HYDROLOGIC INVENTORY OF MEREB-GASH RIVER BASIN IN ERITREA

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
HYDROLOGY

By

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled **"HYDROLOGIC INVENTORY OF MEREB-GASH RIVER BASIN IN ERITREA"** in partial fulfilment of the requirements for the award of the degree of **Master of Technology in Hydrology**, submitted to the department of Hydrology, Indian Institute of Technology Roorkee, Roorkee is an authentic record of my own work carried out during a period from July 2009 to June 2010 under the supervision of Dr. B. S. Mathur, Professor (Retd.) and Emeritus Fellow, Department of Hydrology, Indian Institute of Technology Roorkee, Roorkee.

The matter presented in this thesis has not been submitted by me for the award of any other degree of this or any other institute.

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Candidates signature

This is to certify that the above statement made by the candidate is correct to the best of my knowledge.

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ABSTRACT

Prediction of runoff hydrograph is essential for the assessment of water availability, design of various hydraulic structures and watershed development and management. Different methods have been in practice in surface water hydrology for quite some time back. Derivation of unit hydrograph using rainfall and runoff data [Sherman] as well as synthetic unit hydrograph (SUH) approaches and conceptual models for gauged and ungauged catchments are the efforts in this direction. Nonetheless, most of them have limitations for one or the other reasons. Thus, more recently, use of geomorphologic instantaneous unit hydrograph (GIUH) coupled with other conceptual models has been proved to be the most successful approach for flood prediction from ungauged catchments. In this respect, the Geographic Information System (GIS) and remote sensing image processing tools have been found to be helpful for the determination of geomorphologic characteristics on which entire GIUH development relies upon.

In this study, the applicability of GIUH based Nash model is tested on the Mereb-Gash basin in Eritrea. Due to the uncertainties of the recorded rainfall data at the upper Mereb-Gash, direct surface runoffs are not computed with any of the approaches referred to above. Rather, the GIUH based Nash model unit hydrographs (UH) are developed and compared with the UH [Sherman] for the Debarwa catchment and SUH for other ungauged catchments. However, results indicate that SUH method is applicable for small catchment areas to the likes of Debarwa, Adi-Ghergera, Ethio1 and Ethio2 whereas divergence between the GIUH based Nash model and SUH is clearly observed for the bigger catchments (Tsorona, Tokombia, Adi-Chigono and Tessenei).

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LIST OF ABBREVIATIONS/NOTATIONS

DEM	Digital Elevation Model
DRH	Direct runoff hydrograph
DSRO	Direct surface runoff
EFF	Model efficiency
ERDAS	Earth Resources Data Analysis System
ERH	Effective rainfall hyetograph
ETM+	Enhanced thematic mapper plus
FAO	Food and Agriculture Organizations of the United Nations
GIS	Geographic Information System
GIUH	Geomorphologic instantaneous unit hydrograph
H-Q	stage-discharge
IUH	Instantaneous Unit Hydrograph
MATLAB	Matrix Laboratory
msl	mean sea level
NASA	National Aeronautics and Space Administration
NDVI	Normalized Difference Vegetation Index
RMSE	root mean square error
SCS	Soil Conservation Services
SRTM	Shuttle Radar Topographic System
SUH	Synthetic unit hydrograph
UH	Unit hydrograph
USGS	United States Geological Survey
UTM	Universal Transverse Mercator co-ordinate system
WGS	World Geodetic System
WMO	World Meteorological Organization
WRD	Water Resources Department of Eritrea
A	catchment area
A_i	areas of Thiessen polygons
A_w, $\overline{A}_w$	stream area of the catchment of order w
a, b	rating curve constants

C_m	constant of channel maintenance
C_t, C_p	regional constants representing watershed slope and storage effects
D	duration of unit hydrograph
D_d	drainage density
D_i	distance of estimator station from the estimated station
d	runoff depth
H_R	Sub-basin relief
H, H_o	gauge height and gauge reading corresponding to zero discharge
ha	hectare
i_e	intensity of rainfall
$h_{\min}$	minimum elevation above msl
$h_{\max}$	maximum elevation above msl
K	storage function
L	main channel length
L_{ca}	distance along the main water course from the gauging station to a point opposite to sub-basin centroid
$\overline{L}_o$	length of overland flow
L_Ω	length of the highest order stream
L_w, $\overline{L}_w$	stream lengths
M_1, M_2	first and second moments of the IUH about the origin
M_{DRH1}, M_{DRH2}	first and second moments of inertia of direct runoff hydrograph
M_{ERH1}, M_{ERH2}	first and second moments of inertia of effective rainfall hyetograph
m	number of unit hydrograph ordinates
NIR, R_l	near infrared and red lights
n, $\Gamma(n)$	number of linear reservoirs and gamma function
n_m	Manning roughness coefficient
N_w	number of streams in order w
P	perimeter of the catchment boundary
$\overline{P}$	average rainfall over a catchment

P_i	rainfall magnitudes recorded by individual stations
$Q_{S-Curve}$	S-Curve ordinate
$\overline{Q}_p, \overline{T}_B, \overline{t}_L, \overline{d}, \overline{t}_D$	average peak discharge, time base, time lag, runoff depth and duration
Q, Q_p, q	stream discharge, peak flow and discharge per unit area
$Q_{oi}, \overline{Q}_o, Q_{ci}$	i[th] ordinate of the observed discharge, average of the ordinates of observed discharge and computed discharge
$Q(t), q_p$	IUH of the Nash model and peak discharge of the GIUH
R	estimated rainfall at a station
R_c	hydraulic radius
R_i	rainfall at surrounding stations
R_B, R_L, R_A, R_s	bifurcation ratio, length ratio, area ratio and stream slope ratio
R_h, R_e, R_{sh}, R_f	relief ratio, elongation ratio, shape factor and form factor
S	Average sub-basin slope
$\overline{S}_{cl}, \overline{S}_{cm}, \overline{S}_{ol}$	stream channel slope of order *w,* main channel and overland slope
$t_D, t_{re}, t_{L.alt}$	durations of excess rainfall, required excess rainfall and alternate excess rainfall
t_L, t_p, T_B, T_C	time lag, time peak, time base, time of concentration
V, v	direct runoff volume and unit volume
V_d	dynamic velocity parameter
W_{50}, W_{75}	unit hydrograph widths at 50% and 75% of the peak discharge
β, δ	constants of the relation between the dynamic velocity and rainfall excess intensity
ϕ-index	rainfall abstractions
Δt	time step
α	time base constant
Ω	highest stream order of the catchment

CHAPTER-1

INTRODUCTION

1.1 General

Water is a precious natural resource and is vital for all forms of life. Without water all living creatures on earth cannot survive. It generally refers to that part of fresh water which is renewable annually and includes surface water, soil water and underground water. Water resources technology played an important role in ancient civilisations of the Babylonian, Egyptian, Hittite, Greek, Roman, Chinese and other empires (Yevjevich, 1992). Today, observational records and climate projections provide abundant evidence that freshwater resources are vulnerable and have the potential to be strongly impacted by climate change, with wide-ranging consequences for human societies and ecosystems (IPCC, 2008). As such, a thorough knowledge of the meteorological and hydrological processes influencing the quantity and quality of the water resources and their distribution and variability both in time and space is inevitable for the overall development of any nation.

Stream flow data are the most important hydrological data for surface water analysis. These data are analysed to determine the magnitude and variability of surface waters. They constitute input in planning, design, and operation of surface water projects and are also used in design of bridges and culverts, flood forecasting systems, flood plain delineation, etc. For this purpose, computation of direct surface runoff for a catchment under consideration is essential. Different methods are used for design flood estimation; rational method, empirical method, flood frequency method, deterministic approach, and watershed models. The use of a particular method depends upon; the desired objective, the available data, and the importance of the project. The first three methods are generally used for estimating the magnitude of peak flow whereas the deterministic and watershed models techniques can give the design runoff hydrograph, in addition to the magnitude of design flood peak.

The rational formula is only applicable to small size catchments (less than 50 km^2). The empirical formulae are essentially the regional formulae based on statistical correlation. The frequency analysis approach is the statistical method to predict the flood peaks of a specified return period. The deterministic (unit hydrograph) method is basically a rainfall-runoff relationship normally applicable to moderate size catchments with area less than 5,000 km^2. Unit hydrograph (UH) technique along with routing can be used to estimate the design flood for larger sized basins by dividing the basin into smaller sub-basins. Therefore, the present work aims at the UH technique for the surface water study of the Mereb-Gash river basin in Eritrea with an emphasis on a pilot project selected at its upper stream viz. Ghergera and Debarwa river catchments.

1.2 Overview of East Africa's Water Resources

Apparently, Africa has abundant water resources in large rivers, great lakes, vast wetlands and limited but widespread groundwater. It has 17 rivers with catchment areas greater than 100,000 km^2. Further, it has more than 160 lakes larger than 27 km^2 most of which are located around the equatorial region and sub-humid East African highlands within the Rift Valley. Africa's freshwater resources are distributed unevenly across the continent with western and central Africa having significantly greater precipitation than northern, the eastern and southern Africa. Due to the geographic location of Africa, thereupon its climate, the flow of many African rivers is highly seasonal (Dennis, 2003).

Africa's many lakes stretch along the East African Rift Valley from the Red Sea to the mouth of the Zambezi River (Zambia). Evaporation exceeds their surface rainfall, and as a consequence their outflow is less than the quantities brought in annually by their tributaries. The quantity and quality of groundwater reserves are closely related to geologic structure and the conditions under which groundwater is found. The East African plateaus usually contain little or no quantities of groundwater. In the volcanic areas of this region groundwater may have so high a content of fluorine as to make it unfit for human consumption (Mamdouh, 2003).

The Nile River, which is regarded as the world's longest river, is a south-north flowing river in Africa that empties its contents into the Mediterranean Sea. It has a drainage area 3,254,555 km^2 which is about 10% of the area of Africa. The Nile River and its tributaries traverse ten countries; Tanzania, Uganda, Rwanda, Burundi, D.R Congo, Kenya, Ethiopia, Eritrea, Sudan, and Egypt as per details given in Table 1.1. There are two great tributaries of the Nile joining at Khartoum, i.e. the White Nile starting in equatorial east Africa, and the Blue Nile originating in Ethiopia. Before the Blue and White Nile confluence, the only remaining major tributary is the Atbara River. It flows through the border between Eritrea and Ethiopia (known by Setit River) and joins with Mereb river (Eritrea) in the Sudan.

Table 1.1 Areas of Nile Basin in different countries of Africa (*Source*: FAO, 1997)

Country	Total area of the country (km²)	Area of the country within the basin (km²)	As % of total area of basin (%)	As % of total area of country (%)	Average annual rainfall in the basin area (mm) min.	max.	mean
Burundi	27834	13260	0.4	47.6	895	1570	1110
Rwanda	26340	19876	0.6	75.5	840	1935	1105
Tanzania	945090	84200	2.7	8.9	625	1630	1015
Kenya	580370	46229	1.5	8.0	505	1790	1260
Congo	2344860	22143	0.7	0.9	875	1915	1245
Uganda	235880	231366	7.4	98.1	395	2060	1140
Ethiopia	1100010	365117	11.7	33.2	205	2010	1125
Eritrea	121890	24921	0.8	20.4	240	665	520
Sudan	2505810	1978506	63.6	79.0	0	1610	500
Egypt	1001450	326751	10.5	32.6	0	120	15
For Nile basin		3112369	100.0		0	2060	615

According to the Intergovernmental Panel on Climate Change (IPCC, 2008), the African continent is the most vulnerable to climate change. Runoff and water availability are expected to decline in the northern and southern regions of the continent. The frequency of floods and droughts may increase. As a result, 25 African countries (many east African countries among them) are expected to experience water scarcity over the next 20–30 years. Davidson et al (2003) suggested that the negative impacts associated with climate change are compounded by many factors including widespread poverty, human diseases, and high population density, which is estimated to double the demand for food, water, and livestock forage within the next 30 years.

East African water resources are characterised by the multiplicity of trans-boundary water basins, extreme spatial and temporal variability of climate and rainfall, growing water scarcity, unevenly distributed ground water, increasing demand and low investment water pollution and environmental degradation, and inadequate data and human capacity (Dennis, 2003). Furthermore, the constantly increasing population and economic challenges encountering the region are the main constraints to water resources developments. Nonetheless, to resolve these problems and other possible water conflicts over shared water resources, countries in the region formed a Nile Basin Initiative in 1999. But, further concerted efforts are needed to make the objectives of this initiative tangible and to properly utilize the potentials of the region's water resources. The summarized water availability, utilization and its potentials in different countries within the Nile Basin is given in Table 1.2.

Table 1.2 Irrigation potential, water requirements, water availability and areas under irrigation of the Nile basin (*Source*: FAO, 1997)

Country area within the Nile basin	Irrigation potential	Gross irrigation water requirement		Actual flows		Flows after deduction for irrigation and losses		Area already under irrigation
		per ha	total	inflow	outflow	inflow	outflow	
	(ha)	(m^3/ha.year)	(km^3/yr)	(km^3/yr)	(km^3/yr)	(km^3/yr)	(km^3/yr)	(ha)
Burundi	80000	13000	1.04	0.00	1.50	0.00	0.46	0
Rwanda	150000	12500	1.88	1.50	7.00	0.46	4.09	2000
Tanzania	30000	11000	0.33	7.00	10.70	4.09	7.46	10000
Kenya	180000	8500	1.53	0.00	8.40	0.00	6.87	6000
Congo	10000	10000	0.10	0.00	1.50	0.00	1.40	0
Uganda	202000	8000	1.62	28.70	37.00	23.83	30.51	9120
Ethiopia	2220000	9000	19.98	0.00	80.10	0.00	60.12	23160
Eritrea	150000	11000	1.65	0.00	2.20	0.00	0 55	15124
Sudan	2750000	14000	38.50	117.10	55.50	90.63	31.13	1935200
Egypt	4420000	13000	57.46	55.50	rest to sea	31.13	-26.33	3078000
Sum of countries	10192000		124.08					5078604
Total for Nile basin	< 8000000							

1.3 General Profile of Eritrea

1.3.1 Historical Background

Italy set the boundaries of Eritrea in January 1890 and ruled it as a colony until 1941. The British took over the administration after the Italians were defeated in World War II. Following a decision by the United Nations, Eritrea was federated to Ethiopia in 1952 with a certain amount of autonomy. However, Ethiopia's annexation of Eritrea 10 years later sparked a 30 years bitter armed struggle for independence that eventually ended in May 1991. In an internationally supervised referendum, Eritreans voted for independence that was officially declared on 24th May 1993.

Eritrea has a population of about five million with a growth rate of 3.1%. It is roughly divided in to Muslims and Christians. Though the population density is 41.2 per km^2, 65% of it lives in the highlands and green belt zones which amount to only 16% of the total area (Vasudevu, 2009). The country is divided into six administrative regions called Zobas, 59 sub-zones, and 2,606 village clusters, organized into 701 legally registered administration villages. The capital city, Asmara, lies at an elevation of around 2,350 m above msl.

1.3.2 Geography

Eritrea is located in eastern Africa between latitudes 12° and 18° N and longitudes 36° and 44° E and includes the Dahlak Archipelago and other islands along the Red Sea coast. It is bordering Sudan in the north and west, Ethiopia in the south, the Republic of Djibouti in the south-east and the Red Sea in the north and north-east (Fig. 1.1). It covers an area of 124,324 km^2 comprising a high plateau, lowland and coastal plains.

1.3.3 Topography and Climate

Eritrea's topography can be divided into three broad categories. Firstly, the arid narrow lowland strip along the Red Sea coast. Secondly, the highland which is an extension of the Ethiopian Plateau dissected by river valleys and lastly, the western plain along the Sudanese border. The highest point is Emba Soira, south-east of Asmara, at 3,010 m above msl. The lowest point lies in the Danakil depression, with places at 130 m below msl.

Eritrea has six agro-ecological zones (Negassi et al, 2002) defined by climate, altitude, soil and vegetation (Fig. 1.2). These comprise of the following (i) *the moist highlands* zone,. located in the southern part of the highlands with a small additional area further north have an altitude of over 1,500 m above msl and an average annual rainfall of over 500 mm; (ii) *the moist lowlands* zone located in the southern and south-western parts of the country have an altitude ranging between 500 and 1,600 m above msl, warm to hot semi-arid climate and average annual rainfall between 500 to 800mm; (iii) *the arid highlands* zone situated on the northern part have an altitude starting from 1,600 to 2,600 m above msl, is characterized by a warm climate; (iv) *the arid lowlands* having an altitude ranging from 400 to 1,600 m above msl with hot arid climate and an average annual rainfall of less than 300 mm; (v) *the sub-humid escarpment* (also known by the green belt) lying along the eastern escarpments of the highlands, ranging in altitude between 600 and over 2,600 m above msl, with rainfall of up to 1,000 mm; (vi) *the semi-desert* zone along the Red Sea coast have an altitude that ranges from sea level to 600m with Saharan climate and rainfall fall of less than 200 mm.

The highlands of Eritrea are located on the highest landmass of the African continent and, therefore, have much cooler, damper climate than the semi-arid coast along the Red Sea and the western hills and lowlands. The data obtained from the Water Resources Department (WRD) reveal that the average temperature in Asmara is around 16.3°C where as in Massawa (on the coast) it is around 30.2°C.

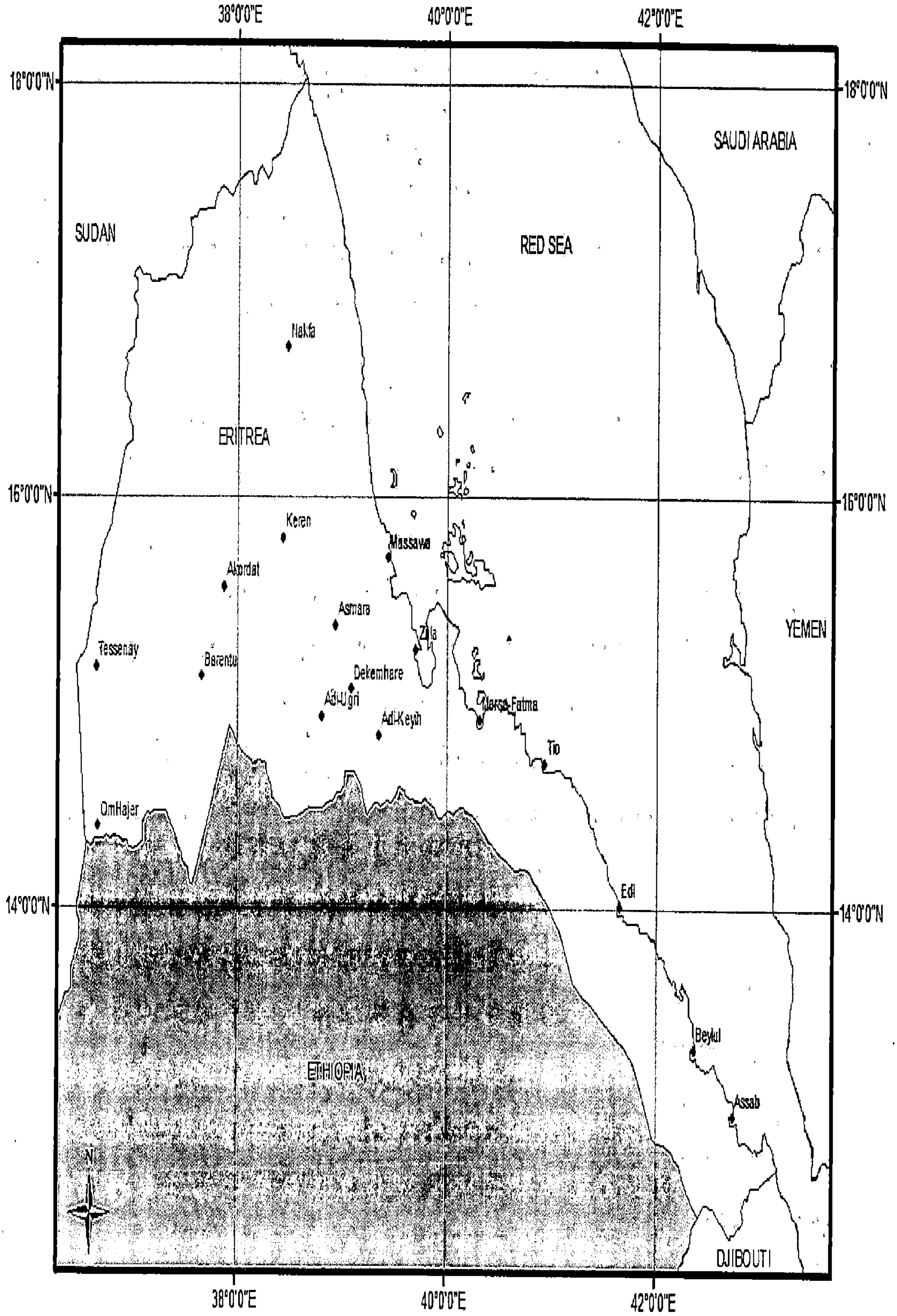


Fig. 1.1 Geographical location of Eritrea

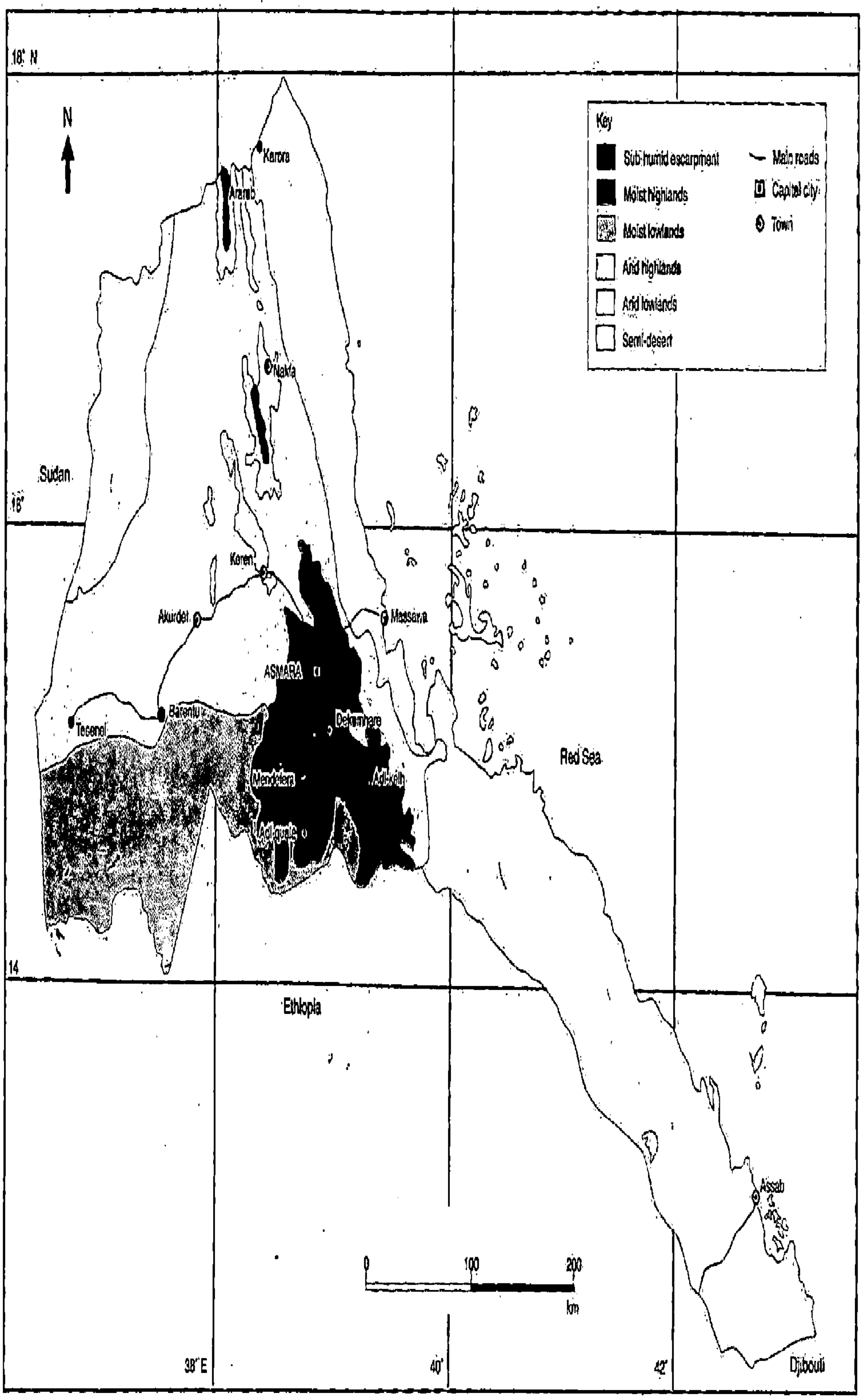


Fig. 1.2 Agro-ecological zones of Eritrea (*Source*: Negassi A., Bein E., Ghebru K. and Tengn□s B., 2002)

1.3.4 Natural Resources

Resources in Eritrea include barite, copper, feldspar, fish, gold, kaolin, potash, salt, zinc, marble, granite. It is endowed with rich tourism potentials and historical relics. On the other hand, owing to being an arid and semi-arid country in the Sahelian Africa it possesses limited water resources. The majority of the population depends on ground water as their main source of water supply for different uses. It is also a country predominantly dependent on rain fed agriculture. Thus, it has been the victim of recurrent and devastating droughts.

1.3.5 Land Use and Environmental Factors

Less than 13.6% of the land in Eritrea is cultivable (0.37 hectare per inhabitant) with an irrigation potential of 11.7% of the cultivable area (FAO, 2005). Although the vast majority of the land is suitable only for pasturage, some areas such as the Red Sea coast and the far north are too arid even for this purpose.

Tesfagiorgis (1993) clearly indicated Eritrea's pressing environmental concerns. Like the other parts of the horn of African region, it has experienced its share of environmental degradation; waters have dried up, forests disappeared, fertile soils eroded precipitously and the expansions of desertification is observed during the past several decades. In respect of this, the National Environmental Management Plan for Eritrea imposes on all Eritreans an obligation to use natural resources frugally; to reuse and recycle resources to the maximum and to minimize the depletion of non-renewable resources.

1.4 Need of Research over Eritrea

When Eritrea was liberated in 1991, all its infrastructures in general and the water resources sector in particular had been completely devastated by the consistent 30 years war for independence. Consequently, this led to interruption in the hydrological records due to destruction of many of the old gauging sites. This is reflected on the presence of large gaps in the records (3 to more than 20 years) of annual rainfall data from the Eritrean cities and towns. Apart from substantial breaks in the data time series, the erratic or unreliable measurements are responsible for the poor quality of available data.

Since independence, the government and people of Eritrea have been working diligently to their level best to develop the area of water resources. It is worth reporting that a number of dams (reservoirs) and diversion structures have been constructed for different uses. But, according to Garry et al (1998), the complete absence of historical stream flow data as well as of climatological records in Eritrea has been yet the primary constraint in the planning and design of water resources development projects. Therefore, to have an adequate up to date hydrometeorological data is a must. To meet this objective, the WRD and other government sectors have made the old stations functional and also established new hydrological and meteorological stations.

The backbone of the Eritrean economy is agriculture. More than 70% of the Eritrean population depends on agriculture and its allied fields which include crop production, livestock, forestry, and traditional fishing. The government's ultimate goal of guarantying food security can be achieved by introducing modern technology based irrigation systems and following rational soil and water conservation practices with less dependence on total rain water irrigation. A proper evaluation of availability of water by carrying out scientific hydrological investigations of all the river basins is the first step in this direction. It will also help in proper runoff computations leading to estimation of flood potential and protection against floods by suitably developing flood forecasting system to strengthen the economy of the country. As a first step towards it, the hydrologic inventory of the Mereb-Gash river basin is taken up as reported in the subsequent sections.

1.5 Purpose of the Study

The computation of direct surface runoff (DSRO) is indispensable for different engineering works which can be obtained from observed stream flow and rainfall data. However, for ungauged catchments or with some uncertainties of the recoded data, DSRO computation ends up with unsatisfactory results. In such circumstances, it is necessary to look for other techniques that use the geomorphologic and physiographic characteristics of the catchment. Both approaches will be attempted using the UH method for DSRO determination to the Mereb-Gash river basin with an emphasis on the upper Mereb-Gash (Ghergera and Debarwa) sub-basins. Specifically, this study has the following objectives:

i) To derive a unit hydrograph based on the observed stage records
ii) To develop a synthetic unit hydrograph based on physiographic characteristics
iii) To develop a UH based on geomorphologic instantaneous unit hydrograph (GIUH) based Nash Model

After deriving the various UHs using the above methods, the one which seems to be suitable for DSRO computations in the specified study area will be recommended.

1.6 Layout of the Thesis

The subject matter of this thesis has been laid out in eight chapters. The first chapter highlights the importance of stream flow data, east Africa's water resources, general profile of Eritrea and the objectives of the study. The second chapter describes briefly about Eritrea's major river basins and it's hydrometeorology.

The methodology adopted in this study is covered under six chapters; description of study area and available data in the third chapter, data processing and analysis in the fourth, UH application, derivation and synthetic unit hydrograph in the fifth, and geomorphologic instantaneous unit hydrograph based Nash model in the sixth chapters. The summary of results obtained from the preceding chapters is presented and discussed in the seventh chapter. Finally, suitable drawn inferences and recommendations are presented in the last chapter.

CHAPTER-2

RIVER BASINS OF ERITREA

2.1 Introduction

A drainage basin is the area bounded by the highest contour called water divide from where precipitated water is collected by surface and subsurface flows and drained out through the natural river. Evaporation, rainfall and stream flow are the most important parameters for hydrometeorological studies of a drainage basin. Evaporation is used to determine water losses from lakes, ponds, and reservoirs as well as in assessing the water requirements of crops. Estimates of evaporation from river basins are used for conceptual hydrological modeling. Rainfall data are used for the design and construction of various water resources projects. By deducting evaporation losses from rainfall surface runoff can be estimated. Long period stream flow data are required for the assessment of the water yield of river basins and for sizing the storage capacity of dams. Keeping view of aforesaid facts, the theme of this chapter is to provide an overview of the major river basins and hydrometeorology of Eritrea.

2.2 Major River Basins

Eritrea has five major river basins viz. Setit, Mereb-Gash, Barka-Anseba, Red Sea and Danakil Depression basins (Fig. 2.1). The first two rivers confluence in the Sudan, and eventually join the Nile River. On the other hand, the Barka-Anseba and the Red Sea drainage basins drain in to the Red Sea. The river systems, with a narrow strip of catchment along the south-eastern bordering Ethiopia, flow into the Danakil basin.

2.2.1 Setit River Basin

The Setit river is the only Eritrea's perennial waterway, albeit a non-navigable one. It originates from north of Lake Tana (the main source of Blue Nile, Ethiopia) joining the western border and flows into the Sudan. Part of the basin area that contributes to this river from the Eritrean territory is estimated to be 7,517 km^2. It has an estimated total annual flow of $8{,}000 \times 10^6$ m^3 (Zerai, 1996).

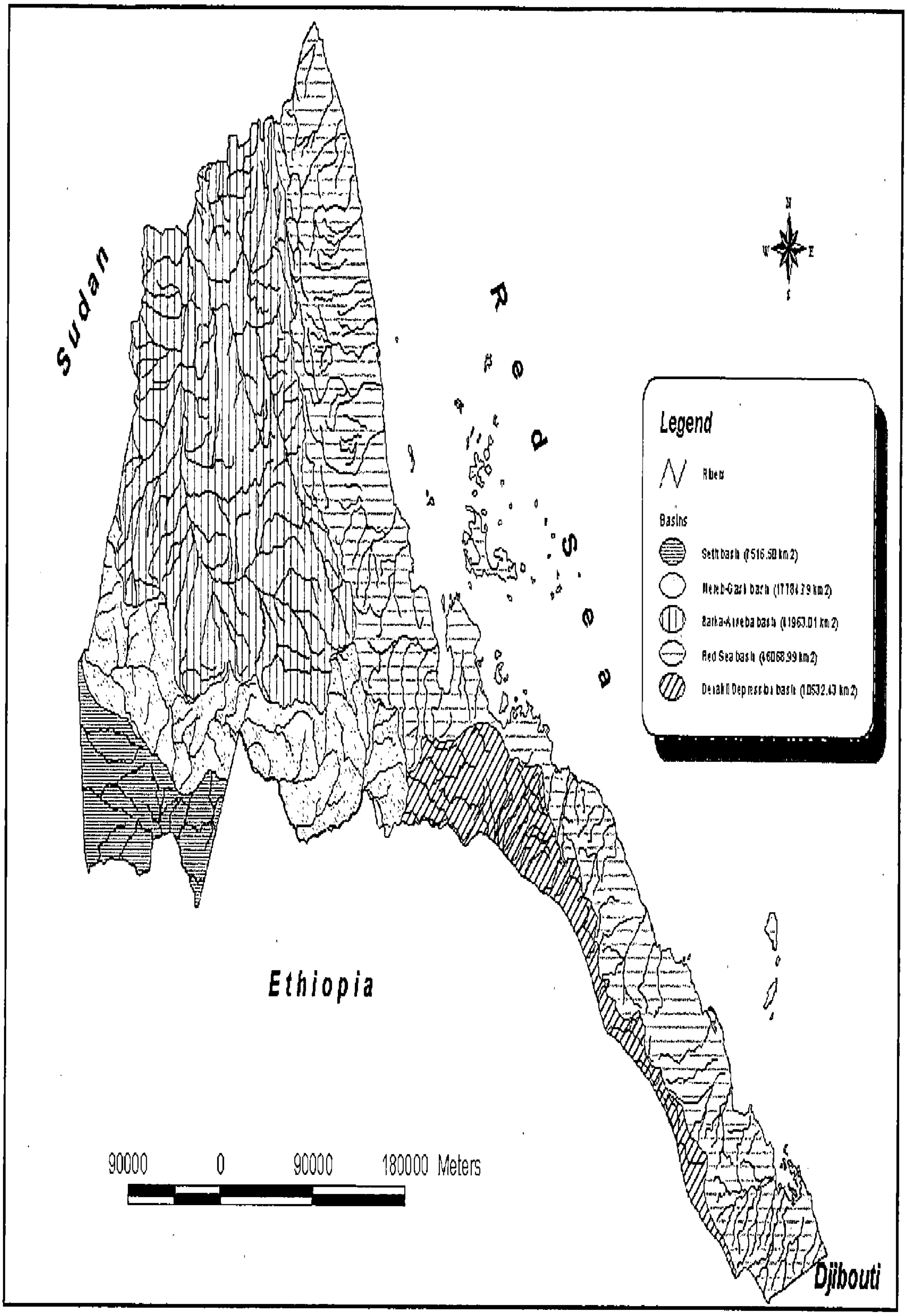


Fig. 2.1 River basins of Eritrea (*Source*: WRD, Eritrea)

2.2.2 Mereb-Gash River Basin

The Mereb-Gash river basin is one of the major river basins in Eritrea. The river flows out of north of Emba-Takara (central Eritrea), which partly forms a frontier between Eritrea and Ethiopia. It further flows south, bordering Ethiopia, then west through western Eritrea to reach the Sudanese plains near Kassala. At its upstream course it is called Mereb and Gash in the downstream western plains. Unlike the perennial Setit River, the Mereb is dry for much of the year subjected to sudden floods during the rainy season. Average annual discharge for the Mereb-Gash at Kassala over the years 1907-1929 was 430×10^6 m^3 (Zerai, 1996). Its main tributaries are Obel, Tsorona, Belesa, and many others. It has an area of 21,805 km^2 (with its outlet near the town of Tessenei) of which more than 82% lies on the Eritrean territory.

