ADDIS ABABA UNIVERSITY

SCHOOL OF GRADUATE STUDIES

APPLICATION OF THE GEOMORPHOLOGIC INSTANTANEOUS UNIT HYDROGRAPH CONCEPT FOR RUNOFF PREDICTION IN UNGAUGED CATCHMENTS

By

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August, 2009

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A Thesis Submitted to School of Graduate Studies, Addis Ababa University, in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering

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October, 2009

ABSTRACT

This study was aimed at development of unit hydrograph by relating the geomorphological characteristics of a catchment with the basic characteristics of the catchment IUH through the concept of Geomorphological Instantaneous Unit Hydrograph (GIUH) to address the problem of predictions in ungauged catchments. Two conceptual unit hydrograph models, ED-GIUH model and the GIUH based Nash model, have been used in the study.

The geomorphological characteristics including the Horton's ratios of the catchments were extracted using GIS software called ILWIS. DEM-hydro processing, a new routine developed in ILWIS, was used for the extraction of the geomorphological characteristics.

The geomorphological characteristics of a catchment were related with the shape and scale parameters of the Nash IUH to derive the complete shape of the GIUH based Nash model. ED-GIUH was developed from geomorphological characteristics of the catchments and probability density function of travel time of rain fall excess to the catchment outlet. These two models were developed in four catchments in Abay basin with range of areas (185-670km^2).

The velocity parameter of the two models was calibrated using unit hydrograph ordinates developed through rainfall-runoff analysis and the optimum velocity was determined. The performances of the calibrated models were evaluated using error functions, namely, Nash-Sutcliffe efficiency (NSE), percentage error in peak discharge (PEP), and percentage error in time to peak (PETP) and the models were compared together with the observed average unit hydrographs.

From the results of the calibration as well as verification of the models it was found that both models were adequately simulate the shape of the unit hydrographs of the catchments. The ED-GIUH was successful in capturing the peak discharge of the hydrographs but the Nash based GIUH model was better in simulating the time to peak as well as the shape of the unit hydrographs.

The optimized velocity parameter of ED-GIUH model was related to the geomorphologic characteristics of the catchments. The linear equation, relating the velocity with the slope and longest flow path of the catchments under the study was found to be best fit. A validation test was carried out to check the reliability of the derived equation in an adjacent catchment which was not used in the calibration of the models and a reasonable result was obtained in four rainfall-runoff events.

Finally, it is clear that the predictability of the developed velocity equation in the region will be improved if the number catchments included in the calibration of the models were increased.

Key words: Geomorphological instantaneous unit hydrograph, ungauged catchments, GIS, Nash model, rainfall-runoff modeling.

ACKNOWLEDGEMENT

I would like to extend my sincere gratitude to the people that helped me in one or the other way which made this thesis possible. Special thanks is due to Dr. Semu Ayalew who bring to me the idea of geomorphologic instantaneous unit hydrograph and also for his guidance and support as supervisor. I also wish to express my gratitude to Ministry of Water Resource, Hydrology Department for providing me the stream flow data and the National Meteorological Service Agency, from providing me the rainfall data.

I would like to acknowledge Ato Godana Seyum, for his invaluable help in obtaining the data.

I would also like to express my deep gratitude to Ato Heruy Zeray for his invaluable help during my whole study period.

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ABRIVATIONS

ED-GIUH : Exponentially Distributed Geomorphologic Instantaneous Unit Hydrograph

IUH : Instantaneous Unit Hydrograph

UH : Unit Hydrograph

DEM : Digital Elevation Model

NSE : Nash-Sutcliff Efficiency

GIS : Geographical Information System

ILWIS : Integrated Land and Water Information System

1. INTRODUCTION

1.1. General

Planning, design and operation of storm water management infrastructures and its components are essentially based of following key characteristics of design flood: total flood runoff volume, magnitude of peak flood runoff at the location to be protected, time to occurrence of peak flood runoff at the location (Maidment, 1992; Chow, 1988):

Selection of appropriate method for estimation of design flood depends upon availability of hydrological information. Several statistical and deterministic methods to estimate design floods and their characteristics are prevalent in applied hydrology.

The application of rainfall based methods is an indirect approach to estimate design flood characteristics, and becomes inevitable if insufficient or no discharge data is available. These methods make use of rainfall data along with additional hydrological information. Rainfall based methods typically employ empirical, conceptual or physically based rainfall-runoff models to transform rainfall histogram into discharge time series. Rainfall runoff modeling is briefly explained in part 2

The rainfall based methods are gaining wide acceptance mainly due to the following advantages:

- Rainfall based methods provide not only the peak runoff but also the additional information such as time to occurrence of peak runoff, flood volume etc.
- They are not restricted by the requirement of very long discharge time series
- They demonstrate ability to tackle regional hydrological changes
- They also exhibit potential to handle climate change scenarios through adaptation of rainfall predictions to climate change.

1.2. Problem of runoff predictions in ungauged catchments

Ungauged catchments are those catchments in which the river discharges have not been measured in the past. Catchments for which available observed discharge time series is inadequate for the calibration are also considered ungauged.

For planning, development and operation of various water resources schemes estimation of run-off response from ungauged catchments is an important subject of research in the sphere of surface water hydrology. Conventional techniques of unit hydrograph derivation require historical rainfall-runoff data. Most of medium and small catchments, however, especially in the developing countries like Ethiopia, lack the adequate discharge measurements.

Hence, indirect inferences through regionalization are sought for such ungauged catchments. In the process of regionalization, the parameters of instantaneous unit hydrograph (IUH) models are related with physiographic and climatologic characteristics for gauged catchments in hydro-meteorologically homogeneous regions. These relationships are then used for run-off estimation for the ungauged catchments of the hydro-meteorologically homogeneous regions. This process of regionalization is a difficult task since it not only requires a good amount of rainfall-runoff data for the gauged catchments, but the hydro-meteorological homogeneity of the region is also difficult to ascertain Kumar *et al* (2007). Further, with the change of the land-use and climate patterns, the model parameters are required to be updated from time to time.

The rainfall pattern, in general is undergoing a change due to global changes in atmospheric conditions. Further, because of different activities in the catchment, its land-use is also undergoing a gradual change and this has an impact on the run-off characteristics of the catchment. Thus, the coupling of the hydrologic characteristics of the catchment with the geomorphologic parameters can provide an insight to the hydrologic behavior of the ungauged catchments in particular.

1.3. Objectives

General objectives

This study was intended to address the problem of predictions of runoff characteristics in ungauged catchments using a set of gauged catchments in Abay basin by relating the characteristics of the instantaneous unit hydrograph (IUH) with the geomorphological characteristics of a catchment through the concept of geomorphological instantaneous unit hydrograph (GIUH)

Specific objectives

- Develop Exponentially Distributed Geomorphological Instantaneous Unit Hydrograph (GIUH) model to derive the complete shape of unit hydrographs.
- Develop the complete shape of the Nash UH by relating the parameters of the Nash cascade model with the peak value and time to peak of the GIUH.
- Evaluate the geomorphological characteristics of the catchment required for derivation of the GIUH by employing GIS.
- Compare the unit hydrographs developed by GIUH based Nash model and ED-GIUH model together with the observed catchment average unit hydrographs using selected error functions.
- Relate the velocity parameter of the GIUH based model with the catchment characteristics to develop a regional equation.
- Check the validity of the models in adjacent nearby catchment.

