

Addis Ababa University
School of Graduate Studies

**DEVELOPMENT OF SYNTHETIC UNIT HYDROGRAPHS FOR WATERSHEDS IN
THE UPPER AWASH AND UPPER TEKEZE BASINS**

By Mulugeta Azeze

October 2004

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**A thesis submitted to School of Graduate Studies, Addis
Ababa University in partial fulfillment for the Degree of
Masters of Science in Civil Engineering**

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ABSTRACT

Hydro-meteorological data are very indispensable for the assessment and development of water resources. However, most catchments in Ethiopia are ungauged and direct streamflow observations are not available at most sites for which rainfall–runoff relationships are required. Therefore, if there are no flow data from a catchment, a technique is needed for estimating the required design parameters, which do not require the availability of hydrological records. One option is to develop models for gauged catchments and link their parameters to physical characteristics, so that the approach can be applied to ungauged basins in the region, whose physical characteristics can be determined. Usually, one resorts to Synthetic Unit Hydrographs (SUHs) if there are no observed discharge hydrographs. This requires that the catchment characteristics be obtained or determined which are then used to adapt the SUH to suit a particular catchment.

In this research work coefficients required for the construction of synthetic unit hydrographs using Snyder's method for watersheds, which have areas in the range from 59.8km^2 to 2449km^2 in the upper Awash and Tekeze basins, have been determined. Moreover equations for estimating the lag time coefficient (C_t) and peak discharge (q_p) from watershed characteristics were also developed.

Thirty concurrent rainfall-runoff events from six gauged watersheds, four in the upper Tekeze and two in upper Awash, were used to estimate the coefficients and develop the equations. 27 rainfall-runoff events were used for calibration and three were used for verification. Due to the lack of getting watersheds with adequate rainfall and runoff records in Tekeze basin two watersheds that have almost similar hydrological characteristics were used to produce a reasonable and largely reliable estimate.

Lag time coefficient (C_t), peak discharge coefficient (C_p), unit hydrograph widths coefficients (C_w) at 50% and 75% of the peak and base time coefficient (C_b) were determined by calibrating Snyder's equations with the available rainfall- runoff data. The mathematical relations for lag time coefficient (C_t) and peak discharge (q_p) with watershed characteristics were established by using regression analyses.

Lag time coefficient (C_t) for the gauged watersheds range from 0.362 to 0.736 with mean value of 0.542 and a standard deviation of 0.157. The peak coefficients of the unit hydrographs of the gauged watershed range from 0.064 to 0.346 with mean value of 0.180 and a standard deviation of 0.112. Coefficients for the base time (C_B) and for the widths of unit hydrograph (C_w) at 50% and 75% of the peak discharge of the gauged catchments are found to be 1.006, 0.20 and 0.108 respectively these values are recommended to construct UH for an ungauged watershed in upper Awash and upper Tekeze basins.

Lag time is an essential input for Snyder's synthetic unit hydrograph model. Strong correlation with $R^2 = 0.90$ is seen between Lag time coefficient (C_t) and slope of a watershed. And hence the equation $C_t = 0.032 * S^{-0.597}$ is recommended to estimate lag time coefficient (C_t) for an ungauged watershed in the upper Awash and Upper Tekeze basins. In addition to this lag time can be estimated directly from the physical characteristics of the watershed by using the equation $t_p = 0.127 * (L L_c / S^{0.5})^{0.352}$. Strong correlation with $R^2 = 0.92$ is seen also between q_p (in m^3/s) and the variable $Z = A/t_1$ and hence the formula $q_p = 8.71 * 10^{-3} * (Z)^{1.78}$

Can be used to estimate the peak discharge.

Therefore, applying the equations and the coefficients give an estimate to the required UH characteristics, which might serve the intended purpose, as long as the hydrometeorological and physical characteristics of the watershed under consideration are within the range of those characteristics for the watersheds in this study.

Chapter One

Introduction

1.1 General

Water is one of the most fundamental substances to the existence of life and also the most prevalent in the earth-atmosphere system.

Knowledge of the general laws governing the distribution and movement of water-the water cycle- is of practical importance for rational use and protection of water resources. Fundamental questions such as “Where does it come from?” “How much water is available?” resulted in the evolution of the field of study called hydrology. Hydrology deals with the continuous circulation, distribution and interactions of water in the atmosphere, on the surface of the earth, and in the ground (Ponce, 1989).

On global scale the concept of water cycle is straightforward. The solar energy causes liquid water to evaporate from oceans, rivers, and lakes as water vapor, which then rises with warm air currents till it reaches the cooler layer of the atmosphere where it condenses. The condensed water droplets often adhere to dust particles and grow to a stage where the falling velocity exceeds the updraft. Depending on the air temperature and other climatic factors, the droplet will fall as rain, snow, sleet or hail. Precipitation that reaches the earth’s surface can either enter the ground via infiltration, contribute to surface streams and lakes as runoff, or return to the atmosphere as water vapor by evaporation. The surface runoff can either percolate in to the ground, or flow back to the sea and ocean, or evaporate to complete the hydrologic cycle.

Moving from the global to smaller spatial scale, e.g. continental, regional and local scales, the complexity and variability of hydrologic processes grow enormously. Even for some climatic regime, we notice the notorious variability of rainfall in time (season) and in space that causes its occurrence very difficult to predict. On the land surface, the timing and

amount of stream flow depends on the precipitation, air temperature, terrain features, vegetation cover, soil types and others factors.

The necessity to estimating river flows from measurable causative factors, principally rainfall, has perhaps provided the most important driving force in developing engineering hydrology. One of the central problems of engineering hydrology is to convert net rainfall into direct surface runoff. Because hydrology phenomena are extremely complex, and may never be fully understood. However, in the absence of perfect knowledge, they may be represented in a simplified way by means of the system concept (Chow, 1988). Hydrologic system analysis is therefore, required to study the system operation and to predict its output. Hydrologic modeling is a procedure in which one or more of the phases of the hydrologic cycle are represented by a simplified system. Developing a hydrologic system model, which is an approximation of the actual system, can do this; its input and outputs are measurable hydrologic variables and its structure is a set of equations linking the inputs and out puts. Although the physical principles underlying the specific hydrologic process should be considered in formulating a model structure, the final design invariably is only an approximation of the process being modeled. In spite of the approximation involved, models can often provide insight into those parts of the physical process in which knowledge of the underlying principle is deficit.

For example a unit hydrograph is a system transfer function that is used to transfer rainfall excess to direct runoff. The unit hydrograph is still recognized as one of the most important techniques in hydrology analysis related to surface runoff predictions. It serves as a basis from which runoff hydrographs can be constructed.

A unit hydrograph represents the basin outflow resulting from 1mm of direct runoff generated uniformly over the drainage area by a uniform rainfall rate during a specified period (Shaw, 1994). By using a unit hydrograph to estimate basin runoff, one assumes that the system is linear and time-invariant.

Although theoretical limitations to the method have been recognized for many years, the unit hydrograph method is still the design method of choice when time-dependent flood flow information is needed and the method is often recommended because of its relative simplicity.

Ethiopia and the Study Areas



Fig 1.1 Maps of Ethiopia and Location of Awash and Tekeze Basins

1.2 Problem Statement

Ethiopia is endowed with abundant water resources. There are 12 river basins, which carry quite a large amount of water annually. Annual surface water flow from these twelve river basins is about 111 billion m³ (IDCOF, 2002). Most of the river basins are suitable both for hydropower and irrigation developments. Yet, the country has not made the best out of the so-called “**white coal**”. This may be for various reasons, but it is known that water resource studies tend to be data intensive. One reason is that the physical systems involved are often large and complex and substantial quantities of data are required for their representation. Another reason is that the investigations themselves are complex, with a variety of interdependent computational elements (e.g. precipitation-runoff simulation, economic analysis, and system analysis e.t.c). The acquisition, processing and management of data required for water resource studies require a substantial amount of resources. Unfortunately, Ethiopia has no sufficient amount of resources to establish the required network for data acquisition, processing and management pertinent to water resource studies. As a result the rainfall-runoff data necessary to search for the existing or forecasting future conditions calibration parameters often are not available. Stream flow data may be missing, rainfall data may be sparse, or the available data may be unreliable, or rainfall-runoff data may not be available at all at some sites where water resources development are sought. For example data, required for deriving directly unit hydrographs that are used usually for determining design discharges for hydraulic structures, are not easy to find in Ethiopia. Therefore unit hydrographs are usually required to be determined synthetically.

This general problem background is the driving force for the inception of the idea of this research work. The research focuses on engineering hydrology to establishing rainfall-runoff relationship to develop synthetic unit hydrograph for catchments in the upper Awash and upper Tekeze basins.

Engineering hydrology seeks to establish relation defining the spatial, temporal, seasonal, annual, regional or geographical variability of water, with the aim of ascertaining societal risks involved in sizing hydraulic structures and systems (Ponce, 1989). To establish the

relationship between the various processes of the hydrologic cycle and to quantify its certain phase or phases, for instance surface runoff, measurements of most or some of the processes is required. Indeed all-hydrological models require input data series and many hydrological models also require output data series either for the purpose of model calibration or for the verification of model structure.

The engineer is dependant upon this information collected from many sources to seek models that represent the behavior of the hydrologic system. Moreover, the choice to develop a model depends on the available data (type and quality), the degree of accuracy required and the reliability of the model for application. In engineering hydrology, four types of mathematical models are in current use: (1) deterministic, (2) probabilistic, (3) conceptual, and (4) parametric (Ponce, 1989). The data requirement for a mathematical model can be divided into two parts.

1. Hydro meteorological data series.
2. Physical parameters describing the catchment

Basic measures such as catchment's area, land slope, vegetal cover, infiltration capacity, and permeability are widely used in hydrological modeling. It is apparent from this list that some parameters are easier to measure.

The collections of hydro meteorological data series require the setting up of a network of recording instruments. However, the network available in Ethiopia is too scant to allow adequate representation of the spatially variable hydrological processes to be modeled. Sufficient stream gauge and meteorological instruments are not available in all the basins of the country. The scarcity of this vital information poses a great difficulty in proper water resources evaluation and development, such as irrigation, water supply, hydropower, flood control etc.

If there are no data observed in a catchment, one has to use methods, which do not require the availability of a long time series of hydrological records. If among many similar and adjacent catchments, some are gauged, and others are not, one may establish regionally

valid relationship for some of the hydro meteorological and physiographic characteristics. One can develop models for gauged catchments and link their parameters to catchment's physical characteristics. Once this is done, the regional approach can be applied to ungauged basins, whose necessary physical characteristics can be determined. Usually synthetic unit hydrographs (SUHs) technique is sought if there are no observed discharge hydrographs. Synthetic unit- hydrograph methods are utilized to describe the entire unit hydrograph for ungauged watershed with only a few hydrograph parameters.

There are various types of synthetic unit hydrograph methods developed for ungauged catchments, such as USSCS (United State Soil Conservation Service), Snyder's synthetic unit hydrograph etc. However, these relations are quite empirical and as such cannot be expected to be universally applicable. In general their application should be restricted to the region in which they were derived. For example, Snyder (1938), based on study of catchments in the Appalachian Highlands of Easter United States develop a set of empirical equations for synthetic-unit hydrographs in those areas.

Therefore, it is very indispensable to develop regional relationships relating hydrological phenomena to physiographic basin characteristics. In this research work, it is intended to accomplish regional analysis to develop empirical equations applicable within upper Awash and upper Tekeze basins. This regional analysis is actually helpful to fill some of the gaps and could be used to alleviate the problem in getting appropriate and reliable model in water resources development and management.

1.2 Objective of the Study

The research work has the following general and specific objectives.

1. General Objective.

In water resources evaluation and development, the first step is the estimation of the quantity of water that can be made readily available or abstracted in the given catchment. This requires the identification or formulation of appropriate and reliable

model that can provide the necessary output using the available input data. Therefore, the general objective of the research work is to develop appropriate model that can be used to develop synthetic unit hydrograph for the ungauged catchments of upper Awash and Tekeze basins.

2. Specific Objectives.

- To develop regional Coefficients like C_t and C_p those are required for the synthesis of unit hydrograph for the ungauged catchments of upper Awash and Tekeze basins using Snyder's method.
- To develop regional predictive equations used to estimate some of the regional coefficients and some of the salient features of unit hydrograph.

1.3 Previous Studies

Snyder (1938) was perhaps the first to have established a set of formulas relating the salient unit hydrograph (UH) characteristics to the physical characteristics of the watershed. These formulas were based on the study of 20 watersheds located mainly in the Appalachian Highlands of the eastern United States, which varied in size from 30km^2 to $30,000\text{km}^2$ (Chew, 1988). The value C_t in Snyder's study ranged from 1.35 to 1.65. The value of the coefficient C_p ranged from 0.56 to 0.69.

Diversified regional studies on hydrology are not made in Ethiopia. Admasu Gebeyehu's study on regional flood frequency analysis (Admasu, 1989) is one of the studies that can be mentioned here. He has made a regional flood frequency analysis in Ethiopia based on flood index method (IM). The IM is based on a widely used regionalization procedure, which, has been adopted for Ethiopia by Admasu (NEDECO, 1998). The maximum flow is a function of catchment area (CA) and slope and a set of regionally determined factors.

No previous studies have been done in this specific area of regionalization in Ethiopia.

1.5. Organization of the thesis

The thesis is presented in eight chapters and eight appendices. The first chapter is an introduction, which gives background information on hydrologic processes and modeling, statement of the problem, objective of the research and previous studies related to the research. Chapter two and three contains brief description of the study area and hydro meteorological data respectively. Chapter four contains detail description of literature review of rainfall-runoff relationship, hydrologic models and their classifications, the concept of unit hydrograph and its derivation. Chapter five deal with the analyses of data for gauged catchments. In this chapter selection of gauged catchments, selection of rainfall-runoff events, separation of hydrographs, derivation of unit hydrograph and estimation of model parameters have been described. Chapter six contains the description used in model calibration and the comparison of coefficients. Chapter seven deal with regression analyses used to development predictive equations. It also contains the discussion in the selection of independent variables and the regression equations developed for estimating lag time, its coefficient and peak discharge. The recommendation and conclusions followed by references are described in chapter eight. The appendices contain some graphs, tables and computer out puts. It also contains data used in the research work.

Chapter Two

Description of the Study Area

2.1 Tekeze Basin

2.1.1 General

Tekeze river basin is located in the northern part of Ethiopia. The basin has an average elevation of 1850m asl and a catchment area of about 63,000km² (excluding tributaries like Angereb and Goang which enter the basin beyond the Ethio-sudanese border). The total area of the basin with Angereb and Goang which joining Tekeze River beyond the border is 86491km². The three rivers, Tekeze, Angereb, and Goang, are the main source for Atbara in the sudan. 12% of Nile flow is from Atbara, from which 9.5%, which corresponds with 80% of Atbara, is from Tekeze Basin. About 70% of the basin lies in the highlands at an altitude of over 1,500masl. Mountains ranges, the elevation of which is over 2000masl, surround the upper reaches of Tezeke (NEDECO, 1998).

The length of Tekeze River from its source at springs near lalibela down to the Sudanese border is more than 750km. The main tributaries of Tekeze, which originate in the highlands of the east side of the Semien Mountain, are Zamra, Tserare, Gheba, and Wori. Large tributaries originating on the east side of the Semien Mountains are the Insia and Zarema. The study is focused on some of these tributaries and their sub-tributaries (NEDECO, 1998).

2.1.2 Land Use and Land Cover

The land use condition in the upper Tekeze catchment includes mainly of cultivated agricultural land, grassland, forestland, and rural settlements. The cultivated land is sub-divided into intensively cultivated, and moderately cultivated. Practically all the land is opened up for cultivation and grazing. Over 70% of the land is under annual crops. Consequently the ground is virtually bare during the dry season. The main land use activities are rain fed cultivation of cereals, oil seeds and pulses and grazing on unimproved pasture and fallow (NEDECO, 1998).

2.1.2 Climate

According to the definition give by the World Meteorological Organization (WMO), Climate is defined as the synthesis of weather condition in a given area characterized by long-term statistics (mean, variance, probabilities of extremes, etc) of the meteorological elements in an area (NEDECO, 1998). The WMO usually accepts 30 years of statistical data series to define climate. The climate of the Tekeze basin in general, comes under the influence of the Inter Tropical Convergence Zone (ITCZ). The meteorological/climatic elements include precipitation, temperature, wind, radiation, humidity, and sunshine hours. The characteristics of some of the above elements important to the study are described below.

i. Precipitation

Precipitation data are an important input in water resources studies, in particular for the study of the rainfall-runoff process through which is determined how much runoff results from a certain amount of precipitation on different catchments. In Ethiopia one differentiates between two distinct precipitation types, the mono- modal and the bimodal type. Each type can again be subdivided into sub-types based on length of the rainy season. In the west of the basin a monomodal type regime can be recognized and a bimodal type in the east (east of 39⁰ 30'). The average annual precipitation decreases from south to north from 1200mm to 600mm. The study area is characterized by a Bimodal type I (quasi double maximum) rainfall pattern, with a small peak in April and a maximum peak in August. Therefore, the region is dominated by a semi – bimodal peak rainfall pattern (NEDECO, 1998). This pattern can be seen from the mean value of the monthly rainfall distribution shown in the Table 2.1 observed from Addis Ababa, Adwa, D/Zeit, and Mekele stations.

ii. Temperature.

In the lowland area (600m asl) average annual temperatures are above 26⁰c. In the largest part of the highlands the average annual temperature is around 22⁰c. Minimum mean

monthly temperature occurs in December-February period, ranging between 3 and 21⁰c while maximum mean monthly values occur in March-April, ranging between 19 and 43⁰c (NEDECO, 1998).

iii. Relative humidity, Sunshine hours, and wind speed.

Minimum mean monthly relative humidity values occur in the dry period (October-March) and can be as low as 40%. High values occur in the main rainy season (July-August) in which period mean monthly values for many stations are above 70%.

Mean monthly sunshine hours for the different stations range between 6.5 and 8.5hrs/day. High monthly values up to 10hrs/day occur during the dry period. Low values to below 4hrs/day occur during the rain season especially in the months of July and August.

Mean Monthly wind speeds up to 300km/day occur during the rain season (June-August). Low values occur during the dry season, some times below 100km/day (NEDECO, 1998).

2.2 Upper Awash Sub Basin

2.2.1 General

The upper Awash catchment is found in the highlands of central Ethiopia with all lands above 1500m asl. The land use condition in the upper Awash catchment includes mainly of cultivated agricultural land, forest land, rural and towns settlement. It is estimated that 67% is intensively cultivated, 25% is moderately cultivated, 5% is bush land or shrub land or wooded grassland, and 3% is urban area and alpine vegetation. The upper Awash River covers the river section from its source up to Koka Reservoir. The upper Awash River drains a catchment area close to 11,300km² and the length of the river up to Koka is around 220km (HALCROW, 1989).

The major tributaries to the upper Awash are Akaki and Mojo rivers. Akaki River starts from the mountainous areas of the northern part of Addis Ababa and join the main Awash River between Melka-Kunture and Melka-Hombple gauging stations. Mojo River, the other main tributary to Awash, originates from the high lands northeast of Addis Ababa. It drains

a catchment area close to 1,900km² and travels a total length of about 105km before joining Awash. Both the tributaries i.e. Akaki and Mojo, are considered in the studying addition to two gauging stations in the main river (HALCROW, 1989).

2.2.2 Land use, land cover, and soil type

The land use condition in the upper Awash catchment includes mainly of cultivated agricultural land, grassland, forestland, rural and towns settlements. It is estimated that 67% is intensively cultivated, 25% is moderately cultivated, 5% is bush land or shrub land or wooded grassland, and 3% is urban area and alpine vegetation. Strictly speaking, even the land use with in the upper Awash is diverse. In the upper most part where there is high rainfall, land use is complete in May with barley and teff. Steeper slopes are heavily wooded with natural acacia and much eucalyptus. On the lower most part, however, rainfall is too unreliable and the sparse dry acacia scrub gives way to wide stretches of bare ground with clumps of coarse grass and occasional thickets of acacia (Paulos, 1998).

The soil type in the upper Awash basin is diverse. The most common soil types are Clay, Sand, Clay-Loam, Silty-Clay-Loam, Sandy-Clay, and Silty-Clay (Paulos, 1998).

2.2.3 Climate

The climate of the upper Awash basin, in general, comes under the influence of the Inter Tropical Convergence Zone (ITCZ). This zone of low pressure makes the convergence of dry tropical easterlies and moist equatorial westerly. The explanation of the seasonal rainfall distribution within the basin lies in the annual migration of the ITCZ across the basin. The ITCZ starts its advance across the basin from the south in March, bringing the small or spring rains. In June and July the ITCZ reaches the its most northerly location beyond the basin which then experiences the heavy or summer rains throughout. The ITCZ returns southwards during August, September and October, restoring drier, easterly air-streams that prevail until the ITCZ resumes its northward migration in March (HALCROW, 1989).

i. Temperature

Temperature varies considerably over the basin and range from a mean annual temperature of 16.7⁰c at Addis Ababa to 29⁰c at Dubti. The mean annual temperature range in the basin is around 15⁰c. The temperature at Addis Ababa ranges from a mean monthly maximum of 22.5⁰c to a mean monthly minimum of 9.6⁰c (HALCROW, 1989).

ii. Wind speed, sunshine and Relative humidity

The wind flow pattern is influenced by the seasonal variation of the ITCZ. The predominant wind direction during June to September is southerly to southwesterly. The wind speed pattern is distinctly bimodal in Addis Ababa region with peaks occurring in March and September and minimum speeds being recorded in July and August. The mean annual wind speed at Addis Ababa is 0.9m/s. The Awash basin experiences 2700hours of sunshine annually. The monthly variation closely follows the rainfall pattern as would be expected with more sunshine hours in the dry months than in the wet months. Sunshine hours vary from a daily mean of 9.4hours in December to 3 hours in July at Addis Ababa. The mean annual relative humidity of the basin is 60.2% measured at Addis Ababa. The monthly variation in relative humidity at Addis Ababa ranges from 50.9% in March to 78.5% in August.

Table 2.1: - Mean value of monthly rainfall distribution for some stations in the study areas.

Station	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Total
A/A	15.4	27.9	52.7	66.9	41.1	73.2	305.5	291.5	105.3	19.7	11.0	4.8	1015
D/zeit	11.2	41.1	37.2	70.3	60.9	91.6	242.7	245.9	119.0	18.0	8.7	4.1	951.0
Adwa	0.0	5.3	26.4	27.6	23.7	76.3	209.7	269.5	158.0	46.7	64	1.2	560.1
Mekele	3.3	7.9	24.8	40.5	33.6	28.3	209.5	210.9	30.9	5.2	4.9	1.1	587.7

Chapter Three

Hydro-Meteorological Data

3.1 Meteorological Data

The National Meteorological Service Agency (NMSA) is the responsible organization for the collection and issuing of meteorological data. One of the most important meteorological data for this study is the rainfall taken from continuous automatic recorder. There are a number of rainfall stations distributed over the upper Awash and Tekeze basins. Most of the stations are non-recording type and few are recording type.

In the Upper Awash basin only stations at Akaki, Mojo, and Addis Ababa at bole and Teklehaimnot are recording type. Besides the limitations in the number of the recording type of stations their spatial distribution is no fairly uniform over the basin. They are concentrated in the northeastern part of the basin. However, the Akaki and Mojo catchments have recording stations that are judged to represent the catchment adequately. These stations are found at Addis Ababa and Debreziet respectively.

In the Upper Tekeze Only stations found at Mekele, Adwa, Senkata, and Adigrat that are pertinent to the study are recording type. Among these stations only Mekele and Adwa are currently found to provide the required data. The rest are not because they are installed recently and have few years of data not more that two years. Even with in these few years of record the unfortunate case is the absence of flood record in the rivers, which can be related to these stations. Besides, the limitations in the number of the stations, the record in the available stations was intermittent especially for the year 1999.

List of the stations and their locations is shown in Table 3.1 and Figures 3.1 and 3.2.

3.2 Hydrological Data Collection and processing

The hydrology department of the ministry of water resources (MWR) is the responsible department for the collection of hydrological data in Ethiopia. Up on an official request to the water resource ministry, hydrological data pertinent to the study have been collected for both the basins. The hydrological data required for this study is the stream flood record taken from automatic continuous recorder. In both the basins streams with water level recorder have been selected and their continuous charts have been collected from the hydrology department of the ministry of water resource for further analysis.

In the upper Awash basin Melka hombole, Melka Kunture, Mojo, and Akaki stations have automatic water level recorder. Out of these stations only Mojo and Akaki are most important to this study because the size of their catchment area is below from the usual recommended upper limit i.e. 5,000km² for unit hydrograph derivation and analysis.

In the Upper Tekeze basin Gheba, Illala, Mariamshewito, Suluh, Zarima, Zamra, Wori, and Genfel are the rivers with automatic water level recorder. But out of these rivers only Gheba near Mekele, Illala, Mariam shewito, and Suluh are found to be important to this research work. Because some of the remaining rivers like Genfel and Zamra have no proper instrument calibration information the other like Zarima has no rainfall recorder with in or near to the watershed.

For all the selected rivers in both the basins hydrographs appropriate for this study have been selected from various years charts and discretized. The selected hydrographs are plots of water level (stage) vs time. Processing of these data involves discretization of the hydrograph and conversion of the stages to discharges using the appropriate rating equations. Hydrology department has developed rating curves/equations for each station. The equations have the general form:

$$Q = a * (H+C)^b \quad (3.1)$$

Where Q is the discharge in m³/s, H is the stage in m and a, b, and c are coefficients.

Some of these equations for the selected watershed are shown in table 3.1. The corresponding rating curves for some of the watersheds is also shown in appendix F.

The Hydrology department has developed rating curves manually using logarithmic paper. This would reduce the accuracy and quality of the curves. The problem can be seen clearly looking at the rating curves in appendix F. It would have been good if the plots were made using computer.

Table: - 3.1 Some of the equations relating stage and discharge

Watershed	Rating curve Equation
Mariam Shewito	$Q = 23.68(h-0.28)^{3.116}$
Illala	$Q = 415.7(h-0.06)^{3.274}$ for $h \leq 0.16\text{m}$ $Q = 41.99(h-0.06)^{2.279}$ for $h > 0.06$
Suluh	$Q = 11.9(h-0.66)^{1.786}$
Akaki	$Q = 20.98(h-0.37)^{1.54}$
Mojo	$Q = 4.64(h-0.10)^{5.168}$ for $h \leq 0.92\text{m}$ $Q = 25.24(h-0.45)^{2.089}$ for $h > 0.92\text{m}$
Gheba	$Q = 7.80 (h-0.70)^{3.033}$

One of the main problems in the collection and processing of time dependent flow data was the absence of sufficient chart with the necessary details and rating curve. Especially the problem was critical in Tekeze basin. The study is done with those rivers which have charts with the necessary calibration information and corresponding rating curves. List of the selected rivers and their watershed characteristics is shown in appendix A.

The location of the gauging stations for both Awash and Tekeze basins is shown the Figures 3.1 and 3.2 respectively.

Table: - 3.2 List of some of the meteorological stations with their coordinate, altitude and mean annual rainfall in the study areas.