2.2.3 Barka-Anseba River Basin

Barka-Anseba river basin is a combined basin of two rivers viz. Barka river and Anseba river both originating from the central highlands of Eritrea which flow north-west. They join at the border with the Sudan eventually emptying their contents into the Red Sea. Like the Mereb-Gash, the river systems in this basin are seasonal and have a total basin area of 41,963 km^2.

2.2.4 Red Sea River Basin

The Red sea basin consists of the entire area along the Red Sea coast. It does not have a definite single main river; instead it has a number of small rivers directly flowing to the Red Sea. It is the biggest basin in the country with an area of about 46,069 km^2 and is characterised by the low and erratic rainfall as it lies along the coastal zones. Garry et al (1998) identified the flows in the rivers of this basin as having high velocities transporting large sediment loads in rocky channels due to the steep, granitic topography of the east-facing escarpment.

2.2.5 Danakil Depression River Basin

The Danakil Depression river basin lies along the south-eastern border of Eritrea and drains into the Danakil depression. Due to its closed topography and arid climate, it is characterized by highly saline soils and has little agricultural potential. The Danakil basin is a very dry basin within the Rift Valley having an area of around10,532 km^2. It receives less than 200mm of annual rainfall, the majority of it receiving less than 100 mm.

2.3 Hydrometeorology of Eritrea

Hydrometeorological data are the national wealth of a nation. They are required to assess, develop and manage the water resources of a country and its water related environment. For instance, these data help to understand the far-reaching effects of climate on the economy and welfare of mankind. Climate is distinguished from other natural resources by its strongly felt temporal as well as spatial variations. As such, climate and its effects cannot be assessed by a single-time survey rather requires a consistent long time records. Trends, which short-term data might show, cannot be easily extrapolated to provide reasonable forecasts of future changes. For these reasons, hydrometeorology monitoring must be continuous, and accumulation of reliable data over long periods is essential for interpreting spatial as well as temporal changes.

Based on the preceding reality and on the fact that a study of water resources rightly commences with rainfall measurement, many of the rainfall monitoring sites in Eritrea were established prior to 1900, for instance, Massawa 1885, Mendefera 1895 and the like. Despite the early establishment of climate monitoring sites, there exists an interruption in the time series of rainfall for a considerable time period possibly due to the war for independence in the proximity of almost all the stations. Currently, the WRD together with other government sectors undertake monitoring of hydrometeorology at different sites in the country on an operational basis. Accordingly, there are 19 stream gauging stations (Fig. 2.2) and 112 agro-meteorological sites established over the entire country (Fig. 2.3).

Rainfall in Eritrea is characterized by high intensity over a short duration, torrential, very unpredictable, and occurs sporadically. Owing to the rugged nature of the highlands, thin soil formations and largely deforested terrain, most of the rain develops in to flash floods. Thus, soil-water interaction is very low. Though there are favourable geological formations in the lowlands, infiltration is also low due to high evaporation rates which may go up to 8,000 mm per year in the Gash-Barka area, being the highest from the Sudano-Sahelian Region (FAO, 2005) and high intensity of rainfall. As it is shown in Fig. 2.4, the southern lowlands and highlands receive rainfall above 500 mm per annum where as the plane areas along the Red Sea coast and the far north receive very small amount of rainfall with a considerable area getting an annual rainfall of less than 100 mm. Even though, there is no information available on the condition of historical stream flow data and different meteorological parameters of the country's river systems at present, they are highly variable among the basins and even within a single basin itself due to its topographical variations, regional and global weather factors.

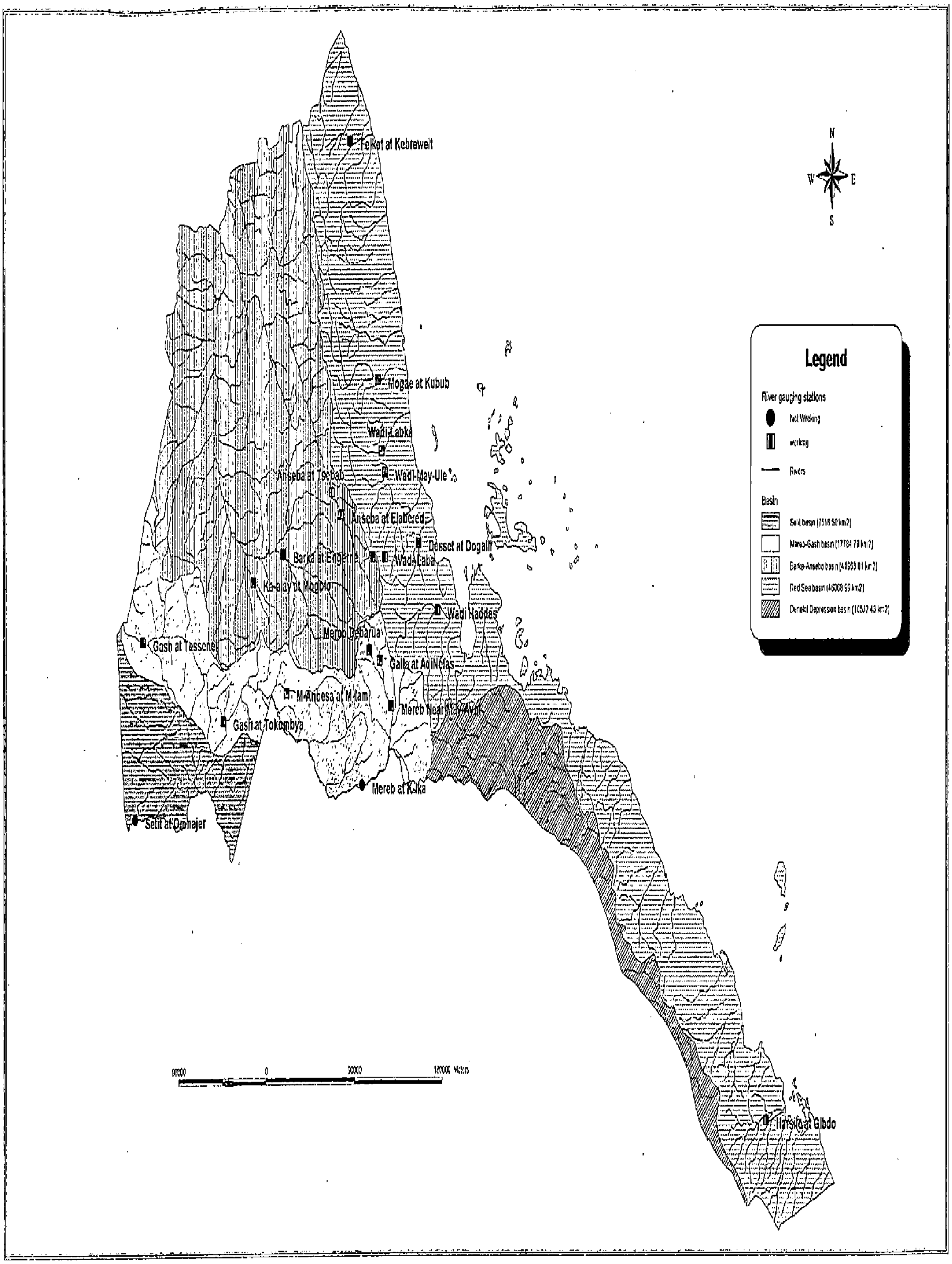


Fig. 2.2 Stream gauging network (*Source*: WRD, Eritrea)

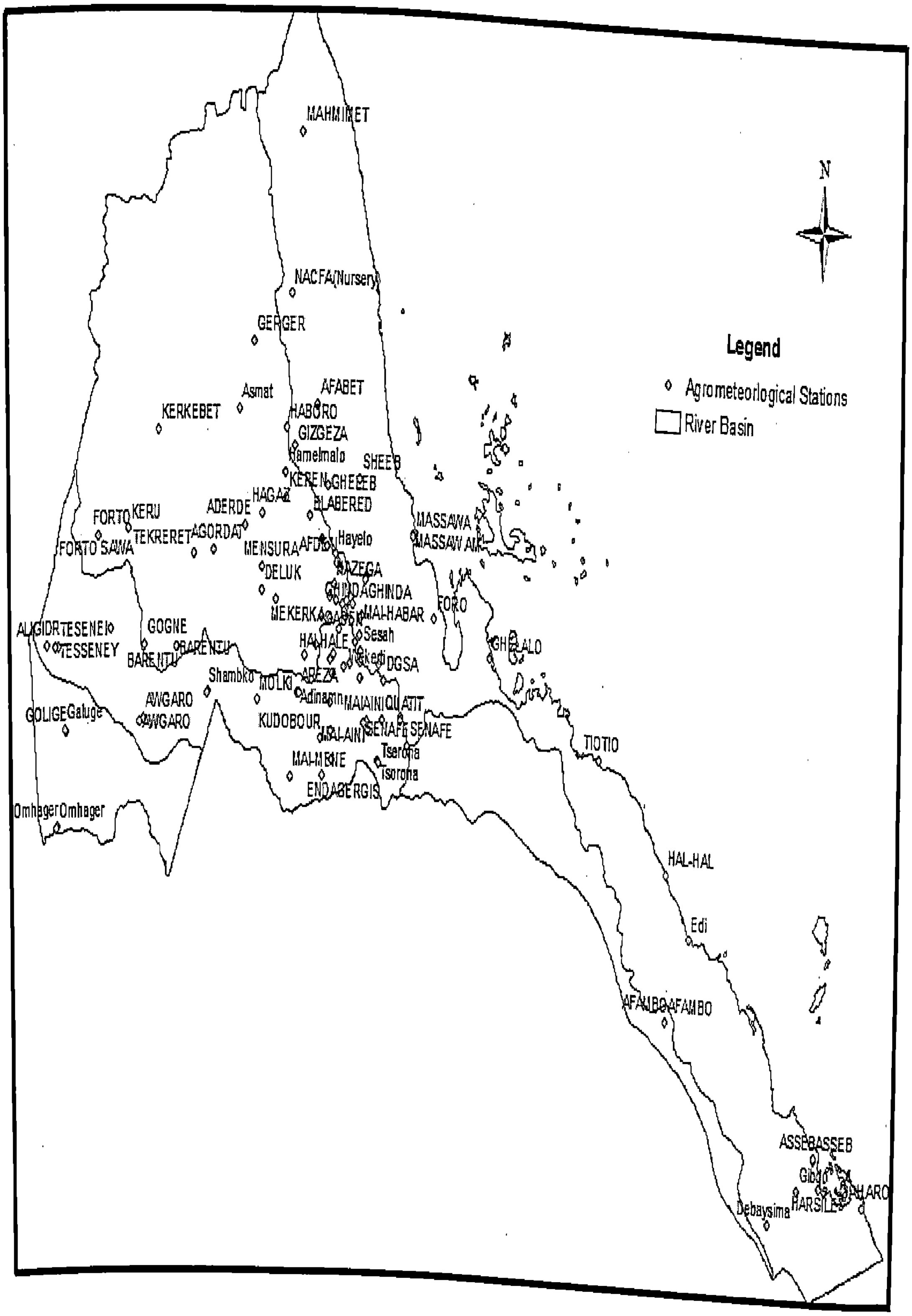


Fig. 2.3 Agro-meteorological sites (*Source*: WRD, Eritrea)

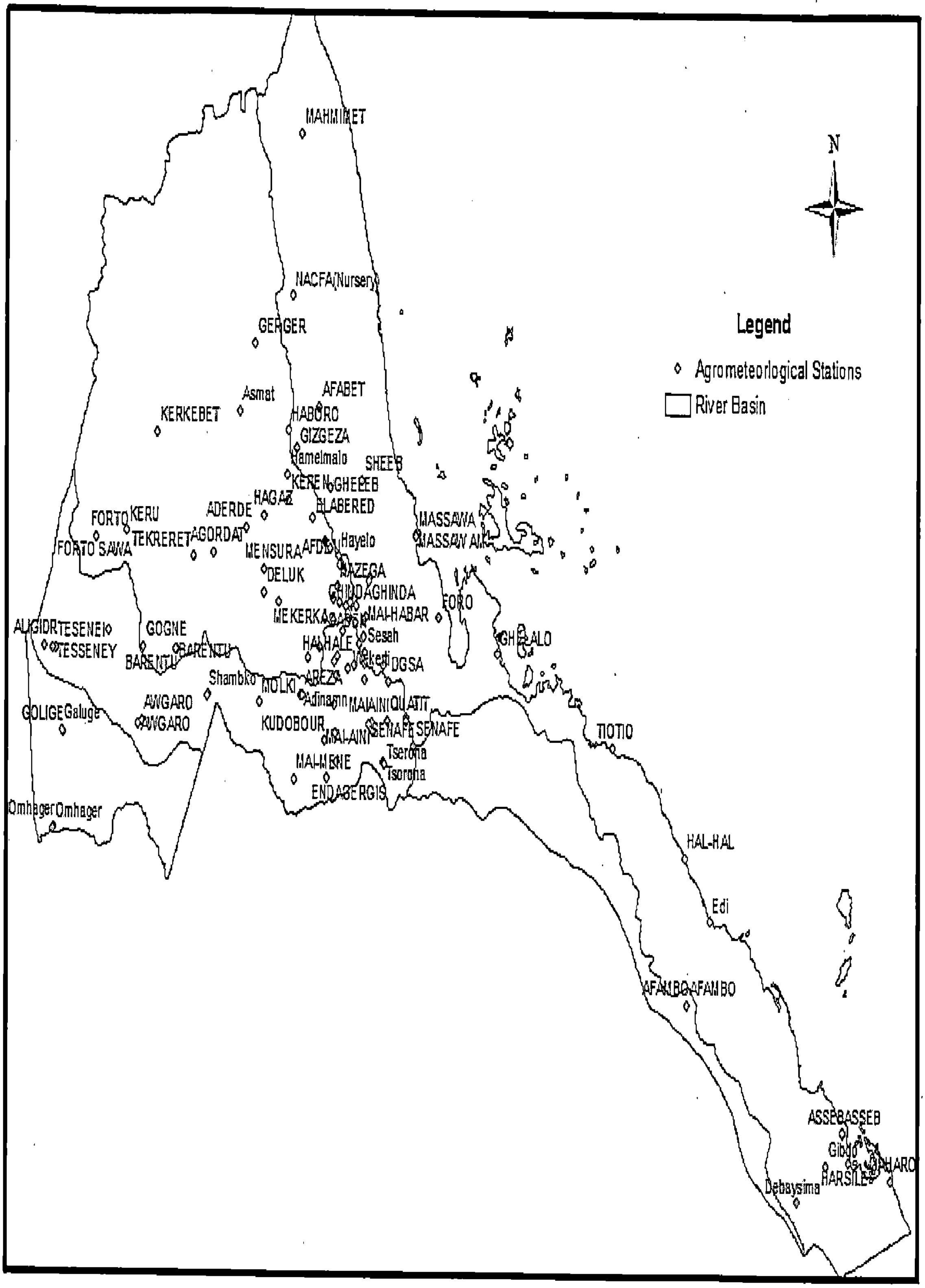


Fig. 2.3 Agro-meteorological sites (*Source*: WRD, Eritrea)

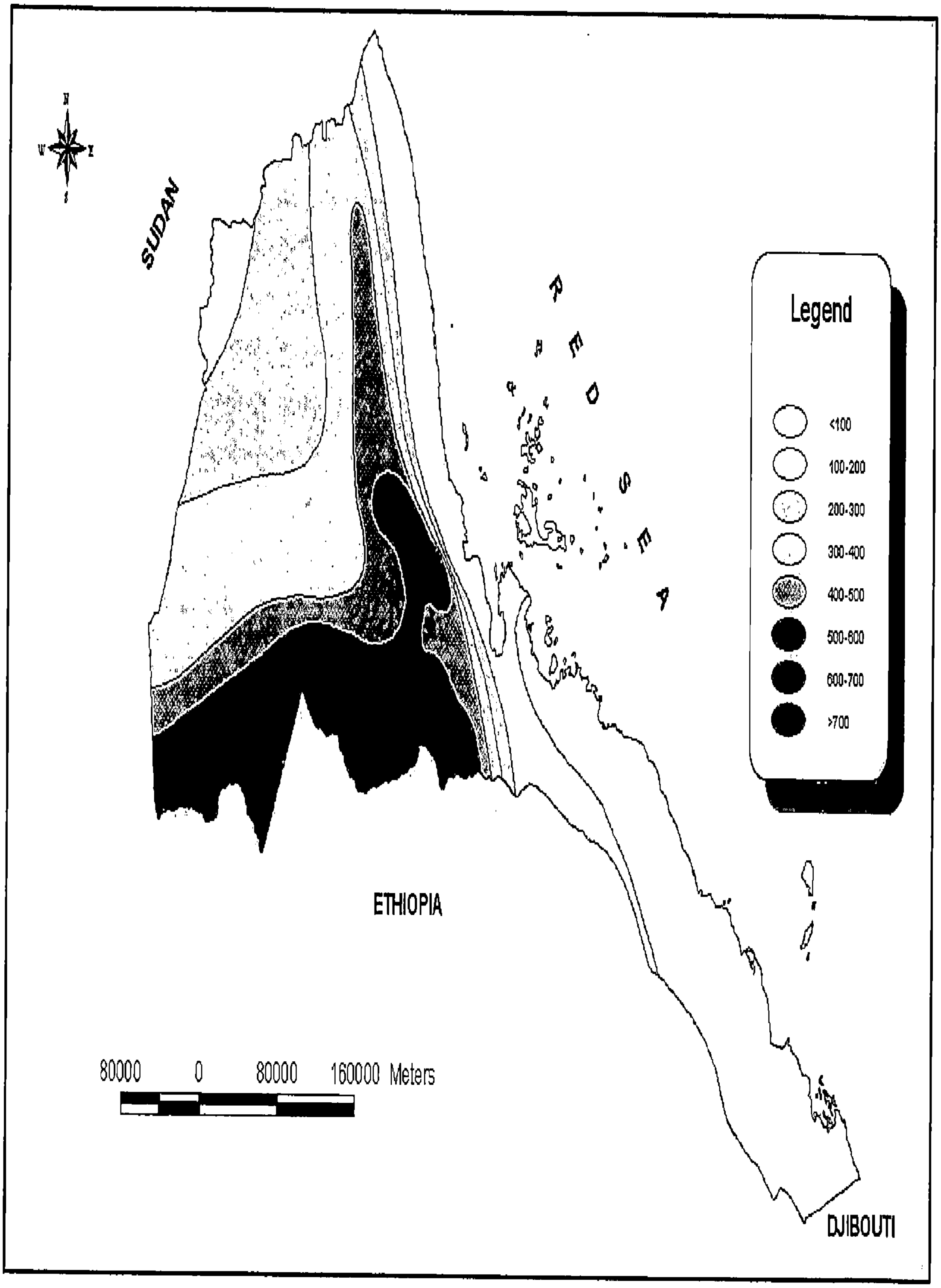


Fig. 2.4 Mean annual rainfall in mm (*Source:* WRD, Eritrea)

CHAPTER-3

DESCRIPTION OF STUDY AREA AND AVAILABLE DATA

3.1 Study Area

As mentioned in chapter 1, the study focuses on hydrological inventory of the Mereb-Gash river basin based on a short time historical data available for a selected pilot project at the upper Mereb-Gash and it's geomorphologic and physiographic characteristics.

3.1.1 Location

The main stream channel of the Mereb-Gash river basin with its outlet near the town of Tessenei (Latitude 11°07'40" N and Longitude 36°40'54" E), is 489 km long having a basin area equal to 21,805 km^2. It has a maximum elevation of 3,260 m above msl at a place located beyond the international boundary of Eritrea (in Ethiopia) and the minimum at Tessenei is 594 m above msl. The basin and main channel average slopes are 11.9% and 0.18%, respectively.

The upper Mereb-Gash at Ghergera is located at south of Asmara and at the eastern part of the basin where the main river originates (Fig. 3.1). Specifically, the outlet at Ghergera is 15°00'17"N Latitude and 38°55'22"E Longitude adjacent to the village - Adi-Ghergera that lies in the Debarwa administrative sub-zone, Southern Region. It has a drainage area of about 526 km^2. According to Dawit (2008), there are three catchments that lie within the upper Mereb; Mereb at Debarwa, Gaul-Mereb and Mai-Ququat. The outlet at Debarwa which is at the upstream of Ghergera, is located on 15° 05'49"N Latitude and 38°50'11"E Longitude at the bridge on the Asmara – Debarwa road having a drainage area of 195 km^2. The other two sub-catchments are very small in size and are beyond the scope of this study.

3.1.2 Climate and Land Cover

Even though sufficient historical hydrometeorological data is unavailable, from the agro-ecological classification of Eritrea as discussed in chapter 1, the Merb-Gash basin lies generally in moist highland and moist lowland zones with a small portion of it being in the arid lowland zone which results the basin to have a highly variable climate. It's temperature varies with a minimum of 0°C in the moist highlands to a maximum of more than 41°C at Tessenei. The pilot study area lies in the moist highlands zone receiving an average annual rainfall of 547 mm at the mining area based on 100 years records of daily amounts of rainfall. Climate in the catchment can be characterized as mild or cool with December-January being

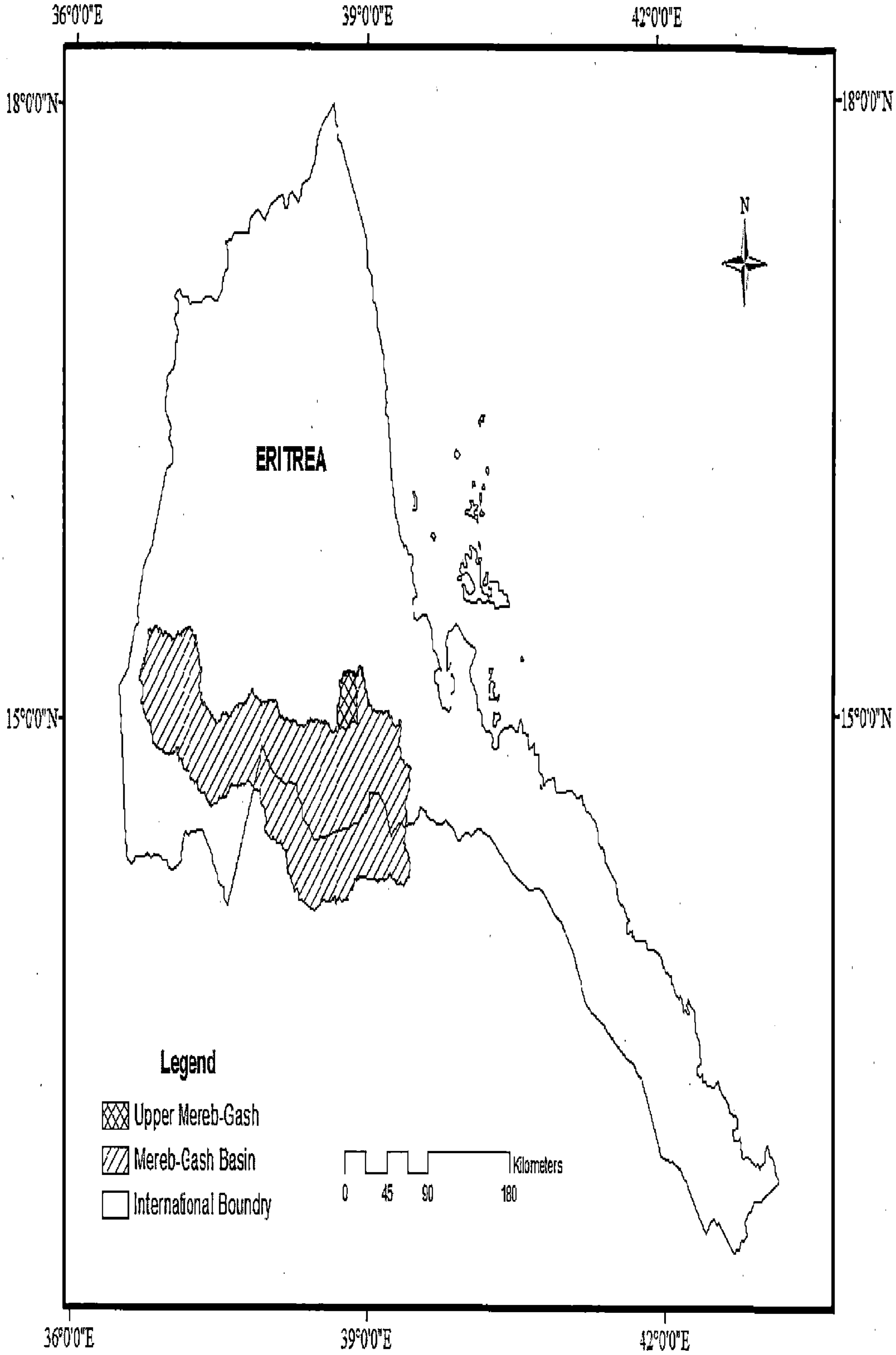


Fig. 3.1 Location map of study area

the coldest and March-April the hottest months. Maximum precipitation occurs in summer season specifically in the months of July and August with a mean rainfall of 185 mm and 175 mm, respectively. The area experiences a minimum temperature of 0°C during the month of January and the mean maximum being 32°C based on the data recorded on the Asmara meteorological station (Dawit, 2008).

The general land cover map is prepared by downloading an Enhanced Thematic Mapper Plus (ETM+) with GeoTIFF type images produced by United States Geological Survey (USGS) which is freely accessible in the web http://glcf.umiacs.umd.edu/index.shtm. The images have a resolution of 30 m, projection: UTM Zone 37, Spheroid and Datum: WGS 84. The layers 2, 3 and 4 (green, red and near infrared) are stacked to obtain false coloured image and thereafter it has been processed with the help of Earth Resources Data Analysis System (ERDAS) for the entire basin and that of upper Mereb-Gash sub-basin is extracted as shown in Fig. 3.2.

3.1.3 Normalized Difference Vegetative Index (NDVI)

Normalized Difference Vegetative Index (NDVI) is a numerical indicator that uses the visible and near infrared bands of the electromagnetic spectrum, and is adopted to analyze remote sensing measurements and assess whether the target being observed contains live green vegetation or not (Rouse et al, 1973). NDVI has found a wide application in vegetative studies as it has been used to estimate crop yields, pasture performance, and rangeland carrying capacities among others. It is often directly related to other ground parameters such as percent of ground cover, photosynthetic activity of the plant, surface water, leaf area index and the amount of biomass.

Generally, healthy vegetation will absorb most of the visible light that falls on it, and reflects a large portion of the near-infrared light. Unhealthy or sparse vegetation reflects more visible light and less near-infrared light. Bare soils on the other hand reflect moderately in both the red and infrared portion of the electromagnetic spectrum (Holme et al, 1987).

An attempt has been done to know the vegetative cover of the basin using the NDVI. From the aforesaid satellite images, the false colours 2, 3 and 4 (green, red and near infrared) are used to obtained NDVI map with the help of Earth Resources Data Analysis System (ERDAS) and ArcGIS softwares. The NDVI is computed as follows.

$$NDVI = \left(\frac{NIR - R_l}{NIR + R_l} \right) \tag{3.1}$$

where NIR - Near infrared light; and R_L - Red light

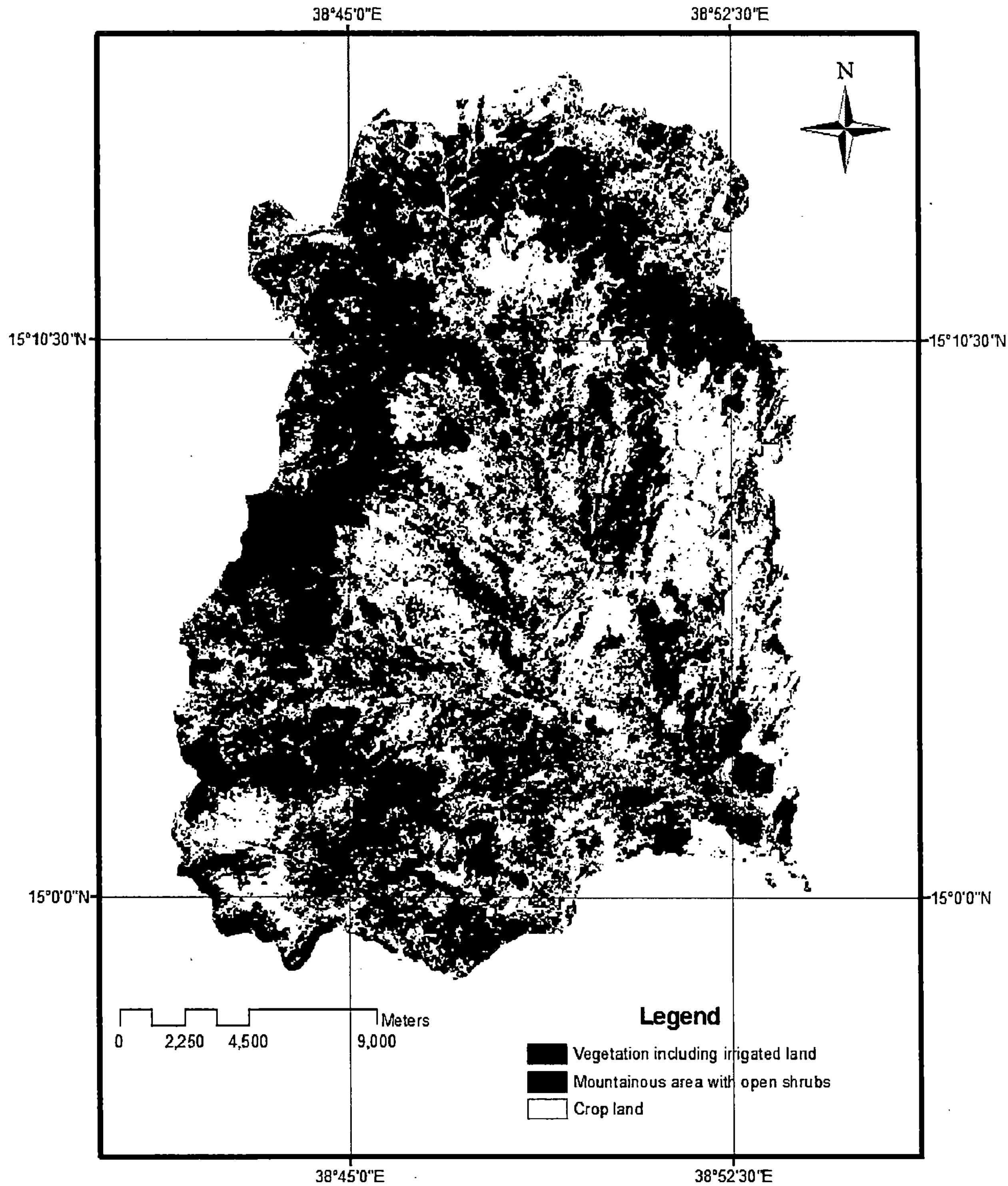


Fig. 3.2 Unsupervised classes of upper Mereb-Gash sub-basin

Theoretically, NDVI values are represented as a ratio ranging from -1 to 1. In practice extreme negative values represent water, values around zero represent bare soil and values over 0.6 represent dense green vegetation. After reviewing a number of literatures for interpretation of the NDVI values, the ones shown in Table 3.1 are adopted for producing the general land cover map of the Mereb-Gash basin (Fig. 3.3). The majority of pixels are found to have an NDVI values that lie below zero which is assumed to represent a barren land owing to satellite images were taken during dry season (most of them in the months of October, November and April) and the absence of large water bodies in the area.