1.4. Previous Studies

Different flow prediction models have been studied in Abay basin to develop runoff prediction model in ungauged catchments. But diversified studies based on unit hydrograph concepts are not made in Ethiopia.

Recently, some runoff prediction methods based on the concept of synthetic unit hydrographs has been developed. Mulugeta Azeze (2004) and Godana Seyoum (2005) studied regionalization of Snyder's unit hydrograph in upper Awash and upper Tekeze basins and Abay basin respectively. An attempt is also made by Zelalem Fisseha to develop a regional equation of instantaneous unit hydrograph in Gibha sub basin of Tekeze basin based on GIUH concept.

The author didn't find any studies on unit hydrograph model development based on the concept of GIUH in Abay basin.

2. LITERATURE REVIEW

2.1. Catchment Runoff Generation and Response Behavior

Basically, runoff generation at a catchment scale in general or hillslope scale in particular includes two main components: (1) surface runoff, (2) sub surface runoff. There are a number of flow processes within each component as illustrated in Figure 2.1.

The surface runoff: Flow processes include overland flow, stream flow, and channel flow which is defined as water flow over the land surface based on the differences on slope gradient. The overland flow is known as infiltration excess overland flow (Horton overland flow) or saturation overland flow (Dunne flow). The Horton overland flow is generated when the rainfall intensity exceeds the infiltration capacity of the soil or by saturation mechanism where the soil becomes saturated by the perennial groundwater rising to the surface or by the lateral or vertical percolation above an impending horizon (Dune, 1982). The overland flow is observed as sheet flow which then generates the rill flow. A number of the rill flow will contribute or create the stream flow which then converges into channel flow.

Subsurface runoff: Flow processes include unsaturated subsurface flow, perched subsurface flow, macro pore flow and groundwater flow. Subsurface runoff is generated since water discharged from the surface into the subsurface system. The unsaturated subsurface flow mostly is in vertical direction while the perched flow moves in lateral direction. The perched flow is generated where the shallow soil layer has much more higher hydraulic conductivity as compared to the lower one. The macro pore flow occurs where the subsurface system has macro pores such as voids, natural pipes, cracks, etc. the flow rapidly contributes to the groundwater system. Ground water flow is produced in the saturated zone which is fed through percolation of infiltrated water or from neighboring system. The ground water contributes to the channel system as rapid groundwater flow in the upper part of the initially unsaturated subsurface domain or as delayed groundwater flow in the lower part of the saturated subsurface domain.

The aforementioned literatures concluded that, the Hortonian or Infiltration Excess Overland Flow (Horton, 1933) is generally assumed to be the dominant mechanism of runoff generation in most arid and semi-arid regions.

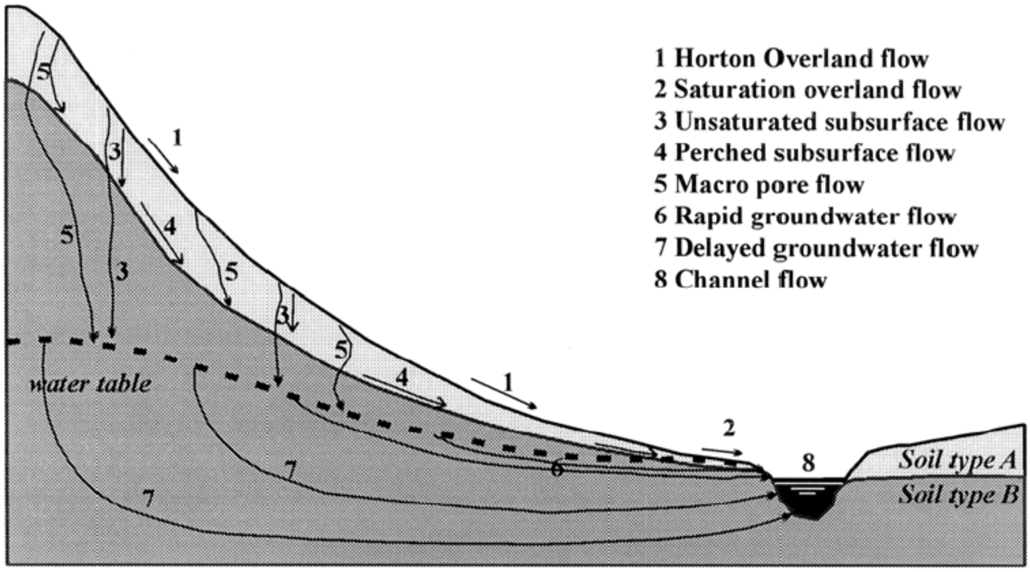


Figure 2-1: Cross sectional presentation of hillslope flow process (after Rientjes, 2004)

2.2. Rainfall – Runoff Modeling

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.

In order to simulate the transformation from rainfall to runoff, rainfall-runoff models have been developed already a long time ago Todini (1988) for his historical review of rainfall runoff modeling.

2.2.1. Approaches for response behavior modeling

The modeling literature is replete with different ways of classifying hydrological models. Chow et al. (1988), Todini (1988). Hydrologic modeling can be linear or non linear, time invariant or time variant, lumped or distributed, continuous or discrete, event driven or continuous process. Figure 2.2 provides a general overview of the hydrological models using the classification criteria randomness, spatial discretization and model structure.

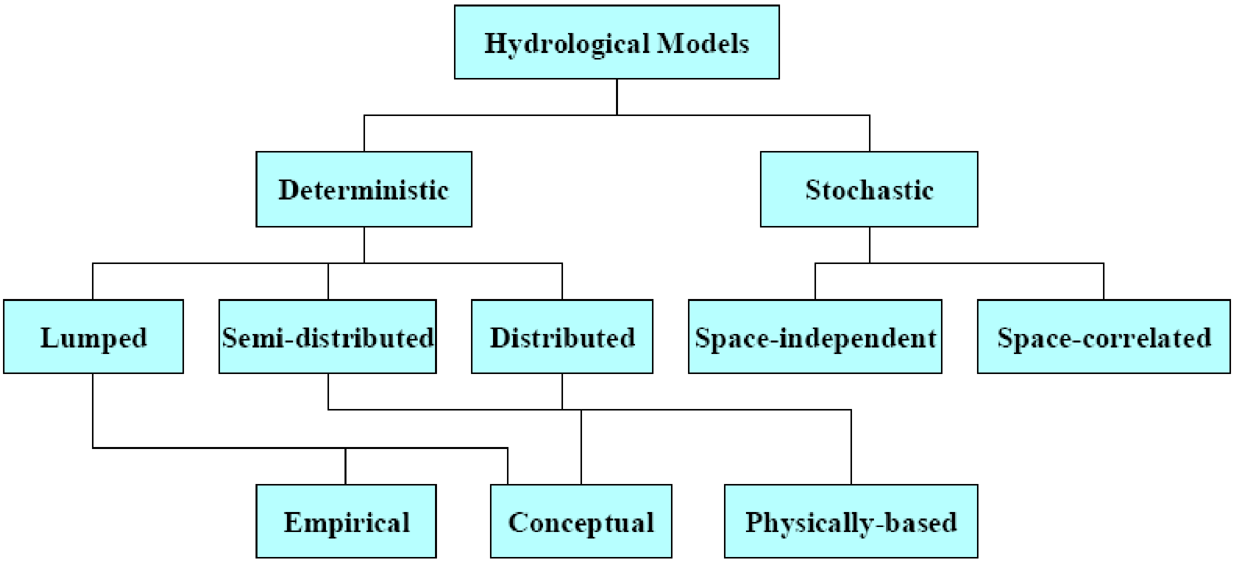


Figure 2-2: Classification of Hydrological models (Chow, 1998, modified)