Station Name	Latitude	Longitude	Altitude	Mean annual rainfall
Adwa	14 ⁰ ,10'	38 ⁰ ,50'	1905	800
Mekele	13 ⁰ ,30'	39 ⁰ ,29'	2070	562
Senkata	13 ⁰ ,57'	39 ⁰ ,36'	2400	580
Hawzen	14 ⁰ ,01'	39 ⁰ ,27'	2280	600
Wukro	13 ⁰ ,50'	39 ⁰ ,40'	1980	530
Addis Ababa	9 ⁰ ,02'	38 ⁰ ,45'	2400	1171
D/Zeit	8 ⁰ ,55'	38 ⁰ ,58'	1955	951
Mojo	8 ⁰ ,37'	39 ⁰ ,08'	1650	923
Melka kunturie	8 ⁰ ,11'	38 ⁰ ,35'	2000	874

Source: (Meteorological Map of Ethiopia, 1979, HALCROW, 1989 and NEDECO 1998)

Selected Gauge Stations in UpperTekeze basin

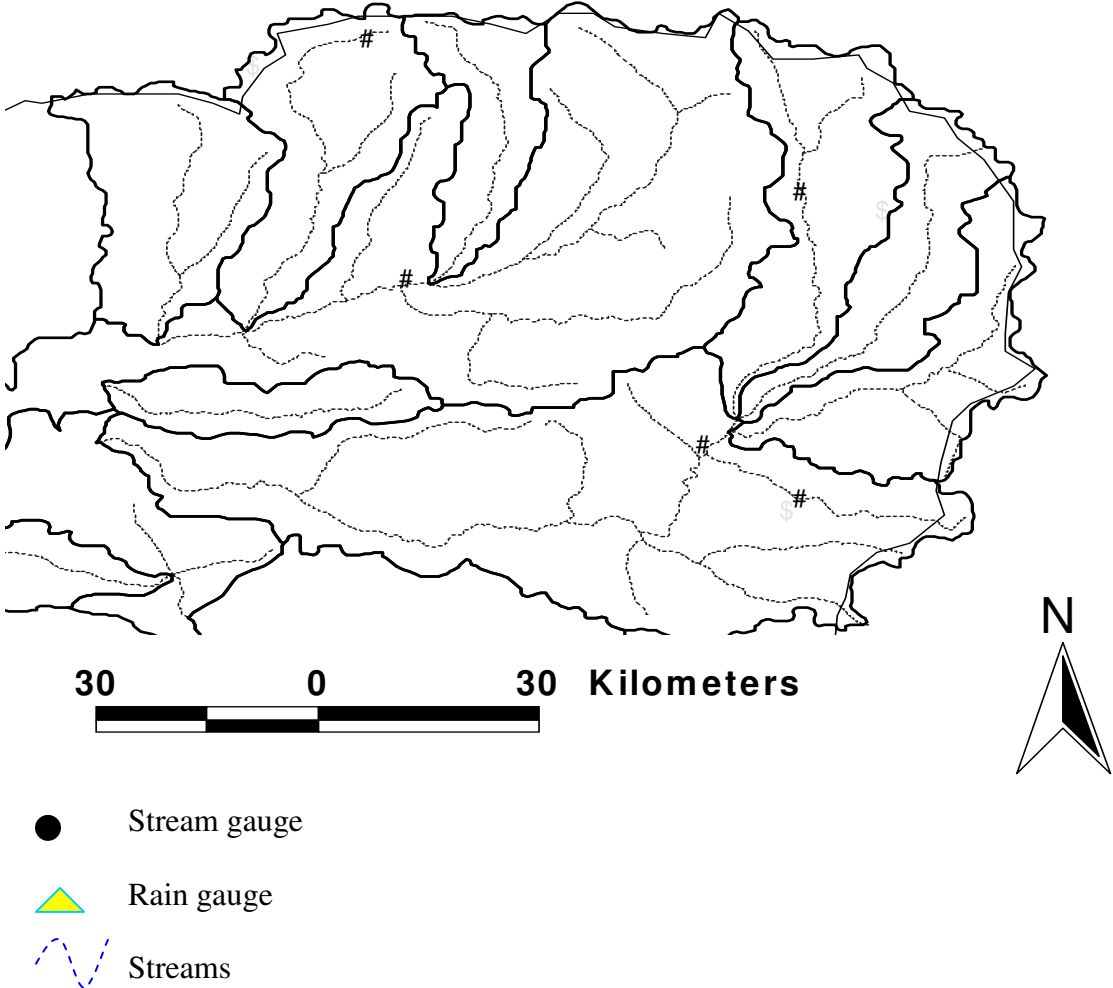


Fig 3.1 Selected Stream Gauging station in Tekeze Basin

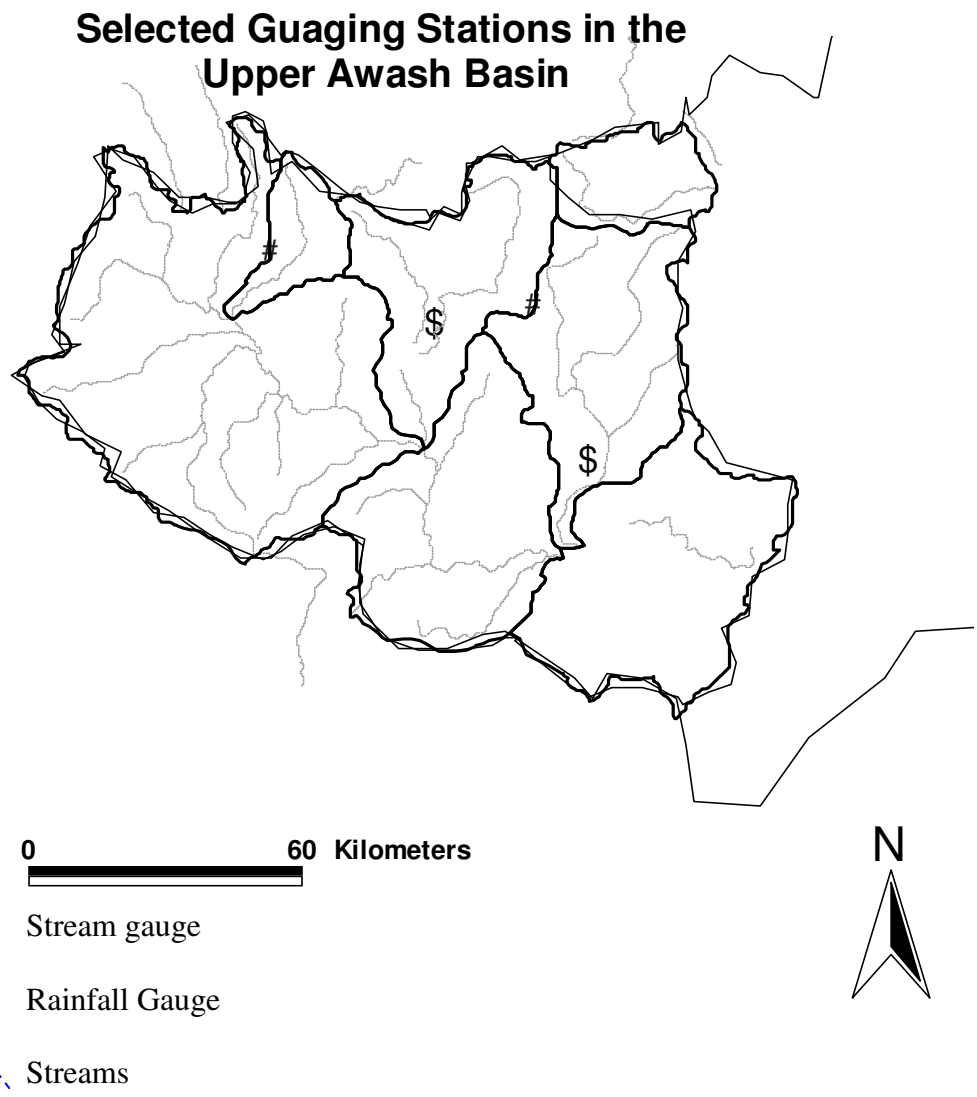


Fig 3.2 Selected hydro meteorological stations in upper Awash Basin

Chapter Four

Literature Review

4.1 Rainfall Runoff Relationship

4.1.1 General

Flood runoff has often been considered to consist of surface runoff produced at the ground surface when the rainfall intensity exceeds the infiltration capacity. While this process, known as Hortonian overland flow, occurs in many situations, two other general storm runoff processes i.e. **Saturated overland flow** and **Throughflow** are now recognized, as a result of observations on natural basins during storm periods and many detailed studies of instrumented plots and small areas (Maidment, 1993). All these processes are discussed in the following paragraphs.

Saturated overland flow occurs when, one part of the drainage basin the surface horizon of the soil becomes saturated as a result of either the buildup of a saturated zone above a soil horizon of lower hydraulic conductivity or the rise of a shallow water table to the surface (Maidment, 1993).

Through flow is water that infiltrates into the soil and percolates rapidly, largely through macro pores such as cracks and root and animal holes, and then moves laterally in a temporarily saturated zone, often above a layer of low hydraulic conductivity. It reaches the stream channel quickly and differs from other subsurface flow by the rapidity of its response and its relatively large magnitude (Maidment, 1993).

Runoff processes operating at any location vary from time to time. Large variations in hydrologic characteristics, and therefore in runoff processes, also occur over small apparently homogeneous areas to the extent that all three runoff processes discussed above may occur during a single storm runoff event. Associated with the recognition of several processes producing storm runoff has been the concept that storm runoff may be generated from only a small part of many drainage basins. In addition, this source area may vary in extent in different seasons and during the progress of a storm.

The type of runoff process and the location of source areas, whether close to the outlet and adjacent to stream channels or on the ridges remote from the channels, has considerable influence on the resulting hydrographs. However, practical methods for estimating storm losses and runoff have not yet been developed to explicitly account for these differences (Maidment, 1993). Uniform or average conditions, at least over subareas, are generally assumed. These new insights into physical processes do, however, give some guidance in assessing the validity and applicability of the practical methods.

4.1.2 Description of Hortonian overland flow Process.

When rain falls, the leaves and stems of the vegetation intercept the first drops of water. This is usually referred to as interception storage.

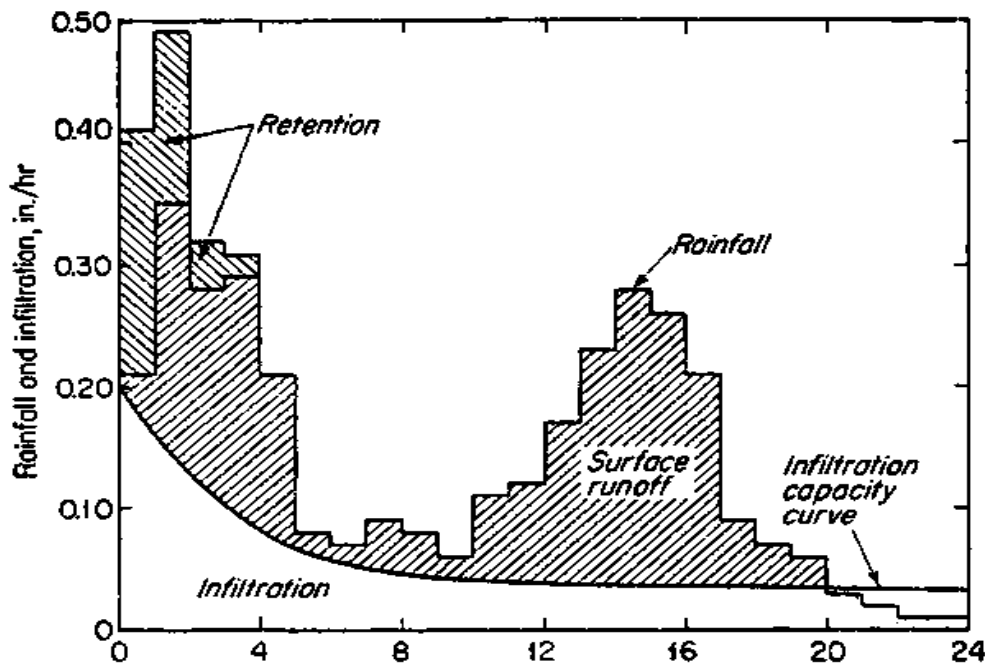


Fig: 4.1 Schematic diagram illustrating relationship between rainfall, infiltration and runoff. (Source: Linsley et al. 1988)

As the rain continues, water reaching the ground surface infiltrates into the soil until it reaches a stage where the rate of rainfall (intensity) exceeds the infiltration capacity of the

soil. Thereafter, surface puddles, ditches, and other depressions are filled (depression storage), after which runoff is generated.

The infiltration capacity of the soil depends on its texture and structure, as well as on the antecedent soil moisture content (previous rainfall or dry season). The initial capacity (of a dry soil) is high but, as the storm continues, it decreases until it reaches a steady value termed as final infiltration rate. The rainfall-runoff relationship is shown in the Fig 4.1 above.

The process of runoff generation continues as long as the rainfall intensity exceeds the actual infiltration capacity of the soil but it stops as soon as the rate of rainfall drops below the actual rate of infiltration.

The rainfall runoff process is well described in many literatures. Numerous papers on the subject have been published and many computer simulation models have been developed. All these models, however, require detailed knowledge of a number of factors and initial boundary conditions in a catchment area, which in most cases are not readily available.

For a better understanding of the difficulties of accurately predicting the amount of runoff resulting from a rainfall event, the major factors, which influence the rainfall-runoff process, are described below.

4.1.3 Factors affecting runoff

Apart from rainfall characteristics such as intensity, duration and distribution, there are a number of site (or catchment) specific factors, which have a direct bearing on the occurrence, and volume of runoff.

i. Soil type

The infiltration capacity is dependent on the porosity of a soil, which determines the water storage capacity and affects the resistance of water to flow into deeper layers.

Porosity differs from one soil type to the other. The highest infiltration capacities are observed in loose, sandy soils while heavy clay or loamy soils have considerable smaller infiltration capacities. The infiltration capacity depends furthermore on the moisture content prevailing in a soil at the onset of a rainstorm. The initial high capacity decreases with time (provided the rain does not stop) until it reaches a constant value as the soil profile becomes saturated.

ii. Vegetation

The amount of rain lost to interception storage on the foliage depends on the kind of vegetation and its growth stage. More significant is the effect the vegetation has on the infiltration capacity of the soil. The root systems as well as organic matter in the soil increase the soil porosity thus allowing more water to infiltrate. Vegetation also retards the surface flow particularly on gentle slopes, giving the water more time to infiltrate and to evaporate.

In conclusion, an area densely covered with vegetation, yields less runoff than bare ground.

iii. Slope and catchment size

Investigations on experimental runoff plots have shown that steep slope plots yield more runoff than those with gentle slopes.

In addition, it was observed that the quantity of runoff decreased with increasing slope length.

This is mainly due to lower flow velocities and subsequently a longer time of concentration (defined as the time needed for a drop of water to reach the outlet of a catchment from the most remote location in the catchment) (Ponce, 1989). This means that the water is exposed for a longer duration to infiltration and evaporation before it reaches the measuring point. The same applies when catchment areas of different sizes are compared.

The runoff efficiency (volume of runoff per unit of area) increases with the decreasing size of the catchment i.e. the larger the size of the catchment the larger the time of concentration and the smaller the runoff efficiency.

4.2 Classification of Hydrological Models

As it is tried to explain in chapter one hydrology phenomena are extremely complex, and may never be fully understood. However, in the absence of perfect knowledge, they may be represented in a simplified way by means of the system concept (Chow, 1988). Hydrologic system analysis is therefore, required to study the system operation and to predict its output. Developing a hydrologic system model, which is an approximation of the actual system, can do this; its input and outputs are measurable hydrologic variables and its structure is a set of equations linking the inputs and out puts.

This may be represented by

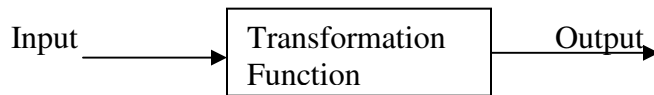


Fig: 4.2. Conceptual representation of the system process

For example a unit hydrograph is a system transfer function that is used to transfer rainfall excess to direct runoff.

The variable may be a function of space and time, and they may also be probabilistic or random variables, which do not have a fixed value at a particular point in space and time, instead are described by probability distributions.

Hydrologic models can be grouped into two general categories: (1) material and (2) formal (Chow, 1988). A material model is a physical representation of the prototype, simpler in structure but with properties resembling those of the prototype. Examples of material models are rainfall simulators and experimental watersheds.

A formal model is a mathematical abstraction of an idealize situation that preserve the important structural properties of the prototype. Since formal models are invariable mathematical in nature, it is customary to refer to them as mathematical models. Many computers catchment models have been developed over the last three decays.

Material models are expensive and of limited applicability. Conversely, formal models are readily available, highly flexible, and comparatively inexpensive to use.

Hydrologic Catchment Models can be:

1. Linear or Nonlinear
2. Time invariant or Time variant
3. Lumped or Distributed
4. Continuous or Discrete
5. Event driven or Continuous process

As mentioned in chapter one, four general types of mathematical models are commonly recognized in catchments modeling practice. These are deterministic, probabilistic, Conceptual, and parametric.

Deterministic models are formulated by following laws of physical and/or chemical processes as described by differential equations. Ideally, a deterministic model should be able to provide the best detail in simulation of physical or chemical processes. In practice, however, the application of deterministic model is often hindered by model's (or modeler's) inability to resolve the temporal and spatial variability of natural phenomena into sufficient small increments (Ponce, 1989).

Probabilistic models are exactly opposite in meaning to deterministic models. A probabilistic model is formulated by following laws of chance or probability. The development of statistical models invariably requires the use of data.

Conceptual models are simplified representations of the physical processes; usually rely on mathematical descriptions (either in algebraic form or by ordinary differential equations), which simulate complex processes in the mean by relying on a few key conceptual parameters.

Parametric models are the simplest of all modeling approaches. The emphasis of parametric model is placed upon the key empirical parameter(s) on which the solution is based. Usually, a parametric model consists of an algebraic equation containing one or more parameters to be determined by data analysis or other empirical means. The applicability of parametric models is restricted to the range of data used in the determination of parameter values. Parametric models are useful where conceptual, deterministic or probabilistic models are judged to be either impractical or too expensive. Regional analysis is a typical example of the parametric approach to hydrologic catchment modeling. In this case, statistical regression techniques are used to develop predictive equations having regional applicability.

4.3 General Concept of Unit Hydrograph

4.3.1 General

Since one of the characteristics of rainfall is its temporal variation, catchment's runoff response shall be described by methods that take explicit account of the temporal variation of rainfall. The most widely used method to accomplish this is the unit hydrograph technique. The unit hydrograph is a simple linear model that can be used to derive the hydrograph resulting from any amount of excess rainfall. It is a means of manipulating a known volume of run-off from a basin to represent the timing when the volume arrives at the basin outlet. Thus, the unit hydrograph is defined as the hydrograph of direct runoff resulting from effective rainfall falling in a unit of time such as 1hr and produced uniformly in space and time over the total catchment area (Shaw, 1994).

For example unit hydrograph resulting from uniform effective rainfall rate over the entire basin for a specified period of rainfall duration such as 1hr is show in the Figure 4.3.

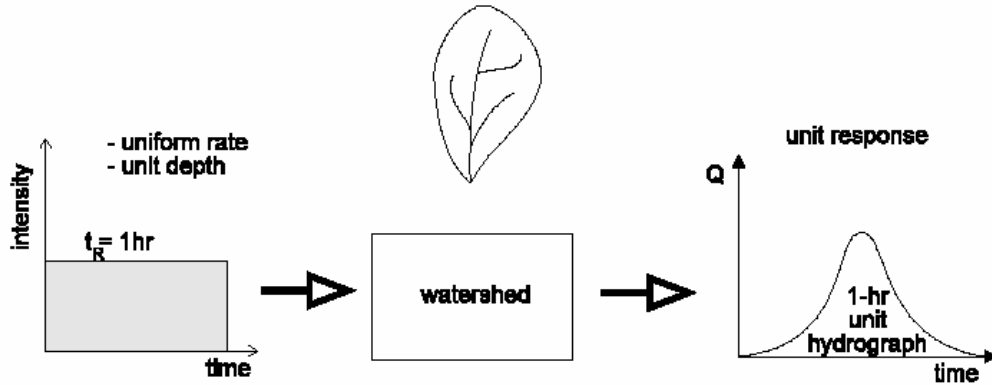


Fig: 4.3 showing unit response of a catchment.

Basin characteristics are reflected in the hydrograph shape. A typical idealized shape with its features is shown in the following Figure 4.4

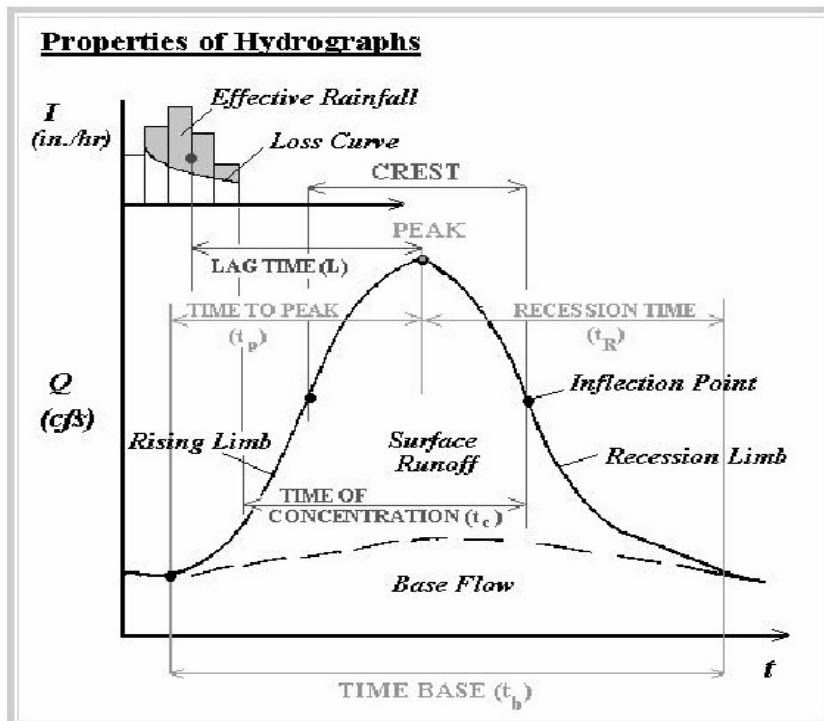


Fig: 4.4 showing some of the properties of an idealized hydrograph shape. (source: U.S Army Corps Of Engineers)

In catchment modeling analyzing basic features of hydrograph is paramount important. The purpose of hydrograph analysis is to analyze measured rainfall and runoff data to obtain an estimate of the transfer function like unit hydrograph.

The unit hydrograph method makes several assumptions that give it simple properties assisting in its application.

4.3.2 Assumptions

Unit hydrograph procedure rests on the following assumptions (Chow, 1988).

1. The excess rainfall has a constant intensity with in the effective duration.
2. The excess rainfall is uniformly distributed throughout the whole drainage area.
3. The base time of the DRH (the direct runoff hydrograph) resulting from an excess rainfall of given duration is constant.
4. The ordinates of all DRH's of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph. This is the principle of superposition and proportionality.
5. For a give watershed, the hydrograph resulting from a given excess rainfall reflects the unchanging characteristics of the watershed. This is the principle of time invariance. Using this assumption of invariance, once a TUH has been derived for a catchment area, it could be used to represent the response of the catchment whenever required.

The unit hydrograph approach can yield questionable results when the above assumptions are not fully met. Under natural condition the above assumptions cannot be perfectly satisfied. For example, the areal distribution of rainfall within a storm is very rarely uniform. However, when the hydrological data to be used are carefully selected so that they come close to meeting the above assumptions. For example, the unit hydrograph may become inapplicable when the drainage area is too large to be covered by a nearly uniform distribution of rainfall. In such cases, the area has to be divided and each sub area analyzed for storms covering the whole sub area.

The unit hydrograph method has the advantage of great simplicity. Once a unit hydrograph of specified duration T has been derived for a catchment area, then for any sequence of effective rainfall in period of T an estimate of the direct runoff can be obtained by adopting the assumptions and applying the simple properties outlined above.

Since a unit hydrograph has meaning only in connection with a given storm duration, it follows that a catchment can have several unit hydrographs, each for a different rainfall

duration. Once a unit hydrograph for a given duration has been determined other unit hydrographs can be derived from it by using one of the following methods.

- Superposition and
- S-hydrograph

4.4 Derivation of Unit Hydrograph

Unit hydrograph for a given catchment can be developed either

- (1) . Directly, by using rainfall-runoff data for selected events or,
- (2) . Indirectly, by using a synthetic unit hydrograph formula.

4.4.1 Development of unit hydrographs: Direct method

To develop unit hydrograph by the direct method it is necessary to have a gauged catchment, i.e., a catchment equipped with rain gages and a stream gage at the outlet, and adequate sets of corresponding rainfall-runoff data.

The rainfall-runoff records should be screened to identify storms suitable for unit hydrograph analysis. Ideally, a storm should have a clearly defined duration; storms should be of uniform rainfall intensity both temporally and spatially. In practice, the difficulty in meeting this latter requirement increases with catchment size. For small or medium sized catchments say up to 500km² a significant rainfall event may extend over the whole area. The upper limit of applicability of the unit hydrograph is not very well defined. Sherman used it in connection with basins varying from 1300 to 8000km². Linsley mention an upper limit of 5000km² in order to preserve accuracy (Subramarya, 1984).

i. Data Requirements

Data collection preparatory to deriving a unit hydrograph for a gaged watershed can be extremely time consuming. To develop a unit hydrograph, it is desirable to have as many rainfall records as possible within the study area to insure that the amount and distribution of rainfall over the watershed are accurately known. A selection of typical single- peaked

hydrograph is taken from the continuous river gauge records and the corresponding rainfall records are taken from automatic recorder.

ii. Estimation of Parameters

The following is the basic procedure for estimating the ordinates of the unit hydrograph at a gauged site. This method yields a series of the-discharge ordinates that defined the unit graph for a specified duration.

- a. For the chosen flood event, determine the duration of effective rainfall. This will become the unit hydrograph duration.
- b. On the recorded hydrograph, separate the base flow from the direct runoff. This can be accomplished by applying one of the base flow separation techniques among the various available.
- c. Determine the total rainfall excess by calculating the volume of direct runoff. Sum the ordinates and multiply each by the unit hydrograph duration to obtain the direct runoff volume. Convert to depth by dividing by the contributing area.
- d. Determine the unit hydrograph ordinates by dividing each ordinate in the observed direct runoff hydrograph by the excess rainfall or by applying deconvolution technique for the case of composite rainfall excess.

4.4.2 Development of unit hydrographs: Indirect method

In the absence of rainfall-runoff data, unit hydrograph can be derived by synthetic means. A synthetic unit hydrograph is a unit hydrograph derived following an established formula, without the need for rainfall-runoff data analysis. Synthetic unit- hydrograph methods are utilized to describe the entire unit hydrograph for a gauged watershed with only a few hydrograph parameters. Needed hydrograph parameters vary among the different synthetic unit-hydrograph methods. These hydrograph parameters can be related to the characteristics of the watershed and the storm from which the parameters were determined. This method can be applied to ungauged watersheds with geomorphology, soils, land cover/land use, and climate similarly to the gauged watersheds. A number of synthetic unit-hydrograph approaches are available, but the ones to have found widespread use are those listed below.

- i. Those relating hydrograph characteristics to watershed characteristic (Snyder, 1938)
- ii. Those based on a dimensionless unit hydrograph (SCS; 1972)
- iii. Those based on models of watershed storage (Clark, 1943).

i. The SCS dimensionless synthetic hydrograph

U.S. Soil Conservation Service developed SCS method in 1957. It is based on dimensionless unit hydrograph, which is developed from a large number of unit hydrographs obtained from basins ranging in size and from different geographic locations. In the SCS dimensionless synthetic unit hydrograph the discharge is expressed by the ratio of discharge q to peak discharge q_p and the time by the ratio of time t to the time of rise of the unit hydrograph, t_p . Given the peak discharge and lag time for the duration of excess rainfall, the unit hydrograph can be estimated from the synthetic dimensionless hydrograph for the given basin. Figure 4.5 shows such a dimensionless hydrograph, prepared from the unit hydrographs of a variety of watersheds. In this method the values of q_p and t_p may be estimated using a simplified model of a triangular unit hydrograph as shown in Figure 4.6 with peak flow Q_p (cfs), time to peak t_p (hr), and time of fall B (hr) for a rainfall duration t_r (hr), mainstream length (L in ft) and average slope of basin (y in %).