The NDVI map is obtained after applying 5 x 5 summary convolution filtering and other necessary image enhancements to the false coloured images. NDVI map for each enhanced image is prepared and thereafter all five NDVI maps are mosaiced out of which area of interest is extracted.

Table 3.1 Land cover based on NDVI values

S. No	Land Cover	NDVI	Pixel Count	% Cover
1	Crop land and Barren Land	-0.32834 – -0.042413	7070740	79.6
2	Mountainous area with open shrubs	-0.042413– 0.1	1652612	18.6
3	Dense vegetation	0.1 – 0.597503	157425	1.8

3.1.4 Geomorphology

The geomorphology which refers to the topographic and geometric properties of channel networks is important as it can be employed in synthesizing and understanding the catchments hydrologic behaviours. Visualization and analysis of spatially oriented data are the primary strengths of a Geographic Information System (GIS). As such, the Mereb-Gash basin characteristics, stream network and slope maps as given in Fig 3.4 and Fig 3.5 are prepared by processing the Shuttle Radar Topographic Mission (SRTM) 90 m resolution Digital Elevation Model (DEM) data produced by NASA using ArcGIS. A mosaiced $5° \times 5°$ tiles are freely available on the web http://srtm.csi.cgiar.org/. Some of the important physiographic parameters of the Mereb-Gash river basin at different outlets excluding pilot study area (Fig 3.6) which are used for analysis purposes are given in Table 3.2. Majority of the selected outlets have an existing stream gauging stations and the remaining ones are selected arbitrarily.

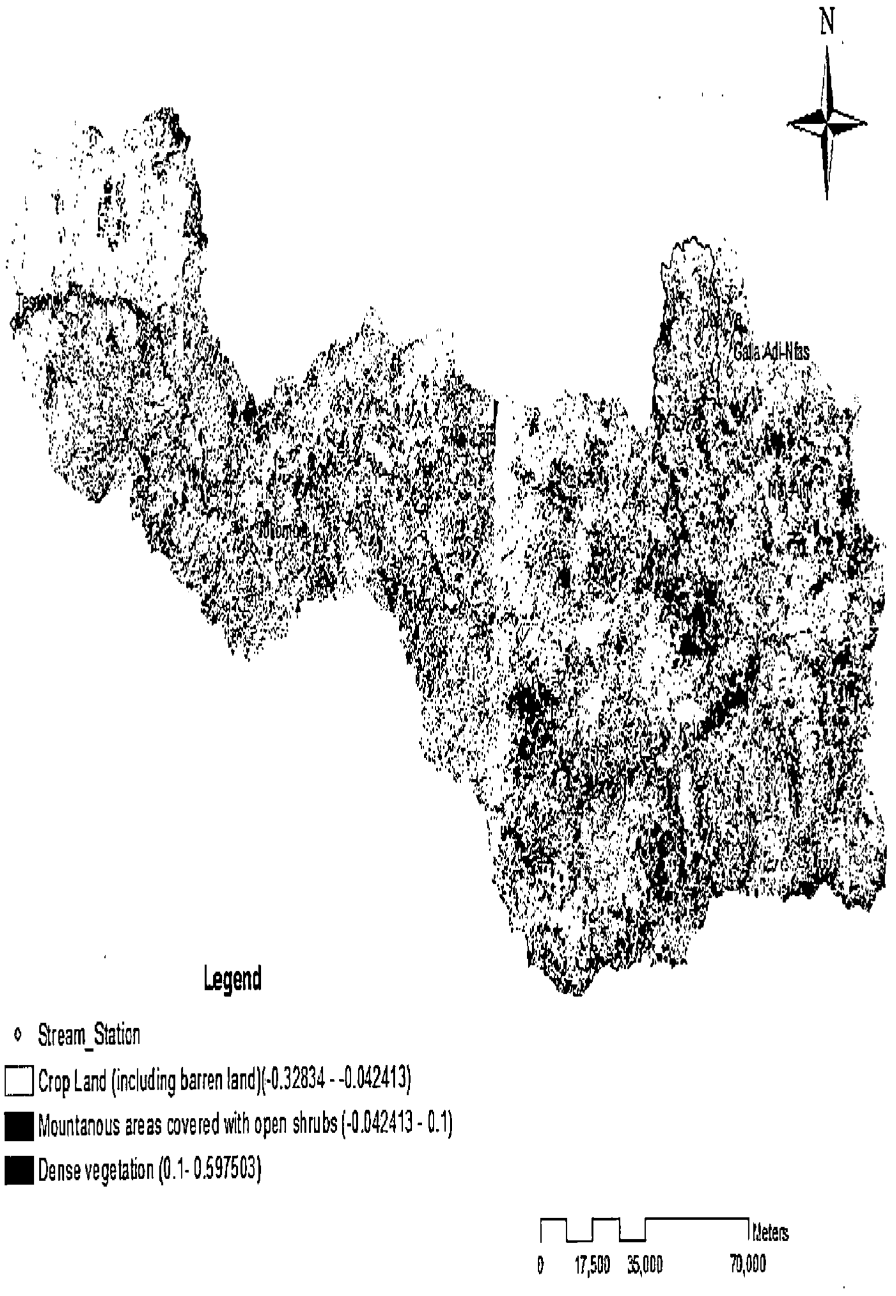


Fig. 3.3 NDVI map of Mereb-Gash basin

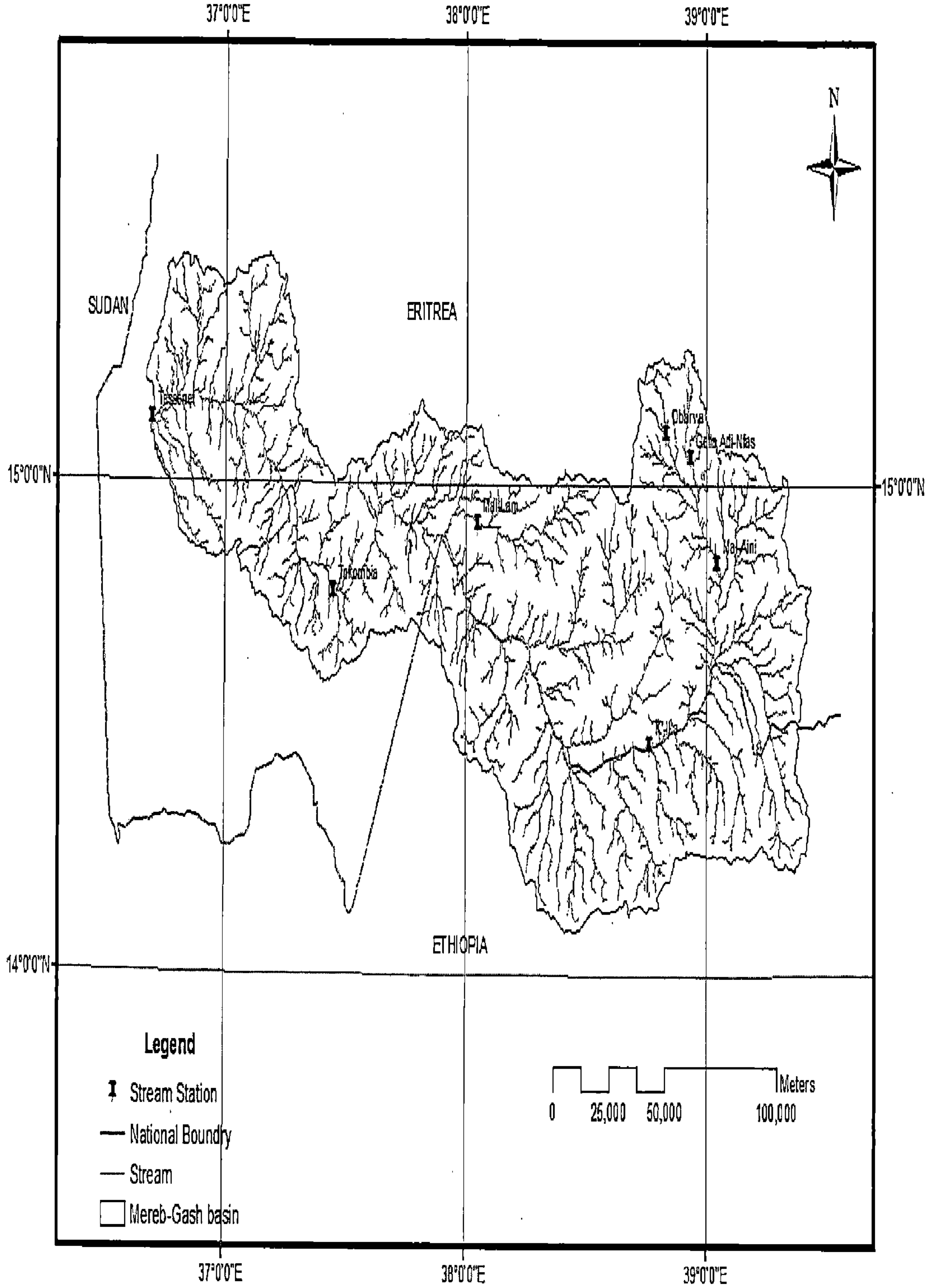


Fig. 3.4 Stream network of Mereb-Gash basin

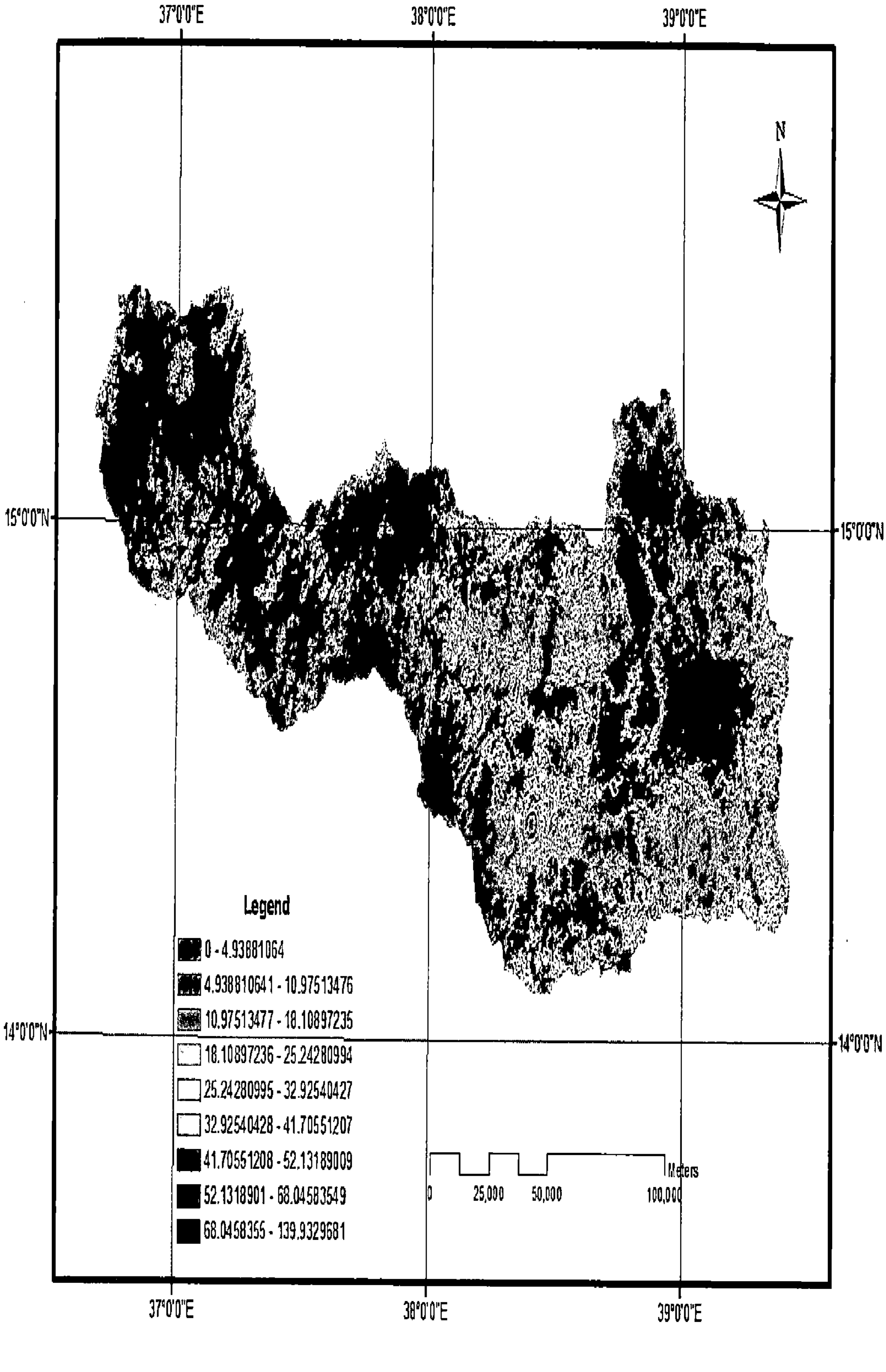


Fig. 3.5 Slope map of Mereb-Gash river basin

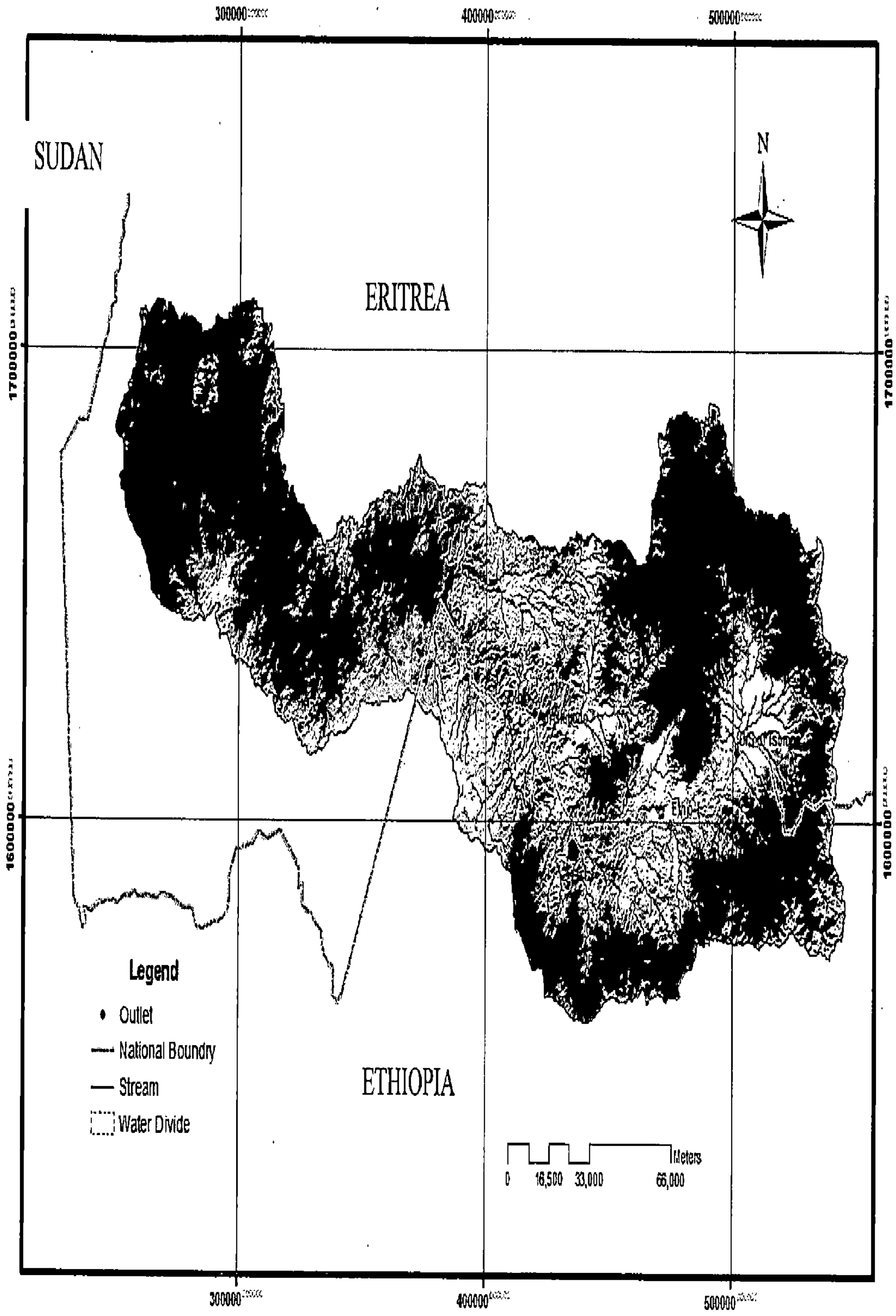


Fig. 3.6 Painted relief map showing the selected outlets for analysis

Table 3.2 Some physiographic parameters of Mereb-Gash basin at selected outlets

Physiographic parameter	*Tsorona	Adi-Chigono	Tokombia	Tessenei	**Ethio1	**Ethio2
A - Drainage area (km^2)	4,652.755	11,553.576	16,873.45	21,805.244	319.481	515.711
L - Main channel length (km)	107.415	229.704	360.125	489.338	43.301	49.047
L_{ca} - distance along the main water course form the gauging station to a point opposite to the centroid (km)	27.329	94.878	167.894	260.629	26.011	27.344
S - Average basin slope (%)	12.6	14.6	13.2	11.9	15.7	9.4
Latitude	14°36′34″	14°40′20″	14°46′37″	11°07′40″	14°48′44″	14°28′22″
Longitude	39°01′08″	38°15′54″	37°27′51″	36°40′54″	38°09′04″	38°48′19″

*at about 25 km downstream of Tsorona town

**Ethio1 and Ethio2 are outlets in the Mereb-Gash representing for sub-basins beyond the international boundary of Eritrea (in Ethiopia).

3.1.5 Delineation

A catchment area is the upslope area contributing flow to a given location. Delineating catchment area is the first step in conducting hydrologic studies. Therefore, the Ghergera catchment is delineated using ArcGIS from a DEM of 50 m resolution obtained from the WRD. On the other hand, the SRTM (DEM 90 m resolution) data is used for delineating the Mereb-Gash basin as the former is available only for the upstream part of the basin. By applying a threshold value of con (> 1000, 1) to the results of the flow accumulation function of both the DEMs, a stream network was delineated. Determining a threshold value that represents where a permanent stream or stream channel begins is affected not only by contributing area but also by climate, slope, and soil characteristics. The aforesaid value is selected arbitrarily and is adopted as the stream network obtained is quite satisfactory. The stream order function used for producing Fig. 3.4 and Fig. 3.7 is the method proposed by Strahler (1957). On the basis of delineated area and its river network, the geomorphologic parameters of the upper Mereb-Gash river basin is obtained and are tabulated in Table 3.3.

3.2 Data Availability

The amount, intensity and areal distribution of rainfall are essential in many hydrological studies. On the other hand, stream flow is required for the efficient day to day management and regulation of a river system. Design, planning and hydrological modeling are some of the important aspects of the water resources projects where the stream flow data are utilized. Specifically, it is needed for reservoir sizing, determining the risk of failure/reliability of water supply for irrigation and hydropower systems, planning future reservoir operation and capacity expansion of water supply systems. Rainfall and steam flow data collected from the field as such cannot be utilized in hydrological studies. However, processing of the raw data is extremely essential in order to reduce in a manner to suit them for analysis as is discussed in the subsequent sections.

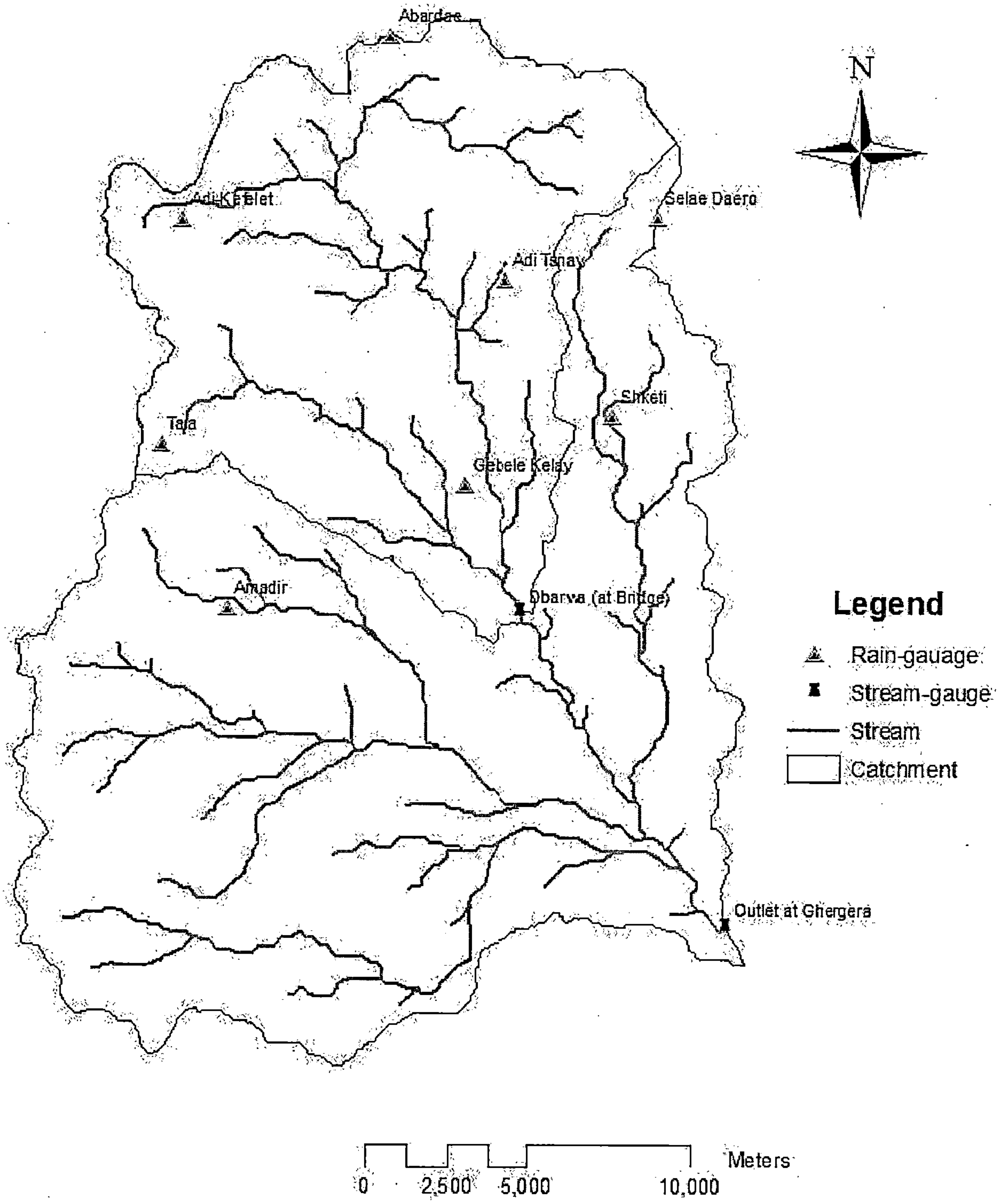


Fig. 3.7 Stream network of upper Mereb-Gash sub-basin

Table 3.3 Geomorphologic parameters of the upper Mereb-Gash sub-basin

Symbol	Description	Unit	Sub-basin	
			Debarwa	Ghergera
A	Drainage area	km^2	194.646	525.726
P	Perimeter of the catchment boundary	km	71.943	118.396
L	Main channel length	km	29.597	42.91
L_{ca}	Distance along the main water course from the gauging station to a point opposite to sub-basin centroid	km	15.191	18.821
S	Average sub-basin slope	%	12.5	11.6
$\overline{S}_{cl}$	Average stream channel slope	%	2.01	1.63
$\overline{S}_{ol}$	Average overland slope	%	10.80	8.57
h_{min}	Minimum elevation above msl (m)	m	1857.00	1714.00
h_{max}	Maximum elevation above msl (m)	m	2585.00	2585.00
H_R	Sub-basin relief	m	728.41	870.71
L_w	Total basin stream length (km)	km	94.926	264.915
D_d	Drainage density	km/km^2	0.4877	0.5039
C_m	Constant of channel maintenance	km^2/km	2.0505	1.9845
R_h	Relief ratio	-	0.0416	0.0322
$\overline{L}_o$	Length of overland flow	km	1.0253	0.9923
R_s	Slope ratio	-	1.7977	1.3285
R_e	Elongation ratio	-	0.4498	0.4793
R_{sh}	Shape factor	-	3.6434	3.2415
R_f	Form factor	-	0.6356	0.7212

3.2.1 Rainfall

Even though the annual rainfall records in the Debarwa area monitoring site reveals that data recording was started back in 1927, the missing data from 1935 to 1942 and 1947 to 1991 makes time series inconsistent. In spite of the presence of quite good rain gauge network in the catchment since 2004 (Table 3.4), both hourly and daily rainfall data are available only for the years 2004, 2005 and 2006. A monthly cumulative rainfall data for the period 1992 to 2007 is also available. It is not clear whether WRD maintains a network of groundwater and quality monitoring stations or not within the catchment.

Table 3.4 Rainfall stations at upper Mereb-Gash sub-basin

S.No	Station	Latitude	Longitude	Elevation (m)	Date of Installation
1	Abardae	15°15'21"	38°47'54"	2219	July 14, 2005
2	Adi-Kefelet	15°12'21"	38°44'23"	2265	June 24, 2004
4	Adi-Tsnay	15°11'23"	38°49'54"	2098	June 22, 2004
3	Amadir	15°05'58"	38°45'11"	2040	Not specified but record started in June 30, 2004
8	Gebele Kelay	15°08'01"	38°49'13"	1991	June 18, 2004
6	Selae Daero	15°12'22"	38°52'31"	2292	July 14, 2005
5	Shketi	15°09'07"	38°51'46"	2046	June 24, 2004
7	Tala	15°08'39"	38°44'00"	2292	Not specified but record started in July 2, 2004

3.2.2 Stream Flow

As mentioned above, the Ghergera sub-basin has two stream gauging stations at Ghergera and Debarwa. No rainfall or stream flow data is available for the other stations along the Mereb-Gash basin and is considered as ungauged. The mean daily flow data at Debarwa is available starting from 1997/1998 to 2007/2008 with the exception of 2004/2005 and 2005/2006 and hence, are considered as missing data. The majority of the flows in this river occurred during the wet season in the months of July through September. Stage levels are also available for both stations only for 2006 and 2007.

CHAPTER-4

DATA PROCESSING AND ANALYSIS

4.1 Rainfall

Rainfall is measured at a number of sample points. The amounts recorded at these points are utilized to form an estimate of mean areal rainfall. However, care must be exercised in employment of these rainfall records. Sometimes, these records are not consistent; at other times, the records at one station may not be complete. Inconsistencies in rainfall records may arise due to records for different gauging stations cover different periods of time, change in the locations of stations, change in observational procedures and change in the exposure of the instrument. To obtain homogeneity among and within measurements of precipitation, adjustment of data becomes indispensable. This is done to make the record homogeneous and to reduce errors by correction for change in location or exposure. Detection of heterogeneity in rainfall records can be identified by graphical methods - double mass curve analysis (Buishand, 1982) or statistical methods. Singh (1989) has discussed both the methods in detail. As the data available for the study area is of a short period, consistency checks are not carried out in this work.

4.1.1 Mean Areal Rainfall

Spatial distribution of rainfall on a catchment and the resulting runoff are affected by many factors. In this regard, determination of the average amount of rainfall that falls on a catchment during a given storm is a fundamental requirement for many hydrologic studies. A number of techniques for estimating mean areal rainfall have been developed. Some of these techniques are simple, well-tried and old, as old as modern hydrology (Horton, 1923), but they tend to be employed without sufficient appreciation of their limitations. An excellent comparison of the various existing methods was done by Singh and Birsoy (1975). They have concluded that there is no particular basis to claim that one method is superior to the other, although in a given situation one method may be preferable to another.

The Thiessen polygon is the most frequently used method (Williams and Haghoe, 1982). The advantage of this method is not only weightage is given to various stations on rational basis but also the influence of stations outside the area can be used effectively. However, various literatures indicate that it gives good results when there is a fair rain gauge network. The World Meteorological Organization (WMO) recommends a density of one station per 100 to 250 km^2 in mountainous regions of temperate, Mediterranean and tropical zones. The pilot study area, having a drainage area of less than 550 km^2, has quite good station density as it comprises six to eight. On the bases of the preceding facts, the Thiessen polygon method is adopted for determining the mean areal rainfall of the study area.

The rain gauge network in 2004 was different from the subsequent years for both the upstream and downstream catchments. Therefore, separate Thiessen polygons along with their weighted rainfalls are prepared for both catchments for the network in 2004 (Fig 4.1) and 2005 afterwards (Fig. 4.2) as given in Table 4.1. These figures are produced using Spatial Analyst tool in ArcGIS.

For n number of stations, the average rainfall over a catchment is given by

$$\overline{P} = \frac{\sum_{i=1}^{n} P_i A_i}{A} \tag{4.1}$$

where $\overline{P}$ - average rainfall over a catchment

P_i - rainfall magnitudes recorded by individual stations

A_i - respective areas of Thiessen polygons

A - catchment area

Table 4.1(a) Thiessen polygon weightages of upper Mereb-Gash in 2004

S.No	Station Name	Sub-area for Debarwa (km^2)	Sub-area for Ghergera (km^2)	Weightage for Debarwa	Weightage for Ghergera
1	Adi Kefelet	48.907	48.650	0.251	0.092
2	Adi Tsnay	74.880	85.014	0.385	0.162
3	Amadir	1.390	189.352	0.007	0.360
4	Gebele Kelay	44.290	114.692	0.228	0.218
5	Shketi	3.487	57.774	0.018	0.110
6	Tala	21.692	30.244	0.111	0.058
	Total	194.646	525.726		

Table 4.1(b) Thiessen polygon weightages of upper Mereb-Gash 2005 afterwards

S.No.	Station Name	Sub-area for Debarwa (km^2)	Sub-area for Ghergera (km^2)	Weightage for Debarwa	Weightage for Ghergera
1	Abardae	28.511	28.640	0.146	0.054
2	Adi Kefelet	39.534	39.023	0.203	0.074
3	Adi Tsnay	42.570	45.735	0.219	0.087
4	Amadir	1.390	188.660	0.007	0.359
5	Gebele Kelay	44.292	116.632	0.228	0.222
6	Selae Daero	13.168	21.673	0.068	0.041
7	Shketi	3.487	54.341	0.018	0.103
8	Tala	21.693	31.022	0.111	0.059
	Total	194.646	525.726		

After preparing the daily rainfall time series, weighted rainfall were computed for both the catchments when all the stations received rain. This is given in Table 4.2 and Table 4.3.

Table 4.2(a) Weighted average rainfall for Debarwa catchment (2004)

Date	Rainfall station along with its respective weightage						Weighted Rainfall (mm)
	Adi Kefelet **0.251**	Amadir **0.007**	Adi Tsnay **0.385**	Gebele Kelay **0.228**	Shketi **0.018**	Tala **0.111**	
04-07-2004	13.50 *3.39*	20.00 *0.14*	38.50 *14.82*	22.00 *5.02*	39.20 *0.71*	9.00 *1.00*	25.07
22-07-2004	10.00 *2.51*	3.30 *0.02*	5.20 *2.00*	24.00 *5.47*	8.20 *0.15*	2.30 *0.26*	10.41
06-08-2004	37.00 *9.29*	3.90 *0.03*	16.20 *6.24*	28.00 *6.38*	7.00 *0.13*	21.60 *2.40*	24.46
13-08-2004	25.80 *6.48*	1.50 *0.01*	91.00 *35.04*	36.20 *8.25*	25.00 *0.45*	5.00 *0.56*	50.78
14-08-2004	3.10 *0.78*	4.90 *0.03*	44.50 *17.13*	14.30 *3.26*	18.50 *0.33*	10.00 *1.11*	22.65
16-08-2004	8.70 *2.18*	2.60 *0.02*	23.70 *9.12*	17.20 *3.92*	23.00 *0.41*	24.00 *2.66*	18.33

Values in italics are weighted rainfall in mm for individual stations.