Empirical "black-box": models use observed discharge data to establish model structure and corresponding model parameters by fitting a function of hydrological characteristics with observed discharge using regression procedures. Empirical models completely ignore the underlying physical processes; hence they solely depend upon the information carried by observed data. Empirical models are still in use mainly due to their simplicity, but their consistency and transferability between catchments is questionable. The presently used empirical models include multiple regression models and artificial neural network models

Conceptual models: are built on simplified concepts derived from physical processes of rainfall-runoff phenomena. In conceptual models the relationships between hydrological characteristics and responses are loosely based on the physical processes and do not use their strict representation. Parameters of conceptual models are derived by fitting the modeled discharge with observed discharge. Due to the incorporation of process knowledge, while keeping a simple structure, these models are relatively robust and reliable. Conceptual models include cascade model (Nash, 1957), geomorphologic instantaneous unit hydrograph (Rodriguez-Iturbeet al., 1979), HBV (Bergstrom, 1995) and IHACRES (Identification of unit Hydrograph And

Component flows from Rainfall, Evaporation and Streamflow data) (Jakeman, 1990)

Physically based models: Which venture to pursue precise representation of the physical processes. These models are usually based on principles of physics such as mass balance or momentum equation. Parameters of physically based models have physical meanings and they can be derived from hydrological characteristics. However, these models are complex, data intensive and computationally demanding. The physically based models include SHE (Abbott *et al.*, 1986).

Another basic distinction between models is whether stochastic or deterministic representations and inputs are to be used. In the stochastic models, the chance of occurrence of the variable is considered thus introducing the concept of probability. In the deterministic models, the chance of occurrence of the variables involved is ignored and the model is considered to follow a definite law of certainty but not any law of probability (Raghunath, 1985).

In stochastic models, some or all of the inputs and parameters are represented by statistical distributions, rather than single values. A range of values is defined instead of a single value. There is then a range of output sets, each derived from different combinations of the inputs and parameters and/or each of them associated with a certain probability of occurrence. Stochastic modeling generally requires the model to be run many times, each with different combinations of parameters or model inputs that are, perhaps, resulting in many outputs that can be analyzed to define a probability distribution of outputs. Stochastic modeling can be very useful, particularly when we are uncertain about the exact values of model parameters or model inputs, but running a model many times can be time consuming. Stochastic models are termed space-independent or space-correlated according to whether or not random variables at different points in space influence each other (Chow *et al.*, 1988).

In a deterministic model, randomness is not considered. This means that a single set of input values and a single parameter set are used to generate a single set of outputs; a given input rainfall always produces the same output runoff (Chow *et al.*, 1988). In terms of spatial domains in catchment modeling, deterministic models can be classified as lumped, distributed or semi-distributed ones. The lumped model ignores spatial distribution of the catchment characteristics; values are spatially averaged and a single value is used for the entire catchment. It may either be conceptual or empirical. Since hydrological processes generally are space dependent, spatial lumping always includes rough conceptualization. In contrast, a distributed model considers the hydrological process taking place at various points in space, in which parameters, inputs and outputs vary spatially. It captures the system by partitioning the catchment into a number of smaller units. It may either be physically-based or conceptually based. A semi-distributed model is something in between the lumped and distributed models that means the catchment is partitioned but in a coarser unit as compared with distributed models. A semi-distributed model may adopt a lumped representation for individual sub catchments.

2.3. The Unit Hydrograph as Catchment Response Function

The Unit Hydrograph (UH) theory is also classified in the conceptual model category and has been widely and successfully used over the past decades. First introduced by Sherman (1932) as a basic tool that represents the hydrologic response of a catchment through which effective rainfall is transformed to direct runoff, the UH is the surface runoff hydrograph resulting from one unit of rainfall excess uniformly distributed spatially and temporally over the catchment for the entire specified duration.

The procedure to develop a unit hydrograph for a storm with a single peak is fairly simple. After the base flow is removed from the total runoff hydrograph, the direct runoff hydrograph remains. The total runoff volume is determined by integrating the direct runoff hydrograph. In order to obtain the unit hydrograph, each ordinate of the direct runoff hydrograph is divided by the runoff volume.

Theoretically, unit hydrographs developed from different storms should be identical; however that is rarely the case in practice. In order to develop an average Linsley *et al*. (1975) suggest that an average response may be determined by calculating the average peak flow rate and time to peak, then sketching a hydrograph shape such that it contains 1 unit of runoff, passes through the average peak, and has a shape similar to the unit hydrographs developed from the individual storm events.

The detail of the derivation of the unit hydrographs from each storm events and the catchment average unit hydrograph is presented in section 3.

The unit hydrograph is assumed to be a constant response function of the watershed as long as there are no major changes in the land use.

A typical idealized shape and its feature of hydrograph for developing unit hydrograph is shown in Figure 2.3.