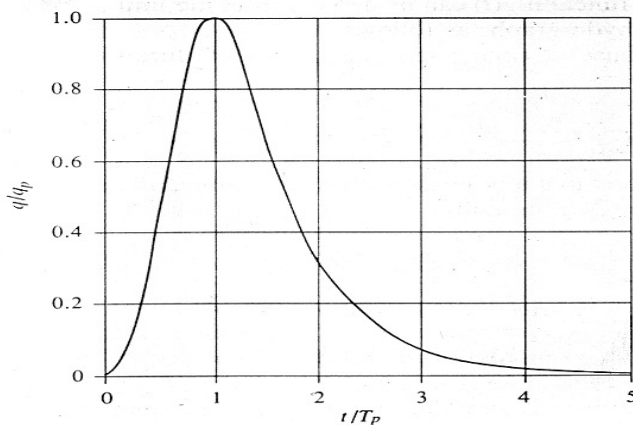


Figure: - 4.5 Dimensionless Unit hydrograph (Source: Chow, 1988)

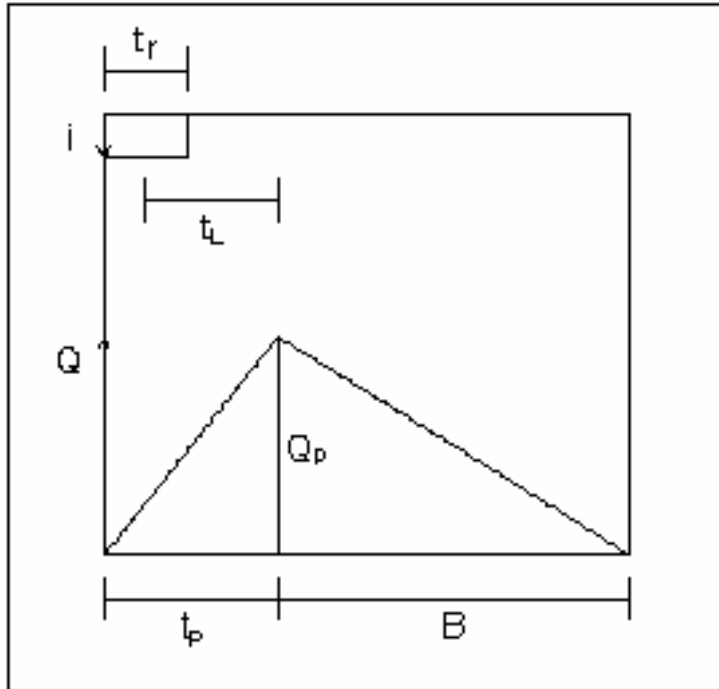


Figure 4.5 Triangular Unit Hydrograph.

From the review of a large number of hydrographs, it was found that for one inch of rainfall excess;

$$Q_p = \frac{484 A}{t_p} \quad (4.1)$$

$$B = 1.67t_p \quad (4.2)$$

$$t_p = (t_r/2) + t_l \quad (4.3)$$

$$T_b = t_p + B \quad (4.4)$$

Where Q_p is peak flow in (cfs)

t_p is time to peak in (hr)

B is time of fall in (hr)

t_r is rainfall duration in (hr), L is mainstream length in (ft) and y is average slope of basin in (%).

Lag time, t_L , is the time from the centroid of rainfall to the occurrence of peak flow and it is estimated from the following equation:

$$t_L = \frac{L^{0.8} (S+1)^{0.7}}{1900 y^{0.5}} \quad (4.5)$$

Where; the parameters are as defined above in the text, with the followings,

$$S = 1000/CN - 10 \quad (4.6)$$

In which CN = Curve number for various soil/land use combination and S, 1000, and 10 are given in inches.

The determination of the curve number (CN), which is a function of soil and land use characteristics of the basin, is essential for this method. In addition to these two characteristics, some other hydrologic conditions of the basin such as the vegetal cover and antecedent moisture situation are important factors in estimation of CN.

In order to determine the representative CN value of a basin, basin area has to be divided into sub-areas, which have same land use and soil type characteristics. Then CN values for every sub-area are determined using appropriate tables. After determination of CNs for particular sub-areas, weighted average of the CN values with respect to their areas will give the representative CN value for the whole basin. In addition to the information necessary for the determination of curve number, other information like area, mainstream length and average slope of the basin are also required for SCS method.

ii. The Clark unit hydrograph

The processes of translation and attenuation dominate the movement of flow through a watershed. Translation is the movement of flow down gradient through the watershed in response to gravity. Attenuation results from the frictional forces and channel-storage effects that resist the flow. Clark (1945) noted that the translation of flow throughout the watershed could be described by a time-area curve, which expresses the curve of the

fraction of watershed area contributing runoff to the watershed outlet as a function of time since the start of effective precipitation. The watershed T_c bound the time-area curve in time. Thus, T_c is a hydrograph parameter of the Clark unit-hydrograph method. Attenuation of flow can be represented with a simple, linear reservoir for which storage is related to outflow as (U.S Geologic survey, 2000):

$$S=R*O \tag{4.7}$$

Where S is the watershed storage,
 R is the watershed-storage coefficient, and
 O is the outflow from the watershed.

Therefore, Clark (1945) proposed that a synthetic unit hydrograph could be obtained by routing 1 unit of direct runoff to the channel in proportion to the time-area curve and routing the runoff entering the channel through a linear reservoir.

The T_c for the Clark unit hydrograph is slightly different than the typical definition applied in storm water management, such as that in the rational method (U.S Geologic survey, 2000). In the typical definition, the time of concentration (t_c) is the travel time for the first drop of effective precipitation at the hydraulically most distant point in the watershed to reach the watershed outlet. In the Clark unit-hydrograph method, T_c is the time from the end of effective precipitation to the inflection point of the recession limb of the runoff hydrograph. The inflection point on the runoff hydrograph corresponds to the time when overland flow to the channel network ceases and beyond that time the measured runoff results from drainage of channel storage. Therefore, Clark's T_c is the travel time required for last drop of effective precipitation at the hydraulically most distant point in the watershed to reach the channel network.

Methods and equations have been developed that relate T_c and R to watershed characteristics. The relation among (T_c+R) , the main channel length, and main-channel slope was determined as

$$(T_c+R) = 35.2*L^{0.39}*S^{-0.78}$$

(4.7)

Where

L is the stream length measured along the main channel from the watershed outlet to the watershed divide, in mi, and

S is the main-channel slope determined from elevations at points that represent 10 and 85 percent of the distance along the channel from the watershed outlet to the watershed divide, in ft/mi.

The equations for estimating T_c and R , in hours, as a function of watershed and storm characteristics

$$T_c = 39.1*A^{0.577} (I+1)^{-1.146} D^{0.781}$$

(4.8)

$$R = 123*A^{0.390}*(I+1)^{-0.722}S^{-0.303}$$

(4.9)

Where

I is the percentage of impervious cover:

A is the watershed area, in mi^2 : and

D is the effective precipitation depth, in inches.

iii. The Snyder unit hydrograph

The synthetic unit hydrograph of Snyder (1938) is based on relationships found between three characteristics of a *standard unit hydrograph* and descriptors of basin morphology. The hydrograph characteristics are the effective rainfall duration, t_r , the peak direct runoff rate, q_p , and the basin lag time, t_l . From these relationships, five characteristics of a *required unit hydrograph* for a given effective rainfall duration may be calculated (Chow, 1988): the peak discharge per unit of watershed area, q_{pR} , the basin lag, t_{lR} , the base time, t_b ,

and the widths, W (in time units) of the unit hydrograph at 50 and 75 percent of the peak discharge.

Standard unit hydrograph: - A standard unit hydrograph is associated with Specific effective rainfall duration, t_r , defined by the following relationship with basin lag, t_l ,

$$t_l = 5.5t_r \quad (4.10)$$

For a standard unit hydrograph the basin lag, t_l , and the peak discharge, q_p , are given by,

$$t_l = C_1 C_t (LL_c)^{0.3} \quad (4.11)$$

$$q_p = \frac{C_2 C_p A}{t_l} \quad (4.12)$$

The basin lag time of the standard unit hydrograph (equation 4.11) is in hours, L is The length of the main stream in kilometers (miles) from the outlet to the upstream divide, L_c is the distance in kilometers (miles) from the outlet to a point on the stream nearest the centroid of the watershed area, and $C_l = 0.75$ (1.0 for English units). The product LL_c is a measure of watershed shape. C_t is a coefficient derived from gauged watersheds in the same region, and represents variations in watershed slopes and storage characteristics.

The peak discharge of the standard unit hydrograph (equation 4.12) is in m^3/s (*cfs*), A is the basin area in km^2 (mi^2), and $C_2 = 2.75$ (640 for English units). As C_t , C_p is a coefficient derived from gauged watersheds in the area, and represents the effects of retention and Storage.

Estimation of Model Parameters C_p and C_t : - As in any model parameter estimation problem, observations of the input (*i.e.*, effective precipitation) and the output (*i.e.*, direct runoff hydrograph) must be available. In addition, the values of L and L_c must also be available (e.g., from surveys, maps, etc.). From the concurrent input-output observations, a unit hydrograph for the basin in question, a so-called *derived unit hydrograph*, can be

developed. From the derived unit hydrograph of the watershed, values of its associated effective duration t_R in hours, its basin lag t_{IR} in hours, and its peak discharge q_{pR} in m^3/s are obtained. If $t_{IR} = 5.5t_R$, then the derived unit hydrograph is a standard unit hydrograph and $t_r = t_R$, $t_l = t_{IR}$, and $q_p = q_{pR}$, and C_t and C_p are computed by the equations for t_l and q_p given above (equations 4.11 and 4.12), corresponding to the standard unit hydrograph.

If t_{IR} is quite different from $5.5t_R$, the basin lag of the standard unit hydrograph for the basin is computed using:

$$t_l = t_{IR} + \frac{t_r - t_R}{4} \quad (4.13)$$

This equation must be solved simultaneously with the equation for the standard unit hydrograph lag time, $t_l = 5.5t_r$, in order to obtain t_r and t_l . With these values of t_r and t_l . The value of C_t is obtained using equation (4.11) for t_l corresponding to the standard unit hydrograph; the value of C_p is obtained using the expression for q_p corresponding to the Standard unit hydrograph, but using $q_p = q_{pR}$ and $t_l = t_{IR}$.

When an ungauged watershed appears to be similar to a gauged watershed, the coefficients C_t and C_p for the gauged watershed can be used in the above equations to derive the **required synthetic unit hydrograph** for the ungauged watershed.

Development of a Required Unit Hydrograph (assumes that C_t , C_p , L , and L_c are

Known):- If a t_R -unit hydrograph is required, that is, if a unit hydrograph whose associated effective rainfall pulse duration is t_R , is required, proceed as follows.

Use equation 4.11 to determine the lag-time, t_l . If t_{IR} meets the criterion for a standard unit hydrograph, that is, if $t_l = 5.5 t_R$ then the **required unit hydrograph is a standard unit hydrograph** and equations 4.11 and 4.12 can be used directly to estimate the peak discharge and the time to peak of the required unit hydrograph. That is,

$$t_{lR} = t_l = C_1 C_t (LL_c)^{0.3} \quad (4.14)$$

$$q_{pR} = q_p = \frac{C_2 C_p A}{t_l} \quad (4.15)$$

If t_R does not meet the criterion of equation 4.10 then the *required unit hydrograph* is not a *standard unit hydrograph* and equations 4.11 and 4.12 can not be used directly to estimate the peak discharge and the time to peak of the required unit hydrograph.

In this case, the lag-time of the required unit hydrograph, t_{lR} , is,

$$t_{lR} = t_l - \frac{t_r - t_R}{4} \quad (4.16)$$

Where t_l is obtained from equation 4.11, t_r is obtained from equation 4.10 and t_R is given. The peak discharge of the required UH, q_{pR} , is,

$$q_{pR} = \frac{q_p t_l}{t_{lR}} \quad (4.17)$$

Where q_p is obtained from equation 3. Assuming a triangular shape for the UH, and given that the UH represents a direct runoff volume of 1 cm (1 in), the base time of the required UH may be estimated by,

$$t_b = \frac{C_3 A}{q_{pR}} \quad (4.18)$$

Where C_3 is 5.56 (1290 for the English system).

As an aid in drawing an adequate UH, the U.S. Army Corps of Engineers developed relationships for the widths of the UH at values of 50% (W_{50}) and 75% (W_{75}) of q_{pR} . The width in hours of the UH at a discharge equal to a certain percent of the peak discharge q_{pR} is given by as (Chow, 1988),

$$W_{\%} = C_w \left(\frac{q_{IR}}{A} \right)^{-1.08} \quad (4.19)$$

Where the constant C_w is 1.22 (440 for English units) for the 75% width and equal to 2.14 (770 for English units) for the 50% width. Usually, one-third of this width is distributed before the peak time and two-thirds after the peak time, as recommended by the U.S. Army Corps of Engineers. However, several other authors have recommended different distribution ratios. For example, Hudlow and Clark (1969) recommend a partition of 4/10 and 6/10, respectively.

Many other investigators have developed similar formulae. Summary of the results of some of the investigators is shown in Table 4.1.

Table: 4.1 Values of C_t and C_p obtained by different investigators.

Name of Investigator	Number of watershed	Regional parameters		Area of watershed in km ²
		Value of C_t	Value of C_p	
1. Miller et al (1983)	27	0.77 to 3.28	0.23 to 0.67	
2. Cordery (1968)	12	0.40 to 2.40	0.40 to 1.10	0.054 to 634.9
3. Linsley (1943)		0.30 to 0.70	0.35 to 0.59	
4. Hudlow and Clark (1969)	13	0.40 to 2.26	0.31 to 1.22	1.28 to 192

Source: Singh Elementary Hydrology.

As shown in the above table, different investigators have reported wide variations in the value of C_t and C_p . Since the coefficients C_t and C_p vary from region to region, in practical

application the values of these coefficients should be determined from gauged catchments and then can be used to hydrological homogeneous basins.

Figure 4.7 below illustrates the form of Snyder's synthetic UH. Note that the time lag is not the same as the time to peak. Also, note that the widths of the hydrograph at 50% and 75% of the peak flow are distributed such that the longer time is to the right of the time to peak.

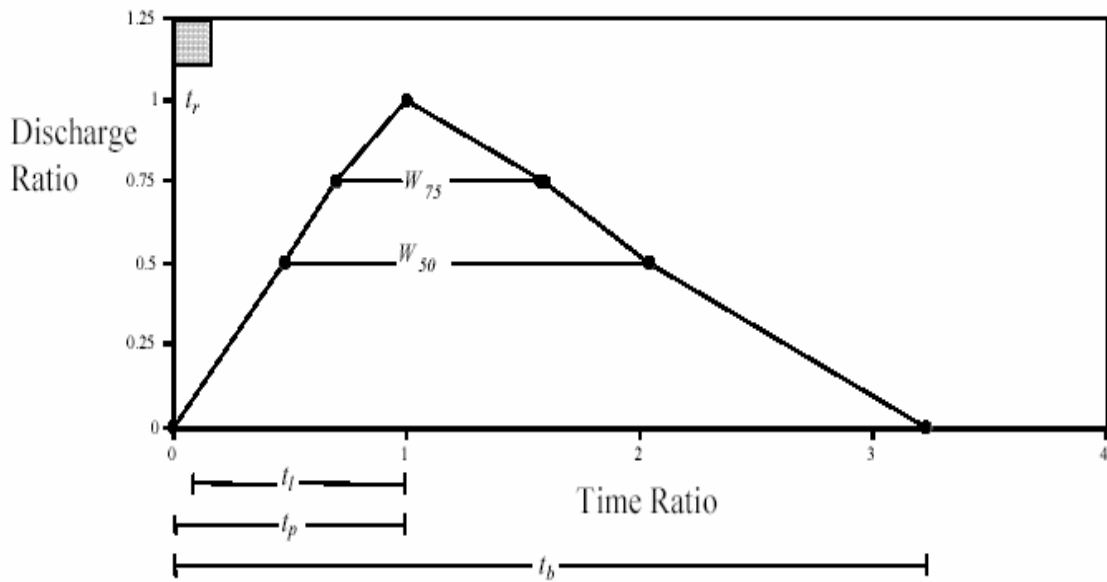


Figure: 4.7 Snyder's Synthetic Unit Hydrograph

Chapter Five

Analysis of Data for Gauged Catchments

5.1 Data requirement

Hydrological data are indispensable for assessment of water resources. Thus one gathers hydrological data in order to assess the water resource and to understand the variables characterizing hydrological processes and the states (storage) of the hydrological systems (drainage basins). If there are no data from a catchment, one has to use methods of estimation of model parameters, which do not require the availability of a long time series of hydrological records. One option is to develop models for the gauged catchments and link their parameters to physical characteristics, so that the approach can be applied to ungauged catchments in the region, whose physical characteristics can be determined. The research work is based on this hydrological regionalization concept. Thus gauged catchments are first identified and their hydrological data is analyzed to estimate calibration parameters. Gauged here referred to the availability of reliable continuous automatic recorder for both rainfall and runoff data sets. Data on Concurrent rainfall runoff events and watershed physical characteristics data are required.

5.2 Selection of Gauged Catchments

The first step required in developing predictive relationships for calibration parameters for a rainfall-runoff model is the collection of rainfall and discharge data for gauged catchments in the region. Considering the location of precipitation gauges relative to stream gauging stations the WMO (1994) states the following (NEDECO, 1998). “To ensure rainfall data are available for extending stream flow records, for flood forecasting purpose, or for hydrological analysis, coordination of the locations of the rainfall gauges with respect to those of the stream gauges is of great importance. Rainfall gauges should be located so that basin rainfall can be estimated for each stream gauging stations. These will usually be located at or near the stream gauge and in the upper part of the gauged drainage basin. Thus, six gauged catchments in both the basins four from Tekeze and two from Awash have been selected. Almost all gauged catchments in the upper Tekeze and upper Awash have

been considered. Except three catchments in Tekeze due to lack of adequate chart calibration information. Most selected catchments have rain gage as per of the WMO recommendation. For the selected gauged catchments rainfall-runoff records have been collected and screened to identify rainfall-runoff events suitable for unit hydrograph analysis.

Table 5.1 List of selected gauged catchments:

Name of River	Catchment Area (km ²)	Basin
Gheba near mekele	2449	Tekeze
Illala near mekele	190	Tekeze
Wori	1770	Tekeze
Mariam-shewito near Adwa	59.8	Tekeze
Suluh	399	Tekeze
M.kunturie	4456	Awash
M.Hombole	7656	Awash
Mojo	1264.4	Awash
Akaki	884.4	Awash

From the above listed selected catchments representative rainfall record is found for Gheba, Illala, Suluh, Mariam shewito, Mojo and Akaki catchments only. For the rest of the catchments i.e. for Wori, .Hombole, and M.Kunturie rainfall record that could represent the whole catchment as per of the unit hydrograph assumption is not found. Thus the research is done based on only the selected six catchments.

5.3 Watershed Characteristics

The concept of watershed is basic to all hydrologic design and analysis. The watershed consists of all land area that shed water to the outlet during a rainstorm. Various parameters that define the characteristics of a watershed are used in both hydrologic analysis and synthesis.

In hydrologic analysis the watershed characteristics are used in defining the nature of the transfer function shown in chapter Four Fig 4.2.

In hydrologic synthesis or design, the characteristics are measured at the design site in order to define the transfer function that is necessary to compute the output function. Watershed characteristics that are often required for hydrologic analysis or synthesis methods include the drainage area, channel length, the shape of the watershed, the slope of the watershed, or channel, the drainage pattern, channel roughness and cross-sectional properties, time of flow parameters and land use (McCuen, 1989).

In this particular study the drainage area, channel length, the shape of the watershed and the slope of the channel are the parameters chosen among the various watershed characteristics listed above for the determination of the transfer function and later for the synthesis of the required outputs.

a). Drainage Area

The drainage area of a watershed requires the delineation of the watershed boundary. All points that will shed water to the outlet define the boundary of a watershed. It is only necessary to decide which points in a region will contribute water to the outlet; the most extreme of these points represents the watershed boundary. This can be done easily and conveniently using 1:50,000 scale topographic map. Hence, based on the above outlined technique, all watersheds for this study are delineated using 1:50,000-scale topographic map. Once the Boundary of a watershed has been defined the drainage area can be estimated. The area is computed by method of counting squares. A grid, usually square is laid over the map and the number of grid blocks within the boundary is counted. The drainage area equals the product of the number of grid blocks and the area of each grid block. The area of each grid block is computed using the scale of the topographic map from which the watershed boundary was delineated.

b). Catchment's Length

The catchment length (or hydraulic length) is the length measured along the principal watercourse. The principal watercourse (or main stream) is the central and largest watercourse of the catchment and the one conveying runoff to the outlet. Other linear measurement required for this research work is the length measured along the principal watercourse, from the catchment outlet to a point located closest to the catchment centroid. In practice, the catchment centroid is estimated as the common point to two or more straight lines that bisect the catchment area in approximately equal subareas (Ponce, 1989). Hence, This has been used in this research to locate the centroid and then to measure the distance between the catchment outlet and the point on the principal watercourse near the centroid of the catchment.

c). Slope of the channel

The channel gradient of a principal watercourse is a convenient measure of catchment relief. A longitudinal profile defines the maximum and minimum elevations and the horizontal distance between them. A measure of channel gradient that takes into account the basin response time is the equivalent slope (Ponce, 1989). To calculate the equivalent slope of the channel, the channel is divided into n number of sub reaches and the slope of each reach is calculated. Based on Manning's equation the time of flow travel through each sub reach is assumed to be inversely proportional to the square root of its slope. Likewise the time of travel through the whole channel is assumed to be inversely proportional to the square root of the equivalent slope. This leads to the following equation.

$$S = \left[\frac{\sum_{i=1}^n L_i}{\sum_{i=1}^n \frac{L_i}{\sqrt{S_i}}} \right]^2 \quad (5.1)$$

In which S = equivalent slope, L_i = each i of n sub reach length, s_i = each i of n subreach slope.

The measured values of the above-discussed characteristics of the selected watershed are shown below.

Table5.2 Values of watershed characteristics.

<i>Name of water shed</i>	<i>Area A (km²)</i>	<i>Length L (km)</i>	<i>Length near the centroid L_c (km)</i>	<i>Equivalent Slope</i>	<i>Minimum elevation masl</i>	<i>Maximum elevation masl</i>
M.Shewito	59.8	9	3.75	0.010986	2040	2160
Illala	190	30	16	0.016001	1990	2440
Suluh	350	36.8	18.5	0.013743	2220	2820
Akaki	884.4	77.6	51.6	0.007287	2060	2800
Mojo	1264.4	76	44	0.004397	1760	2340
Gheba	2449	104	48	0.007955	1720	2900

5.4 Unit Hydrograph from Observed Data

The main problems in the derivation of a unit hydrograph are the assessment of the effective rainfall from the measured rainfall and the separation of the resulting direct runoff from the total hydrograph. Unit hydrograph can be derived in gauged catchments by measuring the concurrent rainfall and runoff amounts for the catchment. Observed storm hyetographs and their corresponding total runoff hydrographs are studied in detail to derive the required unit hydrograph. From continuous river gauge recorders single peaked hydrographs have been selected and then the corresponding rainfalls have been collected from the continuous rainfall recording stations.

After the selection of the appropriate concurrent rainfall-runoff events, the derivation of unit hydrograph has been made. The unit hydrograph derivation from one such storm event proceeds in the following stages.

- a. The first problem is the separation of the base flow from the storm runoff. Fig 5.1 shows several methods. The simplest separation would be given by a horizontal line to a from the start of the rise of the hydrograph. This would assume that the storm has no effect on or makes no contribution to the ground water. The more realistic separation is curve c, which shows a marked storm effect peaking some time after the peak of the stream flows. However, the drawing of this curve would be quite subjective, and different analyst would produce various answers. The straight line to point b, on the recession curve can be easily determined. After point b, the shape of the hydrograph recession is assumed to become exponential, and is fixed uniquely on the semi logarithmic plot (Shaw, 1994). This is a satisfactory compromise and the method is straightforward and gives consistent result. Hence, it is the selected method for this study. Using this method the separation of the base flow from the direct runoff can be accomplished by plotting the hydrograph's recession limb on semi-log paper (flow on the log scale) and identifying the point of inflection i.e. point b as shown in Fig 5.1. At this point it is assumed that direct runoff ceases. To estimate the base flow component of the hydrograph, connect the points representing the beginning and end of the direct runoff. Finally subtract the base flow from the total observed flow to obtain the direct runoff hydrograph.

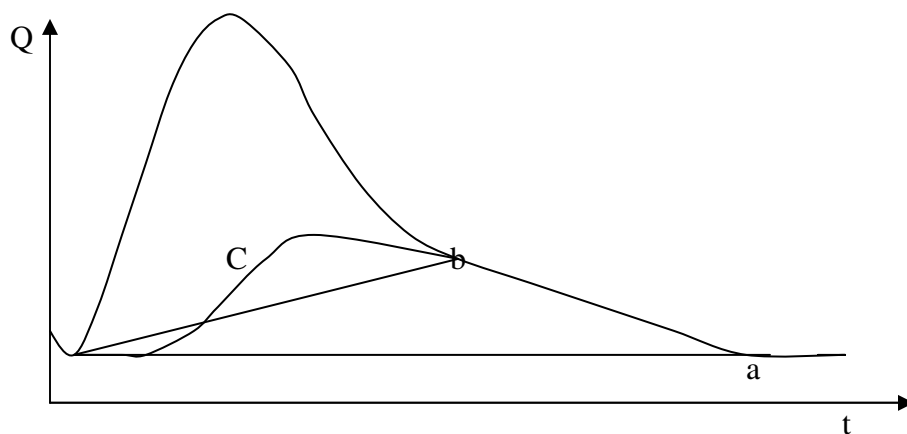


Fig: 5.1 Separation of base flow.

For example log plot of the recession curve and point of inflection for Aug4, 99 flood of Gheba Mekele is shown in Fig 5.2.

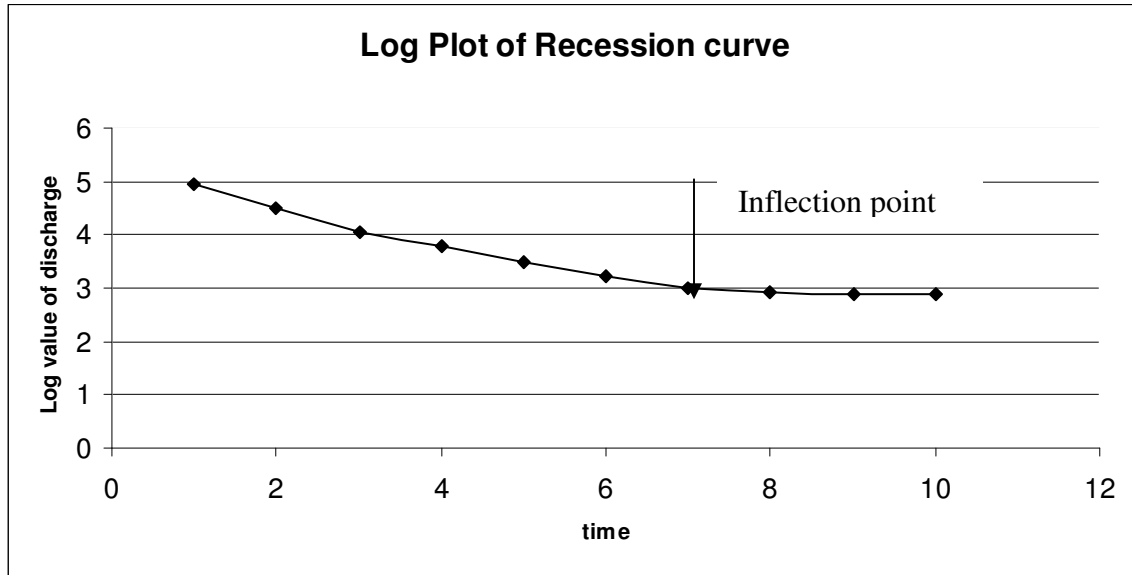


Fig 5.2 Log plot of recession curve and point of inflection for Aug4, 1999 flood of Gheba Mekele.