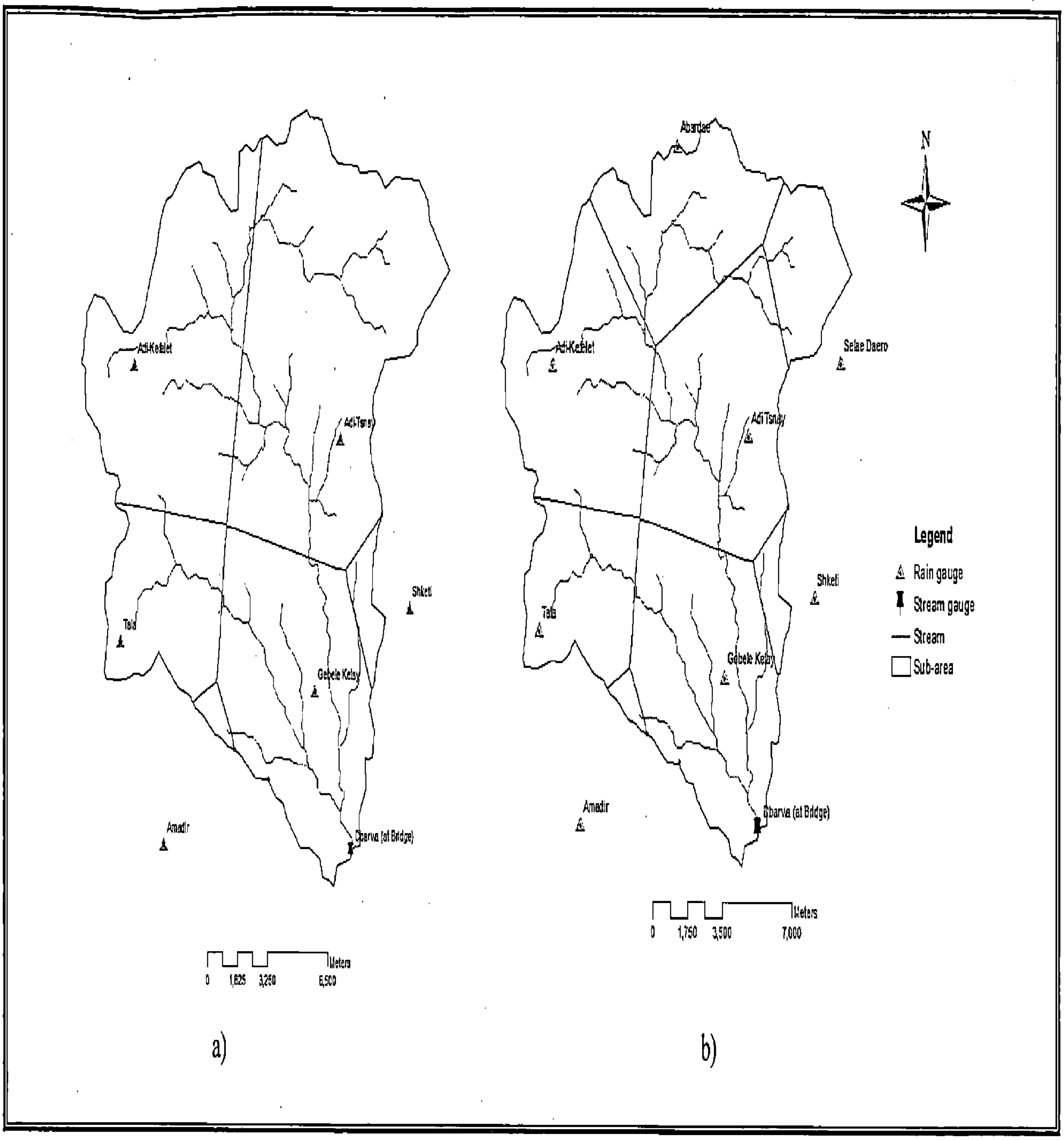


Fig. 4.1 Thiessen polygon for rainfall stations at Debarwa catchment (a) in 2004 (b) 2005 afterwards

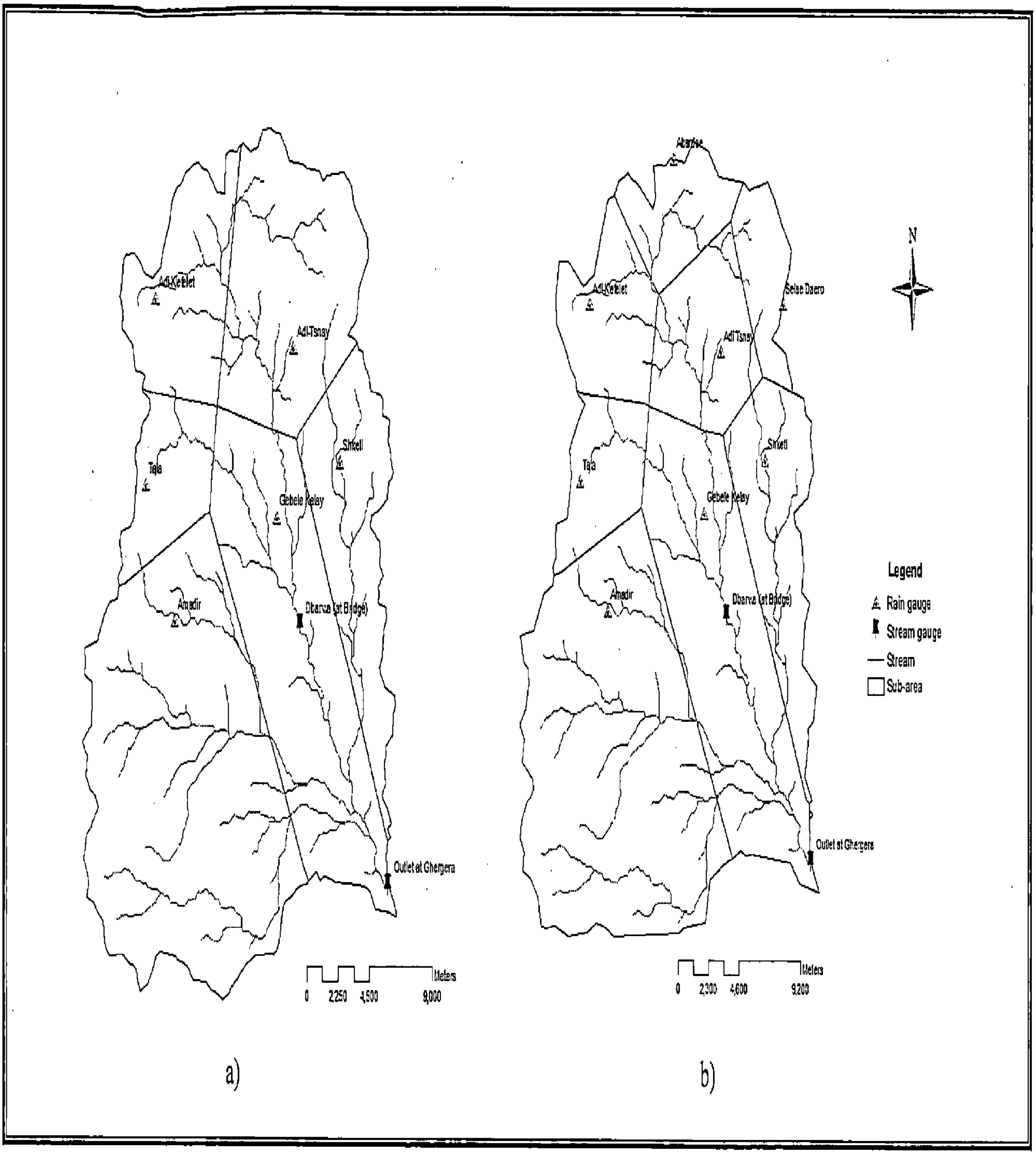


Fig. 4.2 Thiessen polygon for rainfall stations at Ghergera catchment (a) in 2004 (b) 2005 afterwards

Table 4.2(b) Weighted average rainfall for Debarwa catchment (2005)

Date	Rainfall station along with its respective weightage								Weighted Rainfall(mm)
	Abardae	Adi Kefelet	Amadir	Adi Tsnay	Gebele Kelay	Selae Daero	Shketi	Tala	
	0.146	**0.203**	**0.007**	**0.219**	**0.228**	**0.068**	**0.018**	**0.111**	
21-07-2005	7.80	2.40	22.30	3.10	20.00	5.20	3.90	5.40	
	1.14	*0.49*	*0.16*	*0.68*	*4.56*	*0.35*	*0.07*	*0.60*	8.04
22-07-2005	9.80	14.70	4.30	17.30	32.40	30.60	22.00	7.00	
	1.43	*2.98*	*0.03*	*3.79*	*7.39*	*2.08*	*0.40*	*0.78*	18.87
25-07-2005	11.70	2.50	14.80	39.10	16.00	22.30	3.00	3.00	
	1.71	*0.51*	*0.10*	*8.56*	*3.65*	*1.52*	*0.05*	*0.33*	16.43
29-07-2005	21.00	5.50	20.50	3.10	12.30	8.80	2.20	2.00	
	3.07	*1.12*	*0.14*	*0.68*	*2.80*	*0.60*	*0.04*	*0.22*	8.67
04-08-2005	25.50	4.50	4.10	12.90	7.30	2.50	4.90	5.00	
	3.72	*0.91*	*0.03*	*2.83*	*1.66*	*0.17*	*0.09*	*0.56*	9.97
07-08-2005	11.00	5.50	9.00	38.50	23.00	29.00	22.90	20.00	
	1.61	*1.12*	*0.06*	*8.43*	*5.24*	*1.97*	*0.41*	*2.22*	21.07
08-08-2005	13.00	8.00	1.70	6.30	8.80	32.70	6.20	10.00	
	1.90	*1.62*	*0.01*	*1.38*	*2.01*	*2.22*	*0.11*	*1.11*	10.37
27-08-2005	2.80	27.40	6.90	3.50	7.10	13.40	3.50	11.00	
	0.41	*5.56*	*0.05*	*0.77*	*1.62*	*0.91*	*0.06*	*1.22*	10.60
28-08-2005	12.00	39.50	13.60	42.10	20.40	45.50	3.10	13.00	
	1.75	*8.02*	*0.10*	*9.22*	*4.65*	*3.09*	*0.06*	*1.44*	28.33

Values in italics are weighted rainfall in mm for individual stations.

Table 4.2(c) Weighted average rainfall for Debarwa catchment (2006)

Date	Rainfall station along with its respective weightage								Weighted Rainfall(mm)
	Abardae	Adi Kefelet	Amadir	Adi Tsnay	Gebele Kelay	Selae Daero	Shketi	Tala	
	0.146	**0.203**	**0.007**	**0.219**	**0.228**	**0.068**	**0.018**	**0.111**	
05-07-2006	26.90	16.00	3.10	7.00	14.00	7.20	5.50	15.00	
	3.93	*3.25*	*0.02*	*1.53*	*3.19*	*0.49*	*0.10*	*1.67*	14.18
11-08-2006	8.50	10.50	14.00	36.00	11.30	9.80	9.90	10.00	
	1.24	*2.13*	*0.10*	*7.88*	*2.58*	*0.67*	*0.18*	*1.11*	15.89
12-08-2006	25.50	9.50	13.40	28.20	26.00	55.50	10.70	6.00	
	3.72	*1.93*	*0.09*	*6.18*	*5.93*	*3.77*	*0.19*	*0.67*	22.48

Values in italics are weighted rainfall in mm for individual stations.

Table 4.3(a) Weighted average rainfall for Ghergera catchment (2004)

Date	Rainfall Station along with its respective weightage								Weighted Rainfall(mm)
	Abardae	Adi Kefelet	Amadir	Adi Tsnay	Gebele Kelay	Selae Daero	Shketi	Tala	
		0.092	**0.360**	**0.162**	**0.218**		**0.110**	**0.058**	
04-07-2004		13.50	20.00	38.50	22.00		39.20	9.00	
		1.24	*7.20*	*6.24*	*4.80*		*4.31*	*0.52*	24.31
22-07-2004		10.00	3.30	5.20	24.00		8.20	2.30	
		0.92	*1.19*	*0.84*	*5.23*		*0.90*	*0.13*	9.22
06-08-2004		37.00	3.90	16.20	28.00		7.00	21.60	
		3.40	*1.40*	*2.62*	*6.10*		*0.77*	*1.25*	15.56
13-08-2004		25.80	1.50	91.00	36.20		25.00	5.00	
		2.37	*0.54*	*14.74*	*7.89*		*2.75*	*0.29*	28.59
14-08-2004		3.10	4.90	44.50	14.30		18.50	10.00	
		0.29	*1.76*	*7.21*	*3.12*		*2.04*	*0.58*	14.99
16-08-2004		8.70	2.60	23.70	17.20		23.00	24.00	
		0.80	*0.94*	*3.84*	*3.75*		*2.53*	*1.39*	13.25

Values in italics are weighted rainfall in mm for individual stations.

Table 4.3(b) Weighted average rainfall for Ghergera catchment (2005 and 2006)

Date	Rainfall station along with its respective weightage								Weighted Rainfall(mm)
	Abardae	Adi Kefelet	Amadir	Adi Tsnay	Gebele Kelay	Selae Daero	Shketi	Tala	
	0.054	**0.074**	**0.359**	**0.087**	**0.222**	**0.041**	**0.103**	**0.059**	
21-07-2005	7.80	2.40	22.30	3.10	20.00	5.20	3.90	5.40	
	0.42	*0.18*	*8.01*	*0.27*	*4.44*	*0.21*	*0.40*	*0.32*	14.25
22-07-2005	9.80	14.70	4.30	17.30	32.40	30.60	22.00	7.00	
	0.53	*1.09*	*1.54*	*1.51*	*7.19*	*1.25*	*2.27*	*0.41*	15.79
25-07-2005	11.70	2.50	14.80	39.10	16.00	22.30	3.00	3.00	
	0.63	*0.19*	*5.31*	*3.40*	*3.55*	*0.91*	*0.31*	*0.18*	14.48
29-07-2005	21.00	5.50	20.50	3.10	12.30	8.80	2.20	2.00	
	1.13	*0.41*	*7.36*	*0.27*	*2.73*	*0.36*	*0.23*	*0.12*	12.61
04-08-2005	25.50	4.50	4.10	12.90	7.30	2.50	4.90	5.00	
	1.38	*0.33*	*1.47*	*1.12*	*1.62*	*0.10*	*0.50*	*0.30*	6.83
07-08-2005	11.00	5.50	9.00	38.50	23.00	29.00	22.90	20.00	
	0.59	*0.41*	*3.23*	*3.35*	*5.11*	*1.19*	*2.36*	*1.18*	17.42
08-08-2005	13.00	8.00	1.70	6.30	8.80	32.70	6.20	10.00	
	0.70	*0.59*	*0.61*	*0.55*	*1.95*	*1.34*	*0.64*	*0.59*	6.98
27-08-2005	2.80	27.40	6.90	3.50	7.10	13.40	3.50	11.00	
	0.15	*2.03*	*2.48*	*0.30*	*1.58*	*0.55*	*0.36*	*0.65*	8.10
28-08-2005	12.00	39.50	13.60	42.10	20.40	45.50	3.10	13.00	
	0.65	*2.92*	*4.88*	*3.66*	*4.53*	*1.87*	*0.32*	*0.77*	19.60
05-07-2006	26.90	16.00	3.10	7.00	14.00	7.20	5.50	15.00	
	1.45	*1.18*	*1.11*	*0.61*	*3.11*	*0.30*	*0.57*	*0.89*	9.21
11-08-2006	8.50	10.50	14.00	36.00	11.30	9.80	9.90	10.00	
	0.46	*0.78*	*5.03*	*3.13*	*2.51*	*0.40*	*1.02*	*0.59*	13.91
12-08-2006	25.50	9.50	13.40	28.20	26.00	55.50	10.70	6.00	
	1.38	*0.70*	*4.81*	*2.45*	*5.77*	*2.28*	*1.10*	*0.35*	18.85

Values in italics are weighted rainfall in mm for individual stations.

4.1.2 Estimation of Missing Rainfall

Many hydrological records have breaks that make them incomplete which ranges from a day to several months or years. Some of the reasons of discontinuity in record are damage and fault in a rain gauge during a period. It is often necessary to estimate the missing data in order to utilize partial records, especially in data-sparse areas. Among the several methods that can be used to estimate missing data (Clarke, 1981), the distance power method is used here owing to the fact that it has been advocated to be the most accurate of all (Dean and Snyder, 1977). In this method, the rainfall at a station is estimated as a weighted average of observed rainfall at the neighbouring stations and is given by

$$R = \frac{\sum_{i=1}^{n} R_i / D_i^2}{\sum_{i=1}^{n} 1 / D_i^2} \tag{4.2}$$

where R – estimated rainfall at a station; R_i – rainfall at surrounding stations; D_i – distance of estimator station from the estimated station.

Therefore, the missing rainfall data for the Gebele Kelay station in the months of August and September 2005 is filled using equation (4.2). A sample calculation is given in Table 4.4 where as the complete table of the filled missing data is given in Appendix A.

Table 4.4 Sample calculation for filling the missing data on 1st August 2005

Station	Distance from Gebele Kelay, D_i (km^2)	Rainfall R_i (mm)	D_i^2	$1/D_i^2$	Weighted Rainfall R_i/D_i^2
Abardae	13.751	0	189.09	0.005288	0.000000
A.Kefelet	11.835	0	140.0672	0.007139	0.000000
A.Tsnay	6.4	3.6	40.96	0.024414	0.087891
Amadir	8.189	0	67.05972	0.014912	0.000000
S. Daero	10.034	0	100.6812	0.009932	0.000000
Shketi	5.036	1	25.3613	0.03943	0.039430
Tala	9.46	0	89.4916	0.011174	0.000000
	Total			0.112291	0.127321

Rainfall at station Gebele Kelay on 01/08/2005 is estimated to be:

$$\frac{0.127321}{0.112291} = 1.1\,\text{mm}$$

4.1.3 Excess Rainfall

Rainfall intensity is the ratio of rainfall depth with time. When a plot of intensity of rainfall with time is presented in the form of a bar graph then such a graph is known as hyetograph. Hyetograph is very useful for flood studies and calculation of rainfall loss indices. This is the most important part of the data analysis and is discussed here.

Beginning and end of the rainfall are given in the historical rainfall data. A one hour time interval is arbitrarily chosen for hyetograph analysis in this work. If the duration of the rainfall is more than one hour then the rainfall depth is assumed to have a uniform distribution and the amount is divided equally in to the number of hours. The amount of rainfall in a given hour is multiplied by the Thiessen weightage of a specified station. The sum of all the rainfall depths from all the stations in the same hour divided by the time step (one hour in this case) gives rainfall intensity. The same procedure is followed for the entire duration to obtain the average rainfall intensity of a particular storm event. For selected storm events, the computation of average rainfall intensity is presented in Table 4.5 for both catchments.

Another part of the rainfall analysis is the determination of the total abstractions due to infiltration, evaporation, interception, etc. Here the ϕ-index method is adopted. The direct surface runoff depth is computed from the observed data. The line that gives the same depth of rainfall in the rainfall hyetograph is drawn by trial and error. However, the amount of rainfall in excess of the index which is rainfall excess is determined for each storm event.

4.2 Stream Flow

The objective of a flow measurement station is to obtain knowledge of the discharge of a natural or artificial open channel. Regular recording of discharges at short intervals, preferably daily, over a period of time is essential for the correct assessment of natural water resources of river basins and subsequent planning and utilization. Nevertheless, daily discharge observations over long periods are sometimes not feasible, impractical during floods or are too expensive and labourious, while the stages can be easily recorded several times a day during floods. Therefore, stage-discharge (H-Q) relationship or rating curve at a river cross section is a fundamental technique in hydrology employed for computing the discharge using the data of water level records. Rating curve is a very important tool in surface water hydrology because the reliability of discharge data values is highly dependent on a satisfactory H-Q relationship at the gauging station.

Table 4.5(a) Excess rainfall computation for storms selected for analysis at Debarwa catchment

Date	Time	Station and its weightage								Average depth	Intensity	φ-index	Exc rain
		Abardae	AdiKefelet	Amadir	AdiTsnay	GebeleKelay	SelaeDaero	Shketi	Tala				
	(hr)	**0.146**	**0.203**	**0.007**	**0.219**	**0.228**	**0.068**	**0.018**	**0.111**	(mm)	(mm/hr)	(mm/hr)	(mm
	13	0	0	0	0	0	0	0	0	0	0	3.706	-
	14	0.9636	1.1165	0	1.5768	0	0	0	0.333	3.9899	3.9899	3.706	0.28
	15	1.2702	1.1165	0.035	0	0	0	0	0	2.4217	2.4217	3.706	-
07-2006	16	0	0	0	0	0	0	0	0	0	0	3.706	-
	17	0	0	0	0	5.244	0	0	0	5.244	5.244	3.706	1.5
	18	0	0	0	0	0	0	0	0	0	0	3.706	-
	19	0	0	0	0	0	0	0	0	0	0	3.706	-
	13	0	0	0	0	0	0	0	0	0	0	2.606	-
	14	0	0	0	0	0	0	0	0.222	0.222	0.222	2.606	-
	15	1.3578	0.4263	0	2.0586	0	0	0	0	3.8427	3.8427	2.606	1.23
08-2006	16	0	0.8526	0	0	0	0	0	0	0.8526	0.8526	2.606	-
	17	0	0.8526	0	0	2.394	0	0	0	3.2466	3.2466	2.606	0.64
	18	0	0	0	0	2.85	0	0	0	2.85	2.85	2.606	0.2
	19	0	0	0	0	0	0	0	0	0	0	2.606	
	13	0	0	0	0	0	0	0	0	0	0	3.362	-
	14	1.46	0	0.2359	0	2.736	0	0		4.4319	4.4319	3.362	1.06
	15	0	0	0	1.0074	2.736	0	0	0.555	4.2984	4.2984	3.362	0.93
08-2006	16	0	0	0	0	2.508	0	0	0	2.508	2.508	3.362	-
	17	0	0	0	0	0	0	0	0	0	0	3.362	-
	18	0.438	0	0	0	0	0	0	0	0.438	0.438	3.362	-
	19		0.609	0	0	0	0	0	0	0.609	0.609	3.362	-
	20		0	0	0	0	0	0	0	0	0	3.362	-
	11	0	0	0	0	0	0	0	0	0	0	6.607	-
	12	0	0	0	1.0512	0	0	0	0	1.0512	1.0512	6.607	-
	13	0	0.1015	0	6.33348	0	0.952	0	1.11	8.49698	8.49698	6.607	1.8
	14	0.0438	0.203	0	3.1755	0.8208	0.748	0	1.11	6.1011	6.1011	6.607	-
08-2006	15	0.0974	0.203	0	0	0.8208	0	0	0	1.12118	1.12118	6.607	-
	16	0.0974	0.203	0.0385	0	0.8208	0	0	0	1.15968	1.15968	6.607	-
	17	0.0974	0.203	0.0777	0	0.2736	0	0	0	0.65168	0.65168	6.607	-
	18	0.0438	0.1015	0.0777	0	0	0	0	0	0.223	0.223	6.607	-
	19	0	0	0.0196	0	0	0	0	0	0.0196	0.0196	6.607	-
	20	0	0	0	0	0	0	0	0	0	0	6.607	-

Table 4.5(b) Excess rainfall computation for storms selected for analysis at Ghergera catchment

Date	Time	Station and its weightage								Average depth	Intensity	φ-index	Exc rai
		Abardae	AdiKefelet	Amadir	AdiTsnay	GebeleKelay	SelaeDaero	Shketi	Tala				
	(hr)	**0.054**	**0.074**	**0.359**	**0.087**	**0.222**	**0.041**	**0.103**	**0.059**	(mm)	(mm/hr)	(mm/hr)	(mı
	13	0	0	0	0	0	0	0	0	0	0	2.619	
	14	0	0	0	0	0	0	0	0.118	0.118	0.118	2.619	
	15	0.5022	0.1554	0	0.8178	0	0	0	0	1.4754	1.4754	2.619	
)8-2006	16	0	0.3108	0	0	0	0	0	0	0.3108	0.3108	2.619	
	17	0	0.3108	0	0	2.331	0	0	0	2.6418	2.6418	2.619	0.(
	18	0	0	0	0	2.775	0	0	0	2.775	2.775	2.619	0.
	19	0	0	0	0	0	0	0	0	0	0	2.619	
	13	0	0	0	0	0	0	0	0	0	0	13.3023	
	14	0.54	0	12.0983	0	2.664	0	0		15.3023	15.3023	13.3023	2
	15	0	0	0	0.4002	2.664	0	0	0.295	3.3592	3.3592	13.3023	
)8-2006	16	0	0	0	0	2.442	0	0	0	2.442	2.442	13.3023	
	17	0	0	0	0	0	0	0	0	0	0	13.3023	
	18	0.162	0	0	0	0	0	0	0	0.162	0.162	13.3023	
	19		0.222	0	0	0	0	0	0	0.222	0.222	13.3023	
	20		0	0	0	0	0	0	0	0	0	13.3023	
	13	0	0	0	0	0	0	0	0	0	0	8.0	
	14	0	0.2738	5.3491	0	0.5328	0	0	2.36	8.5157	8.5157	8.0	0.5
	15	0	0	6.4261	0	1.6428	0	0	2.419	10.4879	10.4879	8.0	2.4
)8-2006	16	0.054	0	2.6566	0.6264	1.6428	0	0	2.36	7.3398	7.3398	8.0	
	17	0.162	0	0	0	1.6428	0	0	0	1.8048	1.8048	8.0	
	18	0	0	0	0	0.5328	0	0	0	0.5328	0.5328	8.0	
	19	0	0	0	0	0	0	0	0	0	0	8.0	

4.2.1 Stage–Discharge Relationship

When the measured discharge values are plotted against the corresponding stages, it gives a relationship that represents the integrated effect of a wide range of channel and flow parameters. The H-Q relationship at a particular river cross section, even under conditions of meticulous observation, is not necessarily unique as rivers are often influenced by factors not always understood nor easy to quantify. This natural variability in the H-Q relationship is often aggravated by considerable errors of human origin. The procedure for constructing a rating curve is clearly described in Indian Standard (IS: 2914, 1964). This code categorizes open channels in to erodible and inerodible that require different methods of rating curve development. The stream gauging stations in the study area are assumed to fall in the former category and simultaneously possess permanent controls though it needs to be checked for the presence of shifting controls in the future. The relationship given by equation (4.3) can be graphically expressed by plotting observed stage against the corresponding values of discharge in arithmetic or logarithmic plot. For open channels that exhibit a permanent control, the H-Q relationship is expressed in the form of

$$Q = a(H - H_o)^b \tag{4.3}$$

where Q - stream discharge (m^3/s); H - gauge height (m); H_o - a constant which represents the gauge reading corresponding to zero discharge; a and b - rating curve constants.

In case of no shifting controls, the rating curve can be developed using data analysis, physical analysis, double log plot and least square methods. Similarly, there are various methods for determining the value of H_o, for instance, running method (Wisler and Brater, 1959).

In this study, a computer based optimization procedure to estimate value of H_o that gives the best fitting parameter or the coefficient of determination (R^2) is used. Fig. 4.3, Fig. 4.4 and Fig. 4.5 represent rating curves modeled with the help of MATLAB version 7.6 (R2008a) using the discharge values given in Table 4.6 and Table 4.7. Basically, it has a general model of equation (4.3) and the coefficients are determined with a confidence level of 95%.

Measured discharge data are available for the upstream gauging station for the years 2006, 2007 and 2008. It is attempted to put all the data together and develop a rating curve but ended up with a lower R^2. On the contrary, rating curve developed for the data 2007 and 2008 was giving much better result and hence the same curve is adopted for the conversion of stage records to discharge for the upstream station. Simultaneously, different rating curves are developed for the measured discharge data at the downstream gauging station for 2006 and 2007 owing to their incompatibility. The H-Q equations of both stations are used to convert their respective stage records to corresponding discharges thereupon making the data readily available for further hydrograph analysis as has been discussed in the subsequent sections.

Table 4.6 Measured discharges at Debarwa station (2007 and 2008)

S. No	Date	Stage (m)	Average velocity (m/s)	Wetted area (m^2)	Discharge (m^3/s)
1	06/08/2007	0.32	1.01	3.02	3.05
2	09/08/2007	0.40	0.90	4.75	4.27
3	22/08/2007	0.59	1.37	5.98	8.18
4	01/09/2007	0.61	1.24	6.76	8.39
5	02/09/2007	0.62	1.20	6.71	8.05
6	06/09/2007	0.19	0.55	1.50	0.82
7	13/08/2007	0.68	1.37	8.11	11.12
8	27/07/2008	0.31	0.70	2.58	1.81
9	15/08/2008	0.39	0.96	4.53	4.35
10	18/08/2008	0.17	0.56	1.45	0.81
11	20/08/2008	0.26	0.66	1.97	1.30
12	20/08/2008	0.29	0.65	2.37	1.56

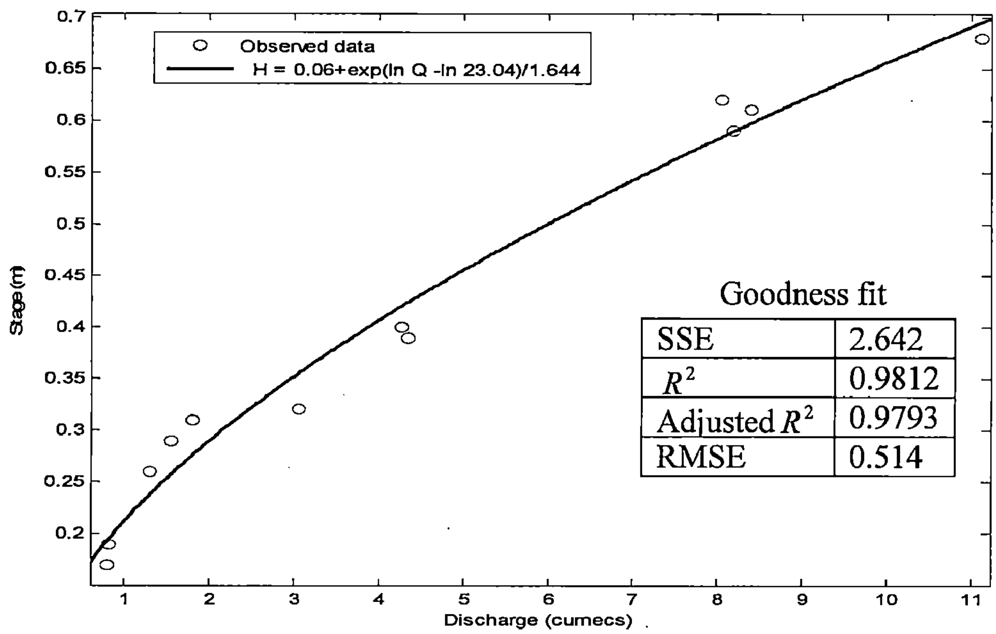


Fig. 4.3 H-Q relationship for the measured discharge in 2007 and 2008 at Debarwa

Table 4.7(a) Measured discharges at Ghergera in 2006

S. No	Date	Stage (m)	Average velocity (m/s)	Wetted area (m^2)	Discharge (m^3/s)
1	19/07/2006	0.24	1.01	4.58	4.63
2	13/08/2006	0.08	0.16	0.99	0.16
3	14/08/2006	0.22	1.17	3.78	4.42
4	18/08/2006	0.36	1.44	5.86	8.44
5	02/09/2006	0.43	1.71	7.06	12.09
6	02/09/2006	0.43	1.62	5.72	9.23
7	07/09/2006	0.23	0.43	3.56	1.53
8	08/09/2006	0.23	0.42	3.44	1.44

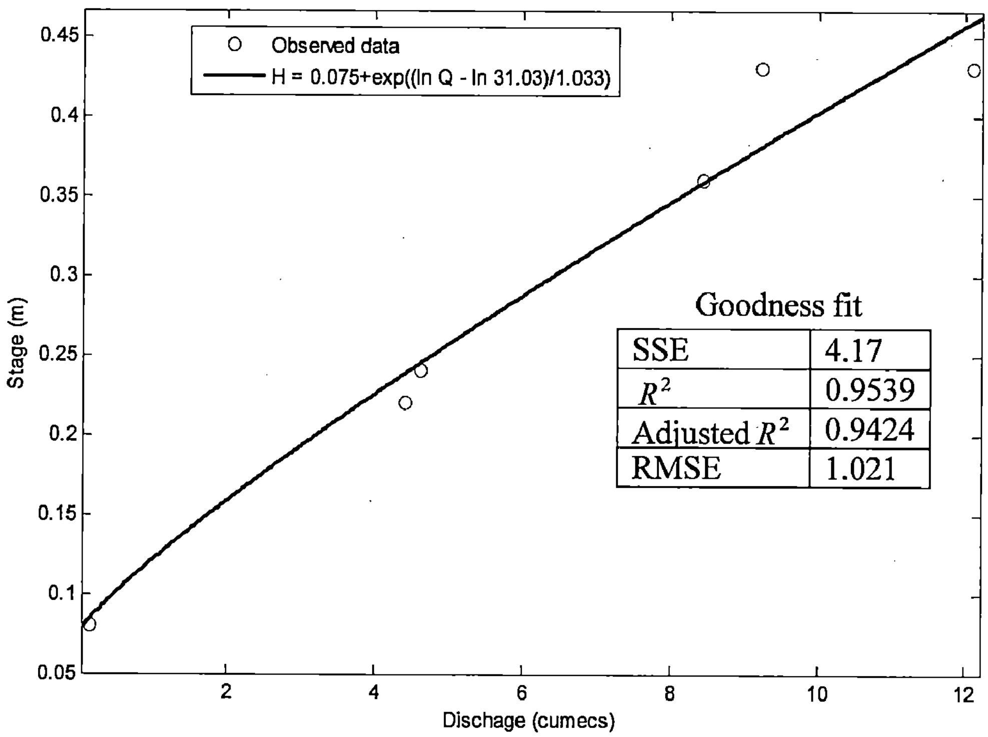


Fig. 4.4 H-Q relationship for the measured discharge in 2006 at Ghergera

Table 4.7(b) Measured discharges at Ghergera in 2007

S. No	Date	Stage (m)	Average velocity (m/s)	Wetted area (m^2)	Discharge (m^3/s)
1	24/07/2007	0.42	0.43	6.730	2.95
2	25/07/2007	0.36	0.49	4.680	2.36
3	26/07/2007	0.23	0.39	3.673	1.44
4	31/07/2007	0.18	0.21	2.083	0.45
5	03/08/2007	0.38	0.44	4.945	2.40
6	03/08/2007	0.31	0.41	3.967	1.74
7	07/08/2007	0.16	0.24	1.950	0.52
8	08/08/2007	0.40	0.46	4.130	2.03
9	08/08/2007	0.36	0.27	4.900	1.45
10	09/08/2007	0.24	0.30	2.403	0.78
11	10/08/2007	0.15	0.17	1.425	0.26
12	11/08/2007	0.19	0.36	1.555	0.56
13	12/08/2007	0.53	0.75	4.593	4.46
14	19/08/2007	0.25	0.42	2.355	1.11
15	22/08/2007	0.29	0.40	4.165	1.80
16	22/08/2007	0.20	0.41	1.765	0.80
17	31/08/2007	0.26	0.57	2.626	1.58
18	02/09/2007	0.21	0.34	2.100	0.82

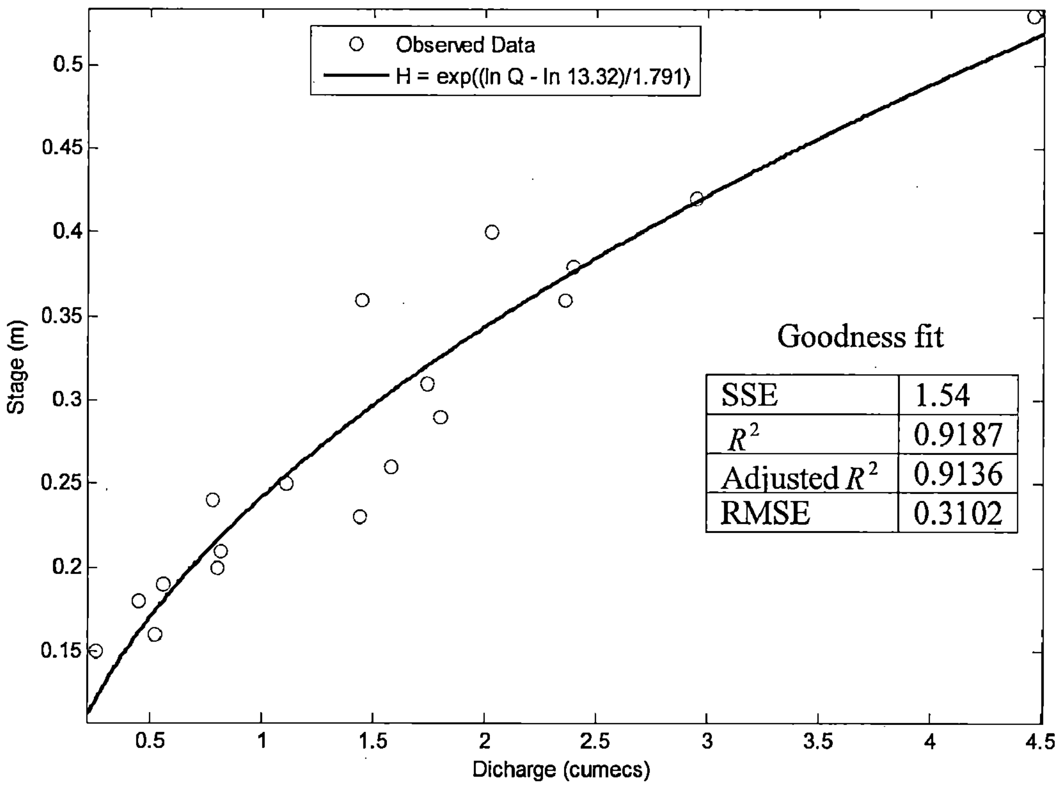


Fig. 4.5 H-Q relationship for the measured discharge in 2007 at Ghergera

CHAPTER-5

UNIT HYDROGRPAH APPLICATIONS

5.1 Introduction

The prediction of stream flow from a catchment has been of concern to hydrologists and water resources professionals since long time back. In this regard, the unit-hydrograph (UH) approach proposed by Sherman (1932) has evolved into one of the most powerful tools in applied hydrology (Dooge, 1959). Chow et al (1988), defined a UH as the direct runoff hydrograph (DRH) resulting from 1 cm depth of effective rainfall hyetograph (ERH) falling uniformly over the drainage area at a constant rate for an effective duration. He further described it as a conceptual linear model that can be used to derive the hydrograph resulting from any amount of excess rainfall. The UH at a specific point on the stream gauging site in a catchment is generally determined by using effective rainfall and surface runoff data by Collins method (Collins, 1939), inversion matrix and least square methods. The assumptions considered in the UH development is that the storms selected for analysis should be of short duration, uniformly distributed over the entire catchment both in space and time and the principles of superposition and homogeneity holds good as it is a linear theory.