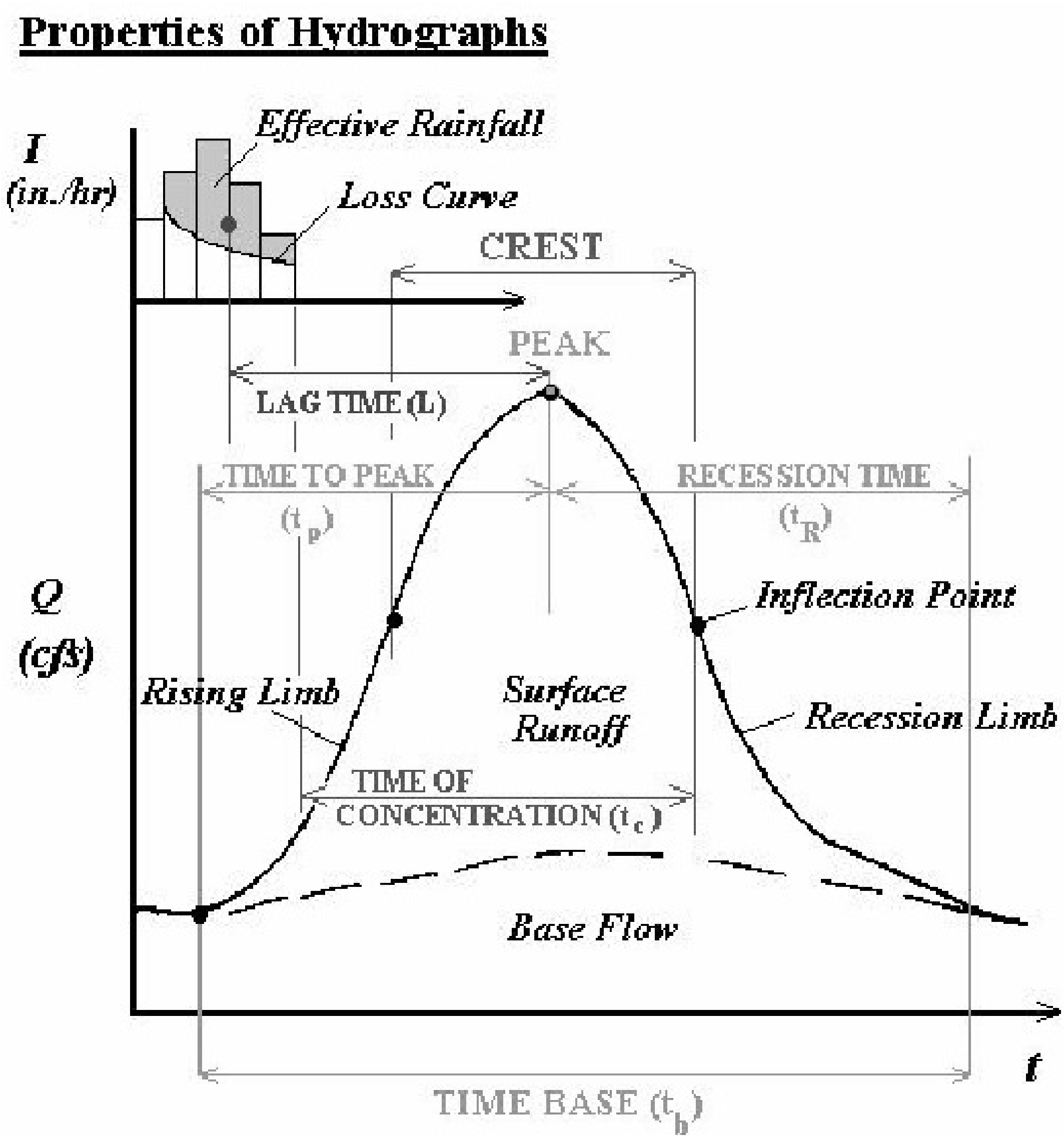


Figure 2-3 Properties of an idealized hydrograph shape. (Source: U.S Army Corps of Engineers)

Unit Hydrograph Theory

The unit hydrograph theory contains the following assumptions that can limit its applications (Chow, 1988).

1. *The excess rainfall has a constant intensity within the effective duration.* When the unit hydrograph is developed using gauged data, the storms selected for analysis should have a short duration because they are the most likely to have a uniform intensity and produce a single-peaked hydrograph.
2. *The excess rainfall is uniformly distributed throughout the entire drainage area.* This assumption may pose difficulties for larger watersheds. For watersheds above a certain size, the assumption of uniform rainfall is no longer valid. Sherman (1932) used the unit hydrograph theory on watersheds ranging from 1300 km^2 to 8000 km^2. Linsley et al. (1975) recommended that the unit hydrograph only be used on watersheds less than 5000 km^2, while Ponce (1989) suggested that it should only be applied on midsize catchments between 2.5 km^2 and 250 km2. Shaw (1999) recommended on catchment sizes not exceeding 1000km^2.Because the unit hydrograph model assumes that rainfall is uniform over an entire area, it is not applicable to large watersheds. Small catchments tend to reflect variations in the rainfall excess more than larger watersheds, because they have less channel storage than larger watersheds, thus the small catchments are less appropriate for unit hydrograph analysis (Huggins and Burney, 1982).
3. *The base time of the direct runoff hydrograph is constant based on a given duration of rainfall.* This assumption implies that the unit hydrograph model cannot account for differences in the watershed response to different rainfall intensities.
4. *The ordinates of all direct runoff hydrographs with the same base time are proportional to the total amount of direct runoff represented by each hydrograph.*
5. *The hydrograph resulting from excess rainfall reflects the unique characteristics of the watershed.* The unit hydrograph model cannot reflect variations in the watershed response due to changes in the season, land use or channel characteristics.

2.3.1. Synthetic Unit Hydrographs

The unit hydrograph developed from rainfall and stream flow data on a watershed applies only for that gauged watershed. Unfortunately, the majority of watersheds are ungauged. Synthetic unit hydrographs attempt to extend the application of unit hydrograph theory to ungauged catchments. Because synthetic methods do not rely on observed runoff data, they may be applied to ungauged watersheds. Chow *et al.* (1988) suggested that there are three major types of synthetic unit hydrographs. They can be:

1. Those based on a dimensionless unit hydrograph (Soil Conservations service. l972).
2. Those based on models of watershed storage (Clark, 1943), and
3. Those relating hydrograph characteristics (peak flow rare, base time, etc.) to watershed characteristics (Snyder, 1938).

2.3.2. Existing Methods for Flood Prediction in Ungauged Catchments

SCS Dimensionless Unit Hydrograph

The dimensionless unit hydrograph developed by the Soil Conservation Service in 1972, has been obtained from the unit hydrographs for a great number of watersheds of different sizes and for many different locations. The SCS dimensionless hydrograph (DUH) is a synthetic unit hydrograph in which the discharge is expressed as a ratio of discharge, q, to peak discharge, q_p and the time by the ratio of time, t, to time to peak of the unit hydrograph, t_p. Given the peak discharge and the lag time for the duration of the excess rainfall, the unit hydrograph can be estimated from the synthetic dimensionless hydrograph for the given basin.The SCS suggests that the dimensionless unit hydrograph can be described in terms of an equivalent triangular hydrograph (TUH). The values of q_p and t_p can then be estimated using this simplified triangular unit hydrograph whose height is equal to q_p and whose time base, t_b, is equal to 2.67 t_p. The time is usually expressed in hours and the discharge in m3/s/cm (or cfs/in). After analysis of a great number of unit hydrographs, the SCS recommends recession duration of 1.67 t_p. Because the volume of direct runoff must equal 1 cm, it can be shown that $Q_p = \frac{CA}{t_p}$ where C = 2.08 (483.4 in the English system) and A is the drainage area in square kilometers (square miles). From a study of many large and small rural watersheds the basin lag is $t_l = 0.6t_c$, where t_c is the time of concentration of the watershed. The time to peak, t_p, is then equal to $\frac{t_r}{2} + t_l$ .where tr is effective rainfall duration.

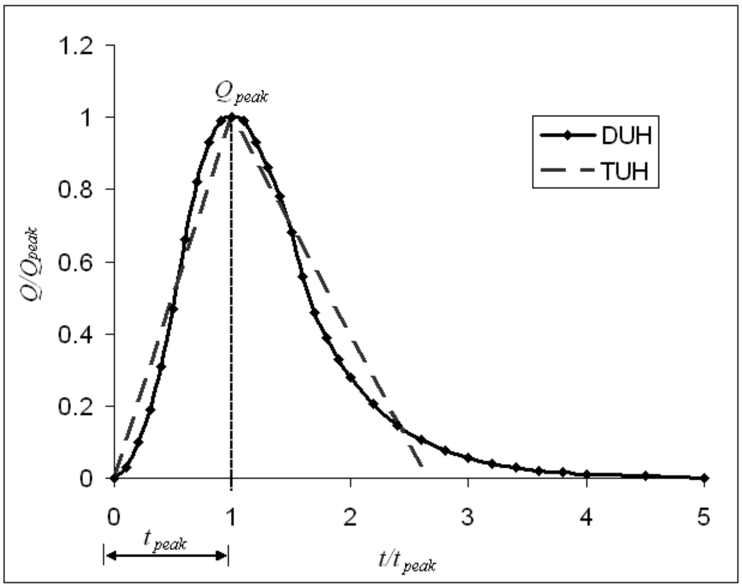


Figure 2-4: SCS dimensionless and triangular unit hydrograph

Limitations of SCS method

The SCS curve number method can be applied only in the case of big storm events. If the total rainfall depth is below 50 mm, the method often underestimates the direct runoff volume (B´ardossy, 2000). Furthermore, SCS dimensionless unit hydrograph or triangular unit hydrograph provides only empirical approximation of flood runoff characteristics; its reliability is limited to the type and the size of the catchments which were used for its derivation.