Similar plots for the other concurrent rainfall-runoff events are shown in the appendix D. For example determination of the direct runoff of Gheba Mekele using the selected method for flood recorded during August 4, 1999 is shown in Fig 5.3.

As shown in the Figure 5.3 the baseflow separation line is almost a straight line for that particular event. But similar plot of the other events have also the same plot i.e. straight line.

- b. The area above the baseflow separation line and enclosed by the hydrograph is then estimated (by discretizing the hydrograph) to give the volume of surface runoff. The equivalent depth of runoff (mm) is evaluated for the catchment area by dividing the estimated direct volume by the area of the watershed. This is then by definition the effective rainfall from the storm

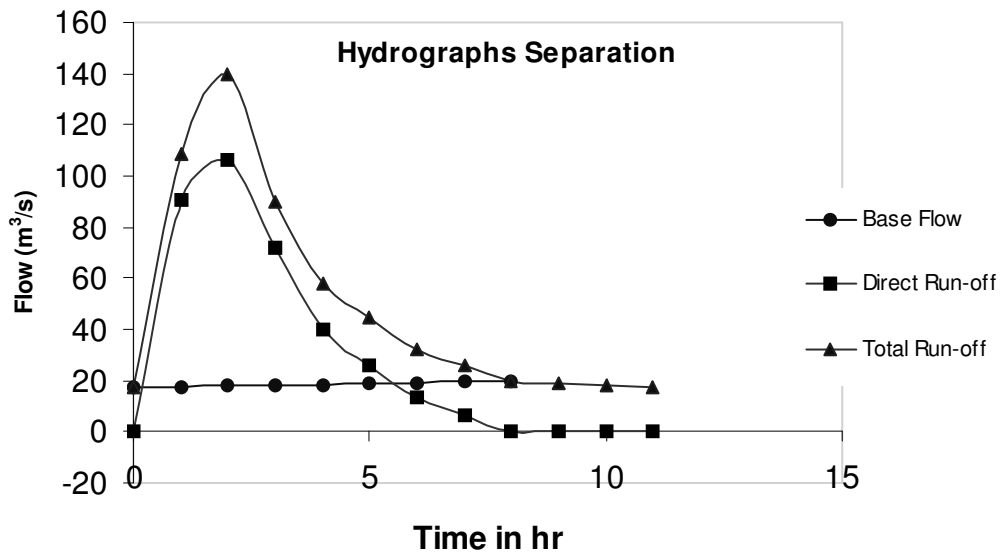


Fig: - 5.3 Separation of hydrographs for Gheba Mekele for Aug4, 1999 flood

- c. The determination of that part of the measured total rainfall that constitutes the effective rainfall is the next step. This requires estimating the abstraction or loss-rate. The loss-rate is dependent on the state of the catchment before the storm and is difficult to assess quantitatively. In the presence of stream flow data the excess rainfall can be determine by a simpler method called the Φ -index. The Φ -index is that constant rate of abstraction (mm/h) that will yield an excess rainfall with a total depth equal to the depth of direct runoff over the watershed (Chow, 1988). As the selected flood hydrographs for most of the catchments are floods occurred during the month of august this method could be used for the estimation of effective rainfall. Because in august there is not significance difference in the state of the catchment from time to time. Thus a constant loss-rate, the Φ -index, would therefore seem to be readily applicable and used to estimate the effective rainfall in this study.

Determination of the effective rainfall requires defining the rainfall duration in addition to the selection of method for the estimation of the loss rate.

For all chosen flood events, 1h effective rainfall duration is used for the simple reason that the continuous chart used to record rainfall is weekly chart that is difficult to process at lower time discrete size than one hour due to the chart scale problem, i.e. to say simply the chart scale is low to discretize it at lower time scale than one hour. Thus the rainfall duration becomes automatically one hour. This implies that we will seek to determine 1h unit hydrograph for all selected flood events.

Once the rainfall duration is defined, the value of Φ -index is determined using the following equation, which is based on the principle that the depths of the direct runoff and excess rainfall are equal.

$$r_d = \sum_{m=1}^N (R_m - \phi \Delta t) \quad (5.2)$$

Sample calculation how Φ -index is estimated using the above equation and principle is shown below for the chosen concurrent rainfall-runoff event of Mariam shewito on Aug5, 1992.

Data: -

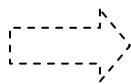
- Size of catchment area -----59.8km².
- Volume of direct run off-----190.255m³.

Rainfall amount in mm	7.9	11.0	0.4
Time in hour	5 to 6 PM	6 to 7 PM	7 to 8 PM

Depth of direct runoff = Volume of direct runoff /area of watershed

$$= 190.25/59.8E6$$

$$= 0.006156m = 6.156mm.$$



$$1.156 = [(7.9*1 + 11*1) - \Phi*1]$$

$$\Phi = 6.37mm/hr$$

Therefore the excess rainfall distribution will be as shown below.

Rainfall amount in mm	7.9	11.0	0.4
Excess rainfall amount in mm	1.53	4.63	0
Time in hour	5 to 6 PM	6 to 7 PM	7 to 8 PM

However, for the chosen concurrent rainfall-runoff events, the occurrence of such kind of rainfall distribution was found to be very rare. Only two such events have been identified one for Mariamshewito used above to shown sample calculation and the other was for Mojo. This is perhaps due to the use of larger rainfall duration i.e. 1hr. In the study area it will be very difficult to see rainfall lasting for more than one-hour duration frequently.

- d. The time axis of the storm hydrograph is then subdivided into convenient time interval. The corresponding ordinates of surface runoff given by the total hydrograph discharge minus the corresponding base flow are then each divided by the effective rainfall or depth of direct runoff to give the ordinates of the unit hydrograph ($\text{m}^3/\text{s}/\text{mm}$) in case of single rainfall event. For the case of multiple rainfall events that have multiple excess rainfalls the deconvolution technique is used to determine the ordinates of the required unit hydrograph. The deconvolution is done easily using MATLAB5.3 software.
- e. The unit hydrograph for effective rainfall of duration of 1h is then plotted and the area under the curve is checked to see if the enclosed volume is equivalent to unit effective rainfall over the area of the catchment.

Using the above outlined procedures 1h unit hydrograph for all the selected concurrent rainfall-runoff events of all catchments are derived.

For exmple for Gheba Mekele for the chosen rainfall-runoff event occurred on Aug4, 1999 the unit hydrograph derived is shown in Fig 5.4.

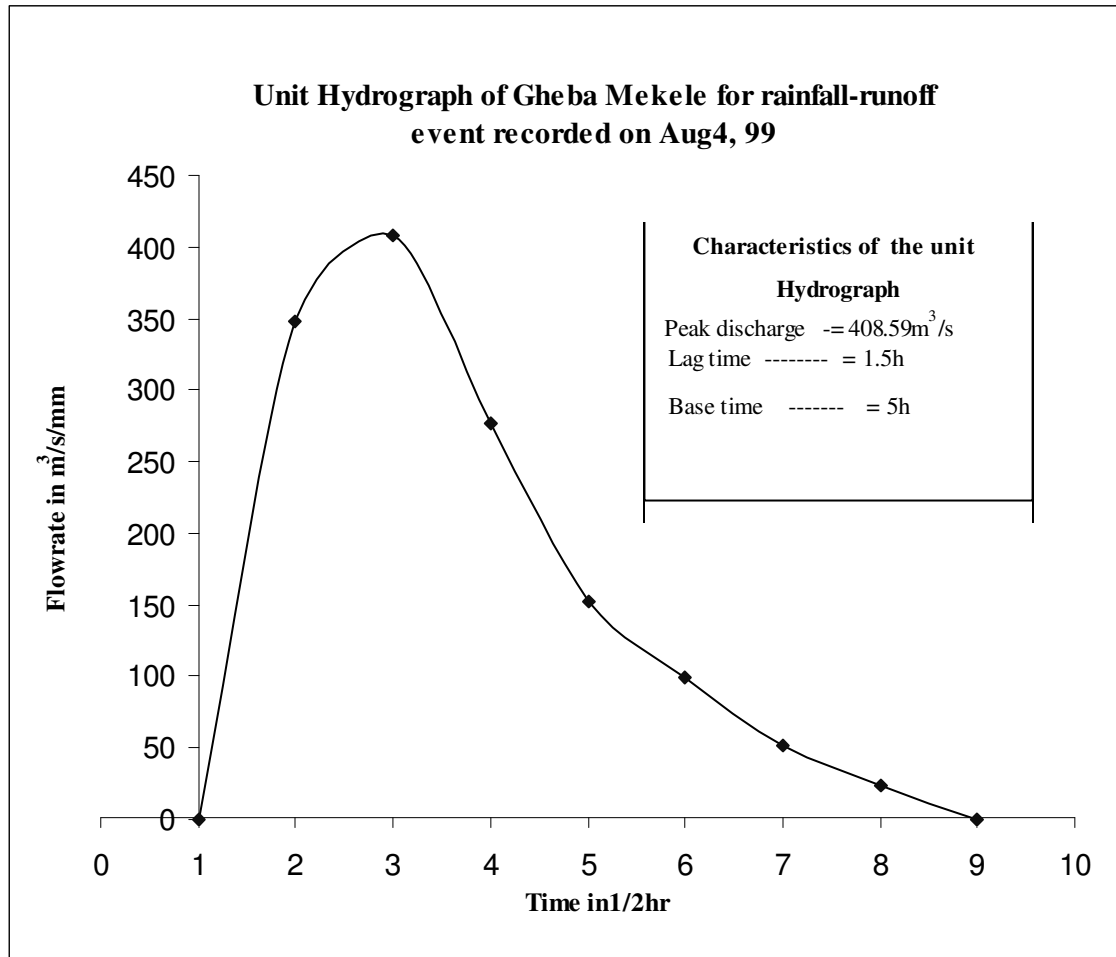


Fig: 5.4 1h unit hydrograph for Gheba Mekele.

- f. When all the singled peaked storms have been analyzed and a corresponding number of unit hydrographs obtained, it will be noted that no two are identical, though they will all have the same general shape. One way that an average unit hydrograph may be constructed is by taking the arithmetic means of the peak flow and the times to peak, plotting the average peak at the appropriate mean value of the time to peak and drawing the hydrograph to much the general shapes of the individual unit hydrographs (Shaw, 1994). For example using this idea the average unit hydrograph of Gheba Mekele resulting from five-unit hydrograph is shown below.

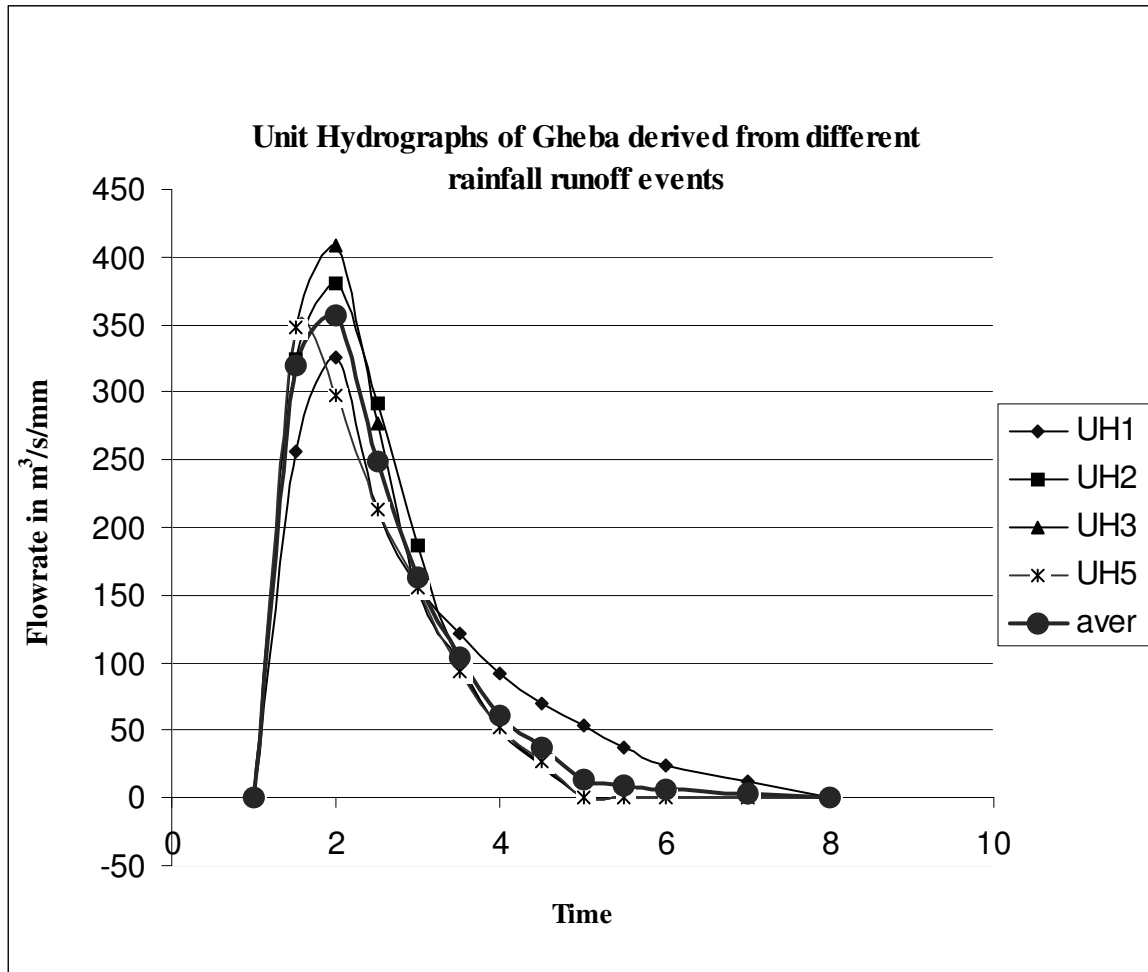


Fig 5.5 Averaging 1 hr UH of Gheba Near Mekele

The resulting average unit hydrograph is then checked to ensure that the enclosed volume of runoff is equivalent to a unit of effective rainfall.

Similarly for all the watersheds the average unit hydrographs have been derived and the necessary model calibration parameters were collected. These parameters are used to calibrate the selected models (Snyder's synthetic unit hydrograph equations). The next chapter deals with the calibration procedure.

Unit hydrograph parameters used for calibration includes Lag time (t_l) in hr, peak discharge (q_p) in m^3/s , width of the unit hydrograph at 50% and 75% of the peak

discharge ($W_{50\%}$ and $W_{75\%}$), and the time base (T_b) in hr. The values of these unit hydrograph parameters for the selected watershed are shown in the next table.

Table: - 5.3 List of unit hydrograph parameters used for model calibration.

Calibration Parameters	<u>Name of Rivers</u>					
	M.Shewito	Illala	Suluh	Akaki	Mojo	Gheba
Peak Discharge q_p (m^3/s)	9.126	35.323	44.243	122.018	79.775	365.656
Peak time t_p (hr)	1.35	1.6	1.5	1.0	2.0	2.0
Lag time t_l (hr)	1.15	1.9	2.333	5.833	6.3	6.375
Base time t_b (hr)	8.0	5.5	7.0	6	14	8
Width of the UH at 50% of q_p $W_{50\%}$ (hr)	1.35	1.3	2.0	1.6	3.75	1.70
Width of the UH at 75% of q_p $W_{75\%}$ (hr)	0.85	0.7	1.00	0.75	1.9	1.0

Chapter Six

Model Calibration for SUH Construction

6.1 General

For ungauged basins, direct development of a unit hydrograph is not possible and techniques for estimating a unit hydrograph from measurable basin characteristics are employed. Generally, a unit hydrograph is represented mathematically as a function of one or two parameters and these parameters are related to basin characteristics by regression analysis or other means. Unit hydrograph developed in such ways is called synthetic unit hydrograph.

A number of parameters are important in determining the shape of the unit hydrograph for a watershed. The discharge parameter, which is mostly used, is the peak discharge (q_p). Lag time (t_l), time to peak (t_p), time of concentration (t_c), and base time (t_b) are often used as the time parameters. Watershed parameters of most concern, influencing the shape of the outflow hydrograph, include area (A in km^2), mainstream length (L in km), length to watershed centroid from the outlet (L_c) and average slope of the basin (s).

Unit hydrographs are synthesized for relationships between relevant parameters affecting the shape of unit hydrographs. These are

- Hydrograph time to peak
- Hydrograph shape
- Basin size, slope,
- Basin length, etc

In general, the development of synthetic unit hydrographs requires relationships for:

- the time to peak
- the peak discharge
- the hydrograph shape

As mentioned before, numerous methods have been developed within the hydrologic literature to generate synthetic unit hydrographs such as Snyder's, SCS, and Clark etc. In this study Snyder's method is used, since it requires watershed characteristics parameters that can very easily be obtained from topographic maps.

The process of deriving characteristics equations constants, weighting factors, and other parameters that serve to define the model for a particular watershed is termed “calibration” (U.S Army Corps, 1994). Calibration here refers to the process of using watershed characteristics data and observed stream flow and rainfall data to develop values for model parameters. In this particular study the model parameters we seek to estimate are Snyder’s synthetic unit hydrograph equations coefficients viz C_t , C_p , and C_w .

Generally calibration of simulation models must be done carefully with due consideration for

1. Selection of reliable rainfall-runoff events: - Generally the largest rainfall-runoff events for which data available provide the best basis for calibration/verification (U.S Army Corps of Engineers, 1994).
2. In addition to the size of the rainfall runoff event, the state of the basin at the time of occurrence is significant. The model must represent land-use and other conditions consistent with the time of occurrence of the observed event.
3. In general, it is desirable to use several events (say, four to six) for calibration. It is also desirable to reserve a couple of events for verification. Sometimes the amount of useful data is limited so that there are few events for calibration and no events for verification.

Verification enables assessment of the reliability of the calibrated model. It is performed by simulating historical events not used for calibration. This procedure will help to ensure that the calibration is not unique and limited to the data set employed for calibration.

6.2 Calibration of Snyder’s SUH Equations

For this particular study the model need to be calibrated is Snyder's SUH equations. As discussed above the parameters required to calibrating these equations are watershed and UH characteristics. The calibration of these equations is done following the steps outlined below.

1. First for the selected watersheds, parameters required for calibration have been determined using the techniques and procedures discussed in chapter five. The value of these parameters is shown in table 5.3 in chapter five.
2. The UHs developed for all the basins are 1hr UHs. Thus, the lag determined has been checked for the requirement of Snyder's standard unit hydrograph. Only three of the catchments i.e. Akaki, Mojo, and Gheba have satisfied this requirement. The remaining three needs an adjusted lag as shown in the appendix A.
3. The equations from 4.10 to 4.19 required for the synthesis of UH as discussed in the previous topics are calibrated.

In the study the calibration process is performed to find direct solution of the coefficients of the equations from 4.10 to 4.19. This is accomplished by treating the required coefficients as dependent variables and the watershed and UH characteristics as independent variables in the respective equations and solving the problem.

6.3 Result of calibrated Coefficients

The calibration process yields the required coefficients of Snyder's SUH equations for the gauged watershed used in this analysis. These values with some of their statistical properties are shown in the table 6.1.

Table 6.1 Results of the Calibration processes

Rivers	Coefficients									
	Modified C_{tl}	C_t	C_p	C_{50}	C_{75}	C_B	T_p/T_b	C_{50}/C_{75}	T_l/T_b	C_t/C_p
M.Shewito	0.150	0.437	0.064	0.177	0.112	1.221	0.169	1.588	0.144	6.854
Illala	0.104	0.362	0.128	0.211	0.114	1.023	0.291	1.857	0.345	2.815
Suluh	0.110	0.411	0.107	0.214	0.107	0.885	0.214	2.000	0.333	3.833
Akaki	0.132	0.646	0.293	0.188	0.088	0.828	0.167	2.133	0.972	2.207
Mojo	0.139	0.736	0.145	0.190	0.096	0.883	0.143	1.974	0.450	5.093
Gheba	0.135	0.661	0.346	0.218	0.128	1.194	0.250	1.700	0.797	1.909
Mean	0.128	0.542	0.180	0.200	0.108	1.006	0.206	1.875	0.507	----
Standard Deviation	0.018	0.157	0.112	0.017	0.014	0.169	0.057	0.203	0.314	----

As shown in the table 6.1 a considerable variation of values from values obtained by Snyder's is observed for all the coefficients. For example the values of C_t obtained by Snyder were in the range from 1.35 to 1.65 but in this study this value is found to be in the range from 0.362 to 0.736. This is actually because the values of these coefficients are depending upon the region under study. Many investigators have observed this and wide variation with value of C_t ranging from 0.3 to 6.0 and value of C_p ranging 0.31 to 0.93 have been reported (Subramarya, 1984,U.S Army Corps, 2000). The average value comparison of these coefficients is shown in the table 6.1.

Table 6.2 Comparisons of Coefficients.

Rivers	Coefficients								
	C_{tl}	C_t	C_p	C_{50}	C_{75}	C_B	T_p/T_b	C_{50}/C_{75}	C_t/C_p
▲▲▲1	---	1.50	0.60	2.14	1.22	5.56	0.33	1.75	2.5
▶▶▶2	0.128	0.542	0.180	0.200	0.108	1.006	0.206	1.875	3.011

Where ▲▲▲1 shows Snyder's Values from Appalachian Mountain region of USA
▶▶▶2 shows values estimated in this research.

It is seen that the variation for C_t and C_p is very high and hence the mean value would not give reasonable estimate when applied to ungauged catchments. For the rest of the coefficients including the modified C_{t1} the mean value is deemed to be adequate for estimating the respective UH characteristics when applied to ungauged catchments. However, hydrological homogeneity of the watersheds should be checked to use the mean values. A brief description on how to check the study areas hydrologic homogeneity is discussed here below.

Linsley point out that if known values of lag are plotted against $LLc/S^{0.5}$ on logarithmic paper the resulting plot should define a straight line for basins of similar hydrologic characteristics (Linsley, 1988). Based on this, logarithmic plot of lag time versus $(LLc/s^{0.5})$ for the gauged catchments is shown on Fig 6.1. The plot look like a straight line and the study watersheds can be considered hydrologically homogeneous. Thus the mean values for the coefficients shown in table 6.1 other than C_t and C_p can be used to develop the required SUH characteristics. However, to estimate all required characteristics of the SUH it is still necessary to seek means to predict the remaining important parameters.

Therefore, attempt should be made to develop predictive equations for C_t and C_p and/or for direct estimation of lag time and peak discharge with out using C_t and C_p . The development of predictive equation for C_t , C_p , t_l , and Q_p is discussed in the chapter seven.

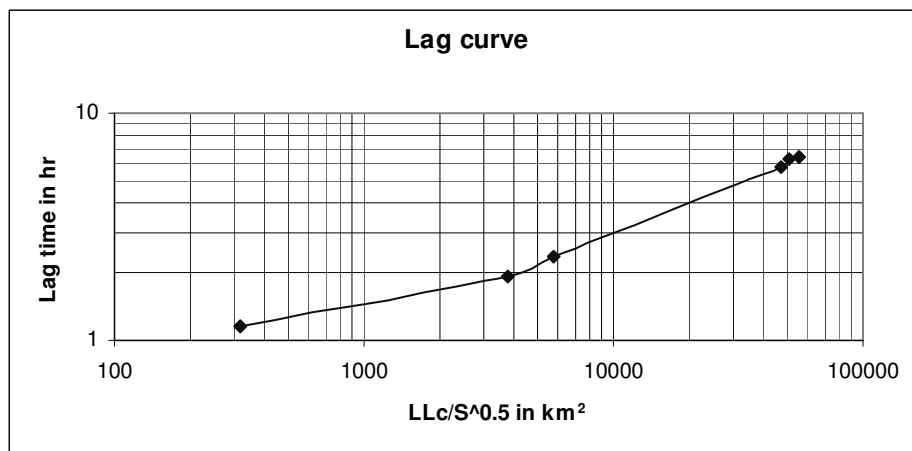


Fig 6.1 Log-Log plot of lag curve

Chapter Seven

REGRESSION ANALYSIS

7.1 Selection of independent variables

To apply a parameter predictive equation for an ungauged catchment the independent variables in the equations should be variables that can be taken or estimated from catchment characteristics. Common sense indicates that lag time must be related to physical characteristics of the watershed. The unit hydrograph peak coefficient could also be dependent on certain watershed characteristics. Regression analyses were performed to determine these relationships. Watershed characteristics considered as independent variables in the regression analysis are:

1. Contributing drainage area (A in km^2)
2. Stream length (L in Km)
3. Watershed shape factor (L_c in km)
4. Average mainstream slope (S)

To examine the relationship among these independent variables and the dependent variables (C_t , C_p , t_l , and Q_p) correlation analyses were performed using MINITAB statistics software. The correlation matrix is shown in table 7.1.

Table 7.1: Correlation matrix for the independent variables and dependent variables

	C_p	C_t	t_l	A	L	L_c	S	Q_p
C_p	1							
C_t	0.651	1						
t_l	0.861	0.887	1					
A	0.873	0.806	0.97	1				
L	0.896	0.792	0.961	0.978	1			
L_c	0.881	0.717	0.957	0.948	0.991	1		
S	-0.415	-0.95	-0.768	-0.653	-0.575	-0.580	1	
Q_p	0.953	0.688	0.907	0.963	0.955	0.918	-0.471	1

Note: Tabulated values are the correlation coefficients, r , for the base -10 logarithmic of the values.

7.2 Regression Analysis for Lag time coefficient (C_t)

Lag time coefficient is seen strongly correlated with channel slope ($r = -0.95$). It has correlations with Contributing area ($r = 0.806$), channel length ($r = 0.792$) and shape factor ($r = 0.717$). All of these correlation coefficients are significantly different from zero at 90% significance level. The correlation coefficient for slope is especially different from zero at 99.5 significance level. Logically channel slope, channel length and shape factors should be important explanatory variables for lag time.

Examination of equation 4.11 reveals that C_t is largely a function of channel slope, since both length and shape have already been accounted for in L and L_c , respectively.

Furthermore this equation implies that when the product (LL_c) is equal to 1, the lag is equal to C_t . Since for two catchments of the same size, lag is a function of slope, and it is unlikely that C_t is constant. Therefore, Lag time should be a function of length, shape, and slope. And hence C_t is largely a function of catchment slope.

To determine the functional relationship between C_t and slope (S) simple linear regression analysis was performed using the MINITAB statistics software. The first step in a regression analysis is the choice of the form of the predictive model, and it should be based on a rational analysis of the problem and on the experiences of other investigators.

Thus based on experiences of other investigations the regression model used in this analysis is the power function

$$C_t = a \cdot S^b \quad (7.1)$$

In which a and b are regression constants and the others are as defined before. Logarithmic transformation of equation 7.1 results in an equation that is linear with respect to the variable slope (S).

$$\text{Log}(C_t) = \text{Log}(a) + b \cdot \text{Log}(S) \quad (7.2)$$

Equation 7.2 was fitted to the data shown in appendix A and the resulting best-fit regression equation was

$$C_t = 0.030 * S^{-0.597} \quad (7.3)$$

Equation 7.3 has a standard error of 0.045 and a coefficient of determination of (r^2) Of 0.902.

Besides, the determination of predictive equation for lag time coefficient, regression analysis was also performed to determine the relationship of Lag time with channel length (L), watershed shape (Lc), and channel slope (S).