Once a UH has been derived for a catchment area, it can be used for the following purposes.

i) Design storm hydrographs for selected recurrence interval storms can be developed through convolution adding and lagging procedures.
ii) Effects of land use/land cover changes, channel modifications, storage additions, and other variables can be evaluated to determine changes in the unit hydrograph.
iii) Effects of the spatial variation in precipitation can be evaluated.
iv) Hydrographs of watersheds consisting of several sub-basins can be produced.

5.2 Derivation of Unit Hydrograph

Once rating curves for the individual discharge measuring stations were developed as discussed in the preceding chapter, the discrete stage records has been converted to discharge. After carefully reviewing the nature of the stream flows of a particular storm, isolated storms occurred at different periods that had produced a single peaked hydrograph at each outlet and their corresponding storm rainfalls have been selected for analysis.

Various methods are available to develop a UH such as using stream flow data, synthetically (SCS (1957), Snyder (1938), time area method (Clark, 1945)), using geomorphologic and fitted distributions. In this section, the UH for the upstream catchment is derived using the available stream flow data. This information in turn has been used for determining it's regional constants by the synthetic Modified Snyder's equation. Assuming the downstream catchment share similar physiographic characteristics to that of the upstream catchment, the constants are used in developing the synthetic unit hydrograph (SUH) for the downstream catchments. UH derivation using geomorphologic parameters will be discussed in the next chapter.

5.2.1 Analysis of Storm Hydrograph

Out of many storm hydrographs available at the outlet, few single peaked hydrographs have been selected for the derivation of UH. Complex storms analyses are beyond the scope of this study. To separate the base flow from direct runoff, a straight line method is used. This is achieved by drawing a straight line which connects the beginning of the surface runoff to a point on the recession limb representing the end of the direct runoff, keeping a suitable time distribution of the base flow in mind. It is preferred to other methods of base flow separation owing to the absence of any other reliable data/information about the base flow distribution. The method assumes the hydrograph recession to become exponential (Shaw, 1994) and is fixed uniquely on the semi-logarithmic plot of discharge (Q) and time (t).

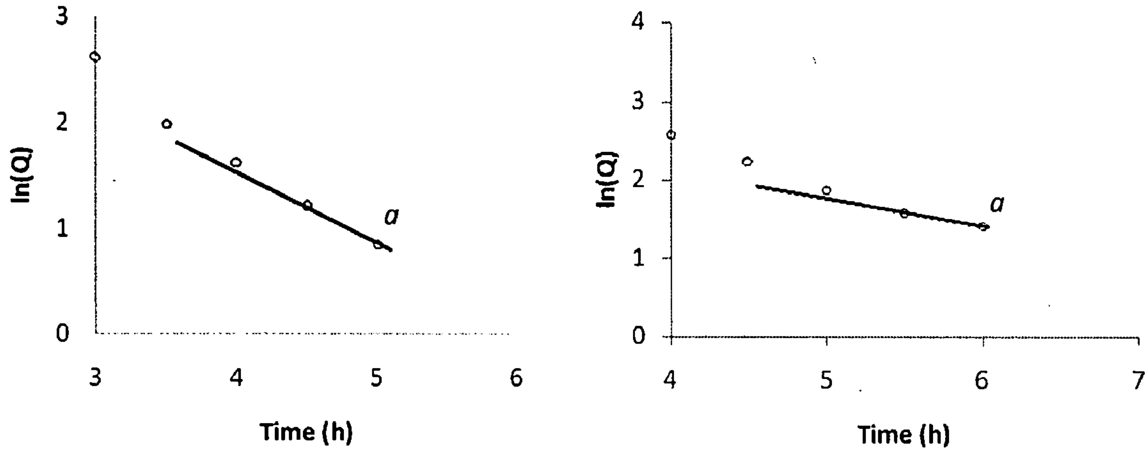
For instance, for the storm hydrograph observed at Debarwa station on August 2, 2006, the recession points beyond five hours are not available. Hence, the selection of the exact end point is not easy. Taking this constraint in to account a personal judgement is applied for selecting the same and thus the recession hydrograph ordinate at point a (Fig. 5.1) is assumed to be the end point. Therefore, it is better to assume a linear relationship for connecting both points (Fig. 5.2) which is prepared using the data given in Table 5.1 and Table 5.2.

Area under the direct runoff hydrograph (DRH) is computed to yield the runoff volume (V) and is given by

$$V = \sum (DRH) \times \Delta t \times 3{,}600 \tag{5.1}$$

where DRH - direct runoff hydrograph ordinate (m^3/s)

Δt - time interval (h).



a) August 2, 2006 b) August 16, 2006

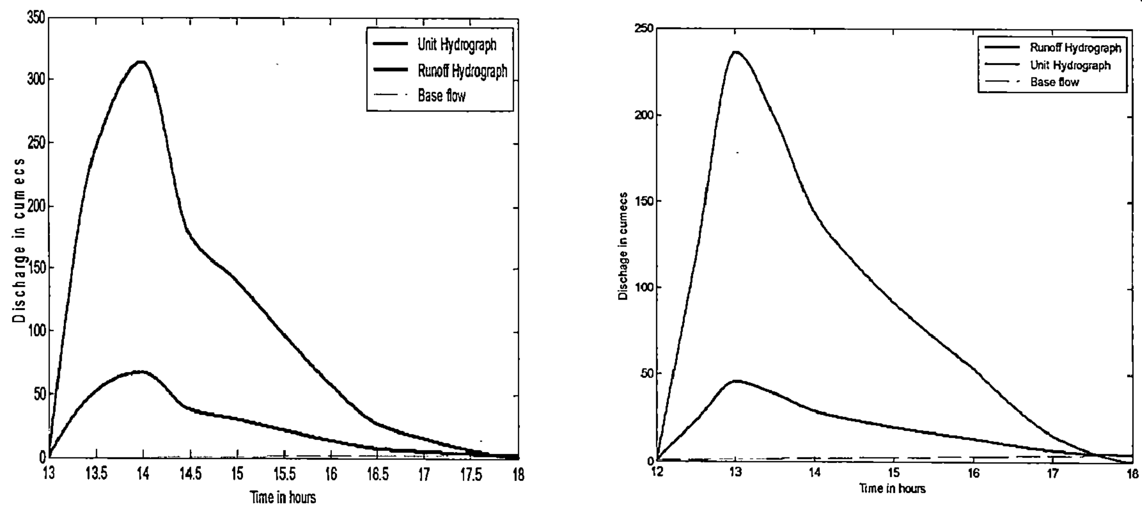
Fig. 5.1 Semi-logarithmic plot of discharge vs time for end point selection

The direct runoff volume V for the runoff observed at Debarwa station on August 2, 2006 is

$$V = 229.61 \times 0.5 \times 3600 = 413{,}306.58\,\text{m}^3$$

The runoff depth (d) is computed by dividing the volume of runoff by the catchment area (A) in km^2.

$$d = \frac{V}{A \times 10^4} \quad \text{(cm)}; \; d = \frac{413{,}306.58}{194.646 \times 10^4} = 0.212\,\text{cm}$$



a) August 2, 2006 b) August 16, 2006

Fig. 5.2 Base flow separation by straight line method

Similarly, the direct runoff volume V for the runoff observed at Debarwa station on August 16, 2006 and its corresponding depth (d) are as follows,

$$V = 204.23 \times 0.5 \times 3{,}600 = 367{,}614 \text{ m}^3$$

and

$$d = \frac{367{,}614}{194.646 * 10^4} = 0.189 \text{ cm}$$

Table 5.1 Derivation of unit hydrograph for the Debarwa station (August 2, 2006)

Time (h)	Discharge (m^3/s)	Base Flow (m^3/s)	Ordinates of DRH (m^3/s)	Ordinates of UH (m^3/s)
13.0	0.00	0.00	0.00	0.00
13.5	52.55	0.24	52.31	246.37
14.0	67.33	0.47	66.86	314.88
14.5	38.20	0.71	37.49	176.55
15.0	30.67	0.94	29.72	139.98
15.5	21.91	1.18	20.73	97.65
16.0	13.73	1.42	12.32	58.01
16.5	7.37	1.65	5.72	26.94
17.0	5.11	1.89	3.22	15.17
17.5	3.36	2.12	1.24	5.82
18.0	2.36	2.36	0.00	0.00
		Total	229.61	1081.37

Therefore, the UH ordinates as shown in column 5 of Table 5.1 are obtained by dividing the DRH ordinates by the runoff depth. The unit volume (v) of the unit hydrograph is the sum of the UH ordinates (m^3/s) multiplied by the time interval (h).

$$v = \sum UH \times \Delta t \times 3{,}600 \text{ (m}^3\text{)} \qquad (5.2)$$

The unit volume for this event is,

$$v = 1{,}081.37 \times 0.5 \times 3600 = 1{,}946{,}466 \text{ m}^3$$

This unit volume should be equal to 1cm rainfall depth multiplied by the catchment area A (km^2).

$$v = 1 \times A \times 10^4 = 1{,}946{,}460 \, \text{m}^3$$

Comparing both results, it is quite reasonable to say the calculations are correct.

Table 5.2 Derivation of UH for the Debarwa station (August 16, 2006)

Time (hr)	Discharge (m^3/s)	Base flow (m^3/s)	Ordinates of DRH (m^3/s)	Ordinates of UH (m^3/s)
12.00	0.71	0.71	0.00	0.00
12.50	23.29	0.9925	22.29	118.04
13.00	45.86	1.275	44.59	236.07
13.50	38.92	1.5575	37.36	197.83
14.00	28.99	1.84	27.15	143.76
14.50	23.83	2.1225	21.71	114.94
15.00	19.73	2.405	17.33	91.73
15.50	16.29	2.6875	13.60	72.03
16.00	13.12	2.97	10.15	53.74
16.50	9.38	3.2525	6.12	32.42
17.00	6.43	3.535	2.90	15.33
17.50	4.85	3.8175	1.03	5.48
18.00	4.10	4.1	0.00	0.00
		Total	204.23	1081.37

Similarly, the unit volume (v) for August 16, 2006 storm,

$$v = 1{,}081.37 \times 0.5 \times 3{,}600 = 1{,}946{,}466 \, \text{m}^3$$

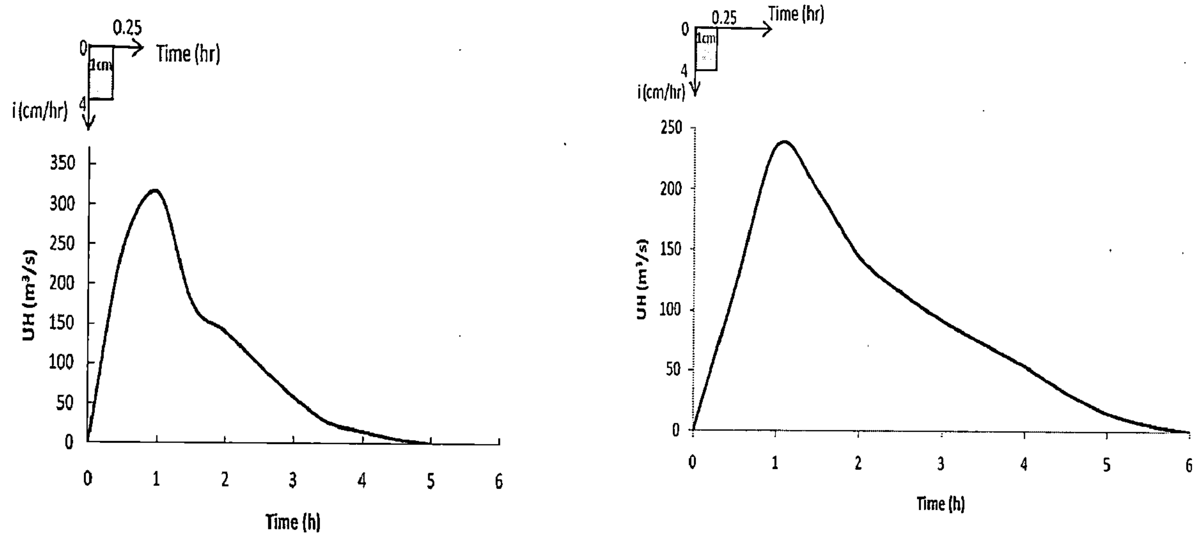
1cm rainfall depth multiplied by A (km^2) gives,

$$v = 1 \times A \times 10^4 = 1{,}946{,}460 \, \text{m}^3$$

Comparing both results, it is quite reasonable to say the calculations are correct.

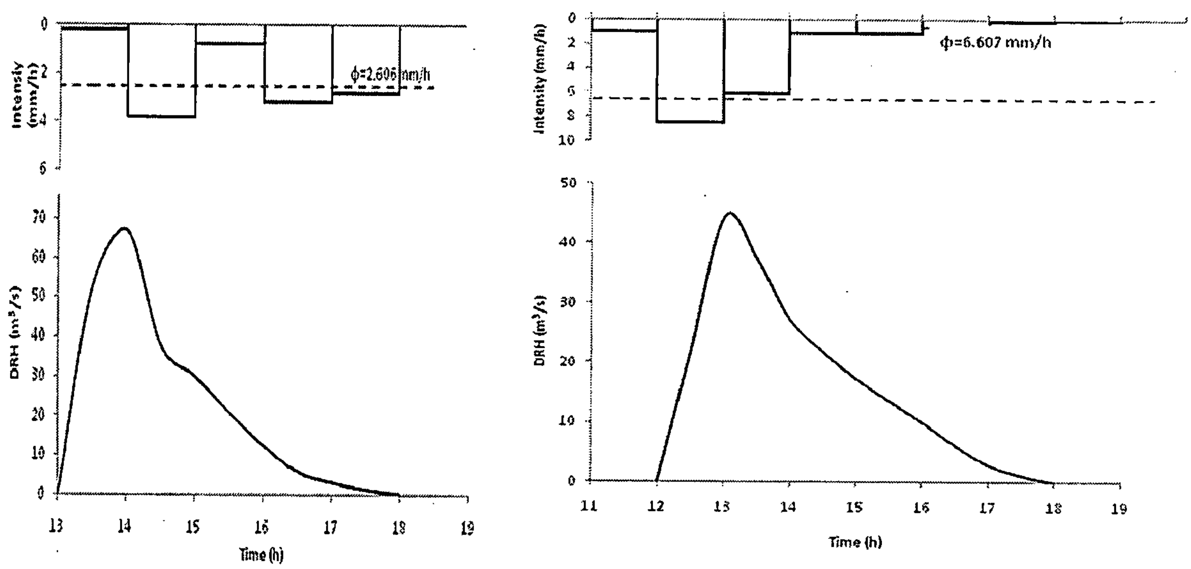
5.2.2 Analysis of Storm Rainfall

The objective of this analysis is to compute the excess rainfall, i.e. component of total rainfall that contributes to direct runoff. From the point rainfall data of different rain gauges in the catchment, the average hyetograph of the storm is constructed. Furthermore, since infiltration capacity curve is unavailable for the site, a suitable value of ϕ-index is obtained so that total rain above the ϕ-index is the same as the runoff depth. Corresponding to unit depth of the UH (1 cm), the ordinates of the DRH are linearly interpolated to obtain the UH ordinates as plotted in Fig. 5.3. The ERH and DRH graphs (Fig. 5.4) are prepared using the values given in Table 4.5 of chapter 4. For the other selected events for analysis, the graphs are presented in Appendix B.



a) August 2, 2006 b) August 16,2006

Fig. 5.3 UH derived from flow observed at Debarwa



a) August 2, 2006 b) August 16, 2006

Fig. 5.4 DRH and excess rainfall for the storms at Debarwa

The unit duration of the rainfall excess is the time during which the unit depth of rainfall excess uniformly falling over the entire catchment. The thumb rule that unit duration of a UH need to be 1/3 to 1/5 of time to peak (t_p) of the UH is applied here. From Fig. 5.3(a), t_p is 1, then,

$$1/3\,t_p = 1/3 \text{ h or 20 minutes; similarly, } 1/5\,t_p = 1/5 \text{ h or 12 minutes}$$

Therefore, the average unit duration is 16 minutes and for convenience 15 minutes (0.25 h) unit duration is considered. Accordingly, UH ordinates at this time interval are given in Table 5.3. As it is shown in Fig. 5.3, the 1 cm depth of rainfall lasted for 0.25 h (15 minutes) which was uniformly distributed over the entire drainage area. The intensity of rainfall (i_e) is equal to the unit depth divided by unit duration is

$$i_e = 1/0.25 = 4 \text{ cm/h}$$

The time of concentration (T_C) is computed from

$$T_C = T_B - t_D \qquad (5.3)$$

where T_B - the time base of the unit hydrograph; and t_D - the excess rainfall duration. Then,

$$T_C = 5.0 - 0.25 = 4.75 \text{ h}$$

Table 5.3 UH ordinates at interval of unit duration at Debarwa station (August 2, 2006)

Time (h)	0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25	2.50
UH (m^3/s)	0.0	151.7	246.4	294	314.9	252.9	176.6	155.9	140	119.0	97.7
Time (h)	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	**4.75**	5.00	
UH (m^3/s)	77.07	58.01	40.25	26.94	20.22	15.17	10.09	5.82	**2.52**	0.00	

Similarly for the August 16, 2006 event, from Fig. 5.3(b) the time to peak, t_p is 1 h,

$$1/3\,t_p = 1/3 \text{ h or 20 minutes; similarly, } 1/5\,t_p = 1/5 \text{ h or 12 minutes}$$

The average unit duration is 16 minutes and for convenience 15 minutes (0.25 h) unit duration is considered. Accordingly, UH ordinates at this time interval are given in Table 5.4. It's corresponding intensity of rainfall (i_e) is equal to the unit depth divided by unit duration, i.e.

$$i_e = 1/0.25 = 4 \text{ cm/hr.}$$

The time of concentration (T_C) is

$$T_C = 6.0 - 0.25 = 5.75 \text{ h}$$

Table 5.4 UH ordinates at interval of unit duration at Dbarwa station (August 16, 2006)

Time (hr)	0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00
UH ordinates (m^3/s)	0.00	59.02	118.04	191.81	236.07	222.55	197.83	169.90	143.76
Time (h)	2.25	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25
UH ordinates (m^3/s)	127.86	114.94	102.78	91.73	81.59	72.03	62.98	53.74	42.99
Time (h)	4.50	4.75	5.00	5.25	5.50	**5.75**	6.00		
UH ordinates (m^3/s)	32.42	23.07	15.33	9.72	5.48	**2.27**	0.00		

5.2.3 S-Curve

The UHs discussed above are derived from selected simple storm hydrographs. At the same time, other UHs derived in a similar fashion for the same catchment has been analysed. It has been observed that their unit durations and other UH parameters are found to be different. Therefore, to develop an average (master) UH for the catchment they need to be brought to the same unit duration. Various methods are available for this purpose such as the method of superposition and S-Curve. Due to the nature of the catchment and short duration rainfalls, the unit durations are in fractions which rules out the use of the former method and therefore, the second method is applied here.

The S-curve is a hydrograph produced by a continuous effective rainfall at a constant rate for an infinite period. It is the sequential accumulation of the ordinates of the unit hydrograph; a graph of cumulative UH ordinates plotted against time. The S-curve shown in

Fig. 5.5 is plotted using the time versus S-Curve ordinates tabulated in Table 5.5 and Table 5.6. Basically, at the time of concentration (T_C = 4.75 h of August 2, 2006), the S-Curve ordinate ($Q_{S-Curve}$) should be equal to the intensity of the excess rainfall times the catchment area, i.e.

$$Q_{S-Curve} = 2.778 \times i_e \times A \, (m^3/s) \tag{5.4}$$

where i_e – intensity of excess rainfall (cm/hr); A – catchment area (km^2).

Table 5.5 Construction of S-Curve for a 0.25 h UH (August 2, 2006)

Time (h)	UH Ordinates (m^3/s)	S-Curve Ordinates (m^3/s)	Smoothened S-Curve (m^3/s)
0.00	0.00	0.00	0.00
0.25	151.70	151.70	151.70
0.50	246.37	398.07	398.07
0.75	294.03	692.09	692.09
1.00	314.88	1006.97	1006.97
1.25	252.95	1259.92	1259.92
1.50	176.55	1436.47	1436.47
1.75	155.94	1592.41	1592.41
2.00	139.98	1732.39	1732.39
2.25	119.03	1851.42	1851.42
2.50	97.65	1949.07	1949.07
2.75	77.07	2026.13	2026.13
3.00	58.01	2084.14	2084.14
3.25	40.25	2124.40	2124.40
3.50	26.94	2151.34	2151.34
3.75	20.22	2171.56	2155.90
4.00	15.17	2186.73	2159.19
4.25	10.09	2196.82	2161.35
4.50	5.82	2202.64	2162.55
4.75	2.52	2205.16	2162.91
5.00	0.00	2205.16	2162.91

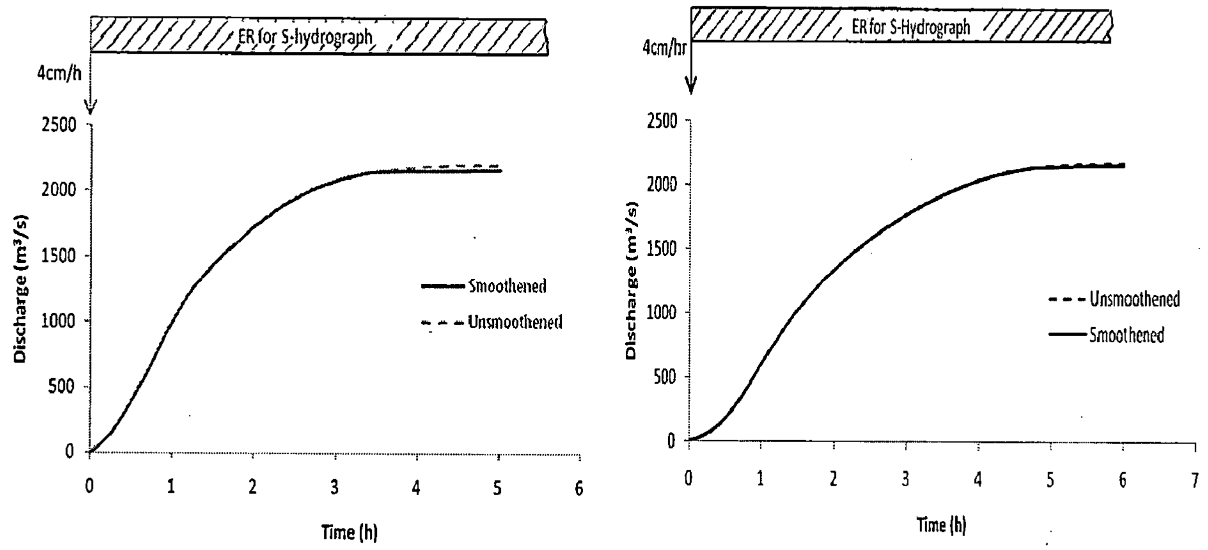
Therefore,

$$Q_{S\text{-}Curve} = 2.778 \times 4 \times 194.646 = 2{,}162.73 \text{ m}^3/\text{s}$$

but from Table 5.5, at $T_C = 4.75$ h, the S-curve ordinate is:

$$Q_{S\text{-}Curve} = 2{,}205.16 \text{ m}^3/\text{s}$$

In actual construction of an S-Curve, it is found that the curve oscillates in the top portion at around the equilibrium value due to magnification and accumulation of small errors in the UH interpolation. When it occurs, an average smooth curve is drawn such that it reaches a value $Q_{S\text{-}Curve}$ at the time base of the unit hydrograph (Subramanya, 2008). Because of this fact, the above values are different and hence, smoothening of the curve was required as shown in Fig. 5.5.



a) August 2, 2006 b) August 16, 2006

Fig. 5.5 S-Curve for two storm events at Debarwa

Similarly, at the time of concentration (T_C = 5.75 h) of August 16, 2006, the S-Curve ordinate ($Q_{S\text{-}Curve}$) should be equal to the intensity of the excess rainfall times the catchment area, i.e.

$$Q_{S\text{-}Curve} = 2.778 \times 4 \times 194.646 = 2{,}162.73 \text{ m}^3/\text{s}$$

but from Table 5.6, at T_c = 5.75hr, the S-curve ordinate is:

$$Q_{S\text{-}Curve} = 2{,}177.91 \text{ m}^3/\text{s}$$

The above values are different and hence, smoothening of the curve is required.

Table 5.6 Construction of S-Curve for a 0.25 h UH (August 16, 2006)

Time (h)	UH Ordinates (m^3/s)	S-Curve Ordinates (m^3/s)	Smoothened S-Curve (m^3/s)
0.00	0.00	0.00	0.00
0.25	59.02	59.02	59.02
0.50	118.04	177.06	177.06
0.75	191.81	368.87	368.87
1.00	236.07	604.94	604.94
1.25	222.55	827.49	827.49
1.50	197.83	1025.32	1025.32
1.75	169.90	1195.22	1195.22
2.00	143.76	1338.98	1338.98
2.25	127.86	1466.84	1466.84
2.50	114.94	1581.78	1581.78
2.75	102.78	1684.56	1684.56
3.00	91.73	1776.29	1776.29
3.25	81.59	1857.88	1857.88
3.50	72.03	1929.91	1929.91
3.75	62.98	1992.89	1992.89
4.00	53.74	2046.63	2046.63
4.25	42.99	2089.62	2089.62
4.50	32.42	2122.04	2122.04
4.75	23.07	2145.10	2145.10
5.00	15.33	2160.43	2152.21
5.25	9.72	2170.16	2158.29
5.50	5.48	2175.64	2161.74
5.75	2.27	2177.91	2162.91
6.00	0.00	2177.91	2162.91

5.3 Average Unit Hydrograph

As discussed in the preceding section, various UHs of a given duration are derived from different storm events. Due to the rainfall variations both in space and time and due to storm departures from the assumptions of the UH theory, the various UHs thus developed are not identical. It is a common practice to adopt a mean of such curves as the UH of a given duration for the catchment. Here, the UHs derived from the storms occurred on 2nd and 16th August, 2006 are used for the development of the average UH (Fig. 5.6). The average of peak flows and time to peaks are first calculated. Then a mean curve of best fit judged by eye is drawn through the averaged peak to close on an averaged base length. The volume of DRH due to the excess rainfall is calculated and any departure from unity is corrected by adjusting the value of the peak. The averaged excess rainfall of unit depth is customarily drawn in the plot of the UH to indicate the type and duration of rainfall causing the UH.