Clark unit hydrograph

Clark (1945) developed his own synthetic unit hydrograph method that incorporated a parameter to model the watershed storage (R) and time of concentration (tc). The Clark synthetic unit hydrograph Method incorporates the processes of attenuation and translation of runoff through the use of the time-area curve. Clark (1945) noted the translation of flow through the watershed was described by a time-area curve that expresses the fraction of watershed area contributing runoff to the watershed outlet as a function of time since the start of effective precipitation (Straub et al. 2000). A linear reservoir was used by Clark to reflect the storage effects of watersheds.

Clark only used gauged catchments in his original work and did not provide guidelines for the estimation or determination of the R value and time-area relationships for ungauged catchments. It is possible to transfer the R value from one catchment to another nearby catchment through regression analysis. Parameters to be considered included (but not limited to) drainage area, lengths, and slopes.

The U.S.Army Corps of Engineers, Hydrologic Engineering Center, has noted that the ratio $K/(T_c+K)$ tends to remain constant for a region (HEC-HMS User's Manual). HEC-HMS also offers a standard time-area curve, it has been shown that these standard curves works reasonably well for variety of conditions.

Limitations of Clark method

Clark did not consider ungauged catchments in his original work, his method of estimating R relies upon gauge information. The transfer of the model parameter to ungauged catchments using regression analysis at a regional scale requires a number of gauged catchments to develop the equations to determine the Clark parameters, but this would be difficult in regions where most of the catchments are ungauged. The standard time-area curve of the HEC-HMS method work reasonably for catchments which have similar nature to the ones used in the development of these curves.

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. These relationships are based on a study of 20 watersheds located in the Appalachian Highlands and varying in size from 10 to 10,000 square miles. 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 et al., 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,

$$t_l = 5.5 t_r \qquad 2.1$$

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

$$t_l = C_l C_t (L L_c)^{0.3} \qquad 2.2$$

$$q_p = \frac{C_2 C_p A}{t_l} \qquad 2.3$$

Where:

The basin lag time of the standard unit hydrograph (Equation 2.2) is in hours, *L* is the length of the main stream in kilometers (miles) from the outlet to the upstream divide, *Lc* 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 *LLc* is a measure of watershed shape.

Ct 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 2.3 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.

To compute C_t and C_p for a gauged watershed, the values of *L* and *Lc* are measured. From a derived unit hydrograph of the watershed, values of its associated effective duration t_R in hours, its basin lag t_{lR} in hours, and its peak discharge q_{pR} in m3/s are obtained. If $t_{lR} = 5.5t_R$, then the derived unit hydrograph is a standard unit hydrograph and $t_r = t_R$, $t_l = t_{lR}$, and $q_p = q_{pR}$, and C_t and C_p are computed by the equations for t_l and q_p given above, corresponding to the standard unit hydrograph. If t_{lR} is quite different from 5.5_{tR}, the standard basin lag is computed using:

$$t_l = t_{lR} + \frac{t_r - t_R}{4} \qquad 2.4$$

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. The value of C_t is then obtained using the equation 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 $t_l = t_{lR}$.

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.

Required unit hydrograph. The peak discharges of the standard and required unit hydrographs are related as follows,

$$q_{pR} = \frac{q_p t_l}{t_{lR}} \qquad 2.5$$

Assuming a triangular shape for the unit hydrograph, and given that the unit hydrograph represents a direct runoff volume of 1 cm (1 in), the base time of the required unit hydrograph may be estimated by,

$$t_b = \frac{C_3 A}{q_{pR}} \qquad 2.6$$

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

As an aid in drawing an adequate unit hydrograph, the U.S. Army Corps of Engineers developed relationships for the widths of the unit hydrograph at values of 50% (W_{50}) and 75% (W_{75}) of q_{pR}.

The width in hours of the unit hydrograph at a discharge equal to a certain percent of the peak discharge q_{pR} is given by Chow et al. (1988) as,

$$W_{\%} = C_w \left(\frac{q_{lR}}{A} \right)^{-1.08} \qquad 2.7$$

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.

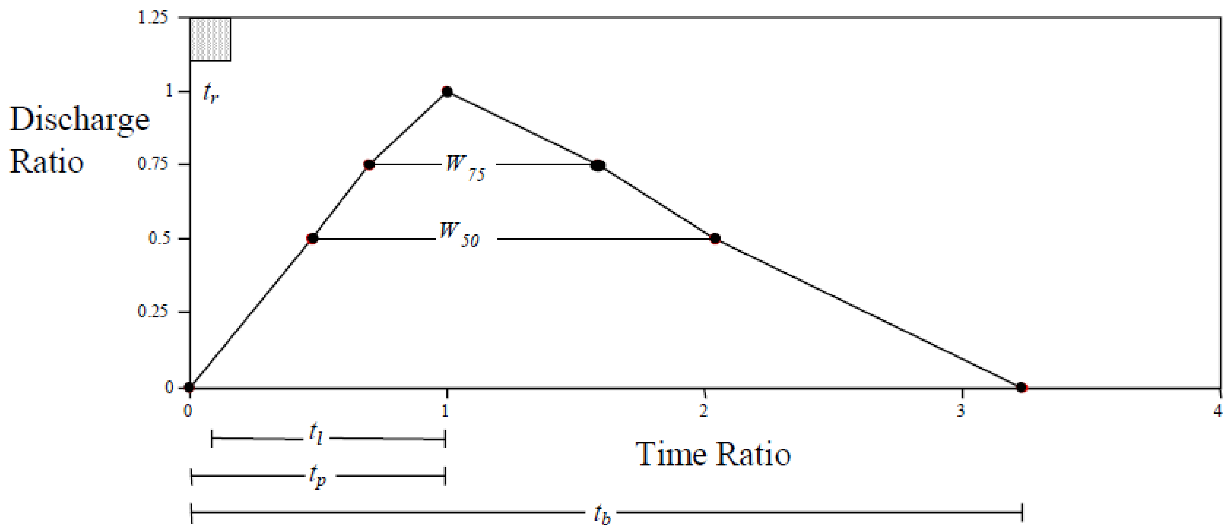


Figure 2-5: The form of the Snyder's unit hydrograph.

Limitations of Snyder unit hydrograph

To develop Snyder parameters for an ungaged basin, it is necessary to identify a nearby and physiographically similar gauged basin from which to transfer parameters. Even if a gauged basin is located near to the ungauged basin of interest, the greatest limitation of the method is the difficulty in verifying that the two basins are sufficiently alike to transfer C_t and C_p from one to the other. Typical hydrologic features that should be evaluated in making this determination include soils and geology, topography, drainage pattern, drainage area, and land cover. However, in the absence of gauge data on both basins, it can never be known with certainty that the transfer is appropriate.

2.3.3. The Proposed Geomorphological Instantaneous Unit Hydrograph (GIUH) Method

The GIUH is also a type of synthetic unit hydrograph which links geomorphological characteristics of the catchment to its response to rainfall. This method does not involve parameter transfer from adjacent gauged and physiographical similar catchment to the ungauged catchment under consideration or does not require development of a regional equation which relates the catchment characteristics to the hydrograph characteristics. Instead, the GIUH models relate the geomorphological characteristics of the catchment to its response to rainfall.