Linsley et al. found that the basin lag t_p better correlated with catchment parameter ($LLc/S^{0.5}$). Hence, a modified form of equation 4.11 was suggested by them as [23].

$$t_p = C_{tl}(LLc/s^{0.5})^n \quad (7.4)$$

Where S is the weighted channel slope.

C_{tl} is modified coefficient of lag time

n is basin constant and others are as defined before.

Furthermore Linsley point out that if known values of lag are plotted against $LLc/S^{0.5}$ on logarithmic paper the resulting plot should define a straight line for basins of similar hydrologic characteristics. Such relation can be used to estimate the lag for ungaged catchments. Based on this, logarithmic plot of lag time versus ($LLc/s^{0.5}$) for the gauged catchments is shown on Fig 6.1.

Considered the plot as straight line and transforming the expression into logarithmic expression and accomplishing simple linear regression analysis using MINITAB statistic software yields the modified equation of lag time of the form

$$t_p = 0.127 * (LLc/S^{0.5})^{0.352} \quad (7.5)$$

Equation 7.5 has a standard error of 0.069 and a coefficient of determination of (r^2) Of 0.962. The value of n and C_{cl} is equal to 0.352 and 0.127 respectively as shown in equation 7.5 above.

The coefficient 0.127 is equal to the mean value of C_{cl} that was calculated from the C_{cl} values of the study watersheds as shown in table 6.1.

For basin in USA the value of n was found to be equal to 0.38 and the value of C_{cl} were 0.83 for mountain drainage area, 0.5 for foothill drainage area, and 0.24 for valley drainage area.

Thus either equation 4.11 or equation 7.4 may be used to estimate lag times (t_l). Because correlation analyses of lag times computed by both formulae with the observed values have resulted correlation coefficients of 0.992 and 0.979 for equations 7.4 and 4.11 respectively. This may show that equation 7.4 has relatively better estimation power. However, both can be used equally because on one hand there is no considerable difference in their correlation coefficients on the other hand the structure of both models are more or less very similar.

7.3 Regression Analysis for peak discharge Coefficient (C_p)

The correlation analysis shows that C_p is strongly correlated with watershed area ($r = 0.873$), channel length ($r = 0.896$), watershed shape factor ($r = 0.881$), and Lag time ($r = 0.861$). But looking at equation 4.12 this variables have been already accounted and hence the correlation has no significance. The correlation analysis show also peak coefficient is weakly correlated with lag time coefficient ($r = 0.651$) and channel slope ($r = -0.415$). This shows that none of the watershed characteristics used in this study are reliable variables to estimate the value of C_p using an equation that could be developed in regression analysis. Thus means to estimate peak discharge have to be sought in order to develop UH for an ungauged catchment.

There are two possible options for this purpose:

1. Use of the mean values of C_p and

2. Developing an equation that uses estimated and/or measured watershed variables

For the case of the first option C_p values for the gauged watersheds range from 0.064 to 0.346. These values have a mean of 0.180 and a standard deviation of 0.112. Therefore, use of the mean value would result in less accurate estimation of the peak discharge. Thus the second option, which is seeking a predictive equation for peak discharge, has to be treated. The required equation is developed using regression analysis.

As pointed out previously the first step in a regression analysis is the choice of the form of the predictive model. For this case a model that has a form

$$Q_p = C * A^q * (t_1)^r \quad (7.6)$$

Where Q_p is the peak coefficient in m^3/s

A is drainage area in (km^2)

t_1 is lag time in hr

C , q , and r are constants to be determined

Which has a general similarity to the model shown in equation 4.12 is chosen. Logarithmic transformation of Eq. 7.6 results in an equation that is linear with respect to the logarithms of the variables

$$\text{Log}(Q_p) = \text{Log}(C) + q * \text{Log}(A) + r * \text{log}(t_1) \quad (7.7)$$

Equation 7.7 was fitted to the data shown in appendix A and the resulting best-fit regression equation was

$$Q_p = 0.054 * A^{1.29} * (t_1)^{-0.786} \quad (7.8)$$

Equation 7.8 has a standard error of 0.1723 and a coefficient of determination of (r^2) of 0.939.

In addition to the regression analysis performed above to express the dependent variable discharge with the independent variables area and lag time. Further regression analysis was performed with only little modification of the form of the selected model. The selected model was modified to have area divided by lag time as a single variable. In the previous case they were considered as separate variable.

Hence, the form of the model becomes

$$Q_p = C * (Z)^r \quad (7.9)$$

Where Q_p is the peak coefficient in m^3/s

$$Z = A/t_1 \text{ (in km}^2\text{/hr)}$$

A is drainage area in (km^2)

t_1 is lag time in hr

C , and r are constants to be determined

Logarithmic transformation of Eq. 7.9 results in an equation that is linear with respect to the logarithms of the variables

$$\text{Log}(Q_p) = \text{Log}(C) + r * \text{Log}(Z) \quad (7.10)$$

Equation 7.10 was fitted to the data shown in the appendix A and the resulting best-fit regression equation was

$$Q_p = 8.71 * 10^{-3} * (Z)^{1.79} \quad (7.11)$$

Equation 7.11 has a standard error of 0.1738 and a coefficient of determination of (r^2) of 0.918.

The regression analyses results show that the length, area, shape and slope are the most important independent variables. From the variables length, shape and slope lag time can be estimated. From the area and lag time peak discharge can be computed.

An important part of a regression model is assessing the adequacy of the model. Testing statistical hypotheses about the model parameters is used to identify the adequacy of the model. The t statistical test and the P- value are used to test the model adequacy. The computer out put containing valuable information including the values used for these tests is attached in the appendixes. The comparisons of statistical value of these tests with the critical value show that all models are adequate except Eq 7.8. For Eq 7.8 the critical value of t is 2.132 for 4 degree of freedom and a 5% level of significance. This value is larger than the t values computed for the constant and the coefficient for the lag time. The P-values for both the constant and the coefficient are much greater than 5%. Therefore this shows that the model does not adequately represent the relationships of the variables. Otherwise, such tests prove the adequacy of the remaining models.

Thus, the single independent variable ($Z = A/t_1$) yields a more reliable regression equation to estimate peak discharge and hence Eq 7.11 is recommended.

7.4 Verification of Calibrated models

Verification refers to the testing of calibrated values, generally with data not used for calibration. It enables assessment of the reliability of the calibrated model. Three rainfall-runoff events that were not used in the calibration process were used for verification. The rainfall-runoff events selected for verification are observed events from the watersheds Mariam shewito, Illala, and Gheba mekele. These watersheds were used in the calibration processes too. It would have been good if the verifications were made with data observed from watersheds that were not used for calibration. But due the scarcity of data it is done with data observed from watershed used in the calibration but with events not used for calibration.

Generally, the quality of fit between the observed and computed hydrographs is judged by reviewing plots of the hydrographs. This requires the construction of both observed and synthetic UHs. The construction of observed UH is direct as discussed in chapter five. But the construction of SUH is outlined below.

7.4.1 Synthetic Unit Hydrograph Construction

To construct the required synthetic unit hydrograph estimating the quantities i.e. time to peak t_p , peak flow rate Q_p , time base T_b and the time width (hours) of the hydrograph at 2 locations.

These are: -

- At 50% of the peak discharge
- At 75% of the peak discharge of the SUH is required.

The **seven points** form the basis of the unit hydrograph construction. The final shape of the curve is adjusted to ensure a rainfall excess of 1 unit. This is shown below.

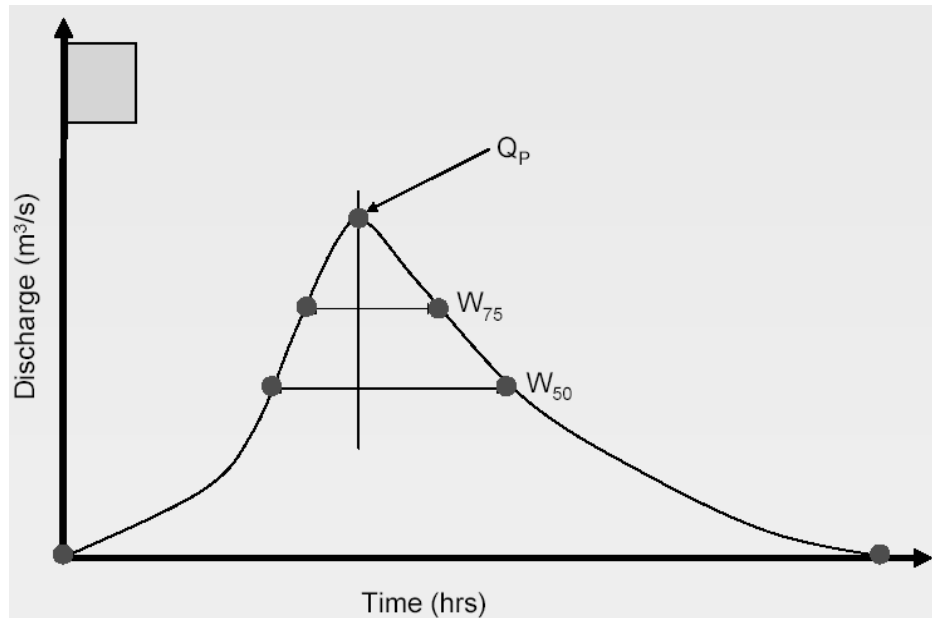
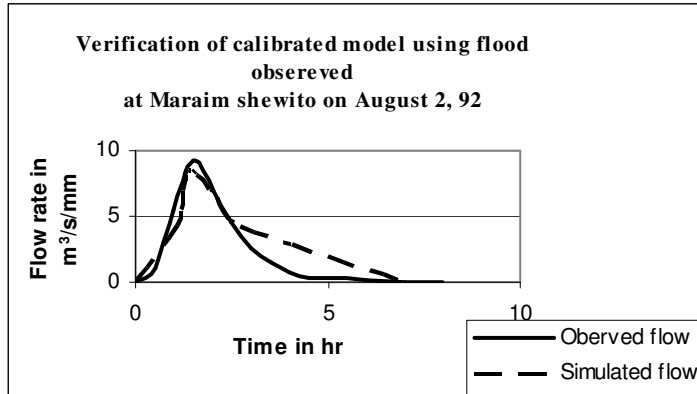
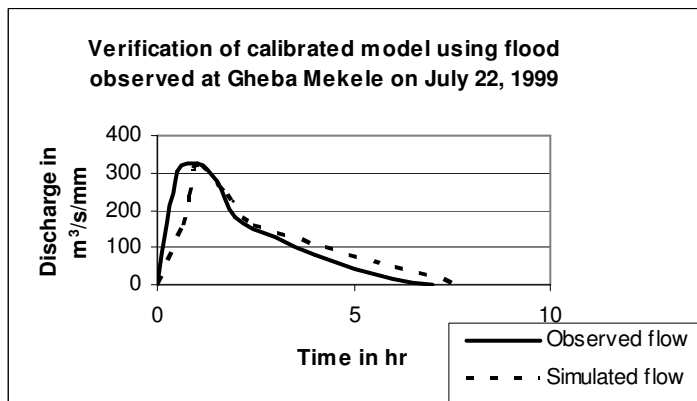


Fig 7.1 Seven points for construction of UH

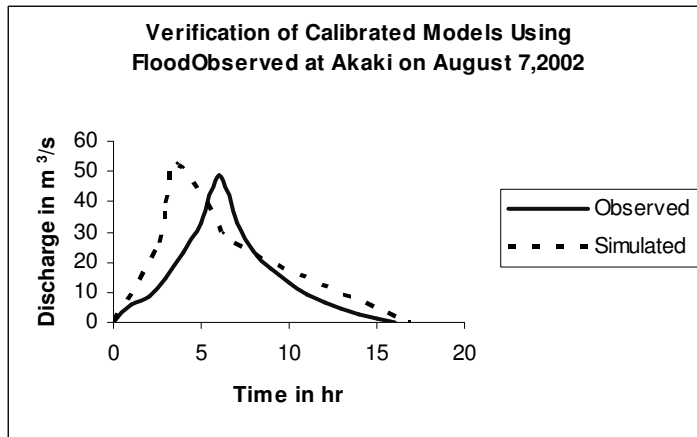
Using the procedures outlined above for SUH construction and following the method discussed in chapter five for deriving UH from observed data, plots of the required hydrographs have been made to visualize the adequacy of the calibrated models. Such plots for Mariamshewito watershed is shown in the Figure 7.2.



(a) Mariamshewito watershed.



(b) Gheba watershed



(c) Akaki Watershed

Figures: -7.2. Observed and simulated UHs

The plots show that the observed and simulated flows are judged to be similar and hence the calibration process is deemed adequate.

7.5 Results and Discussion

The equations and coefficients developed to construct the required SUH have been developed based on averaging various unit hydrographs and hence they are not expected to represent adequately the peak discharge required for design purpose. Besides, the rainfall-runoff events selected for calibration are from the events observed during the wet month i.e. August and therefore the equations and coefficients are not also expected to represent the rainfall-runoff relation adequately for the other periods such as the Belg.

The following summarize some of the results of the research work.

1. The values of lag time coefficient ranges from 0.362 to 0.736 with mean value of 0.542 and a standard deviation of 0.157.
2. The values of peak discharge coefficient (C_p) ranges from 0.064 to 0.346 with mean value of 0.180 and a standard deviation of 0.112.
3. The mean values of the coefficients for the widths of unit hydrograph at 50% and 75% of the peak discharge are found to be 0.20 and 0.108 respectively. The mean value for the base time coefficient is 1.006. The mean value for the ratio of time to peak to base time is 0.206
4. Strong correlation is seen between Lag time coefficient (C_t) and slope of a watershed. And hence the equation $C_t = 0.03 * S^{-0.597}$ is developed using slope (S) as the only independent variable to estimate lag time coefficient (C_t).
5. Strong correlation is seen also between peak discharge and the variable ($Z = A/t_1$) and hence peak discharge q_p can be estimated from the equation $q_p = 0.00871 * (Z)^{1.79}$.

The results of the study are given as a summary in Table 7.2. The characteristics used for comparison are peak discharge (q_p), time to peak (t_p), base time (t_B), and widths of the UH (W_{50} and W_{75}) at 50% and 75% of the peak discharge In this table observed hydrograph

parameters are give as average values for different unit hydrographs derived from different rainfall-runoff events. Beneath the average values the same parameters, which are obtained from Snyder's equations for SUH construction, are given.

Table 7.2 Observed and Synthetic unit hydrographs characteristics

		Parameters				
		t_p in hrs	t_B in hrs	q_p in m^3/s	W_{50} in hrs	W_{75} in hrs
Mariam shewito	Observed	1.35	8.0	9.126	1.35	0.85
	Synthetic	1.4	7.0	8.55	1.6	0.9
Illala	Observed	1.6	5.5	35.323	1.30	0.70
	Synthetic	1.6	7.9	24.32	1.8	1.0
Suluh	Observed	1.5	7.0	44.243	2.0	1.0
	Synthetic	1.4	6.9	51.14	1.6	0.8
Akaki	Observed	1.0	6	122.018	1.6	0.75
	Synthetic	3.4	16.9	52.70	4.2	2.3
Mojo	Observed	2.0	14	79.775	3.75	1.9
	Synthetic	4.0	19.8	64.15	5.0	2.7
Gheba	Observed	2.0	8	365.656	1.70	1.0
	Synthetic	1.5	7.7	318.33	1.8	1.0

As it is seen in the Table 7.2, average values of these characteristics of observed data are almost in harmony with the synthetic ones in the watersheds except Akaki. For Akaki the observed peak discharge is quite very big as compared to the synthetic ones this may be for various reasons. One possible reason could be the uncertainty in the observed data for Akaki catchment. Very big difference from year 1995 to 2002 in the order of 50% and above reduction in the amount of peak flow rate of the 1 hr UH is observed in Akaki watershed.

Chapter Eight

CONCLUSIONS AND RECOMMENDATIONS

8.1 Conclusion

Adequate representation of the rainfall-runoff relationship is one of the most critical tasks in the development and management of water resources. However, the physical systems involved are often large and complex and substantial quantities of data, which require a substantial amount of resources for acquisition, processing and management, are required for their representation. In most case the data are not available in developing countries like Ethiopia with the desired quantity and quality. For example data, required for deriving unit hydrographs to determine design discharges for hydraulic structures, are not easy to find. Therefore unit hydrographs are usually required to be determined synthetically.

In this research, attempt has been made to determine coefficients required to develop synthetic unit hydrograph using Snyder's method for watersheds in upper Awash and Tekeze basins. Besides, efforts were also made to relate lag time coefficient and some of the salient features of unit hydrograph to basin characteristics using regression analyses.

Finally the following concluding results were drawn from the research work.

1. Values of the coefficients C_t and C_p of the Snyder's SUH equations estimated in this study are seen to have quite considerable variation from values obtained by other investigators in some other regions. Thus in practical application it is strongly recommended to estimate these values regionally.
2. For watersheds ($59.8 - 2449\text{km}^2$) in the upper Awash and Tekeze basins lag time coefficient C_t , which is required for the construction of SUH using Snyder's method, can be estimated using the equation $C_t = 0.03*S^{-0.597}$. Or the equation $t_p = 0.127*(LL_c/S^{0.5})^{0.352}$ can be used directly to estimate lag time t_p from watershed characteristics length (L in km), shape (L_c in km), and slope (S).

3. Peak discharge in (m³/s) of 1hr UH can be estimated using $q_p = 8.71 \cdot 10^{-3} \cdot (Z)^{1.78}$ for watersheds (59.8-2449km²) in the upper Awash and Tekeze basins. Where $Z=A/t_i$ in (km²/hr).
4. The mean values of the coefficients for the widths of unit hydrograph at 50% and 75% of the peak discharge are found to be 0.20 and 0.108 respectively. The mean value for the base time coefficient is 1.006. These values can be used to construct the required SUH.

8.2 Recommendation

In developing such models it is very important to have adequate and reliable sets of hydrological and meteorological data. Unless these data are reasonably sufficient and reliable it is difficult to have better estimate of the required coefficients as they vary quite considerably from region to region. Thus it is recommended that

- Applying the equations may result in reliable synthetic unit hydrographs, as long as the physical characteristics of the watersheds under consideration are within the range of those characteristics for the watersheds in this study. However, for more reliable results the coefficients and equations should be tested on hydro meteorologically similar gauged catchment, which has reliable data set and modified accordingly.
- Finally to facilitate model development and their subsequent modification pertinent to water resources development and management it is very imperative to have adequate and reliable data set, specially with due attention to hydrometeorological data. Very big difference from year 1995 to 2002 in the order of 50% and above reduction in the amount of peak flow rate of the 1 hr UH is observed in Akaki watershed. Therefore, It is essential to make continuous monitoring of the stations and checking of the data set to minimize the uncertainty.

References:

1. Admasu Gebeyehu (1989). Regional Flood frequency analysis. The Royal Institute of Technology Stockholm, Sweden.
2. Chow Ven Te, D.R Maidment, and L.W Mays (1988). Applied Hydrology. McGraw hill, New York.
3. Fleming. G (1979). Deterministic models in hydrology. FAO Irrigation and Drainage paper. Rome.
4. Griffith D. F. (1996). An introduction to MATLAB version 2.2. The University of Dundee DD1 4HN Stockholm, Sweden.
5. Haan C. T. (1977). Statistical Methods in Hydrology. The Iowa State University Press.
6. HALCROW (1989). Master Plan for the Development of Surface Water Resources in the Awash Basin for EVDSA Vol 4. Addis Ababa.
7. Herschy R. W. (1995). Streamflow Measurement. E &FN Spon,an Imprint of Chapman &Hall London, UK.
8. IDCOF (2002). Food Security,Sustainable Agriculture and Trade. A report prepared for the Christian Relief and Development Association (CRDA). Addis Ababa
9. Kundzewicz Zbigniew W. (2002). Prediction in Ungauged Basins. Draft of presentation at the kick-off meeting on “Prediction in ungauged basins” (PUB) held in Brasilia, 20-22 November 2002.
10. Linsley R. K., Max A. Kohler, and Joseph L.H Paulhus (1988). Hydrology for Engineers. Metric Edition. McGraw hill London, UK.
11. Maidment David R. Editor-in-Chief (1993). Hand book of Hydrology. McGraw hill, New York.
12. McCuen Richard H (1989). Hydrology Design and Analysis. Prentice Hall Englewood Cliff, New Jersey.
13. McEnroe Bruce M, and H.Zhao (1999). Lag Time and Peak Coefficients for Rural Watersheds in Kansas.University of Kansas
14. NEDECO (1998) Tekeze River Basin Integrated Development Master Plan Project, Vol 5 & 6. Ministry of water resources Addis Ababa.
15. Ponce V.M. (1989). Engineering Hydrology Principles and Practice. Prentice Hall Englewood Cliff, New Jersey.

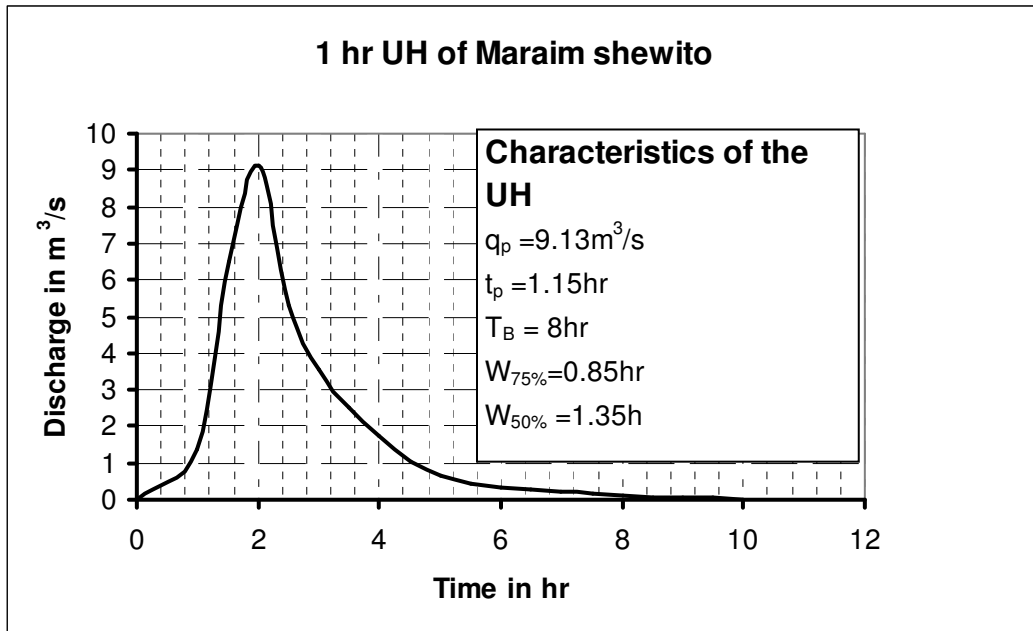
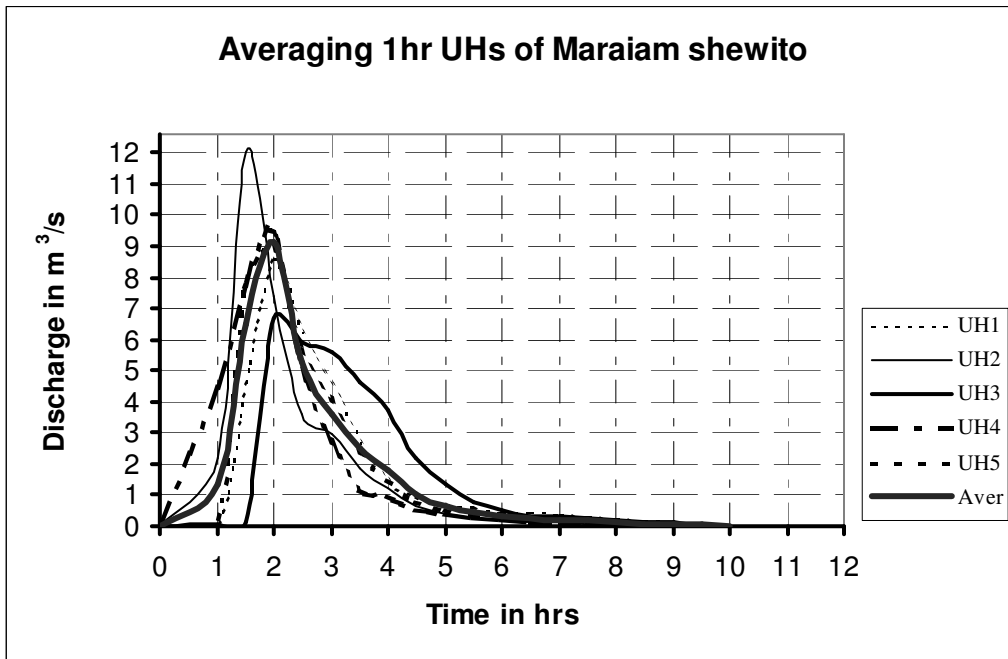
16. Runge G.C.,and D. C.Mongomery (1994). Applied statistics and probability for Engineers. John Wiley & Sons. USA.
17. Paulos Shemelies (1998). Establishing Water Release Rules for Koka Reservoir for Wet Seasons. Addis Ababa University. Addis Ababa.
18. Ramirez Jorge A (2000). Prediction and Modeling of Flood Hydrology and Hydraulics. Colorado State University, Fort Collins, USA.
19. Shaw Elizabeth M (1994). Hydrology in practice. Champan & Hall London, UK.
20. Singh V.P. (1992). Elementary Hydrology. Prentice Hall Englewood Cliff, New Jersey, USA.
21. Singh V.P. (1982). Applied Modeling in catchment hydrology. WRP
22. Solomon Dagnachew (2000). Developments and Application of A Semi Distributed Physically Based Model to Kikuletwa Karangai Catchment. University of Dar-es-salaam,Tanzania.
23. Subramanya K (1984). Engineering Hydrology.Tata McGraw hill New Delhi, India.
24. U.S Army Corps of Engineers (1994) Engineering and design Flood-runoff analysis Manual. Engineer Manual 1119-2-1417 Washington, DC, USA.
25. U.S Army Corps of Engineers U.S Army Corps of Engineers (2000). Hydrologic Modeling System HEC-HMS. Technical Reference Manual.
26. U.S Geological Survey (2000). Equations for Estimating Clark Unit-Hydrograph Parameters for Small Rural Watersheds in Illinios. Water Resources Investigation Report. Urbana, Illinois.