Table 5.7 UH parameters at Debarwa derived from two storm hydrographs

S.No	Duration	Peak Discharge	Time Base	Time Lag	Runoff Depth	Remark
	t_D (h)	Q_p (m^3/s)	T_B(h)	t_L (h)	d (cm)	
1	0.25	314.88	5.00	0.875	0.212	Aug 2, 2006
2	0.25	236.07	6.00	0.875	0.189	Aug 16, 2006
Average	0.25	275.475	5.5	0.875	0.201	

From Table 5.7, Average UH duration, $\bar{t}_D = 0.25$ h; Peak of average UH, $\overline{Q}_p = 275.475\text{m}^3/\text{s}$, Average base length, $\overline{T_B} = 5.5$ h, Average time lag, $\bar{t}_L = 0.875$ h; and Average runoff depth, $\bar{d} = 0.201$ cm

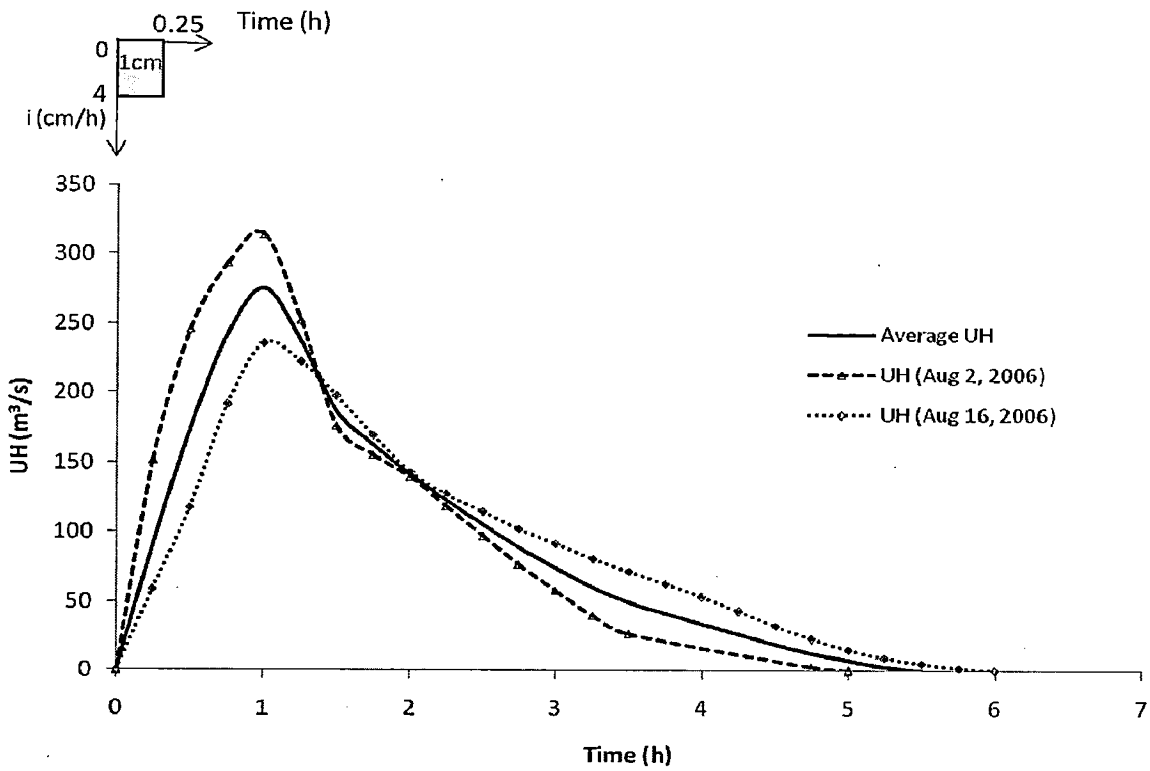


Fig. 5.6 Average UH for Debarwa river catchment

Table 5.8 Average 0.25 h UH for the Debarwa river catchment

Time (h)	Average UH ordinates (m^3/s)	Remarks
0.00	0.00	
0.25	90.49	
0.50	174.00	
0.75	242.92	
1.00	275.48	Peak discharge
1.25	237.75	
1.50	187.19	
1.75	162.92	
2.00	141.87	
2.25	123.45	
2.50	106.30	
2.75	89.93	
3.00	74.87	
3.25	60.92	
3.50	49.49	
3.75	41.60	
4.00	34.46	
4.25	26.54	
4.50	19.12	
4.75	12.79	
5.00	7.67	
5.25	3.00	
5.50	0.00	
total	2162.72	

The unit volume (v)

$$v = \sum (UH) \times \Delta t \times 3{,}600 \text{ m}^3$$

where t - time (h)

$$v = 2185.84 \times 0.25 \times 3{,}600 = 1{,}946{,}448 \text{ m}^3$$

This unit volume should be equal to 1 cm rainfall depth multiplied by the catchment area A (km^2).

$$v = 1 \times A \times 10^4 = 1{,}946{,}460 \text{ m}^3$$

5.4 Synthetic Unit Hydrograph

Developing a UH to a catchment requires detailed information about the rainfall and the resulting runoff hydrograph. However, such information is not always available and/or the data available may be unreliable. As a result, looking for a different approach for UH development is necessary. The use of equations that relate the salient hydrograph characteristics (peak discharge, time lag, time base) to the basin characteristics (basin area, slope of basin, area-elevation curve, and number of major streams in the basin) is available – a synthetic unit hydrograph (Chow, 1964). The term 'synthetic' in synthetic unit hydrograph (SUH) denotes UH derived from catchment characteristics rather than from rainfall-runoff data.

Among the available approximate methods for SUH derivation (Singh, 1988), the method of fitting a smooth curve manually through a few salient points of UH is generally practiced. The method of Snyder (1938) utilize empirical equations for estimation of peak flow (Q_p), time lag (t_L), time base (T_B), time to peak (t_p), UH widths at 50% (W_{50}) and 75% (W_{75}) of the peak flow. The U.S Army Corps of Engineers recommends the widths to be distributed such that one-third is placed before the peak and two-thirds are placed after the peak. Linsley et al (1958) found that the t_L is better correlated with the catchment parameters as shown in equation (5.5). This modified Snyder's method is applied in this study because it uses scaling hydrograph information from one catchment to another catchment with the same physiographic characteristics (Subramanya, 2008).

The fact that the Ghergera catchment data is so scanty and the incompatibility of the rating curves developed for the years 2006 and 2007, to develop SUH based on the UH derived for the upstream catchment was the best option. The assumption is that both catchments share the same peak flow factor (C_p) and lag time factor (C_t). These two parameters are determined first from the previously developed average UH for the upstream catchment and thereafter Q_p, t_L and T_B are computed for the downstream catchment.

The modified Snyder's equation is given by

$$t_L = C_t \left(\frac{LL_{ca}}{\sqrt{S}} \right)^{0.38} \qquad (5.5)$$

where t_L - time lag (h)

L - basin length measured along the water course from the basin divide to the outlet (km)

L_{ca} - distance along the main water course from the gauging station to a point opposite to the catchment centroid (km)

C_t - a regional constant representing watershed slope and storage effects

S - general slope of the catchment

The peak discharge of UH of a standard duration is given by Snyder as

$$Q_p = \frac{2.78 C_p A}{t_L} \tag{5.6}$$

where Q_p - peak discharge of UH (m^3/s); A - Catchment area (km^2); C_p - a regional constant.

The time base (T_B) in days of the UH is given as

$$T_B = 3\left(1 + \frac{t_L}{24}\right) \tag{5.7}$$

5.4.1 Determination of Regional Constants

For the upstream catchment, the physiographic characteristics (Table 3.3) are $L = 29.597\,\text{km}$, $L_{ca} = 15.19\,\text{km}$ and $A = 194.646\,\text{km}^2$. The respective average UH characteristics obtained from Fig. 5.6 are $t_L = 0.875\,\text{h}$, $Q_p = 275.48\,\text{m}^3/\text{s}$ and $T_B = 5.5\,\text{h}$. Therefore, from equation (5.5), the time factor is found to be

$$0.875 = C_t\left(\frac{29.597 \times 15.191}{\sqrt{0.125}}\right)^{0.38}$$

$$C_t = 0.058$$

Substituting the values from the upstream average UH in equation (5.6) gives the peak flow factor as

$$275.48 = \frac{2.78 C_p \times 194.646}{0.875}$$

$$C_p = 0.445$$

Here, it is attempted to determine the constant 3 of equation (5.7) from the observed data (derived average UH) knowing time base and lag time. Replacing it by the constant α and substituting the values in equation (5.7) gives

$$\frac{5.5}{24} = \alpha\left(1 + \frac{0.875}{24}\right)$$

$$\alpha = 0.22$$

5.4.2 Synthetic Unit Hydrograph for Ungauged Catchments

Using the same regional constants, downstream catchment's lag time, peak flow and time base has been computed to acquire a 0.25 h rainfall excess duration to comply with the upstream station for comparison purposes. Using the physiographic parameters of Ghergera (Table 3.3), the following procedures have been followed for its SUH derivation.

From equation (5.5) the time lag is,

$$t_L = 0.058\left(\frac{42.91 \times 18.82}{\sqrt{0.116}}\right)^{0.38} \; ; \; t_L = 1.11\,\text{h}$$

The duration of rainfall excess in hours is computed from

$$t_D = \frac{t_L}{5.5} \qquad (5.8)$$

Substituting,

$$t_D = \frac{1.11}{5.5} = 0.20\,\text{h}$$

The duration computed by equation (5.8) is not the desired duration (t_{re}). Therefore, Snyder proposed the following relationship for calculating alternate durations,

$$t_{L.alt} = t_L + 0.25(t_{re} - t_D) \qquad (5.9)$$

Substituting values, we get,

$$t_{L.alt} = 1.11 + 0.25(0.25 - 0.20) = 1.12\,\text{h}$$

Even though there is very little difference between the initial lag time and the alternate lag time owing to the initial duration is very close to the desired duration, the later is used in the subsequent calculations.

The time to peak (t_p) is equal to

$$t_p = t_{re}/2 + t_{L.alt} \qquad (5.10)$$

Substituting,

$$t_p = \frac{0.25}{2} + 1.12 \approx 1.25\,\text{h}$$

The peak discharge of the unit hydrograph is

$$Q_p = \frac{2.78 \times 0.445 \times 525.726}{1.12} = 580.7 \text{ m}^3/\text{s}$$

A general rule of thumb for determining the time base is to use 3 to 5 times the time to peak as the Snyder's equation for the time base over predicts for small catchments. Therefore, the calculation to be more realistic to the actual problem, the coefficient α is determined using the time base of the observed data at the upstream is used in the calculation of the time base.

$$T_B = 0.22\left(1 + \frac{1.12}{24}\right); \; T_B = 0.23 \text{ days} \quad \text{or} \quad T_B = 5.52 \text{ h}$$

In addition to Debarwa and Ghergera catchments, additional catchments (most of them having stream gauging stations at their outlets) in the Mereb-Gash river basin are arbitrarily selected in this study, namely; Tsorona, Adi-Chigono, Tokombia, Tessenei, Ethio1 and Ethio2. The last two are small catchments with an area less than 400 km^2 that lie outside of Eritrea where as the remaining ones are with areas more or less about 5,000 km^2. The main reason for selecting these catchments is to approximately determine the discharge per unit area of each selected catchment of the basin within Eritrea and outside (Ethiopia), based on the time and flow factors obtained from the gauged Debarwa site. UHs by GIUH based Nash model of these outlets will be discussed in the subsequent chapter.

The way the Ghergera's SUH time base determined seem to work only for small catchments such as Ghergera, Ethio1 and Ethio2 as the value of T_B is more close to the one obtained using the rule of thumb. Thus, the same is adopted for developing the aforementioned catchments' SUH. On the other hand, it ends up with so small value of T_B when it is applied to the moderate and big catchments such as Tsorona, Adi-Chigono, Tokombia and Tessenei. Furthermore, direct use of equation (5.7) gives T_B to be more than three days which is unacceptable for these catchments due to the fact that the majority of the areas possess poor geologic formation that does not enhance infiltration, its rugged nature that develops flash flood and high evaporation rates. Hence, T_B proposed by Taylor and Schwartz (1952) which is five times the time peak given in equation (5.10) is followed.

$$T_B = 5(t_{L.alt} + 0.5t_{re}) \tag{5.11}$$

where t_{alt} is the alternate time lag (h); and t_{re} is the required non standard duration (h).

To assist in the sketching of SUH, the widths of UH at 50% and 75% of the peak have been recommended by the US Army Corps for Engineers as,

$$W_{50} = \frac{5.87}{q^{1.08}} \tag{5.12a}$$

and

$$W_{75} = \frac{W_{50}}{1.75} \tag{5.12b}$$

where $q = Q/A$ (peak discharge per unit catchment area in m^3/s/km^2)

Substituting, we get

$$q = \frac{580.7}{525.726} = 1.105\ \text{m}^3/\text{s/km}^2$$

Thus,

$$W_{50} = \frac{5.87}{1.105^{1.08}} = 5.27\ \text{h}\ (0.50\,Q_p = 290.35\ \text{m}^3/\text{s})$$

$$W_{75} = \frac{5.24}{1.75} = 3.0\ \text{h}\ (0.75\,Q_p = 435.53\ \text{m}^3/\text{s})$$

In plotting Fig. 5.7, the recommendation given by the US Army Corps for Engineers did not hold good and therefore, it is not adopted here. The ordinates at 0.25 h time step are given in Table 5.9.

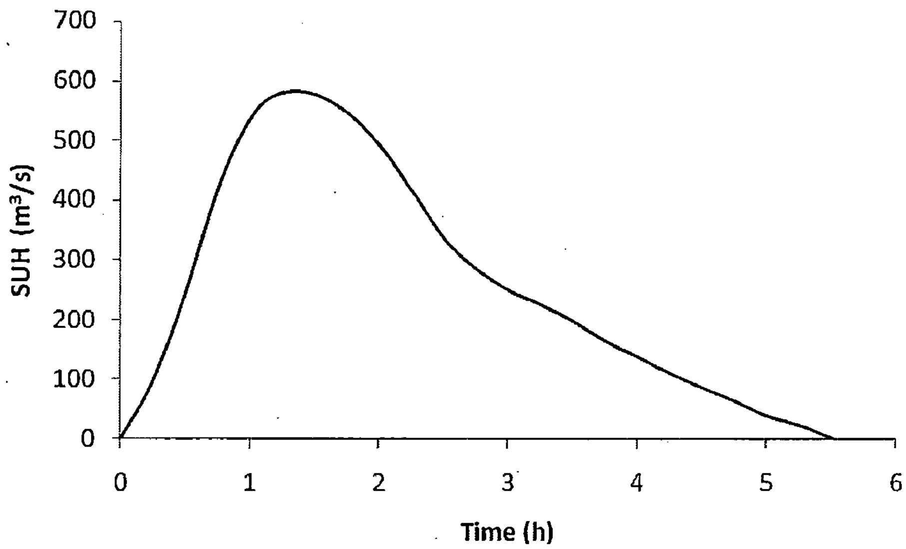


Fig. 5.7 0.25 h duration SUH developed for the Ghergera river catchment

Table 5.9 0.25 h SUH ordinates for Ghergera river catchment

Time (hr)	0.00	0.25	0.50	0.75	1.00	1.25	1.50	1.75	2.00	2.25
SUH (m^3/s)	0	100	243	420	540	580.7	578	545	492	415
Time (hr)	2.50	2.75	3.00	3.25	3.50	3.75	4.00	4.25	4.50	4.75
SUH (m^3/s)	335	287	250	226	198	165	138	111	87	65
Time (hr)	5.00	5.25	5.50	5.52						
SUH (m^3/s)	40	24	2	0						

The sum of SUH ordinates, $\sum(SUH) = 5{,}841.7\ \text{m}^3/\text{s}$

The area under the SUH curve should be equal to the unit volume, i.e. 1 cm depth of rainfall times the catchment area.

$$V = \sum(SUH) \times \Delta t \times 3600\ (\text{m}^3) \qquad \text{where } \Delta t - \text{the time step (h)}$$

$$V = 5{,}841.7 \times 0.25 \times 3{,}600 = 5{,}257{,}530\ \text{m}^3$$

and the unit volume (v)

$$v = 1cm \times A = 1 \times 525.726 \times 10^4 = 5{,}257{,}260\ \text{m}^3$$

5.5 Flow Contribution by Sub-basins in Mereb-Gash

The discharge per unit area, q of the different catchments can help us to draw some inference about the basin characteristics. All the outlets with their respective water divide are shown in Fig. 3.6 of chapter 3. The characteristics of each sub-basin are determined by using C_t and C_p having values 0.058 and 0.445, respectively. From the sub-basins' physiographic parameters discussed in section 3.1.4, the value of each Snyder's parameter for all of the sub-basins is calculated and is given in Table 5.10. The same procedure is followed for all the sub-basins and that of Tessenei is given below for illustration purposes.

The time lag is determined from (5.5) for C_t = 0.058 and taking the values of L, L_{ca} and S from Table 3.2,

$$t_L = 0.058\left(\frac{489.3338 \times 260.629}{\sqrt{0.119}}\right)^{0.38} = 7.57\ \text{h}$$

Duration of rainfall excess for standard storm,

$$t_D = \frac{t_L}{5.5} = \frac{7.57}{5.5} = 1.38\ \text{h}$$

Let the required duration t_{re} be 0.25 h, then the time lag from equation (5.9) is

$$t_{L.alt} = 7.57 + 0.25(0.25 - 1.38) = 7.29\,\text{h}$$

The value of the t_L is replaced by the alternate time lag $t_{L.alt}$ in calculating the peak discharge. From (5.10), we get

$$t_p = \frac{0.25}{2} + 7.29 = 7.41 \approx 7.5\,\text{h}$$

From (5.11), $T_B = 37.05\,\text{h} \approx 37\,\text{h}$

Similarly, the peak flow is calculated as

$$Q_p = 2.78 C_p \frac{A}{t_{L.alt}} \qquad \text{in m}^3/\text{s}$$

$$Q_p = \frac{2.78 \times 0.445 \times 21{,}805.24}{7.29} = 3{,}700.31\,\text{m}^3/\text{s}$$

Therefore, the discharge per unit area, q

$$q = \frac{Q_p}{A} = \frac{3{,}700.31}{21{,}805.24} = 0.17\,\text{m}^3/\text{s.km}^2$$

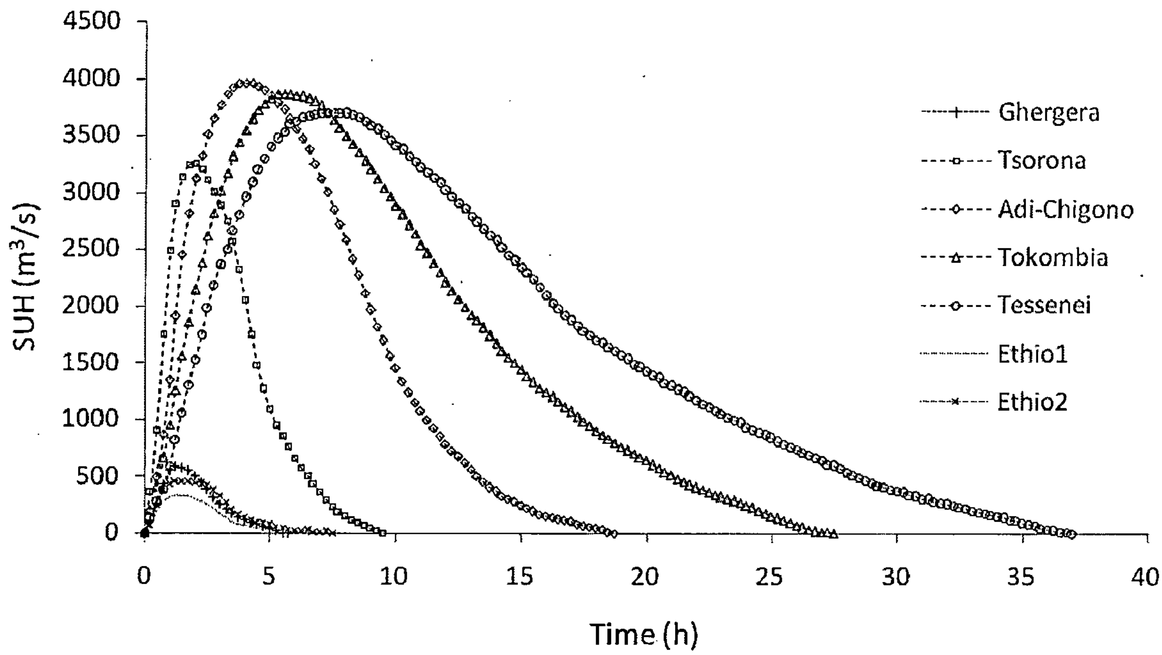


Fig. 5.8 SUH for different outlets in the Mereb-Gash river basin

Table 5.10 Values of q for different sub-basins in the Mereb-Gash river basin

Sub-basin	Area (km^2)	Time Lag t_L (h)	Duration t_D (h)	Alternate time lag $^*t_{alt}$ (h)	Time peak t_p (h)	Time base T_B (h)	Peak Flow Q_p (m^3/s)	Discharge/unit area q (m^3/s.km^2)
Debarwa	194.646	0.875	0.25	-	1.0	5.5	275.48	1.415
Ghergera	525.726	1.0	0.20	1.12	1.25	5.5	580.7	1.105
Tsorona	4,652.755	1.79	0.325	1.77	1.90	9.5	3,251.93	0.70
Ethio1	319.481	1.19	0.22	1.20	1.32	6.5	329.36	1.031
Ethio2	515.711	1.40	0.255	1.40	1.5	7.5	455.70	0.884
Adi-Chigono	11,553.576	3.72	0.68	3.61	3.75	18.75	3,959.26	0.342
Tokombia	16,873.45	5.59	1.02	5.40	5.5	27.5	3,865.58	0.229
Tessenei	21,805.24	7.57	1.38	7.29	7.5	37	3,700.31	0.17

*alternate time lag is calculated for a required duration (t_{re}) of 0.25 h

CHAPTER-6

GEOMORPHOLOGIC INSTANTANEOUS UNIT HYDROGRAPH BASED NASH MODEL

6.1 Introduction

Predicting runoff hydrograph under the arid and semi-arid climate, for countries like Eritrea, is essential for the assessment of water availability, design of various hydraulic structures and watershed development and management. As such, different techniques have been developed for this purpose. The most commonly used practice in applied hydrology being the UH theory introduced by Sherman (1932) which is based on observed rainfall and runoff data from gauged catchments as it has been discussed in detail in the preceding chapter.

A catchment acts as a hydrological system transforming input hydrograph into a runoff hydrograph with the usual characteristics of translation and attenuation of the former's peak (Mathur, 1972a). The transfer function can generally be classified as derived entity, mathematical function and conceptual entity. Furthermore, its nature could be lumped or distributed, linear or non-linear, time variant or time invariant, etc (Singh, 1988). Based on this system transformation approach, numerous conceptual rainfall–runoff models have been developed to simulate the rainfall-runoff process of transformation (Zoch, 1934; Clark, 1945; Nash, 1958; Dooge, 1959; Mathur, 1972a and 1972b). Nash (1957) proposed a conceptual model composed a cascade of linear reservoirs. Dooge (1959) developed a general UH theory which is represented by some combination of linear channels and linear reservoirs. However, due to non-linear relationship between rainfall and runoff, the applications of these models for ungauged catchments have had only a limited degree of success for special cases (Yen and Lee, 1997). Moreover, Nourani et al (2008) suggested that most conceptual models contain large numbers of parameters that cannot be related to physical watershed characteristics, and hence, they must be estimated by calibration using observed data.

Earlier, hydrologists had been using purely empirical equations to develop rainfall-runoff models for ungauged catchments (Snyder, 1938; SCS, 1957). Analytic method and trial-and-error method are some of the common currently available methods to estimate the SUH. In this regard, the technique of regionalization of parameters utilizing the data from the gauged catchments in hydrometeorologically similar regions can be applied. This approach of

SUH derivation has various limitations. However, the shortcomings are overcome by the use of instantaneous unit hydrograph (IUH).

When the unit duration of the rainfall excess of a UH becomes infinitesimally small, the resulting direct runoff hydrograph is called an IUH. It has the advantage of eliminating the main assumption, i.e. uniformity of rainfall excess during the unit duration of the UH. The IUH is widely applied because it can reflect the characteristic of valley flow concentration (Gupta et al, 1980) and much importantly information about the basin characteristics can also be extracted. But, the parameters of IUH are usually evaluated through the statistical moments of effective rainfall and the observed surface runoff.

For one or another reason, the above discussed methods of UH derivation for ungauged catchments have drawbacks and hence, hydrologists have been working to relate catchment response as a function of its geomorphologic properties. Since Rodriguez-Iturbe and Valdes (1979) presented the geomorphologic instantaneous unit hydrograph (GIUH), there have been many attempts to obtain an IUH that incorporates the geomorphologic properties of the catchment (Gupta et al, 1980; Rodriguez-Iturbe et al, 1982; Rosso, 1984; Gupta et al, 1986). Therefore, the main focus of this chapter is to discuss the methodology of developing GIUH based Nash model which could be used for direct surface runoff computation for the Mereb-Gash basin.

6.2 Nash Model

The formulation of Nash's (1957) IUH was obtained under the assumption that catchment behaviour can be associated with a cascade of n linear reservoirs arranged in series (Fig. 6.1). An input of unit rainfall excess over the catchment is applied to the first reservoir instantaneously. The routed outflow from the first reservoir becomes the input to the second reservoir in series and the second reservoir output becomes the input to the third, and so on. Output from the last n^{th} reservoir is the output from the system representing an IUH for the catchment (Chow et al, 1988). This model is not only lumped and time invariant but also provides an explicit equation for the derivation of IUH of a natural catchment as given under.

$$Q(t) = \frac{1}{\Gamma(n)} \frac{1}{K} \left(\frac{t}{K} \right)^{n-1} e^{-t/K} \qquad (6.1)$$

where $Q(t)$ is the IUH of the Nash model; $\Gamma(n)$ is the Gamma function; and K is the storage function.

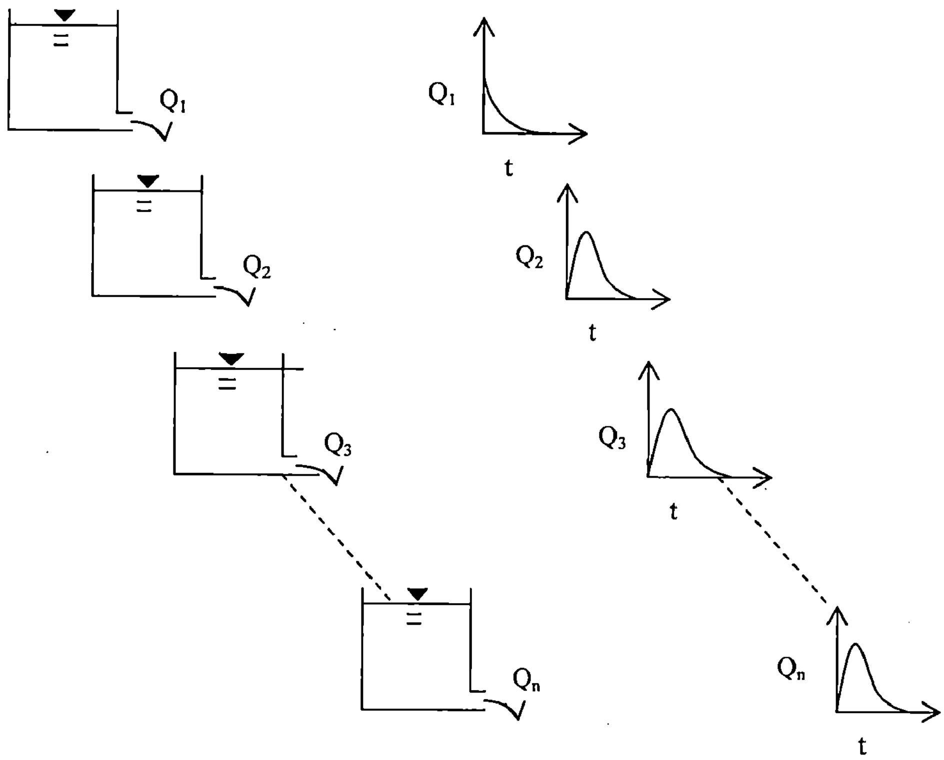


Fig. 6.1 A series of linear reservoirs in Nash Model

6.2.1 Parameter Estimation of the Nash Model

The Nash Model has two parameters n and K which can be determined by a number of ways Singh (1988). The most commonly used one being the method of moments. If a catchment has a total lag time (say T), then each reservoir has a lag time K equal to T/n. Taking the first moment of the IUH equation (6.1) about the origin that is when $t = 0$,

$$M_1 = nK \tag{6.2}$$

M_1 represents the lag time of the centroid of the IUH which is equal to the difference of the first moments of direct runoff hydrograph (DRH) and effective rainfall hyetograph (ERH).

$$M_{DRH1} - M_{ERH1} = M_1 \tag{6.3}$$

Similarly, taking the second moment (M_2) of the IUH (6.1) about the origin, i.e. $t = 0$ is given as follows,

$$M_2 = n(n+1)K^2 \tag{6.4}$$

The difference of the second moments of DRH and ERH about the origin gives,

$$M_{DRH2} - M_{ERH2} = M_2 + (2M_1 \times M_{ERH1}) \tag{6.5}$$

Combining (6.3) and (6.5) and solving, the values of n and K can be determined.

It is worth noting that for the direct determination of the above parameters, reliable historical records of the rainfall and runoff are very much required. On the other hand, the application of the preceding method for the IUH computation for ungauged catchments is difficult. However, hydrologists have been looking for other techniques coupled with the Nash model for IUH derivation for ungauged catchments.

6.3 GIUH Based Nash Model

As discussed in the preceding sections, GIUH has become one of the most recently applied techniques in IUH derivation for ungauged catchments. The basis for GIUH development is the extensive use of stream order laws proposed by Horton (1945) which was later modified by Strahler (1957). Wooding (1965) predicted stream hydrograph using the kinematic wave theory for flow over a catchment and along the stream, assuming the rainfall has constant intensity and finite duration. Yen and Lee (1997) used the geomorphologic ratios of the Horton-Strahler stream ordering systems for GIUH generation. In addition to this, they derived GIUH using the kinematic wave theory to determine the travel times for overland and channel flows in a stream-ordering sub-basin system.

Incorporating GIUH based stream order ratio laws with other linear and non-linear conceptual models for a UH generation has been proved to be successful. Rosso (1984) related the Horton's order ratios to the parameters of the Nash IUH model on the basis of a geomorphologic model of the catchment response through power regression. He showed that the Nash parameter n was dependent on the bifurcation ratio (R_B), length ratio (R_L) and area ratio (R_A) as given in Table 6.1. Similarly, Bhaskar et al (1997) derived a GIUH from watershed geomorphologic characteristics and then related to the parameters of the Nash IUH model for deriving the complete shape of the IUH for Jira river catchment in eastern India. Looking at the facts and the lack of complete data about the study area, the GIUH based Nash model has been tested for the study area.

6.4 Development of GIUH for Upper Mereb-Gash Sub-basin

6.4.1 Estimation of Geomorphologic Parameters

The stream channel networks of the catchments were ordered according to Strahler (1957) in ArcGIS (ARC/INFO Window; version 9.3, 2008) as shown in Fig. 6.2. The delineated catchment is exported to ERDAS Imagine (version 9.1, 1991-2006) software for further image enhancement so that boundary of each contributing sub-area can be seen clearly.

Table 6.1 Geomorphologic parameters used in GIUH derivation (Horton's laws)

Parameter	Description
Bifurcation ration (R_B)	$R_B = N_w / N_{w+1}$ where N_w = number of streams in order w.
Stream length ratio (R_L)	$R_L = \overline{L}_w / \overline{L}_{w-1}$ where $\overline{L}_w$ = mean stream length given by $\overline{L}_w = L_w / N_w$ where $L_w = \sum_{i=1}^{N_w} L_i$ (the total length of all streams of order w).
Stream area ratio (R_A)	$R_A = \overline{A}_w / \overline{A}_{w-1}$ where $\overline{A}_w$ is the mean stream area of the catchment of order w. It is determined from, $\overline{A}_w = A_w / N_w$. A_w of a watershed of order w is the total area projected upon a horizontal plane contributing overland flow to the stream segment of the given order of all segments of lower order.

Thereafter, this image is used for digitizing the same in ArcGIS and extracting its different geomorphologic properties. The highest stream orders are found to be 3 and 5 for Debarwa and Ghergera river catchments, respectively.

Stream channel slope ($\overline{S}_{cl}$) of order w, is the average stream channel slope of all channel segments of order w (Singh, 1989). Accordingly, the average slope of a stream order w is determined by taking the sum of the elevation differences of its stream segments divided by sum of their lengths. Main channel slope ($\overline{S}_{cm}$) is computed using the "85-10" slope factor method proposed by Benson (1962). This is the slope between 85% and 10% of the distance along the stream channel from the basin outlet to the divide. Accordingly, the main channel slopes for Debarwa and Ghergera river catchments are found to be 1.23% and 1.11%, respectively. The overland slope ($\overline{S}_{ol}$) represents the average slope of the overland area contributing to each of the stream order w. The DEM of each area is extracted and it's $\overline{S}_{ol}$ has been determined using the Spatial Analyst tool in ArcGIS. The values of these different parameters are given in Tables 6.2 and Table 6.3.