Rodriguez-Iturbe & Valdes (1979) first introduced the GIUH, which led to the renewal of research in Hydrogeomorphology. The concept was re-stated by Gupta, et al. (1980). They link the hydrologic response of watersheds in terms of the IUH to their geomorphologic parameters, Horton's ratios, including area ratio (R_A), bifurcation ratio (R_B) length ratio (R_L), and to a dynamic parameter (V), and also to the drainage density in case of ED-GIUH, where hillslope velocity is included (Bras and Rodriguez-Iturbe, 1989).

The Horton's ratios and the drainage density of the catchment can be obtained from drainage network, which can be extracted through processing and analysis of Digital Elevation Model (DEM) in GIS environment. A drainage networks consists of a connected set of stream links draining to a common point. Horton (1932, 1945) proposed a numbering system to perform the ordering of stream links. Strahler (1952, 1957) improved upon Horton's ordering system to form the Horton- Strahler ordering system, which is now the most commonly used ordering system in hydrogeomorphology. This ordering procedure, illustrated in Figure 2.6, analyzes networks as follows:

- Channels originating at a source (headwater) are defined to be first-order streams
- When two streams of order i join a stream of order $(i+1)$ is created.
- When two streams of different order merge, the order of the downstream link is the highest of the two stream orders.

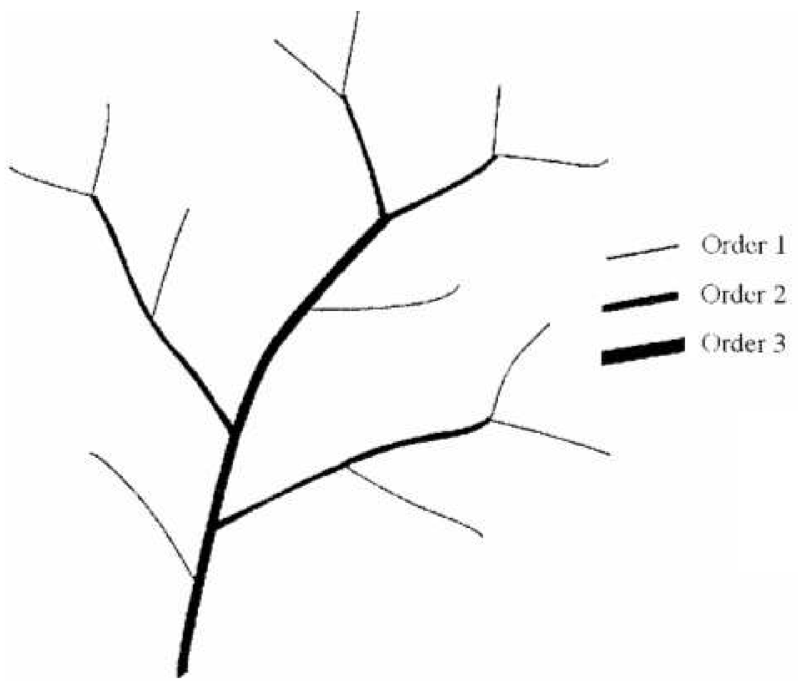


Figure 2-6: Horton-Strahler ordering. (Source: Rodriguez-Iturbe and Rinaldo 1997)

Horton (1932, 1945) defines the drainage density D as:

$$D = \frac{L}{A}$$

Where:

A : the area of the catchment, and

L : the length of the corresponding drainage network segment.

Drainage density is closely associated with the average hillslope length or overland flow length (the distance between the catchment divide to the stream channel). Assuming that D is constant throughout the catchment, the average hillslope length is computed as follows:

$$L_o = \frac{1}{2D}$$

The IUH is interpreted as the probability density function of the travel times to the basin outlet of water droplets randomly and uniformly distributed over the watershed. The resulting IUH for the basin is a function of the probability density function of the travel times of droplets in streams of a given order and of the transition probabilities of water droplets going from a given state to another of higher order. The states of the process are defined to be the hillslope region, a_i, or streams, $r_{i,}$ of a given order where the drop is travelling. The travel of the drop is governed by the following rules, Rodriguez-Iturbe & Valdes (1979):

Rule. 1. When the drop is still in the hillslope phase, the state, a_i, is of the order of the stream towhich the land drains directly.

Rule. 2. The only possible transitions out of state a_i, are into the corresponding $r_{i.}$ From r_i transitions of the form, $i \rightarrow j$ for some $j > I\ (j = i+1, \ldots, \Omega)$ are possible.(Ω is the highest order).

Rule 3. Defining the outlet as a trapping state, $\Omega + l$, the final state of the drop is $\Omega + l$ from which transitions are impossible.

The travel times in the streams or hillslope are assumed to be exponentially distributed and independent of one another. The initial probabilities of a drop landing anywhere on the basin, as well as the transition probabilities which are required in order to define the probability function of a particular path that a drop may take to the outlet, can be defined as functions of only geomorphologic and geometric parameters.

The ED- GIUH yields fully analytical, but complicated expression for the IUH. Rodriguez-Iturbe & Valdes (1979) suggested that it is adequate to assume a triangular IUH and only specify the time to peak and peak of the IUH. These characteristics have simple expressions obtained by regression of the peak and time to peak of the analytic solution for a wide range of parameters:

$$q_p = 1.31 R_l^{0.43} \left(\frac{v}{L_\Omega} \right) (hour^{-1}) \qquad 2.8$$

$$t_p = 0.44 R_l^{-0.38} \left(\frac{R_B}{R_A} \right)^{0.55} \left(\frac{L_\Omega}{v} \right) (hour\ ) \qquad 2.9$$

Where:

L_Ω : the length in kilometers of the highest order

V : the expected peak velocity in m/s

Since the first introduction of the ED-GIUH in 1979, a number of publications have been released. These include the improvement of analytical solution, use of the time to peak and peak discharge of the above equations to estimate parameters of various IUHs, parameterization of the velocity parameters.

Using the above equations Rosso (1984) relate the shape and scale parameters of the Nash IUH to Horton order ratio. In GIUH models the dynamic parameter, V makes the model inapplicable. This parameter has been explained by different researchers in different way. The suggested values include: average flow velocity Rodriguez-Iturbe & Valdes, (1979), expected peak velocity Bras and Rodriguez-Iturbe.(1989), Franchini and O'Connell (1996) and Al-Wagdany and Rao (1998) considered purely as a calibration parameter. In This study the ED- GIUH model development was done mainly based on the reference of the works of Rodriguez-Iturbe & Valdes, (1979), Gupta et al, (1980), Bras and Rodriguez-Iturbe.(1989), Al-Wagdany and Rao (1998), and the GIUH based Nash model development is based on the work of Roso, (1984), The detail of model development of the ED-GIUH and GIUH based Nash IUH models are briefly discussed in Section 4.

2.3.4. Use of GIS and Digital Elevation Model (DEM) in Rainfall-Runoff Modeling

GIS processing becomes a critical step, in hydrologic modeling in general and rainfall runoff modeling in particular, since it contributes to generation model parameters distribution in spatial manner. Typical examples on applying GIS in rainfall runoff modeling can be found in Maidment (1993), Schumann et al (2000), Maidment and Djokic (2000). In these applications, the GIS processing steps such as data storing, map overlying, map analysis etc. have helped to drive hydrologic parameters from soil, land cover, rainfall maps etc.

With respect to GIS processing products, Digital Elevation Models (DEM) are more important in rainfall runoff modeling. The development of DEM processing algorithms as well as relevant software to extract hydrologic information from DEM is increasing and makes it widely applied. For example, Tarboton et al (1991) introduced criteria to properly extract drainage networks, Moore et al (1992) reviewed many application of DEM in different disciplines including hydrology, while he also Moore (1996) introduced different algorithms to extract catchments from DEM.