Appendix A: Watershed and their 1hr Unit hydrograph Characteristics

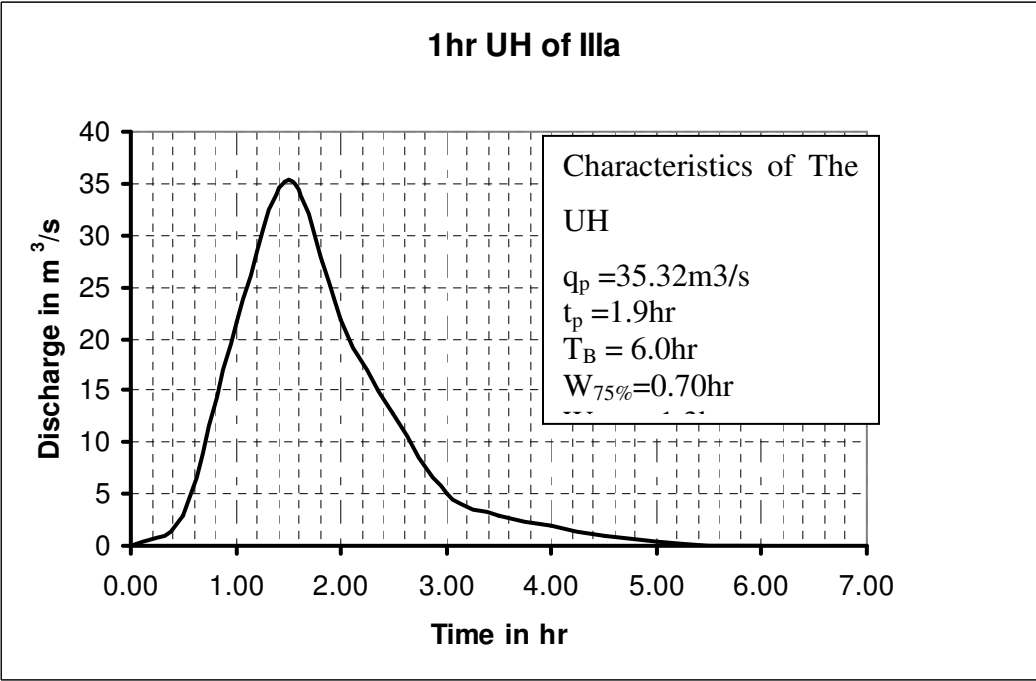
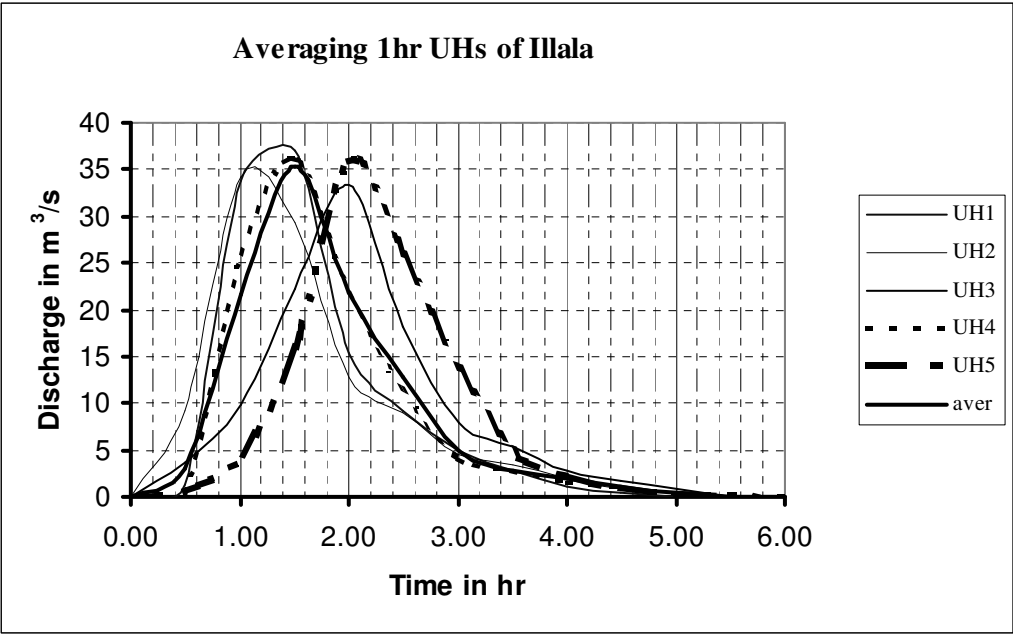
Parameters	Name Rivers					
	M.Shewito	Illala	Suluh	Akaki	Mojo	Gheba
Area (A) in km ²	59.8	190	350	884.4	1264.4	2449
Length (L) in km	9	30	36.8	77.6	76	104
Length (Lc) in km	3.75	16	18.5	51.6	44	48
Slope (s)	0.0109862	0.0160	0.013743	0.007287	0.004397	0.007955
Peak discharge (q _p) in m ³ /s	9.126	35.323	44.243	122.018	79.775	365.656
Peak time t_p (hr)	1.35	1.6	1.5	1.0	2.0	2.0
Observed Lag time (t _l) in hr	1.15	1.9	2.333	5.833	6.3	6.375
Adjusted Lag time (t _{lR}) in hr	0.943	1.728	2.182	5.849	6.338	6.417
Time base (T _b) in hr	8.0	5.5	7	6	14	8
Time width at 50% of q _p (W ₅₀) in hr	1.35	1.3	2	1.6	3.75	1.70
Time width at 75% of q _p (W ₇₅) in hr	0.85	0.70	1.0	0.75	1.9	1.00
Coefficient for lag time C _t	0.437	0.362	0.411	0.646	0.736	0.661
Coefficient for peak discharge C _p	0.063	0.128	0.107	0.292	0.144	0.344
Coefficient for time width of W _{50%} C ₅₀	0.177	0.211	0.214	0.188	0.190	0.218
Coefficient for time width of W _{75%} C ₇₅	0.112	0.114	0.107	0.088	0.096	0.128
Coefficient for time width of base time T _B	1.221	1.022	0.884	0.828	0.883	1.194
Ratio of peak time to base time (T _p /T _b)	0.169	0.291	0.214	0.167	0.143	0.250
Ratio of lag time to base time (T _l /T _b)	0.144	0.316	0.330	0.486	0.450	0.797

Appendix B: Derived Unit Hydrographs

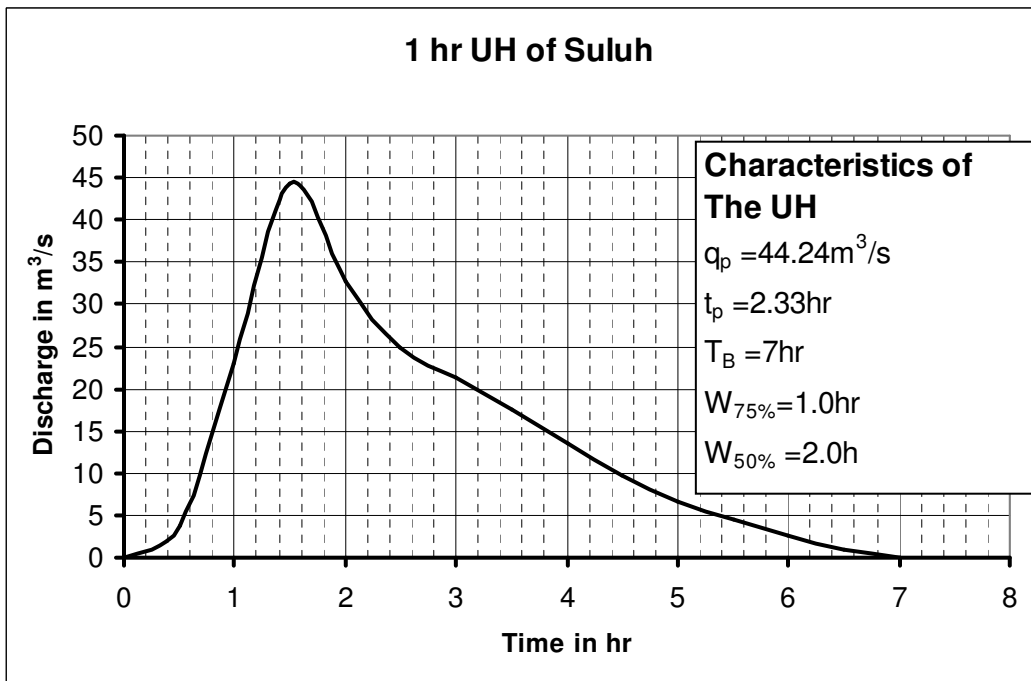
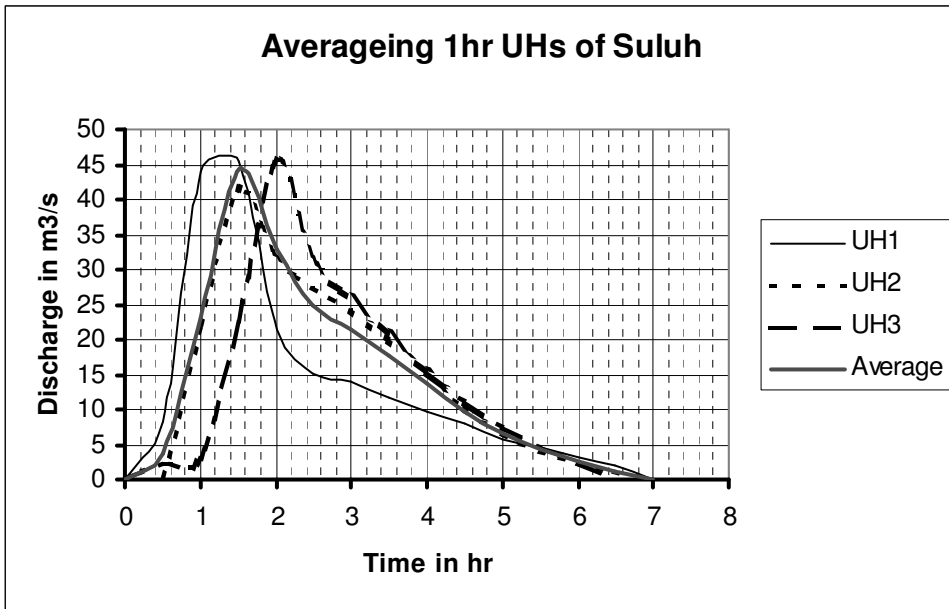
Appendix B1: Mariam Shewito



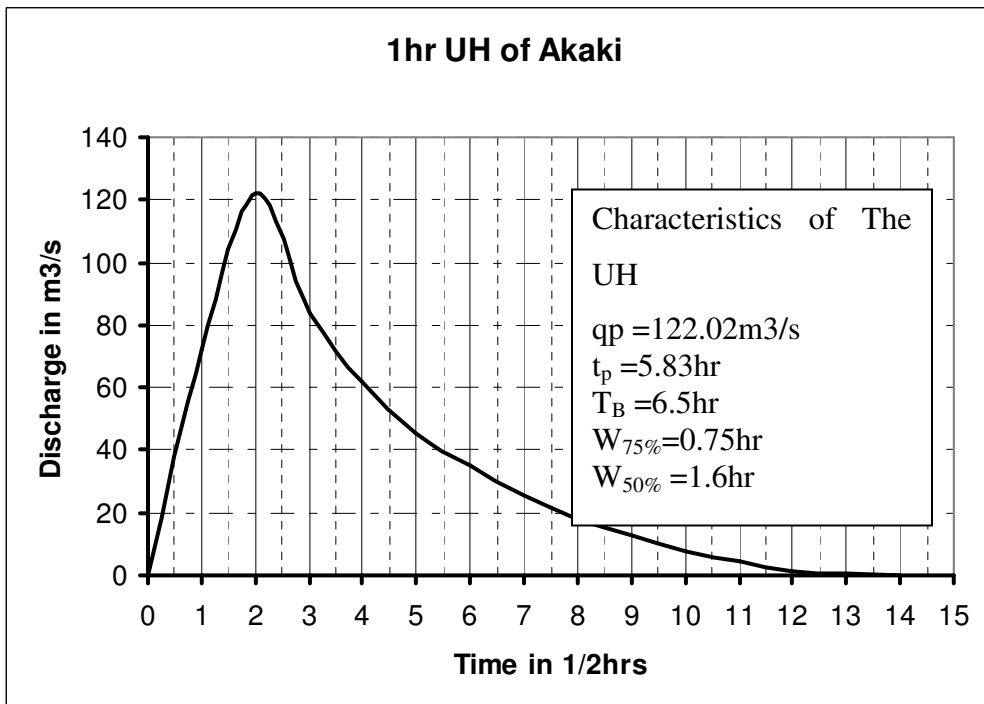
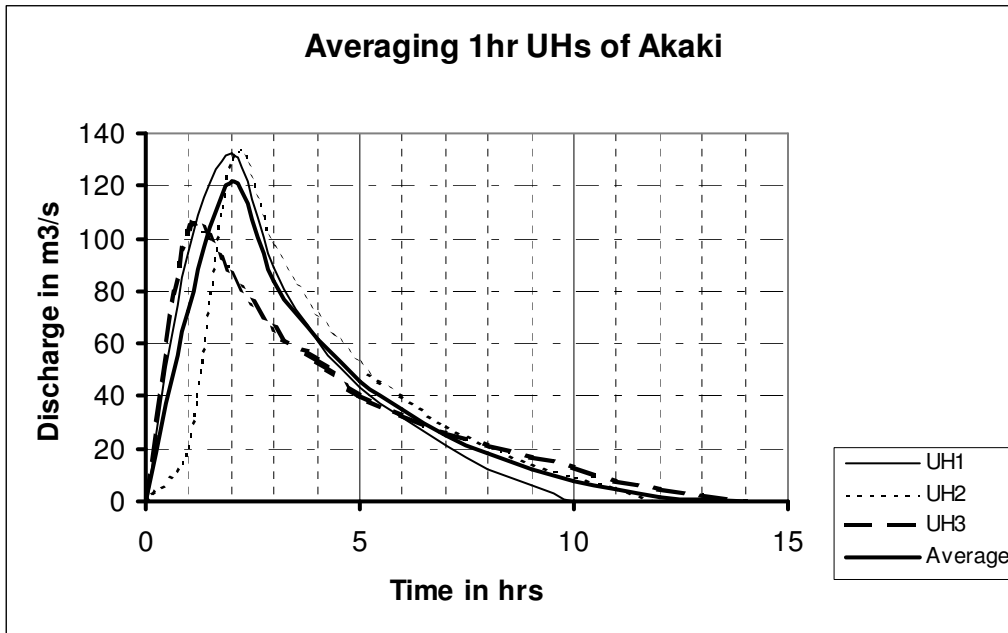
Appendix B2: Illala



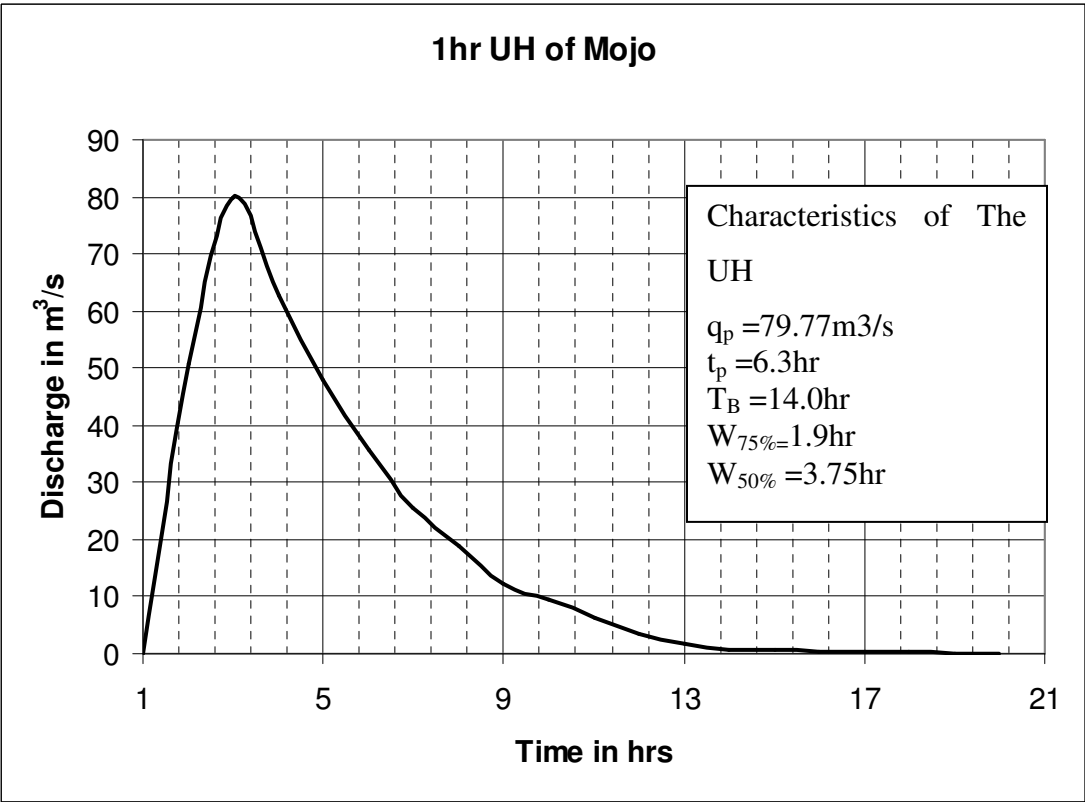
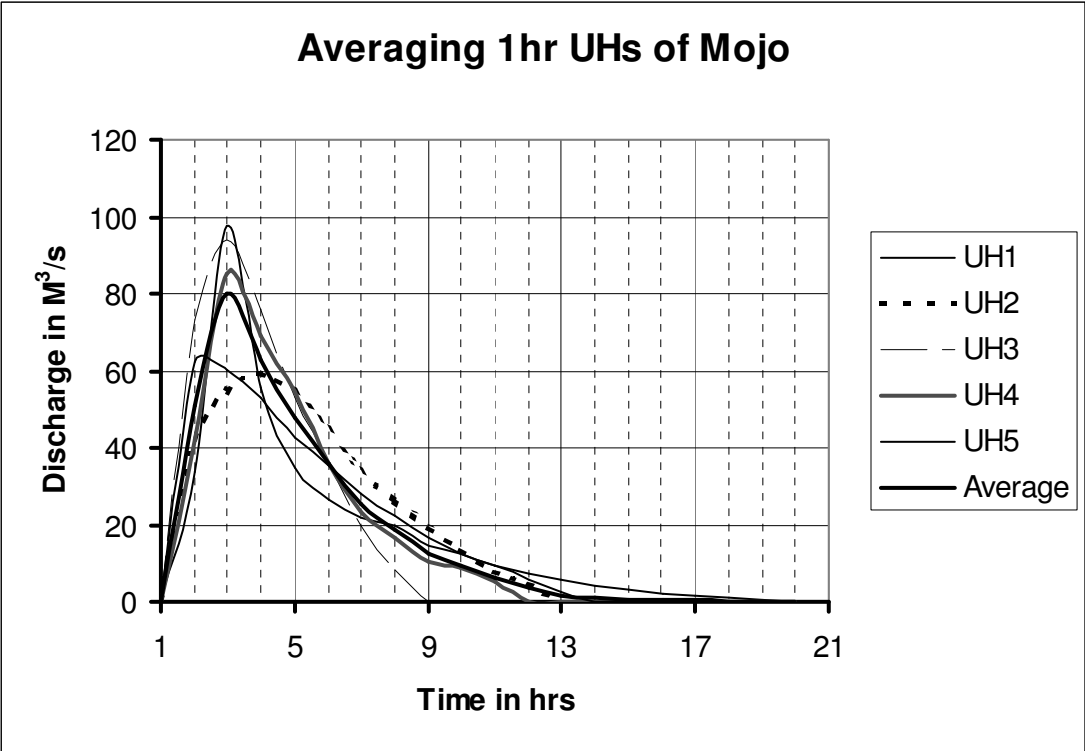
Appendix B3: Suluh

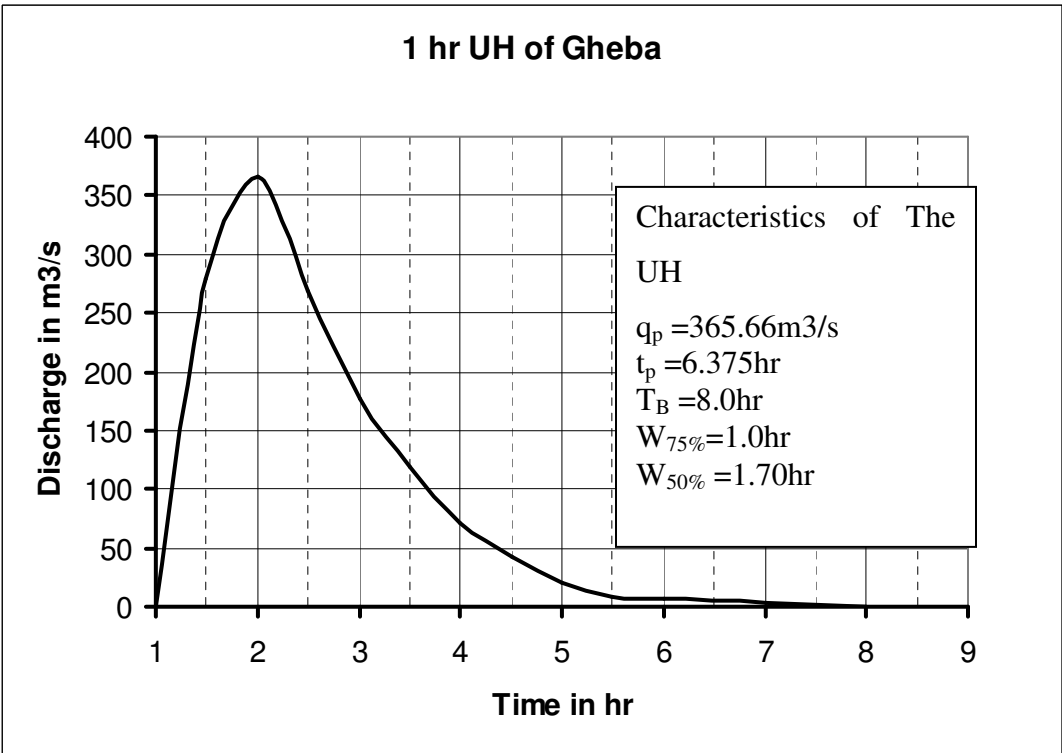
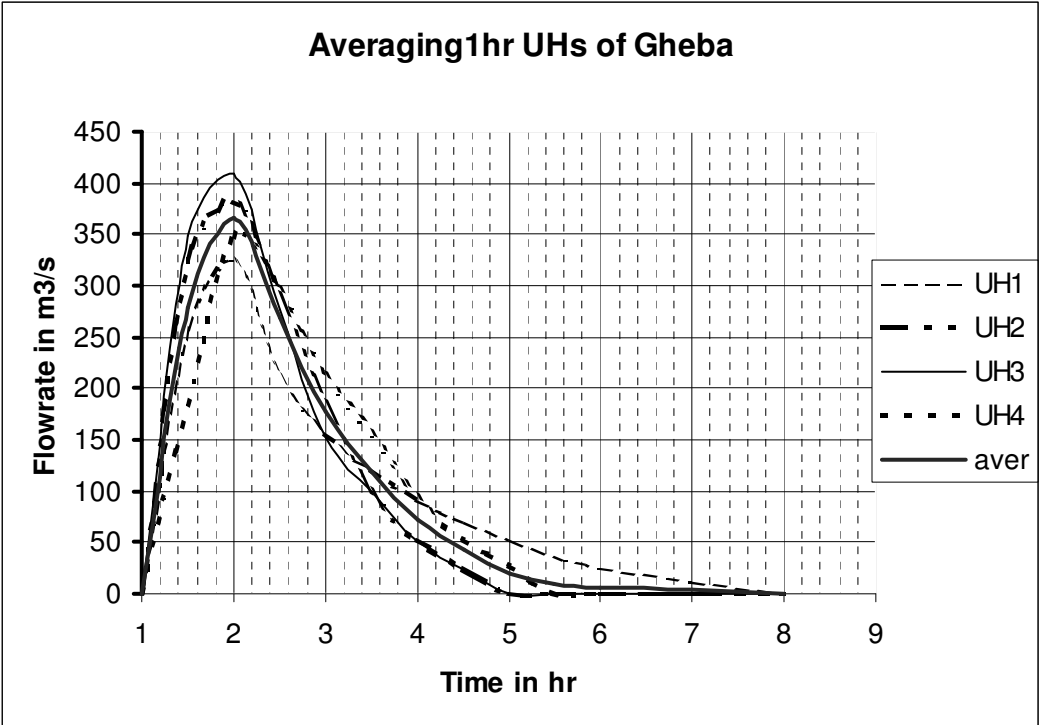


Appendix B4: Akaki



Appendix B5: Mojo





Appendix: C

Computer out put of regression Analysis

Appendix C1:

Regression Analysis: Ct

versus S

The regression equation is
 $Ct = - 1.50 - 0.597 S$

Predictor	Coef	SE Coef	T	P
Constant	-1.4962	0.2008	-7.45	0.002
S	-0.59683	0.09827	-6.07	0.004

S = 0.04515 R-Sq = 90.2% R-Sq(adj) = 87.8%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	1	0.075199	0.075199	36.88	0.004
Residual Error	4	0.008155	0.002039		
Total	5	0.083354			

Appendix C2:Regression Analysis: t₁ versus y

Where $y = (L * L_c)/(S^{0.5})$

The regression equation is
 $t_1 = - 0.895 + 0.352 y$

Predictor	Coef	SE Coef	T	P
Constant	-0.8953	0.1417	-6.32	0.003
y	0.35244	0.03476	10.14	0.001

S = 0.06920 R-Sq = 96.3% R-Sq(adj) = 95.3%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	1	0.49225	0.49225	102.81	0.001
Residual Error	4	0.01915	0.00479		
Total	5	0.51140			

Appendix C3:Regression Analysis: Qp versus t1, A

The regression equation is
 $Q_p = - 1.27 - 0.786 t_1 + 1.29 A$

Predictor	Coef	SE Coef	T	P
Constant	-1.2725	0.9563	-1.33	0.275
t ₁	-0.7863	0.9970	-0.79	0.488
A	1.2945	0.5394	2.40	0.096

S = 0.1723 R-Sq = 93.9% R-Sq(adj) = 89.9%

Analysis of Variance

Source	DF	SS	MS	F	P
Regression	2	1.37648	0.68824	23.19	0.015
Residual Error	3	0.08905	0.02968		
Total	5	1.46552			

Source	DF	Seq SS
t ₁	1	1.20554
A	1	0.17094

Appendix C4:Regression Analysis: Qp versus z

Where $Z = (A/t_1)$

The regression equation is
 $Q_p = - 2.06 + 1.79 z$

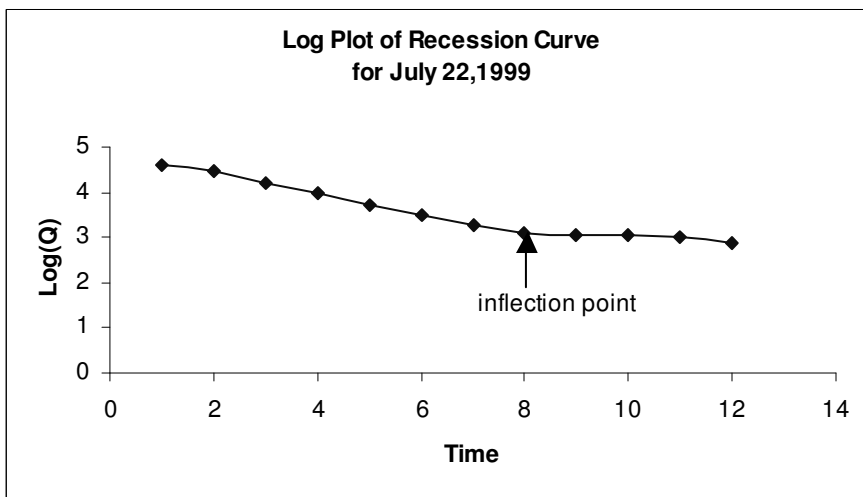
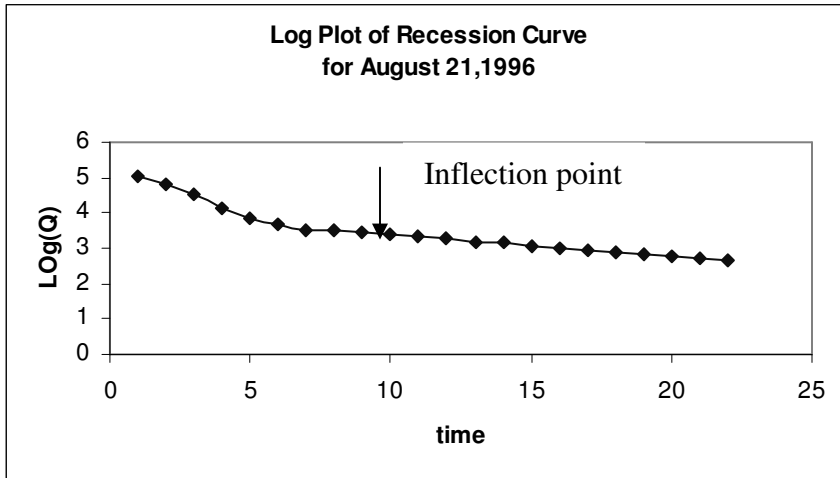
Predictor	Coef	SE Coef	T	P
Constant	-2.0626	0.5809	-3.55	0.024
z	1.7810	0.2669	6.67	0.003