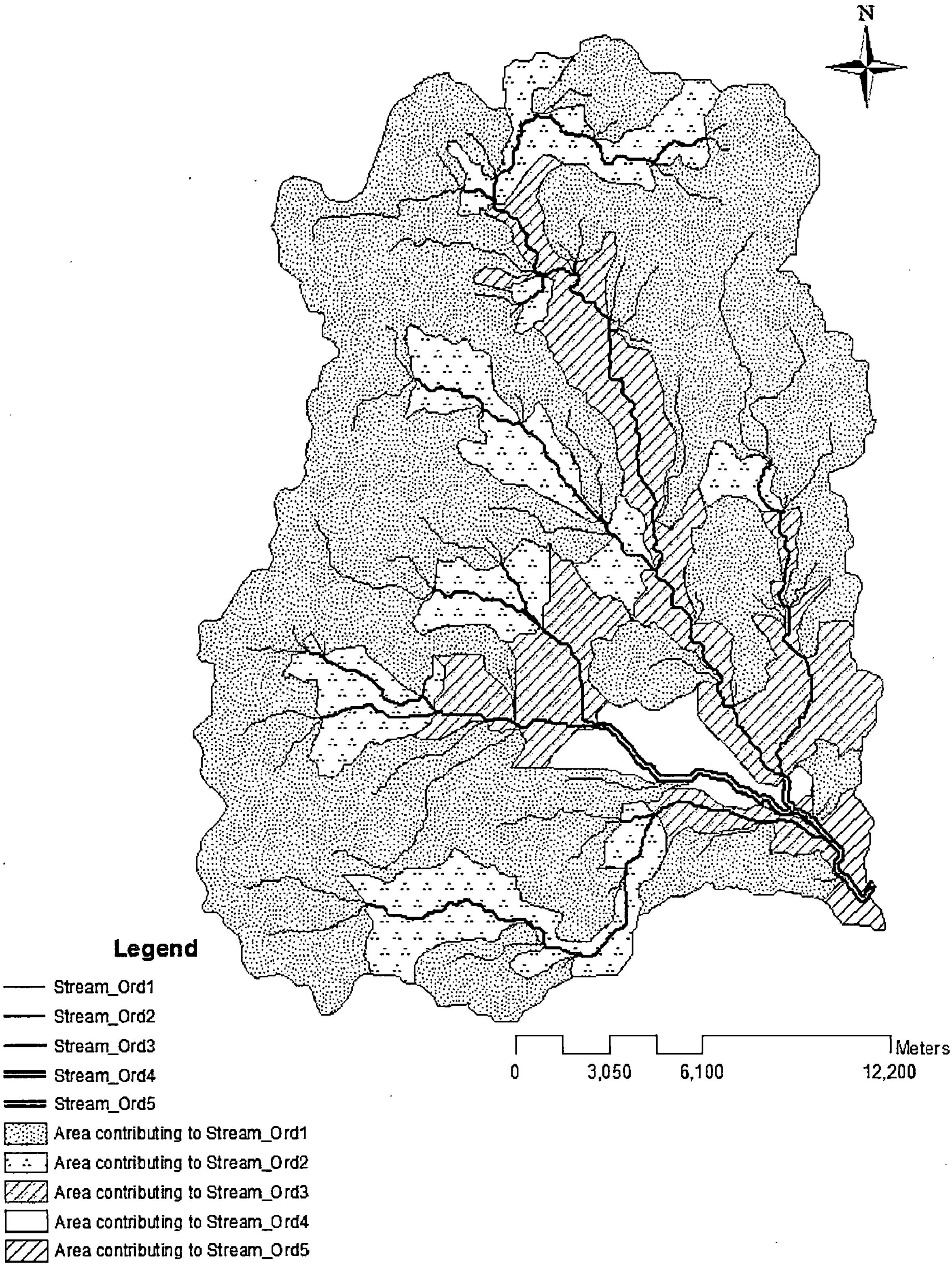


Fig. 6.2 Stream orders and their respective contributing areas

Table 6.2 Length and area for the upper Mereb-Gash sub-basin

Sub-basin	Stream Order	Total number of streams N_w	Total length of streams L_w (km)	Total drainage area of streams A_w (km^2)
Ghergera	1	58	142.089	344.022
	2	14	58.65	428.367
	3	5	51.04	505.037
	4	2	8.942	519.756
	5	1	4.194	525.728
Debarwa	1	23	51.97	128.189
	2	6	25.284	166.983
	3	1	17.672	194.646

After the computation of R_B, R_L and R_A for consecutive stream orders using the preceding methods, the same for the whole sub-basins are obtained using two methods; graphical method (Fig. 6.3 and Fig. 6.4) and average method as shown in Table 6.2 and Table 6.3. For the development of the GIUH, the values obtained using the later is adopted here.

6.4.2 Parameter Estimation of the GIUH based Nash Model

The relationship between the peak discharge q_p and time peak t_p of the IUH as a function of the geomorphologic characteristics of the catchment (Rodriguez-Iturbe and Valdes, 1979) is given as follows,

$$q_p = 1.31 R_L^{0.43} (V_d / L_\Omega) \tag{6.6}$$

and

$$t_p = 0.44 \left(\frac{L_\Omega}{V_d} \right) \left(\frac{R_B}{R_A} \right)^{0.55} R_L^{-0.38} \tag{6.7}$$

where Ω - stream order of the catchment; L_Ω - length of the highest order stream (km); V_d - dynamic velocity parameter (m s^{-1}); R_B, R_L and R_A are bifurcation ratio, length ratio and area ratio, respectively. The parameters q_p and t_p have units (h^{-1}) and (h), respectively. Further, the assumption of the IUH to have a triangular shape gives a satisfactory result as long as q_p and t_p are correctly estimated.

Table 6.3 Geomorphologic parameter values of the upper Mereb-Gash sub-basin

Sub-basin	Stream order	Mean stream Length $\bar{L}_w$ (km)	Cumulative mean stream length (km)	Mean stream area $\bar{A}_w$ (km^2)	Mean stream slope $\bar{S}_{cl}$	Mean overland slope $\bar{S}_{ol}$	Bifurcation ratio R_B	Stream length ratio R_L	Cumulative mean stream length ratio	Stream area ratio R_A	Stream slope ratio R_s
Gherghera	1	2.450	2.450	5.931	0.0265	0.136	4.1429	-	-	-	1.4586
	2	4.189	6.639	30.598	0.0181	0.087	2.8000	1.7100	2.7100	5.1586	1.8543
	3	10.208	16.847	101.007	0.0098	0.064	2.5000	2.4367	2.5376	3.3011	0.6281
	4	4.471	21.318	259.878	0.0156	0.025	2.0000	0.4380	1.2654	2.5729	1.3730
	5	4.194	25.512	525.728	0.0113	0.117	-	0.9380	1.1967	2.0230	-
				Average	0.016	0.086	2.8607	1.3807	1.9274	3.2639	1.3285
Debarwa	1	2.260	-	5.573	0.032	0.135	3.8333	-	-	-	1.7547
	2	4.214	-	27.831	0.018	0.091	6.000	1.8650	-	4.9934	1.8406
	3	17.672	-	194.646	0.010	0.098	-	4.1936	-	6.9940	-
				Average	0.020	0.108	4.9167	3.0293		5.9937	1.7977

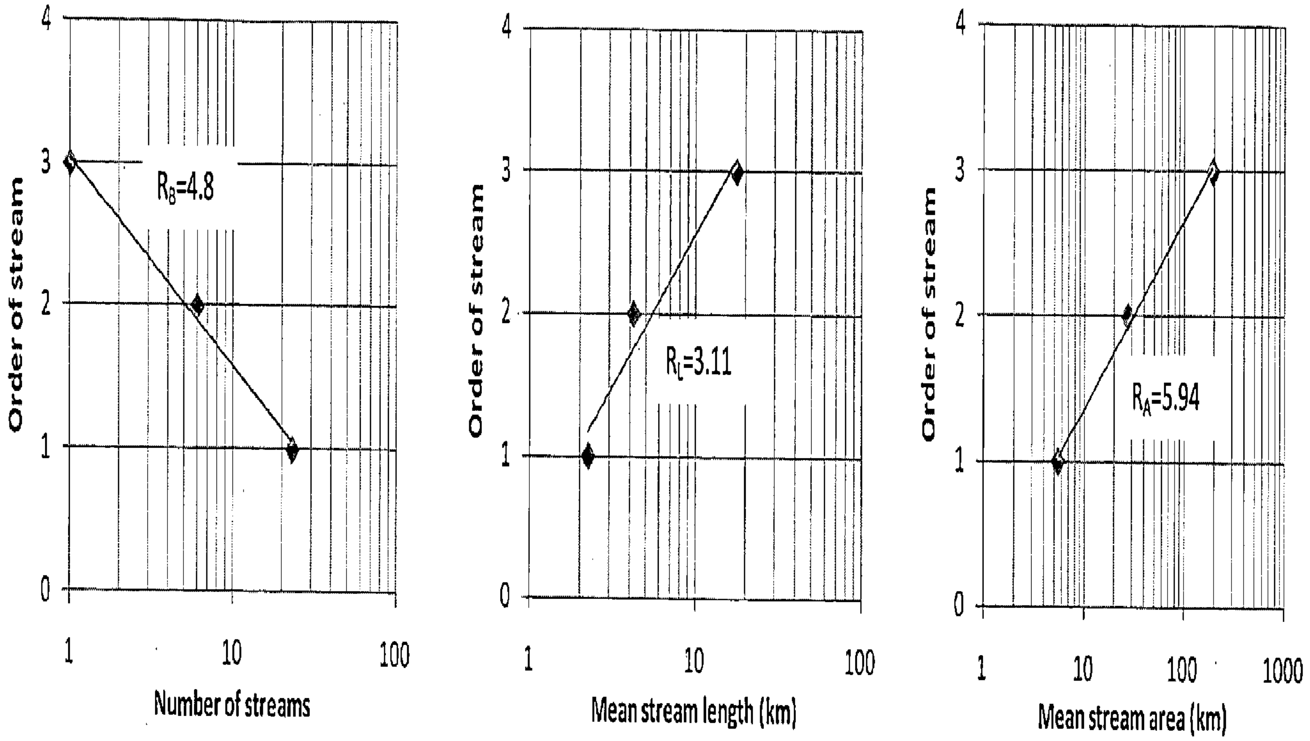


Fig. 6.3 Graphical method of determining the Horton's stream order ratios for Debarwa catchment

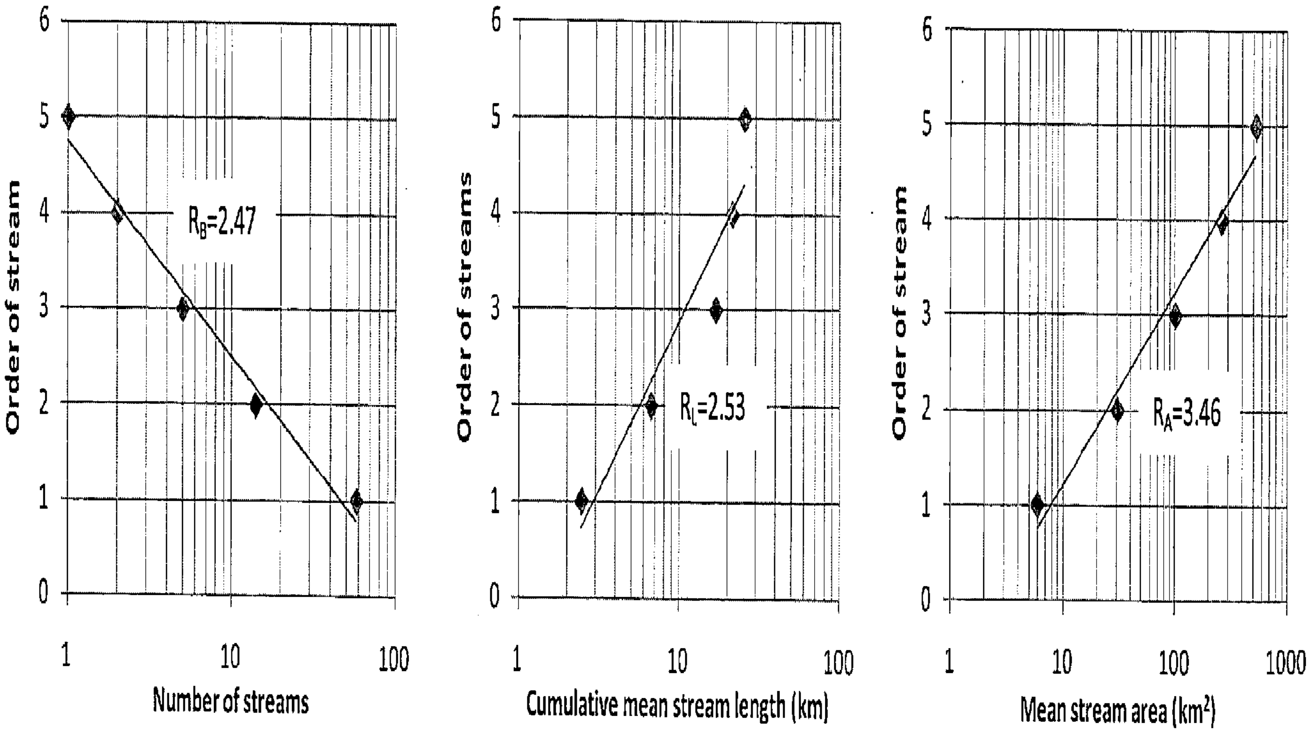


Fig. 6.4 Graphical method of determining the Horton's stream order ratios for Ghergera catchment

Multiplication of (6.6) and (6.7) gives a non-dimensional term (Rosso, 1984) which is independent of the dynamic velocity and storm characteristics. It is purely a function of the geomorphologic characteristics which is as follows.

$$q_p \times t_p = 0.5764(R_B / R_A)^{0.55} R_L^{0.05} \tag{6.8}$$

On the other hand, the derivative of (6.1) gives the time to peak as follows,

$$t_p = (n-1) \times K \tag{6.9}$$

Substituting this value for t_p in (6.1), the peak discharge q_p of the IUH is obtained as,

$$q_p = \frac{(n-1)^{n-1}}{K \times \Gamma(n)} \times e^{-(n-1)} \tag{6.10}$$

The product of (6.9) and (6.10) gives a function of the Nash parameter n. Thus,

$$q_p \times t_p = \frac{(n-1)^n}{\Gamma(n)} \times e^{-(n-1)} \tag{6.11}$$

Equating (6.8) and (6.11), the following relationship is arrived at.

$$\frac{(n-1)^n}{\Gamma(n)} \times e^{-(n-1)} = 0.5764 \left(\frac{R_B}{R_A} \right)^{0.55} R_L^{0.05} \tag{6.12}$$

Using the MatLab optimization techniques of (6.12), we obtain the Nash model IUH parameter n. Re-arranging (6.9) and substituting the right hand side of (6.7) for t_p, the value of K could be solved from,

$$K = \frac{0.44}{n-1} \times \left(\frac{L_\Omega}{V_d} \right) \left(\frac{R_B}{R_A} \right)^{0.55} R_L^{-0.38} \tag{6.13}$$

A proper estimate of dynamic velocity is needed in (6.13).

6.4.3 Estimation of Dynamic Velocity

The dynamic velocity proposed by Rodriguez-Iturbe and Valdes (1979) is the velocity corresponding to the peak discharge for a given rainfall-runoff event in the catchment. Two ways of determining the velocity parameter are discussed as follows.

i) The equilibrium velocity can be obtained with the help of Manning equation. Thus,

$$V_d = \frac{1}{n_m} R_c^{2/3} \overline{S}_{cm}^{1/2} \tag{6.14}$$

where n_m - Manning roughness coefficient; R_c - hydraulic radius; and $\overline{S}_{cm}$ - slope of the main channel (Bhaskar et al, 1997).

ii) When a Manning's roughness coefficient is unavailable but the velocity and its corresponding discharges at different water levels at the gauging station are available, then a regression equation can be drawn between the velocity and the rainfall excess intensity ($V_d = \beta i_e^{\delta}$) where $i_e = P_e / \Delta t$ in (mm/h) β and δ are constants (Sarkar et al, 2009).

The available rainfall data does not give the detailed information about a particular storm's duration; it only gives the beginning and the end of the rainfall. Rainfall storms with duration greater than one hour are considered to have a uniformly distributed rainfall depth throughout the entire period. On the other hand, rainfall storms with duration less than one hour are considered to have occurred during the one hour duration regardless of how short the actual duration of the storm was. Consequently, the rainfall-runoff analyses of the recorded data show, in most cases, the peak discharges occur either before or at the same time with the rainfall excess which is far from reality as there exist time lag between the ERH and DRH. This is possibly due to the fact that the catchment experiences a short duration and high intensity rainfall in most cases, and therefore, a one hour time step of the rainfall excess hyetograph which is used in the rainfall analysis may not be the correct approach.

Because of the preceding fact, obtaining the regression equation between the dynamic velocity and the rainfall excess intensity has not been easy. Instead, the velocity at the peak discharge of a selected storm is calculated by applying (6.14). Here, the Manning's roughness coefficient adopted for this study is 0.035 and 0.04 for upstream and downstream, respectively. The exact channel section profiles at the gauging stations were not available. Based on DEM, Google Earth and available information an approximate trapezoidal sections having side slopes 1:1 and bottom widths of 11 m and 15 m for Debarwa and Ghergera, respectively may be appropriate for Manning's equation applications.

Finally, the GIUH is developed with the help of the Nash model (6.1) for different values of t. This represents an IUH of 1cm of rainfall excess which was taken as an instantaneous input to the system. Therefore, multiplying this IUH by the catchment area yields the discharge (m^3/s).

6.5 GIUH Development for Mereb-Gash Basin

One of the limitations of UH applications is that it cannot be used for large catchments. It is recommended to be applied for small catchments though there is no fixed value as catchment conditions vary from place to place. It is not known which limiting value is applicable to Eritrean catchments. Thus, two approaches have been used here for UH derivation at different outlets using the GIUH based technique as is discussed in the preceding sections of this chapter; considering the entire area contributing to the selected outlet no matter how big the area is, and taking limiting value in to consideration. For the later approach, literatures recommend UH to be applied to catchments having an area less than or equal to 5,000 km^2. Accordingly, the entire basin is divided in to four sub-basins so that this limiting value approach is more or less maintained as is shown in Fig. 6.5. The results are compared to SUH developed by using the Debarwa observed data. By convoluting these UHs and their respective severe most rainfalls, the design runoffs can be computed. For outlets consisting of more than one sub-basin, runoff can be determined by applying hydrologic routing techniques to each sub-basin and superimposing them. The highest stream order of the basin is five as given in Table 6.4.

6.6 UH Development from GIUH

The ultimate objective of GIUH development is to derive a UH of required duration which in turn can be used for computation of direct surface runoff. Thus, the following equation is used for this purpose:

$$(D\text{-hour UH})_t = \frac{1}{2}[(IUH)_t + (IUH)_{t-D}] \qquad (6.15)$$

where D – is the duration of the UH

The D-hour UHs were obtained by lagging the IUHs by their respective duration (D-hour) and corresponding ordinates are summed up and divided by two. The duration of the predicted UH is taken to be equal to the duration of the observed UH.

Table 6.4 Length and area for the selected outlets of Mereb-Gash basin

Sub-basin	Stream Order	Total number of streams N_w	Total length of streams L_w (km)	Total drainage area of streams A_w (km^2)
Tsorona	1	142	621.7685	2833.387
	2	34	460.9406	4057.767
	3	6	152.5531	4492.699
	4	2	67.4864	4643.675
	5	1	3.5454	4652.755
Adi-Chigono (excluding Tsorona)	1	197	895.4215	4012.122
	2	43	410.3495	5232.942
	3	12	282.3845	6208.796
	4	2	61.6543	6413.243
	5	1	153.9113	6902.206
Adi-Chigono (including Tsorona)	1	339	1517.1899	6845.5090
	2	77	871.2901	9290.7090
	3	18	434.9376	10701.4950
	4	4	129.1407	11056.9180
	5	1	157.4568	11554.9610
Tokombia (excluding Adi-Chigono)	1	170	765.678	3162.947
	2	48	384.114	4119.73
	3	14	167.128	4634.515
	4	2	56.744	4867.067
	5	1	130.425	5318.489
Tokombia (including Adi-Chigono)	1	509	2282.8679	10007.0710
	2	125	1255.4041	13410.4390
	3	32	602.0656	15336.0100
	4	6	185.8847	15923.9850
	5	1	287.8818	16873.4500
Tessenei (excluding Tokombia)	1	165	792.242	3153.4870
	2	44	436.133	4176.6890
	3	10	113.839	4449.6160
	4	1	14.189	4497.7840
	5	1	129.424	4931.7900
Tessenei (including Tokombia)	1	674	3075.1099	13160.5580
	2	169	1691.5371	17587.1280
	3	42	715.9046	19785.6260
	4	7	200.0737	20421.7690
	5	1	417.3058	21805.2400
Ethio-1	1	11	43.705	183.999
	2	2	15.523	226.298
	3	1	28.607	319.481
Ethio-2	1	17	56.197	280.477
	2	4	52.512	432.817
	3	2	14.904	475.707
	4	1	14.568	515.711

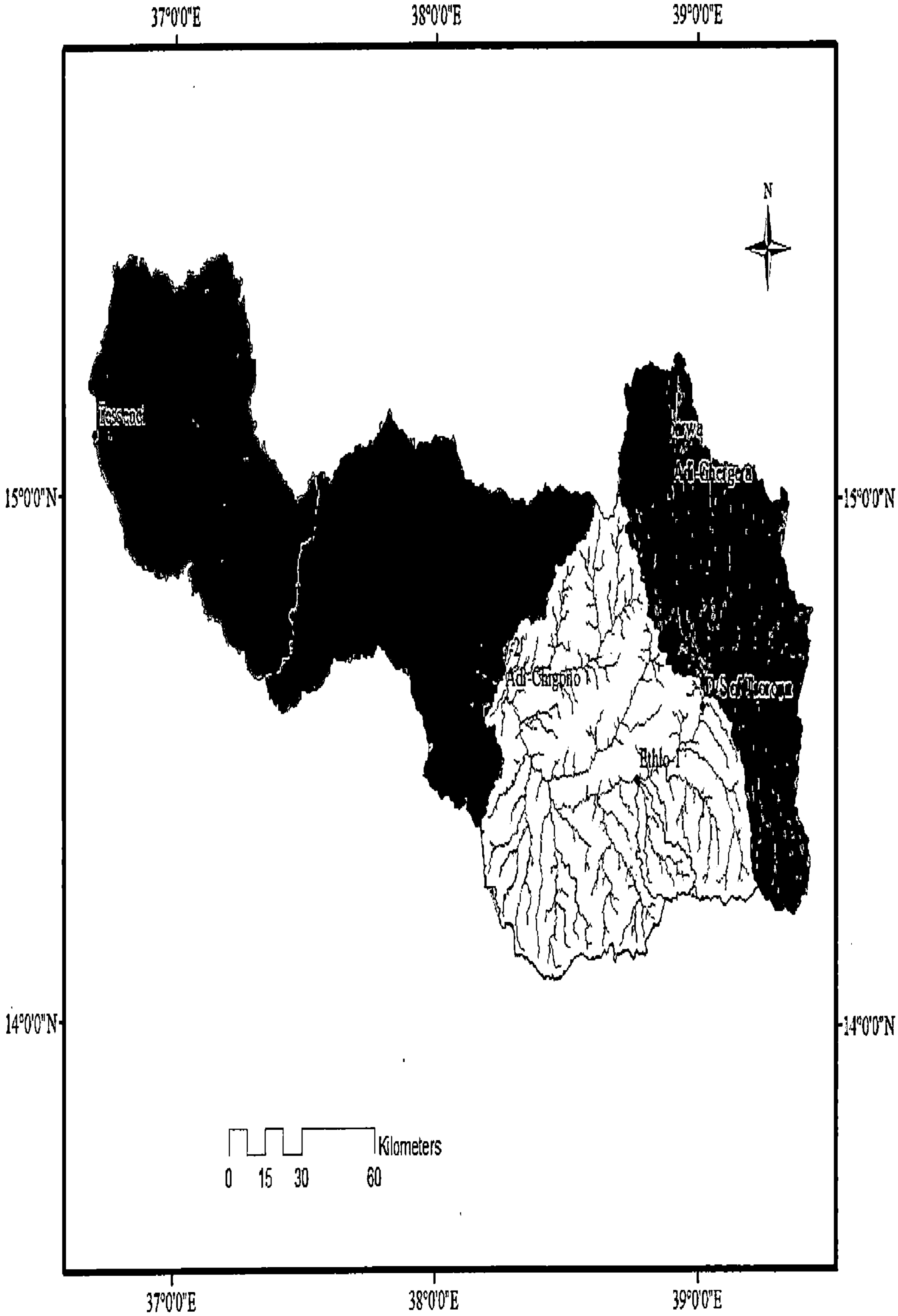


Fig. 6.5 Sub-basins of the Mereb-Gash river basin

Table 6.5 Geomorphologic parameter values for selected sub-basins

Sub-basin and length of highest order	Stream order	Mean stream Length $\bar{L}_w$ (km)	Cumulative mean stream length (km)	Mean stream area $\bar{A}_w$ (km^2)	Bifurcation ratio R_B	Stream length ratio R_L	Cumulative mean stream length ratio	Stream area ratio R_A
Tsorona $L_\Omega = 20km$	1	4.3787	4.3787	19.9534	4.1765	-	-	-
	2	13.5571	17.9357	119.3461	5.6667	3.0962	4.0962	5.9812
	3	25.4255	43.3612	748.7832	3.0000	1.8754	2.4176	6.2740
	4	33.7432	77.1044	2321.8375	2.0000	1.3271	1.7782	3.1008
	5	3.5454	80.6499	4652.7550	-	0.1051	1.0460	2.0039
				Average	3.7108	1.6010	2.3345	4.3400
Adi-Chigono (excluding Tsorona) $L_\Omega = 153.911km$	1	4.5453	4.5453	20.3661	4.5814	-	-	-
	2	9.5430	14.0883	121.6963	3.5833	2.0995	3.0995	5.9754
	3	23.5320	37.6203	517.3997	6.0000	2.4659	2.6703	4.2516
	4	30.8271	68.4475	3206.6215	2.0000	1.3100	1.8194	6.1976
	5	153.9113	222.3588	6902.2060	-	4.9927	3.2486	2.1525
				Average	4.0412	2.7170	2.7095	4.6443
Adi-Chigono (including Tsorona) $L_\Omega = 157.457km$	1	4.4755	4.4755	20.1932	4.4026	-	-	-
	2	11.3155	15.7909	120.6586	4.2778	2.5283	-	5.9752
	3	24.1632	39.9541	594.5275	4.5000	2.1354	-	4.9274
	4	32.2852	72.2393	2764.2295	4.0000	1.3361	-	4.6495
	5	157.4568	229.6961	11554.9610	-	4.8771	-	4.1802
				Average	4.2951	2.7192	-	4.9330

Table 6.5 cont...

Sub-basin	Stream order	Mean stream Length $\bar{L}_w$ (km)	Cumulative mean stream length (km)	Mean stream area $\bar{A}_w$ (km^2)	Bifurcation ratio R_B	Stream length ratio R_L	Cumulative mean stream length ratio	Stream area ratio R_A
Tokombia (excluding Adi-Chigono) $L_\Omega = 130.425km$	1	4.5040	4.5040	18.6056	3.5417	-		-
	2	8.0024	12.5064	104.4333	3.4286	1.7767		4.6130
	3	11.9377	24.4441	435.4701	7.0000	1.4918		3.8570
	4	28.3720	52.8161	2869.0036	2.0000	2.3767		7.3512
	5	130.4250	183.2411	8187.4926	-	4.5970		2.1855
				Average	3.9926	2.5605		4.5017
Tokombia (including Adi-Chigono) $L_\Omega = 287.882km$	1	4.4850	4.4850	19.6603	4.0720	-		-
	2	10.0432	14.5282	107.2835	3.9063	2.2393		5.4569
	3	18.8146	33.3428	479.2503	5.3333	1.8734		4.4671
	4	30.9808	64.3236	2653.9975	6.0000	1.6466		5.5378
	5	287.8818	352.2053	16873.4500	-	9.2923		6.3577
				Average	4.8279	3.7629		5.4549
Tessenei (excluding Tokombia) $L_\Omega = 129.424km$	1	4.8015	4.8015	19.1120	3.7500	-		-
	2	9.9121	14.7136	94.9248	4.4000	2.0644		4.9668
	3	11.3839	26.0975	444.9616	10.0000	1.1485		4.6875
	4	14.1890	40.2865	4497.7840	1.0000	1.2464		10.1083
	5	129.4240	169.7105	4931.7900	-	9.1214		1.0965
				Average	4.7875	3.3952		5.2148

Table 6.5 cont...

Sub-basin	Stream order	Mean stream Length $\bar{L}_w$ (km)	Cumulative mean stream length (km)	Mean stream area $\bar{A}_w$ (km^2)	Bifurcation ratio R_B	Stream length ratio R_L	Cumulative mean stream length ratio	Stream area ratio R_A
Tessenei	1	4.5625	4.5625	19.5261	3.9882	-		-
(including	2	10.0091	14.5716	104.0658	4.0238	2.1938		5.3296
Tokombia)	3	17.0453	31.6169	471.0863	6.0000	1.7030		4.5268
$L_\Omega = 417.306km$	4	28.5820	60.1989	2917.3956	7.0000	1.6768		6.1929
	5	417.3058	477.5046	21805.2400	-	14.6003		7.4742
				Average	5.2530	5.0435		5.8809
	1	3.9732	3.9732	16.7272	5.5000	-		-
Ethio-1	2	7.7615	11.7347	113.1490	2.0000	1.9535		6.7644
$L_\Omega = 28.607km$	3	28.6070	40.3417	319.4810	-	3.6858		2.8235
				Average	3.7500	2.8196		4.7940
	1	3.3057	3.3057	16.4986	4.2500	-		-
Ethio-2	2	13.1280	16.4337	108.2043	2.0000	3.9713		6.5584
$L_\Omega = 14.568km$	3	7.4520	23.8857	237.8535	2.0000	0.5676		2.1982
	4	14.5680	38.4537	515.7110	-	1.9549		2.1682
				Average	2.7500	2.1646		3.6416

CHAPTER-7

RESULTS AND DISCUSSIONS

7.1 GIUH for Upper Mereb-Gash Sub-basin

The values of the Nash model IUH parameters are determined from (6.12) and (6.13) with the help of MatLab (version 7.6, 2008). Accordingly, the values of n are found to be 3.034 and 3.085 for Debarwa and Ghergera, respectively. The parameter K is inversely proportional to the dynamic velocity as it can be seen from equation (6.13) and in the meantime it is very sensitive to small variation of the same. The dynamic velocity computed using the Manning's equation is also in turn sensitive to the Manning's coefficient (n_m). Thus, the value of n_m as recommended by Chow et al (1988) for natural channels has not been found suitable here as it yields very high velocities. The computed velocities of the different storms are large probably due to the higher slopes of main channels and small values of K (less than 0.6 h).

As mentioned earlier, the Nash parameters could have been obtained from the observed ERH and DRH. Basically, this is helpful for validating the GIUH based Nash model parameters. Nevertheless, due to the mismatch and uncertainties of the ERH and DRH this sort of analysis is excluded. For the selected storm events, different parameters are computed, yielding an average t_p and Q_p of 1.093 h and 269.41m^3/s and 1.039 h and 721.394 m^3/s for catchments of the rivers Debarwa and Ghergera, respectively. The time to peak and peak discharge are very much dependent on L_Ω. The parameter L_Ω of the downstream catchment is much smaller than that of upstream catchment and its effect has been reflected in the values of time to peak as shown in Table 7.1. Even though the time to peak of both catchments is nearly the same the peak discharge of the downstream catchment is approximately twice to that of the upstream catchment.

The UHs shown in Fig. 7.1 and Fig. 7.2 reveal that for shorter durations the GIUH and UH are approximately the same. It is, therefore, quite acceptable to consider the GIUH equivalent to UH for durations less than one hour. Furthermore, the GIUH based Nash model and that of Sherman's UHs for the Debarwa catchment have no significant difference among the corresponding UH parameters. The same is true for the outputs of Ghergera catchment in 2007 with the exception of July 17, 2007 where the peak is higher with smaller time peak.

This could be due to the non-uniformity of the rainfall, i.e. the rainfall might have occurred in the vicinity of the outlet. On the contrary, a significant difference in the output of both UHs for the 2006 data at Ghergera is clearly noticed. The reason for this irregularity could be due to errors in using the appropriate discharge computation techniques.

Table 7.1 UH parameters of different storm events

Sub-basin	Storm date	Stage (m)	V_d (m/s)	K (h)	t_p (h)	Q_p (m^3/s)
Debarwa (n = 3.034)	July 17, 2006	1.96	4.2173	0.5335	1.085	279.730
	August 2, 2006	2.20	4.4895	0.5012	1.019	283.292
	August 4, 2006	2.10	4.3780	0.5139	1.045	277.495
	August 16,	1.58	3.7451	0.6008	1.222	237.123
Ghergera (n = 3.085)	August 2, 2006	2.20	3.8730	0.5809	1.182	660.843
	August 4, 2006	2.10	3.7736	0.5962	1.213	623.295
	August 22,	4.00	5.344	0.4210	0.856	801.458
	August 5, 2007	2.37	4.6126	0.4878	0.992	776.425
	August 10,	3.50	4.8024	0.4685	0.953	744.947

7.2 Comparison of Various UHs

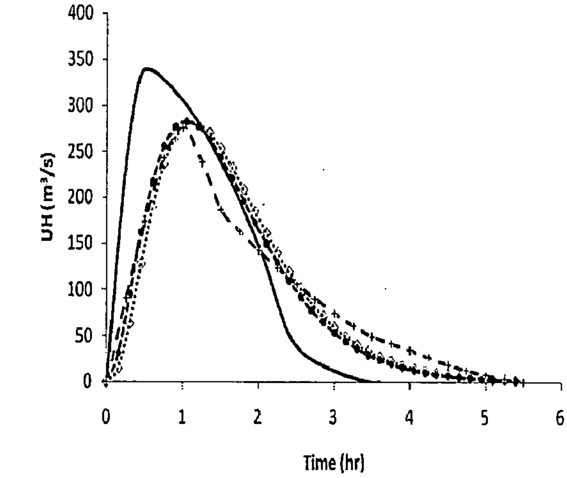
The UH derived by Sherman's method and Snyder's SUH method are compared with one obtained by GIUH based Nash model using different performance evaluation methods; model efficiency (EFF) (Nash and Sutcliff, 1970) and root mean square error (RMSE) given as follows,

$$EFF = \left(1 - \sum_{i=1}^{m}(Q_{oi} - Q_{ci})^2 / \sum_{i=1}^{m}(Q_{oi} - \overline{Q}_o)^2\right) \times 100 \tag{7.1}$$

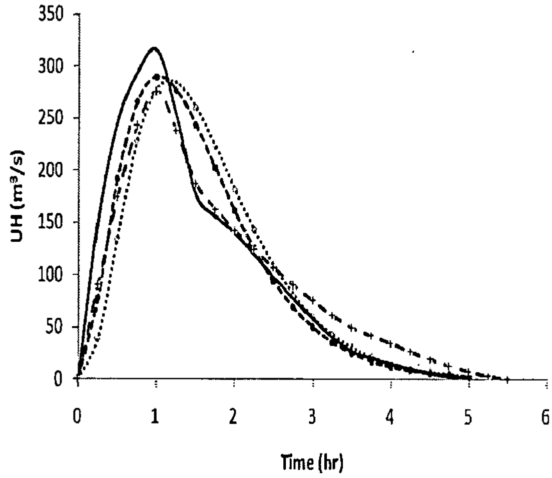
and

$$RMSE = \sqrt{\frac{\sum(Q_{oi} - Q_{ci})^2}{m}} \tag{7.2}$$

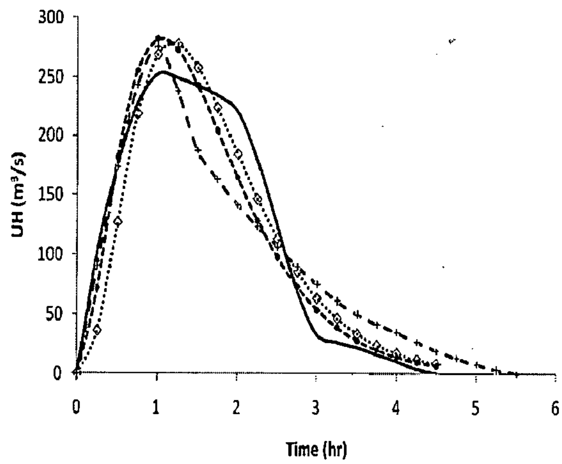
where Q_{oi} = ith ordinate of the observed discharge (m^3/s); $\overline{Q}_o$ = average of the ordinates of observed discharge (m^3/s); Q_{ci} = computed discharge (m^3/s); m = number of ordinates. The computed values are given in Table 7.2 and Table 7.3.