DEM is popularly processed in Arcgis, Arcview (with Hec-Geo-HMS), (Doan, 2000), ILWIS (Maathuis, 2005), Tardem (Tarboton, 1997), etc. to extract hydrologic parameters or physical characterstics of a catchment and can serve for model simulation.

3. STUDY AREA AND DATA COLLECTION

Introduction

Stream flow and precipitation data are the most important parameters in the hydrological rainfall-runoff modeling. Obtaining reliable data over time and space was an essential step before modeling the rainfall-runoff.

3.1. Selection of Gauged Catchments

In developing relationships for calibration of parameters, for a rainfall-runoff model, collection of rainfall and stream flow data for gauged catchments in the region is very important. The first step in collection of the rainfall and stream flow data was selection of the gauged catchments in the basin. About twenty two gauging stations with automatic water level recorders are available in Abay basin. (BCEOM, 1999). Following the recommendation of Shaw (1999), Catchments their size not exceeding 1000km^2 were selected. Further refinement was made based on the availability of automatic or hourly rainfall recording stations in or near the boundary of the stream gauged catchments and the length of their recordings.

Based on the above criteria and the correspondence of rainfall recordings with the collected stream flow data, in four catchments, namely, Andassa and Azuari in North Gojam sub basin and Jedebe and Chemoga in South Gojam Sub basin, data were collected adequately and used in the modeling work. Stream flow recording stations with corresponding rainfall recording stations are summarized in Table 3.1 and Table 3.2.

Table 3. 1: The Selected Catchments and their Locations

Hydrological stations					
Station No	River Name	Site Name	Latitude	Longitude	catchment area(km2)
112018	Azuari	Nr. Motta	10.97	38.02	185.4
113011	Jedebe	Nr. Ammanuel	10.40	37.57	249.8
113008	Chemoga	Nr. D/Markos	10.30	37.73	310.4
112004	Andassa	Nr. BDR	11.50	37.48	670
112017	Muga	Nr. Dejen	10.17	38.25	452

3.2. Stream Flow Data

The Ethiopian Ministry of Water Resource Hydrology Department (MoWRHD) is the responsible body in collection of hydrological data in Ethiopia. The hydrological data required for this study is the stream flood record taken from automatic and continuous water level recorder. The continuous water level recordings of the stream flow, which is a chart, were collected from MoWRHD for different years. The corresponding rating equations also obtained from the same source. The water level readings from charts were discretized for each flood event selected at each of stations. Then the discharge hydrograph was obtained using the rating equations provided. The storms included in this study were selected to met the criteria that isolated peak storm event with significant rainfall amount and observable runoff peaks and occur between may and October.

3.3. Rainfall Data

Rainfall data were collected from the National Meteorological Service Agency (NMSA) that is the responsible organization for the collection and issuing of meteorological data in Ethiopia. Data includes discretized hourly rainfall intensity values and chart readings from which hourly values were discretized. .The obtained rainfall data cover a range of years from 1982-1990 depending on the availability of stream flow records particular to the catchment being considered. The collected data stretched over these years is to obtain adequate match between the rainfall and the resulting stream flow records. Figure 3.1 shows the location map of selected gauging stations

Table 3. 2: The Selected Catchments and their Corresponding Rainfall Stations

Station No	River Name	Rainfall Stations	Latitude	Longitude
112018	Azuari	Mota	11.08	37.87
113011	Jedebe	Debere Markos	10.33	37.67
113008	Chemoga	Debere Markos	10.33	37.67
112004	Andassa	Bahirdar (met)	11.60	37.45
112017	Muga	Yetnora	10.20	38.15

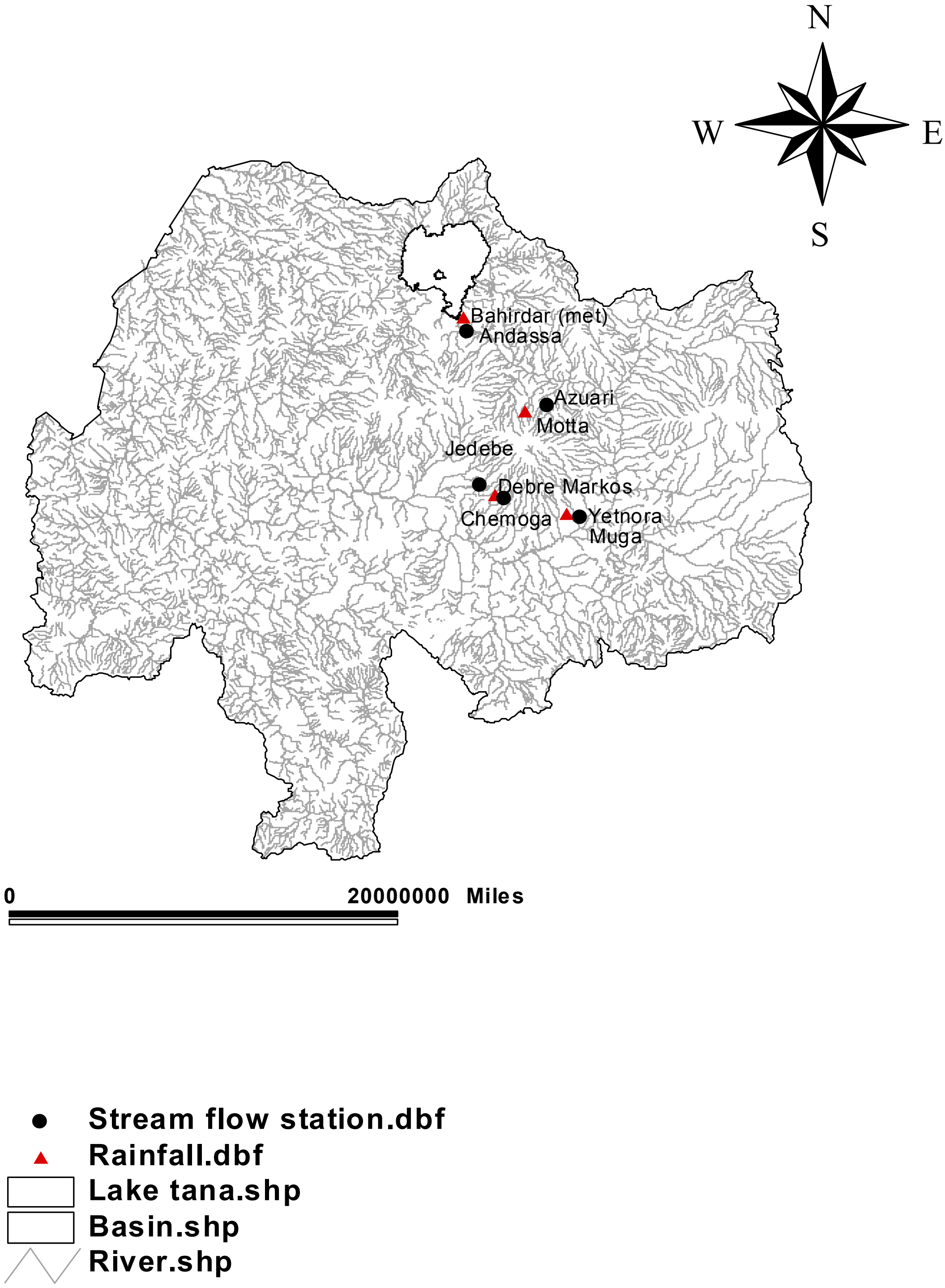


Figure 3-1: Location Map of Selected Gauging Stations

4. METHODOLOGY

4.1. Development of Catchment Average Unit Hydrograph

Each particular rain event results in a unique hydrograph, which shape can vary even if the duration of effective rainfall is similar. It is necessary to build an average catchment unit hydrograph that can be used to forecast the discharge flow for any given rainfall hyetograph. Even if catchment average unit hydrograph gives less accurate predictions of time to peak and peak flow rates than a unit hydrograph that was developed with an intensity that was close to the average intensity of the test event (Kildore, 1997), the averaging process is a necessary step that has to be carried in order to obtain an unit hydrograph that may be applicable to the widest range of rainfall events and antecedent soil moisture conditions.