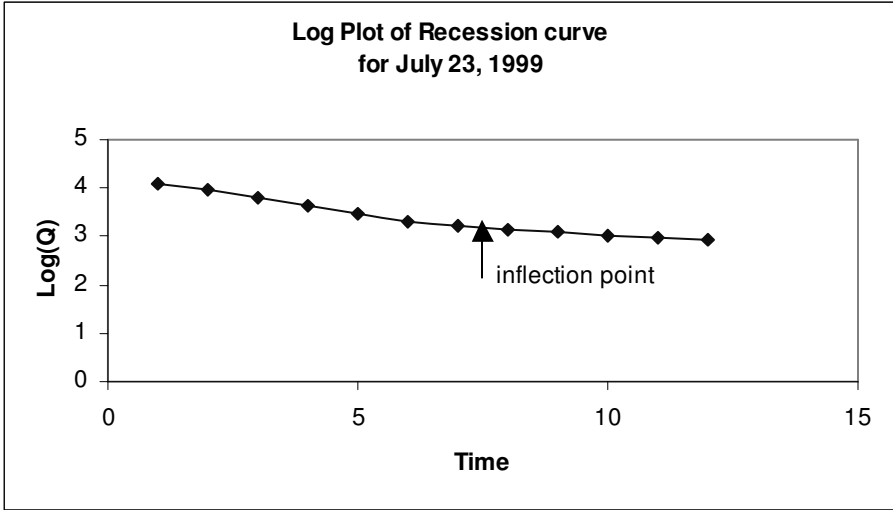
S = 0.1738 R-Sq = 91.8% R-Sq(adj) = 89.7%

Analysis of Variance

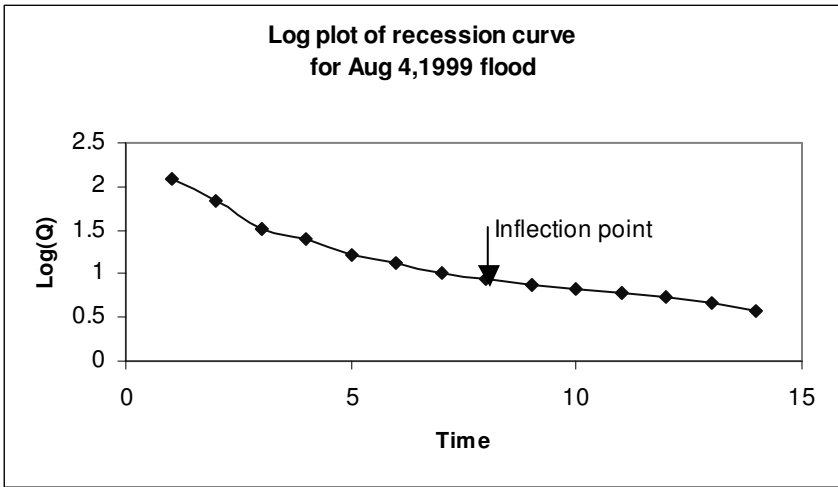
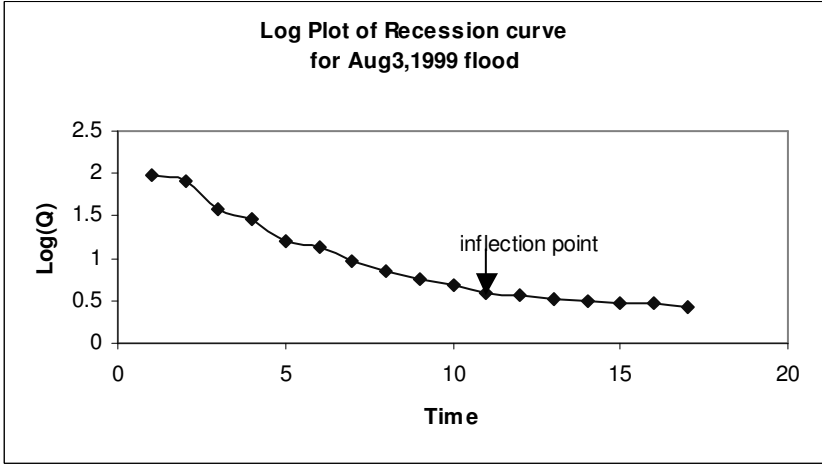
Source	DF	SS	MS	F	P
Regression	1	1.3447	1.3447	44.52	0.003
Residual Error	4	0.1208	0.0302		
Total	5	1.4655			

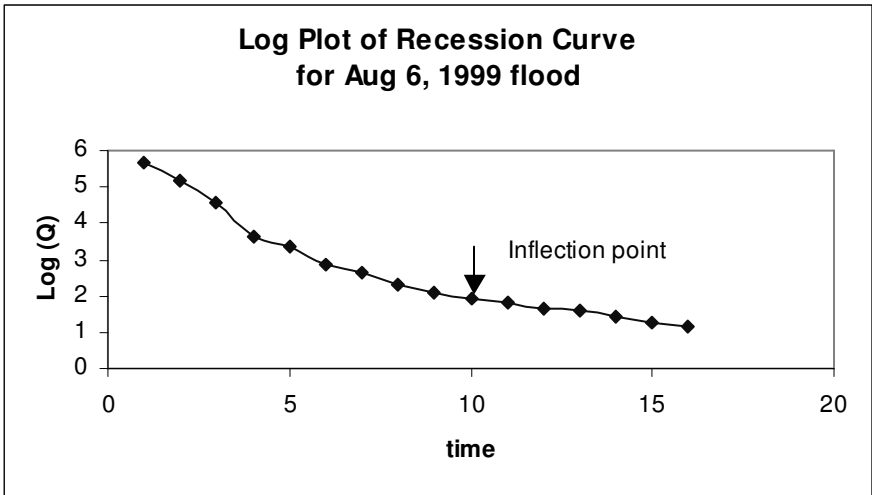
Appendix: - D Log plot of Recession Curves (Samples)
Gheba Mekele D1



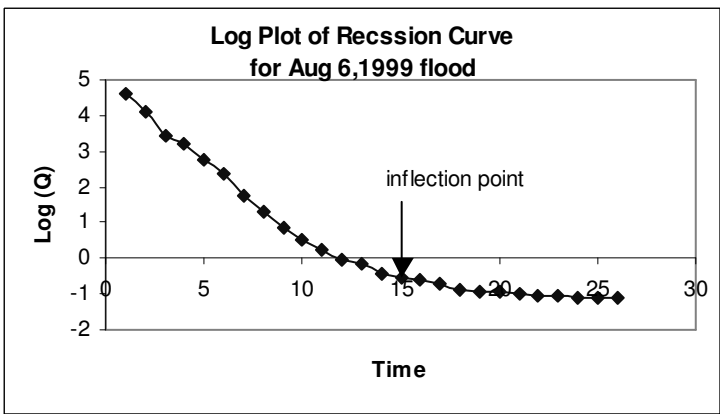
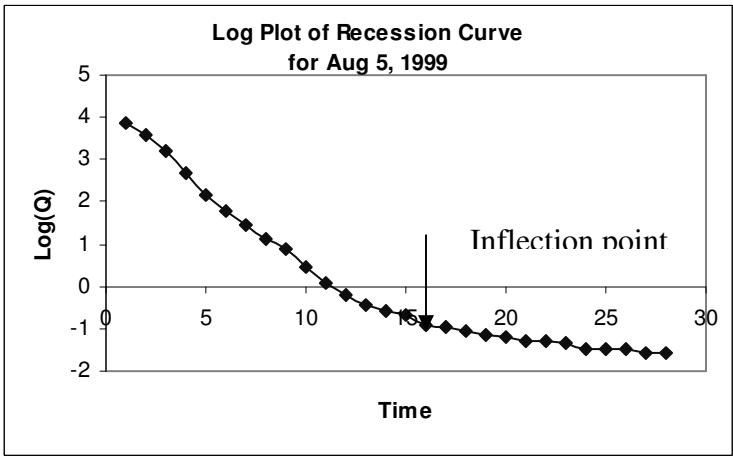


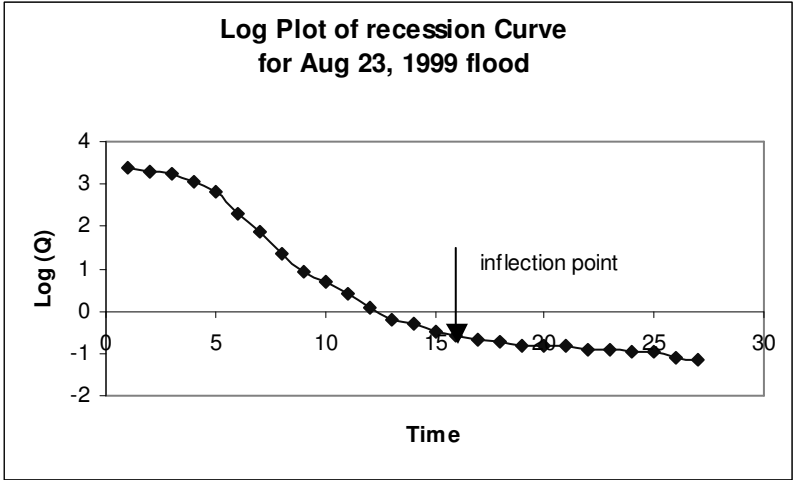
Illala D2



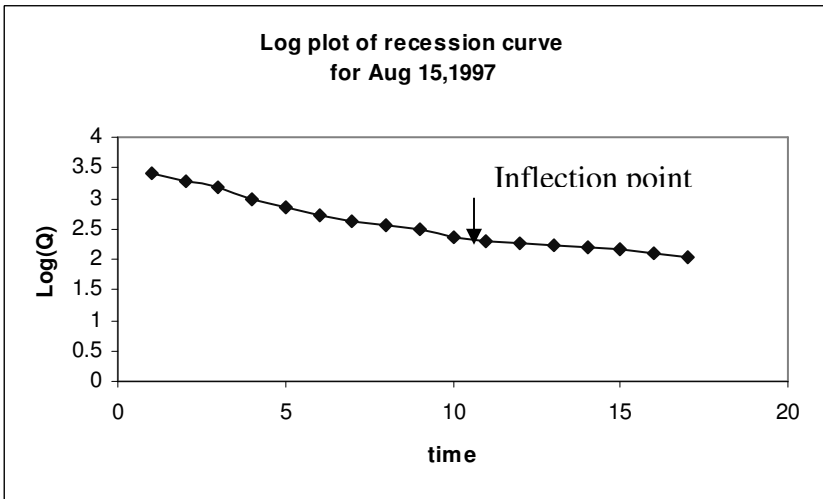
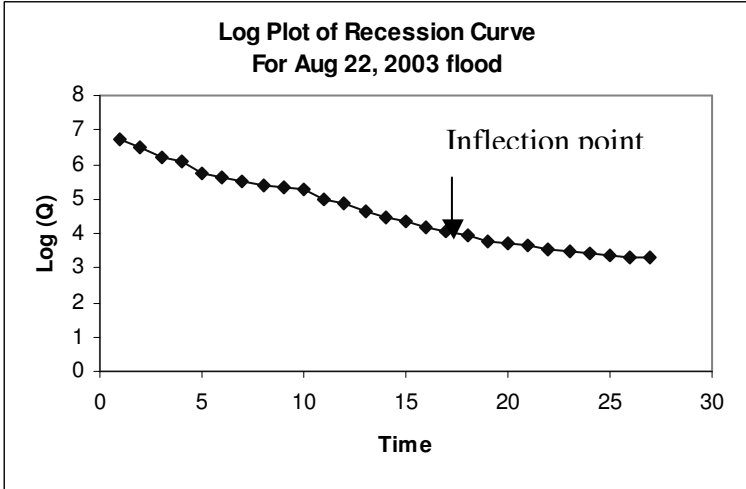


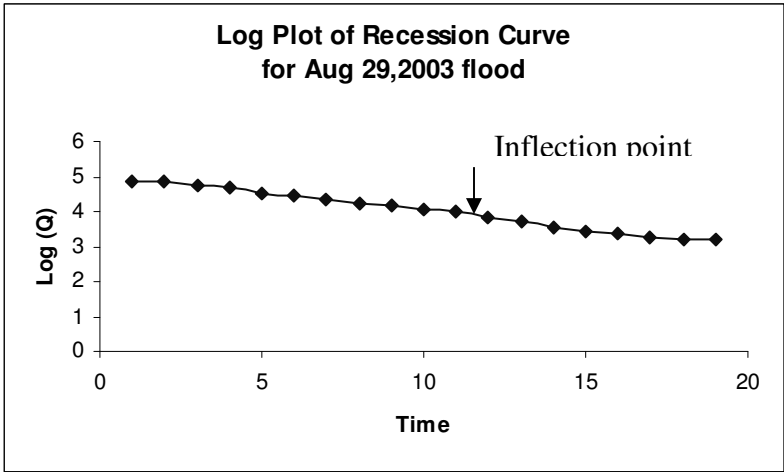
Mariam Shewiot D3



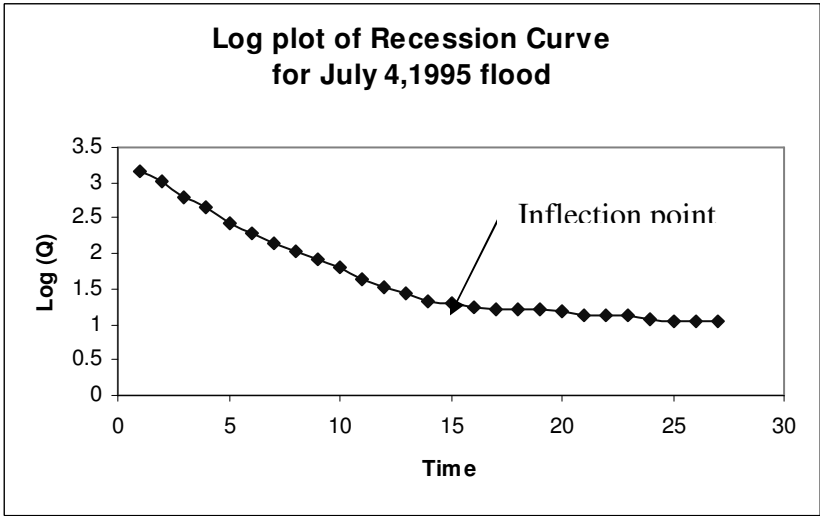


Mojo D4

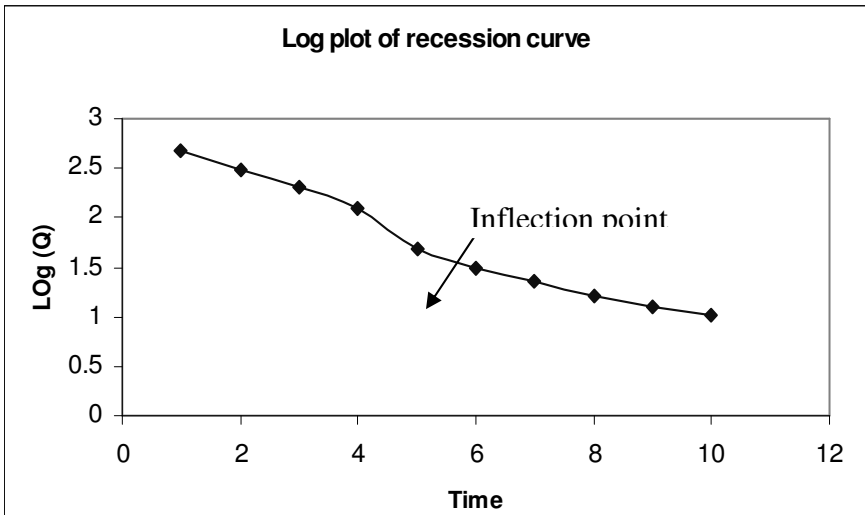




Akaki D5

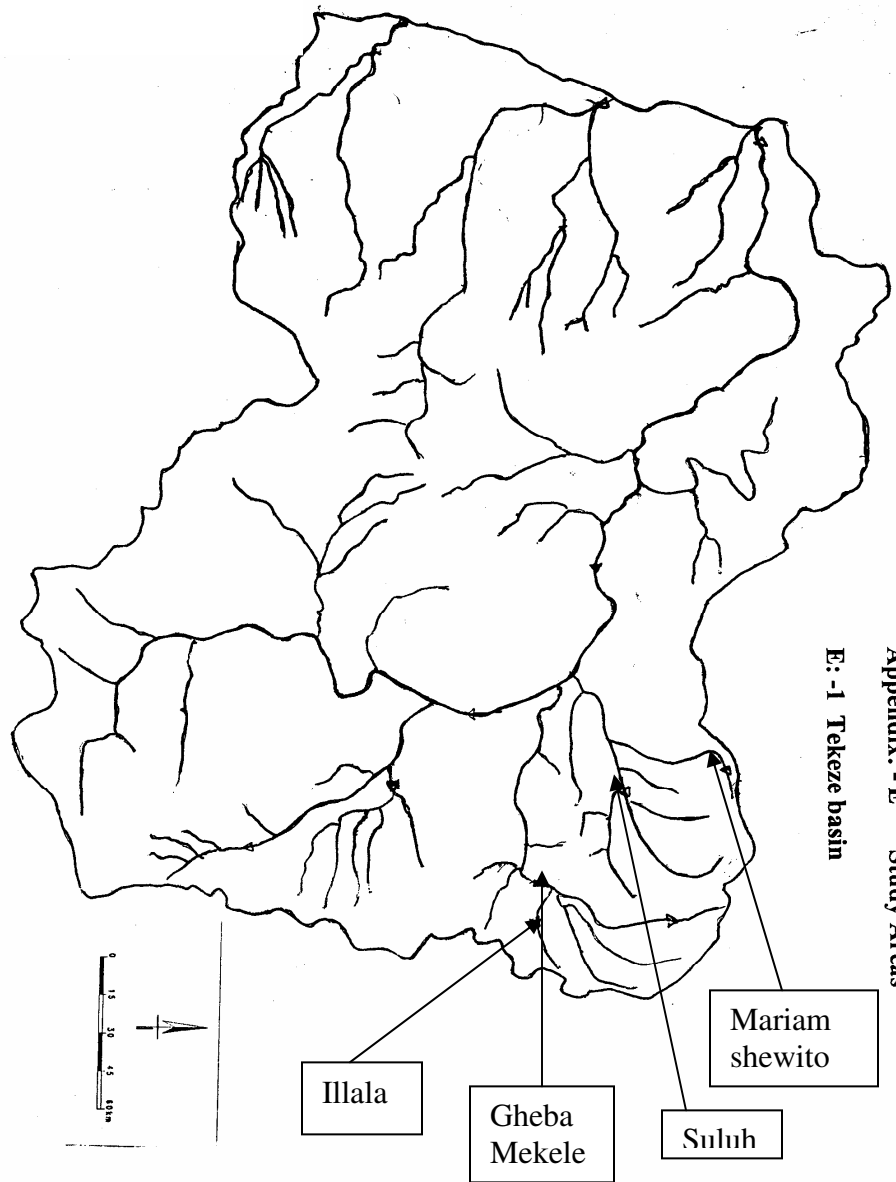


Suluh D6

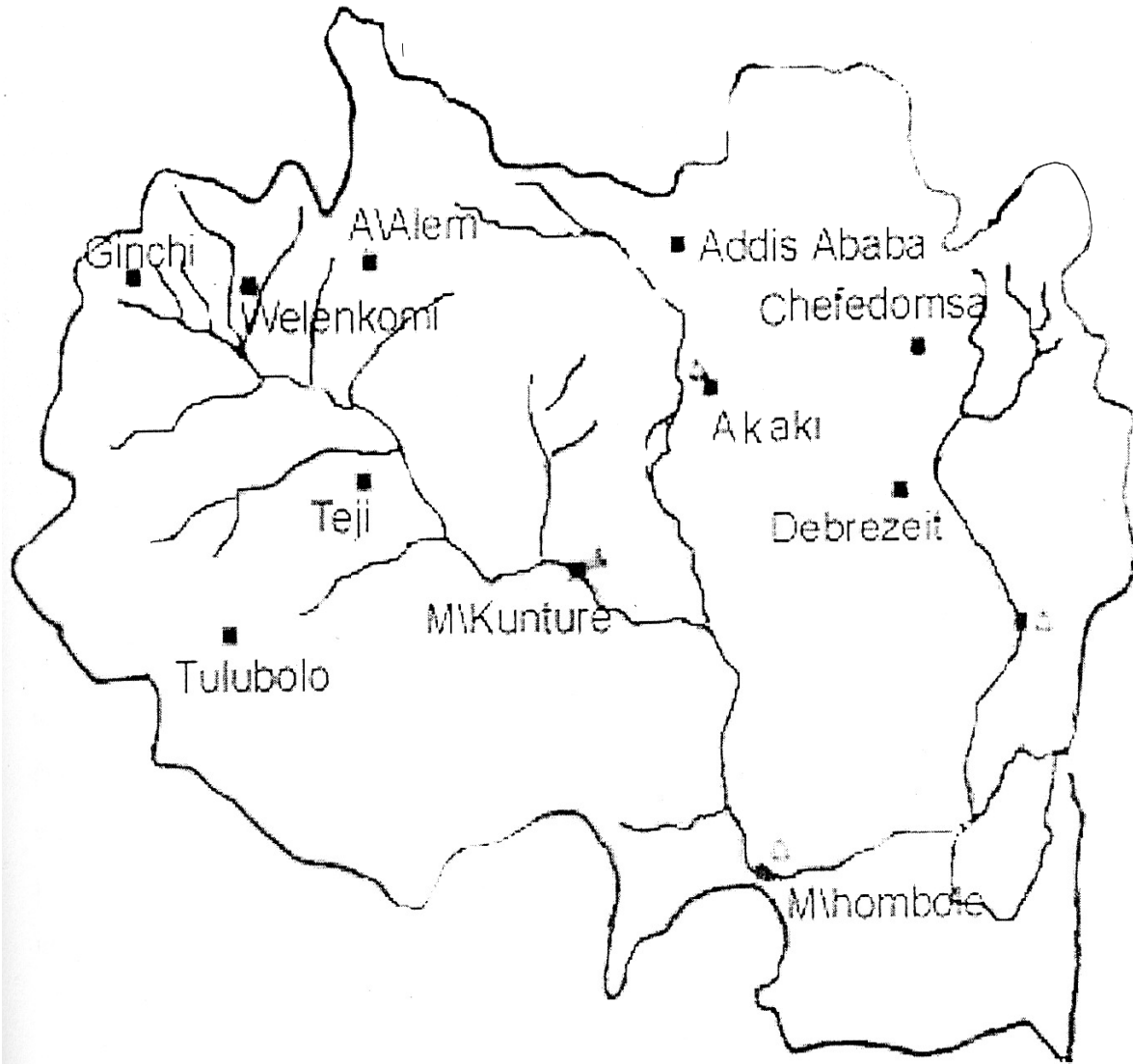


Appendix E: -Study Areas

E: - 1 TekeZe Basin



E: - 2 the Upper Awash Basin



▲ Stream gauging stations

■ Rainfall gauging stations

Appendix F Rating Curves (Samples)

**Appendix: -G Streamflow Data
Gheba Mekele G1**

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug7,1996	8:30P.M	1	14.51
	9:00	1.72	65.36
	9:20	1.86	79.35
	10:00	1.64	57.96
	10:30	1.52	47.67
	11:00	1.44	41.36
	11:30	1.37	36.19
	12:00	1.32	32.71
Aug8,96	12:30AM	1.28	30.05
	1:00	1.24	27.50
	1:30	1.21	25.65
	2:00	1.18	23.87
	2:30	1.15	22.16
	3:00	1.13	21.05
	3:30	1.12	20.50
	4:00	1.11	19.97
	4:30	1.08	18.40
	5:00	1.06	17.38
	5:30	1.04	16.40

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug21,96	10:30P.M	1	14.51
	11:00	2.3	131.91
	11:30	2.46	154.22
	12:00	2.25	125.29
	12:30	1.96	90.16
	1:00	1.68	61.60
	1:30	1.52	47.67
	2:00	1.43	40.60
	2:30	1.34	34.08
	3:00	1.32	32.71
	3:30	1.3	31.36
	4:00	1.28	30.05
	4:30	1.25	28.12
	5:00	1.22	26.26
	5:30	1.19	24.46
	6:00	1.17	23.30
	6:30	1.14	21.60
	7:00	1.12	20.50
	7:30	1.1	19.44
	8:00	1.08	18.40
	8:30	1.05	16.89
	9:00	1.03	15.92
	9:30	1.02	15.44
	10:00	1	14.51

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug4,1999	5:00P.M	2	17.29
	5:30	3.08	108.25
	6:00	3.19	124.15
	6:30	2.94	90.07
	7:00	2.64	58.23
	7:30	2.48	44.85
	8:00	2.3	32.46
	8:30	2.18	25.62
	9:00	2.06	19.82
	9:30	2.04	18.95
	10:00	2.02	18.11
	10:30	2.01	17.69

DATE	TIME IN HR	GAGE HEIGHT METER	DISCHARGE IN M ³ /S
July22,1999	7:40	2	17.29
	8:00	2.98	95.03
	8:10	3.02	100.18
	8:30	2.93	88.85
	9:00	2.72	65.82
	9:30	2.58	52.93
	10:00	2.43	41.13
	10:30	2.3	32.46
	11:00	2.19	26.15
	11:30	2.12	22.60
	12:00	2.1	21.65
July23,99	12:30A.M	2.08	20.72
	1:00	2.06	19.82
	1:30	2.02	18.11

DATE	TIME IN HR	GAGE HEIGHT	DISCHARGE IN M ³ /S
July,23,99	11:10P.M	2.02	18.11
	11:30	2.62	56.43
	11:35	2.65	59.14
	12:00	2.58	52.93
July,24,99	12:30A.M	2.47	44.09
	1:00	2.39	38.32
	1:30	2.29	31.84
	2:00	2.22	27.78
	2:30	2.18	25.62
	3:00	2.13	23.08
	3:30	2.12	22.60
	4:00	2.08	20.72
	4:30	2.05	19.38
	5:00	2.03	18.53

Illala

G2

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
July25,1999	8:00P.M	0.16	0.221
	9:00	0.18	0.335
	10:00	0.2	0.476
	10:30	0.36	2.702
	11:00	1.24	61.228
	11:30	1.29	67.301
	12:00	0.9	28.222
July26,99	12:30	0.74	17.437
	1:00	0.58	9.462
	1:30	0.48	5.816
	2:00	0.36	2.702
	2:30	0.3	1.625
	3:00	0.26	1.072
	3:30	0.24	0.843
	4:00	0.22	0.645
	4:30	0.21	0.557
	5:30	0.19	0.402
	6:00	0.175	0.304
	7:00	0.16	0.221
	8:00	0.15	0.174

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug4,1999	3:00P.M	0.4	3.593
	3:30	0.39	3.357
	4:00	0.38	3.130
	4:30	0.72	16.290
	5:00	1.02	38.260
	5:30	1.4	81.807
	6:00	1.65	120.806
	6:30	1.3	68.554
	7:00	0.96	33.027
	7:30	0.85	24.539
Aug4,99	8:00	0.72	16.290
	8:30	0.66	13.110
	9:00	0.6	10.312
	9:30	0.56	8.653
	10:00	0.53	7.515
	10:30	0.51	6.806
	11:00	0.49	6.136
	11:30	0.465	5.353
	12:00	0.44	4.630
Aug5,99	1:00	0.41	3.839