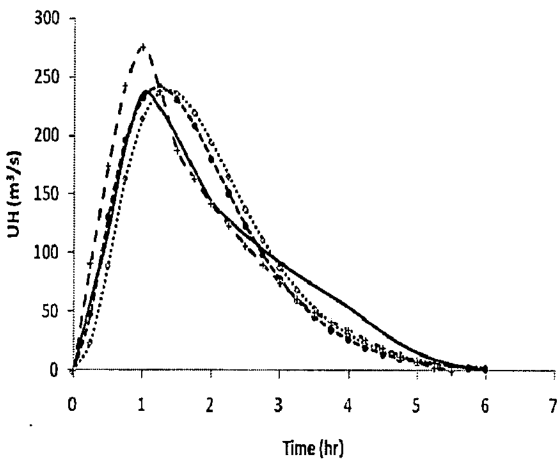
a) 0.15hr-UH for July 17, 2006



b) 0.25hr-UH for August 2, 2006

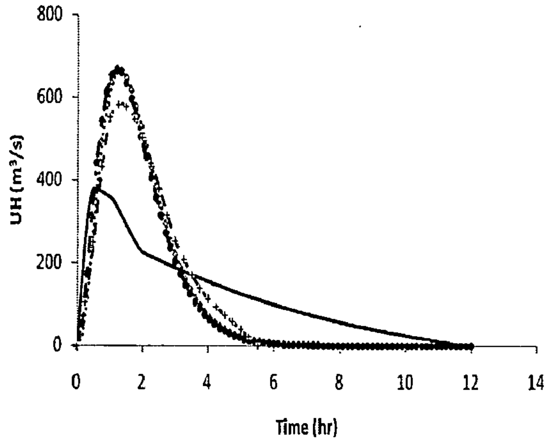


c) 0.25hr-UH for August 4, 2006

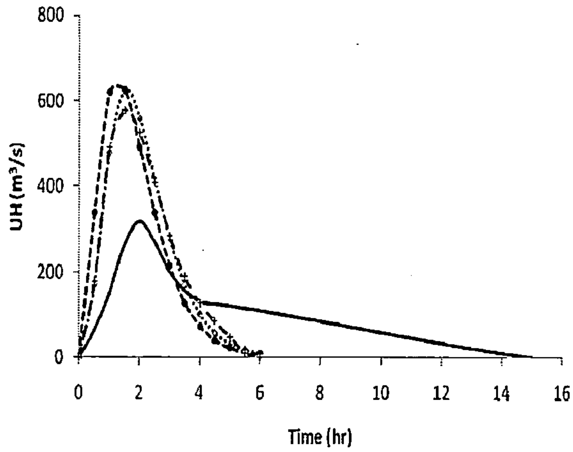


d) 0.25hr-UH for August 16, 2006

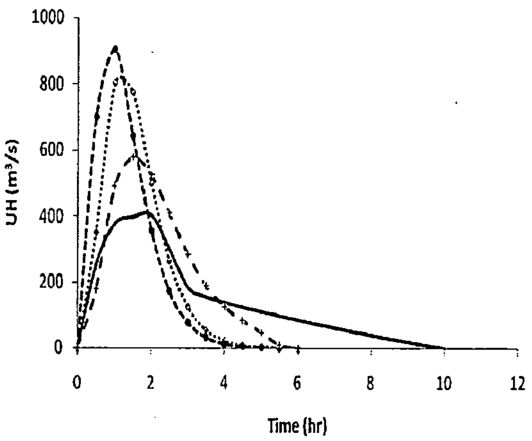
Fig. 7.1 UHs derived for the Debarwa river catchment



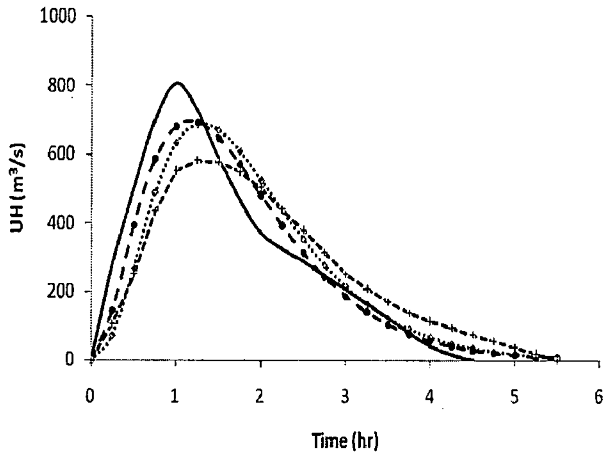
a) 0.15hr-UH for August 2, 2006



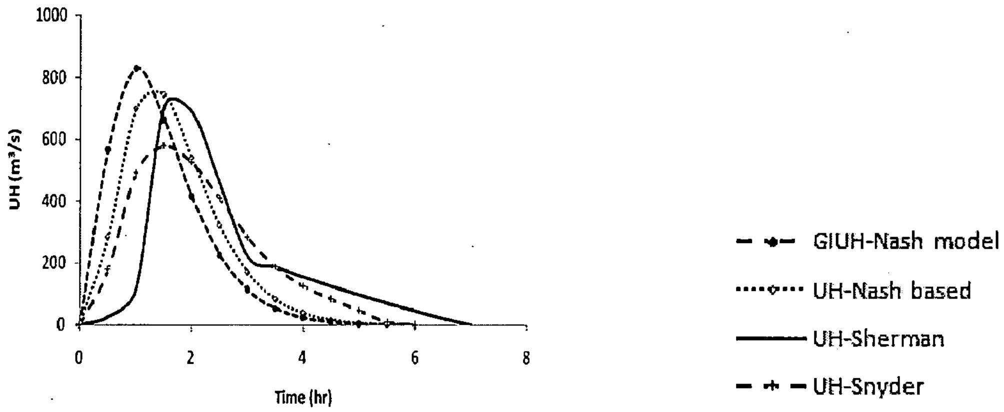
b) 0.50hr-UH for August 4, 2006



c) 0.50hr-UH for August 22, 2006



d) 0.25hr-UH for August 5, 2007



e) 0.5hr-UH for August 10, 2007

Fig. 7.2 UHs derived for the Ghergera river catchment

Table 7.2 Performance evaluation of the GIUH based Nash model

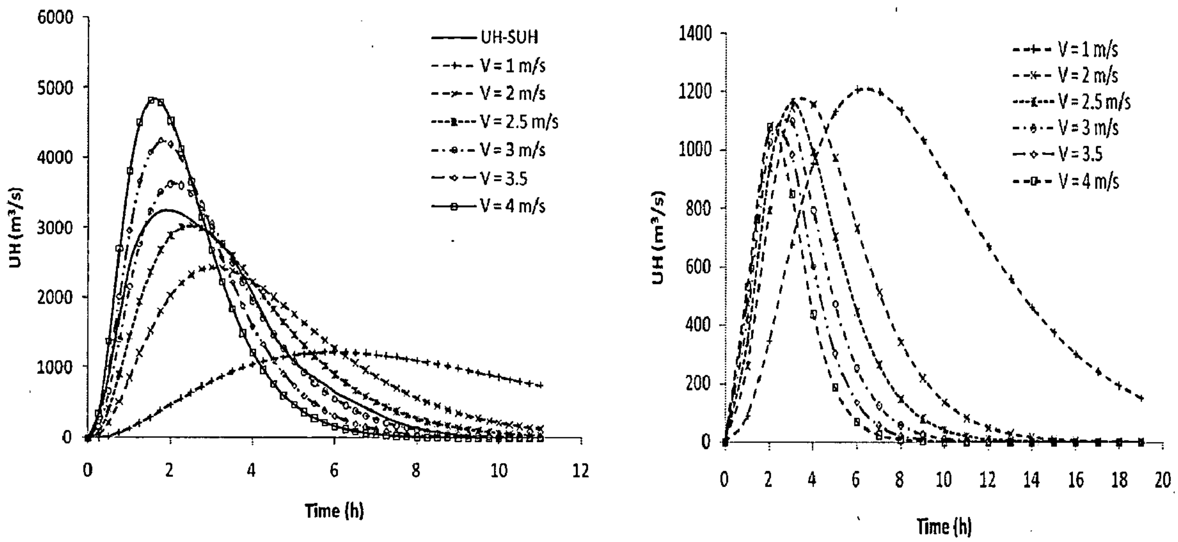
S.no	Storm date	Debarwa		Ghergera	
		RMSE	EFF (%)	RMSE	EFF (%)
1	July 17, 2006	16.25	58.33	-	-
2	August 2, 2006	10.2	78.44	6.14	91.10
3	August 4, 2006	5.94	93.09	5.75	98.96
4	August 16, 2006	4.79	88.92	-	-
5	August 22, 2006	-	-	38.16	54.02
6	August 5, 2007	-	-	9.81	94.32
7	August 10, 2007	-	-	27.32	76.43

Table 7.3 Percentage (%) variations in UH parameters for upper Mereb-Gash sub-basin

S.no	Storm Date	Debarwa		Ghergera	
		t_p	Q_p	t_p	Q_p
1	July 17, 2006	20.57	-10.95	-	-
2	August 2, 2006	1.93	-10.03	0	10.8
3	August 4, 2006	4.53	9.89	0	7.77
4	August 16, 2006	22.19	0.45	-	-
5	August 22, 2006	-	-	-33.33	38.58
6	August 5, 2007	-	-	0	18.13
7	August 10, 2007	-	-	0	28.81

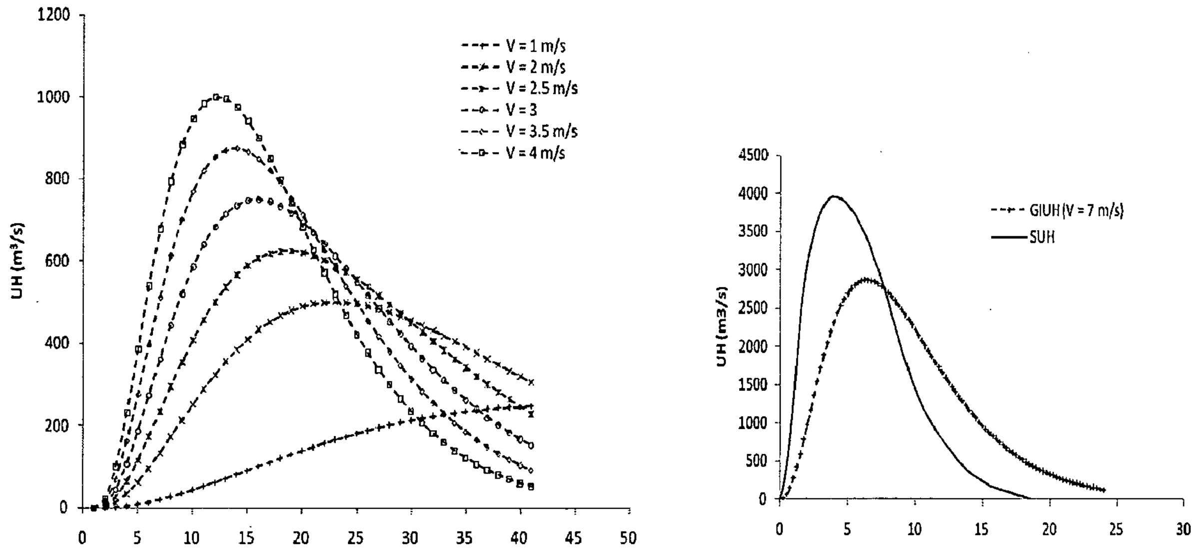
7.3 UHs of Various Sub-basins

As discussed in the previous chapter, two cases are considered in this analysis. Firstly, when the basin is divided into smaller sub-basins and secondly, when the entire area above the selected outlet is considered. The fact that almost all of the selected catchments don't have historical data along the Mereb-Gash river basin with the exception of upper Mereb-Gash, their GIUHs are developed by assigning values to the dynamic velocity for the computation of the GIUH parameters. The smaller the dynamic velocity, the smaller is the peak discharge, the larger time peak and time base and vice versa. The UHs developed for the second case are compared to the previously derived SUH. The results indicate (Fig. 7.3, Fig. 7.4, Fig. 7.5 and Fig. 7.6) that with the increase of catchment area the SUH and GIUH diverge as in the case of Tessenei, Tokombia and that of Adi-Chigono. Furthermore, the peak discharge is higher and time peak is smaller in the case of SUH compared to the GIUH of these catchments. On the other hand, it is observed that when the catchment area is small like that of Ethio1 and Ethio2, both approaches give more or less similar outputs (Fig. 7.7).



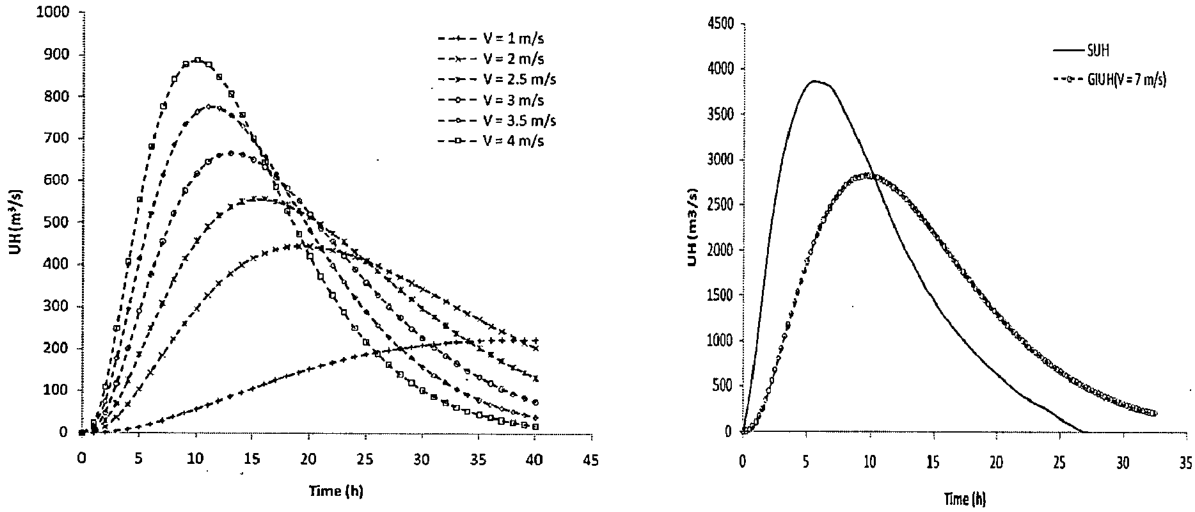
a) b)

Fig. 7.3 UHs for Tsorona catchment a) 0.25 h UH b) 1 h UH



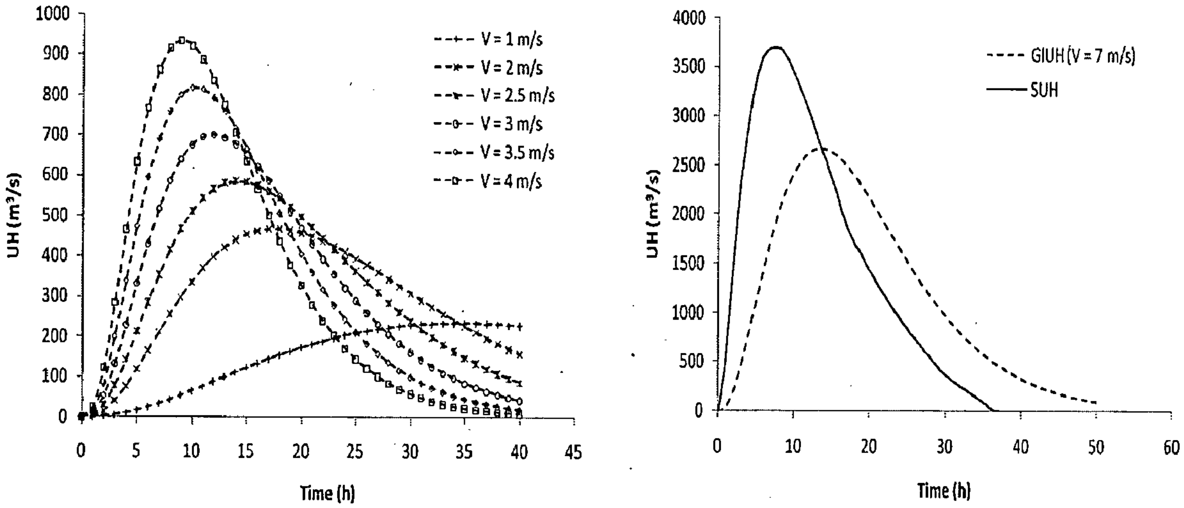
a) b)

Fig. 7.4 UHs for Adi-Chigono a) 1 h UH excluding Tsorona b) 0.25 h UH including Tsorona



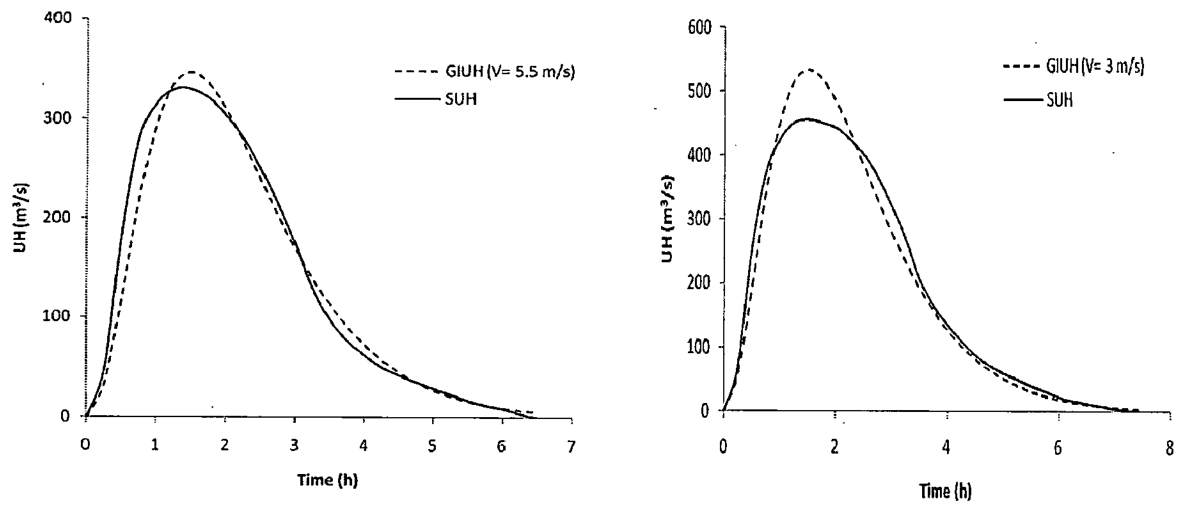
a) b)

Fig. 7.5 UHs for Tokombia a) 1 h UH excluding Adi-Chigono b) 0.25 h UH including Adi-Chigono



a) b)

Fig. 7.6 UHs for Tesseni a) 1 h UH excluding Tokombia b) 0.25 h UH including Tokombia



a) b)

Fig. 7.7 UHs for areas beyond the boundry a) 0.25 h UH for Ethio1 b) 0.25 h UH for Ethio2

When the basin is divided in to smaller sub-basins, different parameters of each sub-basin are computed as indicated in Table 7.4 for an assumed dynamic velocity parameter. A dynamic velocity of 4 m/s and 1h duration are selected for Tsorona, Adi-Chigono, Tokombia and Tessenei. On the other hand, a velocity that gives a peak more or less equal to that of SUH for Ethio1 and Ethio2 and duration of 0.25 h are assumed.

As explained earlier, L_Ω is found to be the most senstive parameter and is reflected here in the sub-basin of Tsorona. The fact that the outlet is selected such that a major tributary joins the main river of this sub-basin near the outlet is the cause of area to have the smallest length of the highest order (L_Ω). As a result, it has very much influenced it's time peak and storage coefficient (K).

Table 7.4 Different parameter values of the selected outlets (Case 1)

Parameter	Tsorona	Adi-Chigono	Tokombia	Tessenei	Ethio1	Ethio2
n	3.0705	3.1380	3.1700	3.3063	2.9257	2.8140
V (m/s)	4.0	4.0	4.0	4.0	5.5	3.0
K (h)	0.7064	5.022	4.330	3.701	0.7002	0.7526
t_p (h)	1.463	10.16	9.40	8.54	1.35	1.37
q (m^3/s)	1078.041	999.001	886.906	934.258	346.149	534.654
Duration (h)	1.0	1.0	1.0	1.0	0.25	0.25

CHAPTER-8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusion

The UHs derived from GIUH based Nash model for the Debarwa river catchment can be used for computation of direct surface runoffs as there is no significant difference among the UH derived by Sherman method and the GIUH based Nash model method. Similarly, due to the closeness of SUH and GIUH based Nash model, the measured discharge data of Ghergera in 2007 seems to be correct where as it's 2006 data is either wrong due to discharge computational errors or there was a shift/change of gauging station. However, after comparing the results of GIUH based Nash model with SUH, even though the peak discharge obtained by the former is slightly greater than that of the later, it gives satisfactory results and could be used for direct surface runoff computations.

It has become clear that SUH method is applicable for small catchment areas to the likes of Debarwa, Adi-Ghergera, Ethio1 and Ethio2 which are smaller than 550 km^2. The major assumption of homogeneity of catchments does not hold well with the increase of catchment area which in turn affects the values of the regional constants. As a result, for the bigger catchments (Tsorona, Tokombia, Adi-Chigono and Tessenei), divergence between the GIUH based Nash model and SUH is observed. Therefore, further study is required for ensuring the application of GIUH based Nash model is as good as when it is applied to small catchments by comparing with observed discharges.

Predicting runoff from existing rainfall measurements depends largely on the length of time interval considered. The rainfall-runoff analyses of the recorded data show, in most cases, the peak discharges occur either before or at the same time with the rainfall excess which is far from reality as there is time lag between the ERH and DRH. This is possibly due to the fact that the catchment experiences a short duration and high intensity rainfall in most cases, and therefore, a one hour time step of the rainfall excess hyetograph which is used in the rainfall analysis may not be the correct approach.

8.2 Recommendation

The Landsat satellite images that are used for the NDVI map making were taken on different dates, in different months and even some of them in different years. This difference in time has been the main cause of the mismatch between the pixels rendering unavoidable reduction in the quality of the output map. Therefore, for better land use/land cover investigations it is worthy to deal with the non-public domain images.

Better results of UH of a particular area can be achieved using the GIUH based Nash model provided stage at the time of peak discharge and the river cross section profiles at gauging station.

The length of the highest order is found to be the most sensitive parameter. Hence, when selecting the outlets in the derivation of UH by GIUH based Nash model, it is highly recommended that the outlet should be in such a way that no major river joins the main river. Almost all of the stage records for a particular runoff at the upper Mereb-Gash stations were not fully recorded. As a result, this has affected not only the number of stream flows selected for analysis but also it is found to be a source of error in getting complete shape of the runoff hydrograph (particularly their time bases) which are achieved by interpolation between two measured discharges at the required interval. Therefore, establishment of automatic stage recorders is recommended to avoid this limitation.

8.3 Future Course of Work

The following activities are badly needed to be accomplished to make this work more meaningful and applicable.

i) Computation of design flood at the different outlets based on design storms from the nearby historical rainfall gauging stations as most of the rain gauges used in this study were recently installed.
ii) Studying the effect of DEM resolution on GIUH based Nash model.
iii) Determining water availability at different outlets of the river basin.
iv) The above work should be supplemented by other conceptual models.

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APPENDICES

Appendix A

Table A-1(a) Missing data filled using distance power method for August 2005

Date	Daily rainfall series for the month of August 2005							
	Abardae	Adi Kefelet	Amadir	Adi Tsnay	Gebele Kelay	Selae Daero	Shketi	Tala
1	0.0	0.0	0.0	3.6	**1.1**	0.0	1.0	0.0
2	9.9	15.6	0.0	0.0	**4.4**	12.4	3.5	6.3
3	1.5	0.6	0.0	8.4	**1.9**	0.0	0.0	0.0
4	25.5	4.5	4.1	12.9	**7.3**	2.5	4.9	5.0
5	3.1	0.0	0.0	6.0	**3.4**	21.5	0.0	0.0
6	33.2	0.0	0.0	0.0	**1.6**	0.0	0.0	0.0
7	11.0	5.5	9.0	38.5	**23.0**	29.0	22.9	20.0
8	13.0	8.0	1.7	6.3	**8.8**	32.7	6.2	10.0
9	2.5	15.2	0.0	43.9	**19.9**	71.4	4.2	15.0
10	0.0	0.0	0.0	0.0	**0.1**	1.0	0.0	0.0
11	37.0	7.8	20.8	5.6	**9.4**	8.6	0.0	24.0
12	0.0	9.3	0.0	0.5	**3.2**	0.0	7.0	0.0
13	0.0	0.0	0.0	0.0	**0.0**	0.0	0.0	0.0
14	0.0	16.0	0.0	2.4	**3.2**	0.0	3.8	3.2
15	0.0	6.0	0.0	1.3	**2.2**	0.0	0.0	15.3
16	0.0	0.0	25.3	0.4	**16.7**	2.0	37.1	0.0
17	0.0	0.0	0.0	0.0	**0.7**	0.0	2.0	0.0
18	3.3	3.8	0.0	0.0	**1.6**	0.0	0.0	12.0
19	8.9	0.0	0.0	5.3	**1.6**	0.0	0.0	0.0
20	4.5	0.0	0.0	15.0	**8.2**	19.6	8.4	0.0
21	2.5	0.0	0.0	1.0	**1.8**	7.1	2.1	1.5
22	0.0	0.0	0.0	0.0	**3.7**	16.0	6.4	0.0
23	27.5	10.3	9.4	0.0	**8.9**	0.0	13.0	11.0
24	0.0	0.0	0.0	22.0	**4.8**	0.0	0.0	0.0
25	0.0	0.0	0.0	0.0	**0.0**	0.0	0.0	0.0
26	14.2	11.5	10.3	5.5	**5.4**	9.1	0.0	6.4
27	2.8	27.4	6.9	3.5	**7.1**	13.4	3.5	11.0
28	12.0	39.5	13.6	42.1	**20.4**	45.5	3.1	13.0
29	1.8	0.0	2.5	0.0	**0.4**	0.0	0.0	0.0
30	1.0	0.0	0.0	0.0	**0.0**	0.0	0.0	0.0
31	9.9	0.0	0.0	0.0	**0.5**	0.0	0.0	0.0

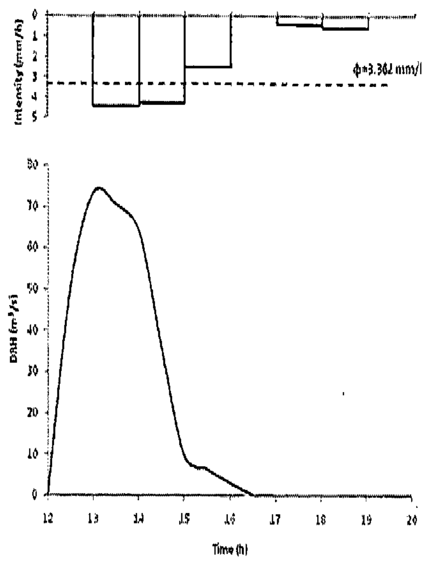
Bold figures are filled data.

Table A-1(b) Missing data filled using distance power method for September 2005

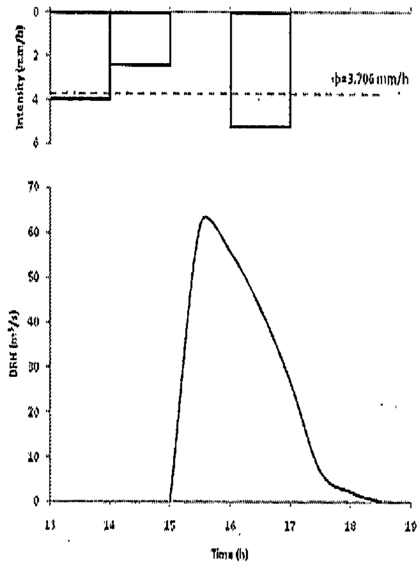
Date	Daily rainfall series for the month of September 2005							
	Abardae	Adi Kefelet	Amadir	Adi Tsnay	Gebele Kelay	Selae Daero	Shketi	Tala
1	2.0	0.2	0.0	2.2	**1.3**	0.0	2.1	0.0
2	0.0	0.0	0.0	0.0	**0.5**	0.0	0.0	4.6
3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
6	0.0	10.0	0.0	0.0	0.0	0.0	0.0	3.0
7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
8	0.0	0.0	2.7	0.0	0.0	0.0	0.0	2.0
9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
10	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
11	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
12	0.0	0.0	0.0	0.0	2.0	0.0	0.0	0.0
13	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
14	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
15	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
16	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
17	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
18	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
19	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
20	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
21	0.0	0.0	1.9	0.0	6.0	0.0	0.0	6.0
22	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
23	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
24	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
25	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
26	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
27	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
28	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
29	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
30	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Bold figures are filled data.

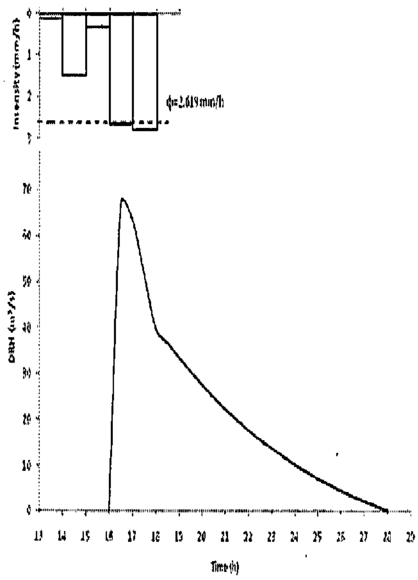
Appendix B Effective rainfall hyetograph and direct runoff hydrograph for selected events at upper Mereb-Gash stations



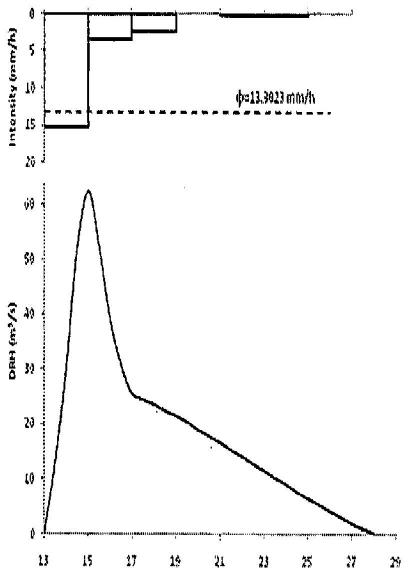
August 4, 2006 at Debarwa



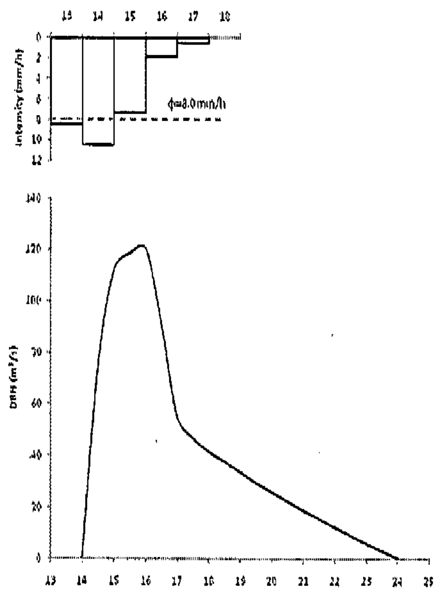
July 17,2006 at Debarwa



August 2, 2006 at Ghergera



August 4, 2006 at Ghergera



August 22, 2006 at Ghergera