Alternative[12]     0.499     1.000     0.750
Alternative[13]     0.490     1.000     0.745
```

Table AV- 12 MCDSS output file showing the ranking of different alternatives for the 80% drawdown

```
>>>> RANKING OF ALTERNATIVES <<<<
**************************************
Alternative [11]     0.800
Alternative [ 7]     0.755
Alternative [12]     0.750
Alternative [13]     0.745
Alternative [ 8]     0.721
Alternative [ 4]     0.720
Alternative [ 9]     0.706
Alternative [10]     0.698
Alternative [ 6]     0.674
Alternative [ 3]     0.296
Alternative [ 5]     0.262
Alternative [ 2]     0.259
Alternative [ 1]     0.256
```

APPENDIX VI

LIST OF PUBLICATIONS

This appendix lists the following peer reviewed conference and journal papers written relating to this study:

Jarrar A., Jayasuriya N., Othman M., Al-Masri M., Jayyousi A.., Kaluarachchi J., McKee M., 2005a. Decision Support System for Integrated Water and Land Management in Agriculture-Dominated Watersheds: A conceptual study to Faria watershed, Palestine. Proceedings of the 2005 World Water and Environmental Resources Congress, EWRI, American Society of Civil Engineers ASCE, May 15-19, 2005, Anchorage, Alaska, pp. 1-11.

Jarrar A., Jayasuriya N., Othman M., Jayyousi A., 2005. Integrated Natural Resources Management Framework in Semi-arid Regions. Proceedings of the 2005 XXXI IAHR Congress, International Association of Hydraulic Engineers, KWRA. September 11-16, 2005. Seoul, Korea, pp. 249-260.

Jarrar A., Jayasuriya N., Jayyousi A., Othman M., 2007. Applicability of the GIUH model to estimate flood peaks from ungauged catchments in arid areas - a case study for the West Bank. IAHS Publ. 313, 2007, 346-356.