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug3,1999	8:00P.M	0.41	3.839
	8:30	0.9	28.222
	9:00	1.49	94.869
	9:30	1.4	81.807
	10:00	1.02	38.260
	10:30	0.9	28.222
	11:00	0.72	16.290
	11:30	0.67	13.613
	12:00	0.58	9.462
Aug4,99	1:00A.M	0.52	7.155
	1:30	0.48	5.816
	2:00	0.45	4.912
	3:00	0.41	3.839
	4:00	0.4	3.593
	5:00	0.39	3.357
	6:00	0.38	3.130
	7:00	0.37	2.911
	8:00	0.37	2.911
	9:00	0.36	2.702

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug6,1999	3:30P.M	0.39	3.357
	4:00	0.46	5.204
	4:30	2.1	213.172
	4:50	2.42	297.124
	5:00	1.96	181.288
	5:30	1.5	96.388
	6:00	1.02	38.260
	6:30	0.9	28.222
	7:00	0.74	17.437
	7:30	0.68	14.127
	8:00	0.6	10.312
	8:30	0.55	8.264
	9:00	0.51	6.806
	9:30	0.49	6.136
	10:00	0.46	5.204
	10:30	0.45	4.912
	11:00	0.42	4.093
	12:00	0.4	3.593
	1:00A.M	0.38	3.130

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug16,1999	10:30P.M	0.26	1.072
	11:00	0.5	6.466
	11:30	1.02	38.260
	12:00	1.7	129.638
Aug17,99	12:30A.M	2.38	285.772
	1:00	2.08	208.440
	1:30	1.6	112.323
	2:00	1.08	43.928
	2:30	0.86	25.253
	3:00	0.72	16.290
	3:30	0.64	12.135
	4:00	0.56	8.653
	4:30	0.52	7.155
	5:00	0.48	5.816
	5:30	0.45	4.912
	6:00	0.42	4.093
	7:00	0.37	2.911
	8:00	0.34	2.308
	9:00	0.32	1.950
	10:00	0.3	1.625
	11:00	0.3	1.625

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
sep1,1998	4:00P.M	0.34	2.308
	4:30	0.42	4.093
	5:00	1.54	102.598
	5:30	1.38	79.051
	6:00	1	36.468
	6:30	0.86	25.253
	7:00	0.7	15.187
	7:30	0.64	12.135
	8:00	0.56	8.653
	8:30	0.53	7.515
	9:00	0.48	5.816
	10:00	0.44	4.630
	11:00	0.42	4.093
	12:00	0.39	3.357
sep2,98	1:0A.M	0.38	3.130
	2:00	0.37	2.911
	3:00	0.37	2.911
	4:00	0.36	2.702

Suluh G3

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug29,1999	3:30P.M	1.1	2.766
	4:00	1.4	7.002
	4:30	2.15	24.446
	5:00	2.18	25.332
	5:30	1.74	13.757
	6:00	1.61	10.940
	6:30	1.59	10.532
	7:00	1.54	9.542
	7:30	1.5	8.781
	8:00	1.46	8.048
	8:30	1.41	7.172
	9:00	1.38	6.667
	9:30	1.35	6.179
	10:00	1.32	5.708
	11:00	1.27	4.958
	12:00	1.25	4.672
Aug30,99	1:00A.M	1.23	4.392
	2:00	1.22	4.256
	3:00	1.2	3.988
	4:00	1.19	3.857
	5:00	1.17	3.601

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
July15,1999	6:00P.M	1.1	2.766
	6:30	1.12	2.995
	7:00	1.52	9.158
	7:30	1.78	14.681
	8:00	1.66	11.990
	9:00	1.57	10.131
	10:00	1.46	8.048
	11:00	1.3	5.402
	12:00	1.23	4.392
July16,99	1:00A.M	1.19	3.857
	2:00	1.15	3.353
	3:00	1.12	2.995

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
July23,99	2:00P.M	1.1	2.766
	3:00	1.15	3.353
	3:30	1.17	3.601

Continued-----			
	6:00	1.15	3.353
	7:00	1.14	3.231
	8:00	1.13	3.112
	9:00	1.12	2.995
	10:00	1.1	2.766

Continued-----			
	4:00	1.44	7.6923
	4:20	1.68	12.4218
	5:00	1.55	9.737
	6:00	1.44	7.692
	7:00	1.33	5.863
	8:00	1.25	4.672
	9:00	1.19	3.857
	10:00	1.16	3.476
	11:00	1.13	3.112
	12:00	1.1	2.766

Mariam Shewito G4

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug5,1992	4:00P.M	0.47	0.134
	4:30	0.54	0.356
	5:00	0.86	4.336
	5:30	0.9	5.338
	6:00	1.4	33.710
	6:30	1.54	48.660
	7:00	1.42	35.622
	7:30	1.3	25.187
	8:00	1.14	14.800
	8:30	1	8.507
	9:00	0.92	5.893
	10:00	0.86	4.336
	11:00	0.8	3.086
	12:00	0.76	2.404
Aug6,92	1:00A.M	0.7	1.586
	2:00	0.65	1.068
	3:00	0.62	0.821
	4:00	0.595	0.647
	5:00	0.58	0.556
	6:00	0.57	0.500
	7:00	0.55	0.400
	8:00	0.545	0.378
	9:00	0.54	0.356
	10:00	0.53	0.315
	11:00	0.525	0.296
	12:00	0.52	0.277
	1:00P.M	0.52	0.277
	2:00	0.515	0.260
	3:00	0.505	0.227
	4:00	0.505	0.227
	5:00	0.505	0.227
	6:00	0.5	0.211
	7:00	0.5	0.211

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug6,1992	8:00P.M	0.5	0.211
	8:30	1.2	18.261
	9:00	1.87	100.465
	9:30	1.64	61.737
	10:00	1.36	30.098
	10:30	1.3	25.187
	11:00	1.16	15.899
	11:30	1.05	10.487
	12:00	0.92	5.893
Aug7,92	12:30A.M	0.83	3.675
	1:00	0.76	2.404
	2:00	0.71	1.707
	3:00	0.67	1.259
	4:00	0.64	0.981
	5:00	0.625	0.859
	6:00	0.595	0.647
	7:00	0.585	0.585
	8:00	0.58	0.556
	9:00	0.57	0.500
	10:00	0.555	0.424
	11:00	0.55	0.400
	12:00	0.55	0.400
	1:00P.M	0.545	0.378
	2:00	0.54	0.356
	3:00	0.54	0.356
	4:00	0.535	0.335
	5:00	0.535	0.335
	6:00	0.535	0.335

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug23,1992	5:00P.M	0.52	0.277
	5:30	0.56	0.448
	6:00	0.64	0.981
	6:30	1.36	30.098
	7:00	1.32	26.759
	7:30	1.3	25.187
	8:00	1.24	20.851
	8:30	1.18	17.052
	9:00	1.04	10.068
	9:30	0.94	6.486
	10:00	0.84	3.887
	10:30	0.77	2.564
	11:00	0.73	1.966
	12:00	0.69	1.471
Aug24,92	1:00A.M	0.65	1.068
	2:00	0.62	0.821
	3:00	0.61	0.748
	4:00	0.59	0.616
	5:00	0.58	0.556
	6:00	0.57	0.500
	7:00	0.565	0.474
	8:00	0.56	0.448
	9:00	0.56	0.448
	10:00	0.56	0.448
	11:00	0.55	0.400
	12:00	0.55	0.400
	1:00P.M	0.545	0.378
	2:00	0.545	0.378
	3:00	0.535	0.335
	4:00	0.53	0.315

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug29,1992	3:00P.M	0.55	0.400
	3:30	0.55	0.400
	4:00	1.28	23.680
	4:30	1.47	40.721
	5:00	1.55	49.874
	5:30	1.32	26.759
	6:00	1.14	14.800
	6:30	0.95	6.798
	7:00	0.91	5.611
	7:30	0.82	3.471
	8:00	0.78	2.731
	9:00	0.73	1.966
	10:00	0.69	1.471
	11:00	0.67	1.259
	12:00	0.64	0.981
Aug30,92	1:00A.M	0.635	0.939
	2:00	0.62	0.821
	3:00	0.61	0.748
	4:00	0.61	0.748
	5:00	0.6	0.680
	6:00	0.595	0.647
	7:00	0.59	0.616

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug21,92	6:00P.M	0.52	0.2773
	6:30	0.9	5.3382
	7:00	1.34	28.3952
	7:30	1.53	47.4670
	8:00	1.42	35.6221
	8:30	1.28	23.6800
	9:00	1.12	13.7533
	9:30	1	8.5069
	10:00	0.86	4.3364
	10:30	0.77	2.5639
	11:00	0.73	1.9663
	12:00	0.69	1.4712
Aug22,92	1:00A.M	0.65	1.0684
	2:00	0.62	0.8209
	3:00	0.61	0.7480
	4:00	0.6	0.6796
	5:00	0.59	0.6155
	6:00	0.58	0.5558
	7:00	0.58	0.5558

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug17,1997	1:00P.M	1	7.239
	1:30	1.92	56.444
	2:00	2.05	67.375
	2:30	2.1	71.848
	3:00	2.14	75.534
	3:30	2.17	78.362
	4:00	2.14	75.534
	4:30	2.12	73.679
	5:00	2.06	68.257
	5:30	2	63.051
	6:00	1.94	58.060
	6:30	1.86	51.738
	7:00	1.8	47.245
	7:30	1.76	44.368
	8:00	1.68	38.896
	8:30	1.65	36.940
	9:00	1.6	33.798
	9:30	1.56	31.388
	10:00	1.49	27.395
	10:30	1.46	25.770
	11:00	1.43	24.197
	11:30	1.4	22.675
	12:00	1.37	21.205
Aug18,97	1:00A.M	1.31	18.419
	2:00	1.26	16.252
	3:00	1.22	14.621
	4:00	1.18	13.079
	5:00	1.16	12.342
	6:00	1.12	10.934
	7:00	1.1	10.263
	8:00	1.07	9.298
	9:00	1.04	8.383
	10:00	1.01	7.517

Mojo G5

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug15,1997	1:00P.M	1.26	16.252
	2:00	1.25	15.836
	3:00	1.24	15.425
	4:00	1.4	22.675
	5:00	1.54	30.219
	6:00	1.48	26.848
	7:00	1.42	23.684
	8:00	1.34	19.786
	9:00	1.28	17.102
	10:00	1.24	15.425
	11:00	1.2	13.839
	12:00	1.18	13.079
Aug16,97	1:00A.M	1.15	11.981
	2:00	1.11	10.595
	3:00	1.09	9.936
	4:00	1.08	9.614
	5:00	1.07	9.298
	6:00	1.06	8.988
	7:00	1.05	8.683
	8:00	1.03	8.089
	9:00	1.01	7.517

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug22,03	7:00A.M	1.16	12.342
	7:30	1.18	13.079
	8:00	4	356.054
	8:30	5.48	737.335
	9:00	5.8	838.727
	9:30	5.2	654.188
	10:00	4.6	493.392
	10:30	4.36	435.659
	11:00	3.8	315.434
	11:30	3.6	277.370
	12:00	3.44	248.752
	12:30	3.3	225.040
	1:00	3.2	208.860
	2:00	3.08	190.273
	3:00	2.8	150.401
	4:00	2.64	129.801
	5:00	2.44	106.266
	6:00	2.28	89.197
	7:00	2.15	76.471
	8:00	2.02	64.763
	9:00	1.92	56.444
	10:00	1.84	50.217
	11:00	1.76	44.368
	12:00	1.7	40.229
Aug23,03	1:00A.M	1.66	37.586
	2:00	1.61	34.415
	3:00	1.57	31.982
	4:00	1.55	30.801
	5:00	1.53	29.642
	6:00	1.5	27.948

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
Aug29,03	8:00A.M	1.39	22.180
	9:00	1.39	22.180
	9:30	1.64	36.300
	10:00	2.44	106.266
	10:30	2.6	124.897
	11:00	2.66	132.289
	11:30	2.62	127.337
	12:00	2.52	115.386
	12:30	2.44	106.266
	1:00	2.32	93.318
	1:30	2.26	87.173
	2:00	2.16	77.414
	2:30	2.08	70.041
	3:00	2	63.051
	3:30	1.93	57.249
	4:00	1.88	53.283
	5:00	1.78	45.795
	6:00	1.7	40.229
	7:00	1.62	35.037
	8:00	1.56	31.388
	9:00	1.51	28.507
	10:00	1.47	26.306
	11:00	1.44	24.716
	12:00	1.44	24.716

DATE	TIME IN HR	GAGE HEIGHT IN METER
Aug27,97	10:00A.M	0.95
	11:00	0.95
	12:00	0.95
	1:00P.M	1.71
	2:00	1.7
	3:00	1.65
	4:00	1.57
	5:00	1.51
	6:00	1.45
	7:00	1.4
	8:00	1.35
	9:00	1.31
	10:00	1.29
	11:00	1.255
	12:00	1.23
Aug28,97	1:00A.M	1.21
	2:00	1.195
	3:00	1.19
	4:00	1.18

Akaki G6

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
July1,95	5:00P.M	0.67	3.285
	5:30P.M	0.94	8.828
	6:00	1.82	37.180
	6:30	1.6	28.858
	7:00	1.42	22.617
	7:30	1.29	18.452
	8:00	1.2	15.747
	8:30	1.11	13.195
	9:00	1.05	11.584
	9:30	1	10.299
	10:00	0.96	9.309
	10:30	0.93	8.590
	11:00	0.9	7.892
	11:30	0.88	7.438
	12:00	0.86	6.994
	12:30	0.85	6.775
	1:00	0.84	6.559
	1:30	0.82	6.134
	2:00	0.81	5.925
	3:00	0.79	5.516
	4:00	0.77	5.117
	5:00	0.75	4.728
	6:00	0.74	4.538
	7:00	0.72	4.166
	8:00	0.71	3.984
	9:00	0.7	3.805
	10:00	0.69	3.629
	11:00	0.69	3.629

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
July4,95	7:00P.M	0.64	2.793
	7:30	1.455	23.789
	8:00	1.35	20.337
	8:30	1.22	16.335
	9:00	1.14	14.028
	9:30	1.04	11.323
	10:00	0.98	9.800
	10:30	0.93	8.590
	11:00	0.89	7.664
	11:30	0.85	6.775
	12:00	0.82	6.134
July5,95	1:00AM	0.77	5.117
	1:30	0.74	4.538
	2:00	0.72	4.166
	3:00	0.7	3.805
	4:00	0.69	3.629
	5:00	0.68	3.455
	6:00	0.675	3.370
	7:00	0.675	3.370
	8:00	0.675	3.370
	9:00	0.67	3.285
	10:00	0.66	3.118

DATE	TIME IN HR	GAGE HEIGHT IN METER	DISCHARGE IN M ³ /S
June29,95	5:30P.M	0.62	2.481
	6:00	1.72	33.306
	6:30	2.04	46.216
	7:00	1.72	33.306
	7:30	1.49	24.981
	8:00	1.34	20.019
	8:30	1.25	17.231
	9:00	1.16	14.593
	9:30	1.08	12.381
	10:00	1.04	11.323
	10:30	0.99	10.048
	11:00	0.94	8.828
	11:30	0.98	9.800
	12:00	1.04	11.323

Appendix: -H Rainfall Data

1. Station Mekele **2. Region** Tigray **3. Element** Rainfall intensity

4. Month August **5. Year** 1996

H1

Time	Date															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0__1																
1__2																
2__3																
3__4												0.4				
4__5																
5__6																
6__7																
7__8																
8__9																
9__10																
10__11																
11__12																
12__13																
13__14																
14__15	4.5							4.0							3.5	
15__16								6.2	0.3							
16__17									0.4				6.4			
17__18							5.0		0.2							
18__19					2.0	3.2										
19__20					0.4						5.0					
20__21										4.8						
21__22																
22__23																
23__24																
H in 1 hr	4.5	-	-	-	2.0	3.2	5.0	6.2	-	4.8	5.0	0.4	-	6.4	3.5	-

From Day 2 to 4 no Rain fall

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Mekele **2. Region** Tigray **3. Element** Rainfall intensity

4. Month August 5. Year 1996

Time	Date															
	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	
0__1								10.0								
1__2																
2__3																
3__4																
4__5				3.0												
5__6																
6__7																
7__8					0.8											
8__9				4.5	4.0											
9__10																
10__11																
11__12																
12__13																
13__14													0.3			
14__15															6.0	
15__16																
16__17				2.4												
17__18																
18__19						1.4										
19__20				2.4									2.0			
20__21												1.0				
21__22																
22__23																
23__24																
H in 1 hr				4.5	4.0	1.4		10.0	--	--	--	1.0	2.0		6.0	

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Mekele **2. Region** Tigray **3. Element** Rainfall intensity

4. Month July **5. Year** 1999

Time	Date															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
0__1																
1__2																
2__3															4.4	
3__4																
4__5																
5__6											2.0				0.3	
6__7											2.2					
7__8																
8__9											0.5					
9__10																
10__11											2.0					
11__12																
12__13											3.0					
13__14																
14__15					2.6											
15__16																
16__17								5.7	10.0	0.4						
17__18										0.8						
18__19						18.2					1.0					
19__20																
20__21																
21__22															7.0	
22__23						1.8										
23__24																
H in 1 hr	--	--	--	--	2.6	18.2	--	5.7	10.0	0.8	3.0	--	--	--	7.0	--

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Mekele **2. Region** Tigray **3. Element** Rainfall intensity

4. Months July **5. Year** 1999

Time	Date															
	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	
0__1									0.3							
1__2							0.5		0.4							
2__3																
3__4																
4__5																
5__6																
6__7																
7__8																
8__9																
9__10																
10__11																
11__12																
12__13																
13__14																
14__15																
15__16										0.9						
16__17					1.0						6.7	5.8			1.5	
17__18				7.0												
18__19	0.4	0.4	2.0				2.0		5.0		0.8			12.0		
19__20			1.0							8.0						
20__21							6.3					12.5				
21__22																
22__23							4.0			2.4						
23__24																
H in 1 hr	0.4	0.4	2.0	7.0	1.0	6.3	2.0	--	5.0	8.0	6.7	12.5	--	12.0	1.5	--

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Mekele **2. Region** Tigray **3. Element** Rainfall intensity

4. Month August **5. Year** 1999

Time	Date																
	2	3	4	6	7	8	10	11	12	13	15	16	17	24	26	28	29

0__1												10.4	9.8				
1__2																	
2__3													0.4				
3__4																	
4__5																	
5__6																	
6__7																	
7__8																	
8__9																	
9__10																	
10__11																	
11__12																	
12__13											0.6						
13__14								1.0									
14__15	3.4							1.8						1.0	2.4		
15__16						10.0											0.8
16__17			17.8	13.5				1.8		7.5	8.6						
17__18																	
18__19		8.0	0.6			11.5											
19__20													0.2				
20__21		2.0	2.0										0.4				
21__22					11.0							11.8					
22__23													6.0				
23__24																	1.0
H in 1 hr	3.4	8.0	17.8	13.5	11.0	11.5	10.0	1.8	1.8	7.5	8.6	10.0	9.8	1.0	2.4	1.0	0.8

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Mekele 2. Region Tigray 3. Element Rainfall intensity
4. Month September 5. Year 1999

Time	Date															
	2	3	4	7	26											
0__1																
1__2																

2__3																			
3__4																			
4__5																			
5__6																			
6__7																			
7__8																			
8__9																			
9__10																			
10__11																			
11__12																			
12__13																			
13__14	1.4																		
14__15																			
15__16																			
16__17																			
17__18																			
18__19																			
19__20																			
20__21		2.4	1.0	0.6															
21__22																			
22__23																			
23__24																			
H in 1 hr	1.4	2.4	1.0	0.6															

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Adwa 2. Region Tigray 3. Element Rainfall intensity

4. Month August 5. Year 1992

H2

Time	Date																
	1	2	3	4	5	6	7	9	10	11	12	13	14	15	16	17	
0__1																	
1__2									0.8								
2__3									0.6								
3__4																	
4__5																	

5__6								0.6									
6__7								0.5									
7__8								1.0									
8__9												1.0					
9__10																	
10__11													1.2				
11__12													0.5				
12__13													10.0				
13__14													2.6				
14__15		0.3								10.1		0.3					
15__16			13.3	8.8							18.0	0.1				0.6	
16__17				1.4			1.5				0.7				0.4		
17__18					7.9				1.4		5.0	0.1					
18__19	2.5	3.3		0.5	11.0		4.4				5.0						
19__20	2.8				0.4					0.7	1.5					0.2	0.5
20__21																	
21__22					0.9	11.8					7.0						
22__23									1.2	2.0							
23__24								0.3			3.0						
H in 1 hr	2.8	3.3	13.3	8.8	11.0	11.8	4.4	1.0	1.4	10.1	18.0	1.0	10.0	0.4	0.6	0.5	

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Adwa 2. Region Tigray 3. Element Rainfall intensity

4. Month August 5. Year 1992

Time	Date										
	18	19	20	21	22	23	24	27	28	29	31
0__1		5.6		0.8							
1__2				1.2							
2__3											
3__4											
4__5				0.5							
5__6											

6__7												
7__8												
8__9												
9__10												
10__11	0.3											
11__12	0.2											
12__13					0.2							
13__14	4.7			0.3								
14__15	0.4									11.8	15.3	
15__16	1.5							2.6	12.2	0.8		
16__17				1.5		14.7				0.5		
17__18		12.6	1.8									
18__19		1.0	11.7	8.2						1.2	4.0	
19__20										1.0		
20__21							1.6	1.2				
21__22												
22__23		0.7										
23__24		1.3										
H in 1 hr	4.7	12.6	11.7	8.2	0.2	14.7	1.6	2.6	12.2	11.8	15.3	

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station Adwa **2. Region** Tigray **3. Element** Rainfall intensity

4. Month September **5. Year** 1992

Time	Date															
	1	2	3	5	7	8	9	10	11	12	13	16	17	24	30	
0__1																
1__2																
2__3																
3__4																
4__5																
5__6																
6__7																
7__8																

8__9																
9__10																
10__11																
11__12																
12__13																
13__14											8.0					
14__15										0.9	11.7					
15__16						1.0		7.8	7.3							
16__17				0.6		20.5	18.3								3.8	
17__18	4.9	0.5	11.0	0.4	6.5	2.5								2.0		
18__19			5.4													
19__20												2.0				
20__21												0.5				
21__22												0.7				
22__23																
23__24																
H in 1 hr	4.9	0.5	11.0	0.6	6.5	20.5	18.3	7.8	7.3	0.9	11.7	2.0	4.5	3.0	0.0	

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station D/Zeit **2. Region** Shewa **3. Element** Rainfall intensity

4. Month August **5. Year** 1997

H3

Time	Date															
	1	2	3	5	8	11	12	13	14	15	16	17	18	19	20	
0__1								0.6					0.7			
1__2					21.4											
2__3																
3__4									0.6	2.8						
4__5			2.0				3.0					1.6		1.4		
5__6																
6__7	4.0		1.4									0.5				
7__8																
8__9	2.8		1.5							3.8						

9__10		0.3														
10__11						0.6										
11__12																
12__13																
13__14																
14__15																
15__16				0.2							0.3					
16__17					0.2									3.7		
17__18																
18__19																
19__20																
20__21				0.4		3.9	2.2									
21__22																
22__23									4.0					0.4		
23__24							0.4		0.6							
H in 1 hr	4.0	0.3	2.0	0.4	21.4	3.9	3.0	0.6	4.0	3.8	0.3	1.0	0.7	3.7		

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station D/Zeit **2. Region** Shewa **3. Element** Rainfall intensity

4. Month August **5. Year** 1997

Time	Date															
	23	24	25	27	29	30	31									
0__1	0.6		3.8		0.2	1.6	1.4									
1__2			2.2													
2__3					0.2											
3__4																
4__5							0.4									
5__6																
6__7		1.3														
7__8				28.8												
8__9		4.4														
9__10		3.6														
10__11																

11__12																
12__13																
13__14																
14__15																
15__16																
16__17																
17__18	9.8															
18__19			1.1													
19__20																
20__21																
21__22																
22__23																
23__24																
H in 1 hr	9.8	4.4	3.8	28.8	0.2	1.6	1.4									

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station D/zeit **2. Region** Shewa **3. Element** Rainfall intensity

4. Month August **5. Year** 2003

Time	Date															
	1	2	3	6	7	8	9	10	11	12	13	14	16	17	18	19
0__1						1.7		0.9								
1__2						1.1	0.3	0.8						0.2		
2__3						4.0	0.5	2.2								
3__4					32.1	4.0	0.5	0.9								
4__5						0.2	0.9	1.0								
5__6			6.7				0.1									
6__7	1.1		2.3													
7__8	4.7					0.9				1.0	1.0					
8__9	1.5	2.8		0.3		0.8					11.0					
9__10	2.0	4.0				0.5					0.1					
10__11	1.3	2.0				1.0		0.5								
11__12						1.0										
12__13	0.3					0.8										

13__14						2.1										
14__15																
15__16								17.5								
16__17										17.1	6.1	0.8				
17__18									14.1							
18__19									1.7				0.6			
19__20													0.6			
20__21																
21__22																
22__23							0.5									
23__24					5.8		1.2									
H in 1 hr	4.7	4.0	6.7	0.3	32.1	4.0	1.2	17.5	14.1	17.1	11.0	0.8	0.6	0.2		

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station D/Zeit **2. Region** Shewa **3. Element** Rainfall intensity

4. Month August **5. Year** 2003

Time	Date															
	20	21	22	23	24	25	26	27	29	30						
0__1		3.6						0.1								
1__2		1.0														
2__3										1.1						
3__4			0.7													
4__5		0.4								0.5						
5__6		1.1			7.7	3.9										
6__7		2.5			1.2	5.1										
7__8		0.5	8.7	1.2												
8__9		0.7	15.4													
9__10		1.3	4.5		0.2											
10__11		1.5	0.6													
11__12		1.1	0.9													
12__13							10.1									
13__14																
14__15																

15__16																
16__17	5.5															
17__18									2.1							
18__19	0.6															
19__20																
20__21																
21__22									0.2							
22__23							0.3									
23__24							0.3									
H in 1 hr	5.5	3.6	15.4	1.2	7.7	5.1	10.1	0.1	2.1	1.1						

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station A/A Bole **2. Region** Shewa **3. Element** Rainfall intensity

4. Month July **5. Year** 1995

H4

Time	Date															
	1	2	4	6	7	8	10	11	12	13	14	16	18	20	21	22
0__1		0.0										0.9				
1__2																0.4
2__3												2.8				
3__4																
4__5										1.6						
5__6																
6__7																
7__8																
8__9						1.8		0.6		0.4	0.8					
9__10																
10__11										1.2	3.0					
11__12	2.8			2.0												
12__13			1.9	0.7						2.5	0.8			2.8		
13__14																
14__15					2.3		20.0							0.6		0.3
15__16					0.8		0.3									0.7

16__17													1.7			0.2
17__18																
18__19													2.8			
19__20													0.8			
20__21																0.2
21__22												0.9				
22__23															3.0	0.3
23__24																
H in 1 hr	2.8	0.0	1.9	2.0	2.3	1.8	20.0	0.6	2.5	1.6	3.0	2.8	2.8	0.0	3.0	0.7

Time is local standard time

H = Highest rain fall in mm

hr = hour

1. Station A/A Bole **2. Region** Shewa **3. Element** Rainfall intensity

4. Month July **5. Year** 1995

Time	Date															
	23	25	26	28	29	30	31									
0__1																
1__2																
2__3																
3__4																
4__5				0.9												
5__6																
6__7																
7__8																
8__9																
9__10																
10__11		1.0	0.3													
11__12	0.8															
12__13			5.8													
13__14																
14__15																
15__16																
16__17							1.8									
17__18																

18__19					0.7		0.2									
19__20																
20__21					1.4	0.3										
21__22			1.3													
22__23																
23__24																
H in 1 hr	0.8	1.0	5.8	0.9	1.4	1.8	0.2									

Time is local standard time

H = Highest rain fall in mm

hr = hour