After the selection of the appropriate concurrent rainfall-runoff events, the derivation of unit hydrograph has been made. The first steps in the derivation of a unit hydrograph are the determination of the effective rainfall from the observed rainfall and the separation of the resulting direct runoff from the total hydrograph.

Base flow separation

Base flow separation is considered to be somewhat arbitrary and there is no reliable method to accurately separate base flow from surface runoff (Bedient and Huber, 1992). For this study a *straight line method* is used. An inclined line was drawn to connect the beginning point of the surface runoff with a point on the recession limb of the hydrograph where normal base flow resumes or straight line drawn from the point of starting of rising limb to the point on recession limb representing the end of direct runoff

Determination of excess rainfall

The area above the base flow separation line and enclosed by the hydrograph is then 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.

The determination of that part of the measured total rainfall that constitutes the effective rainfall is the next step. 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).

Procedures in determining ERH from RH and stream flow hydrograph by ϕ - index

1. Calculate the rainfall data as pulse data and stream flow data as sample data for single storm.
2. Separate the base flow from stream flow and calculate the DRH by subtracting the base flow from the stream flow
3. Compute the volume of direct runoff (Vd) and the equivalent depth of direct runoff rd.

Volume of direct runoff (Vd) = $\sum_{n=1}^{n} Q_n \Delta t$ 4.1

Depth of direct runoff rd = $\dfrac{Volume\ of\ Direct\ runoff\,(V_d)}{Watershed\ area}$

4. Estimate the rainfall abstraction by infiltration and surface storage in the watershed.

$$r_d = \sum_{m=1}^{M} \left(R_m - \phi \Delta t \right) \qquad 4.2$$

Where:

r_d : depth of direct runoff over the catchment (mm)

Δt : time interval length (hr)

R_m : observed rainfall (mm) in time interval m

$\O$: constant rate of abstraction (mm/hr)

Any rainfall prior to the beginning of direct runoff is taken as initial abstraction. The abstraction rate ϕ, and M, the number of non-zero pulses of excess rainfall, are found by trial and error.

- That is, If the value of Ø gives positive result and greater than the initial abstraction, the value of Øis satisfactory.

- If the value of ϕ gives negative result this is not physically possible. To solve for a new trial value of ϕ, taken the value of the highest rainfall by increasing the number of intervals, M

Calculate the excess rainfall hyetograph (ERH). The ordinate of the ERH are found by subtracting $\Phi\Delta t$ from the ordinate of the observed rainfall hyetograph, neglecting all time intervals in which the observed rainfall depth is less than $\Phi\Delta t$. Then check the duration of ERH and total depth of excess rainfall equal to the total depth of direct runoff.

4.1.1 Unit Hydrograph Development

The first and simplest unit hydrograph development approach is to be used only with simple storm event, in which the storm hydrograph has a smooth shape. The 1-hour unit hydrograph was computed by multiplying each ordinate of the direct-runoff hydrograph by the reciprocal of the depth of rainfall excess, which equals the depth of direct runoff (McCuen, 2004). Since the unit hydrograph must have a depth of 1mm and the direct-runoff hydrograph has a volume equivalent to the depth of rainfall excess, the reciprocal of the depth of rainfall excess can be used as a proportionality constant to convert direct-runoff hydrograph to a unit hydrograph.

For the case of complex multi peaked rainfall events that have multiple excess rainfalls the deconvolution technique was used to drive the ordinates of the required unit hydrograph, but the possibility of errors or non linearity in the data is greater than for a single peaked hydrograph. The least square procedures can be used to minimize the error. The underlying criterion in this method consists of the minimization of the sum of the squares of the difference between the measured data and the calculated values (Brutsaert, 2005). These differences are called the *residuals*.

The usual way of deriving the least-squares solution U is to express the linear convolution equation that links excess rainfall, unit hydrograph ordinates and direct runoff response as some matrix equations.

$$Y_i = \sum_{k=1}^{i} X_{i-k+1} . U_k , \qquad 4.3$$

The above equation is then written as $Y = X.U$ where X, U and Y are respectively the excess rainfall matrix, the unit hydrograph ordinates and the direct runoff response. If the excess rainfall input is composed with p bursts, the direct runoff response with m ordinates, the size of the unit hydrograph is $n = m - p + 1$ and the matrixes X, U and Y are constructed as follows:

$$X = \begin{bmatrix} x_1 & 0 & 0 & 0 & 0 \\ x_2 & x_1 & 0 & 0 & 0 \\ . & x_2 & . & 0 & 0 \\ . & . & . & . & 0 \\ x_{p-1} & . & . & . & x_1 \\ x_p & x_{p-1} & . & . & x_2 \\ 0 & x_p & . & . & . \\ 0 & 0 & . & . & . \\ 0 & 0 & 0 & . & x_{p-1} \\ 0 & 0 & 0 & 0 & x_p \end{bmatrix} , \quad U = \begin{bmatrix} u_1 \\ u_2 \\ . \\ . \\ u_{n-1} \\ u_n \end{bmatrix} \quad \text{and} \quad Y = \begin{bmatrix} y_1 \\ y_2 \\ . \\ . \\ y_{m-1} \\ y_m \end{bmatrix}$$

The matrix X having a strictly positive rank, it is reversible and the matrix X^{-1} therefore exists.

When X and Y are known, the solution U is obtained by writing

$$U = (X^T.X)^{-1}.X^TY \qquad 4.4$$

This unit hydrograph derivation is applicable to a wide range of rain events and can be written in a Matlab code, attached in Appendix-A, using Matlab 7.0, in order to make the calculations easier. Finally, application of the least-squares method to derive a unit hydrograph from an observed event often produces an unrealistic solution (unit hydrograph showing oscillation or negative values) and therefore has to be smoothed in order to provide an acceptable shape. The smoothing process was done by temporarily fixing the unit hydrograph extremities to null values, and by running a neighbor to neighbor averaging coupled with a shifting operator.

The unit hydrograph for effective rainfall of duration of 1-hr was then plotted and the area under the curve was checked to see if the enclosed volume is equivalent to unit effective rainfall over the area of the catchment.

4.2. ED-GIUH Model Development

Basins are assumed to respond as linear systems. That is, the discharge at the outlet of the basin as a result of a precipitation event of intensity *i(t)* can be expressed as a convolution integral as follows

$$Q(t) = \int_{o}^{t} i(\tau).u(t-\tau)d\tau \qquad 4.5$$

Where the function *u(t)* is the impulse response function of the system or Instantaneous Unit Hydrograph (IUH)*)* . Rodriguez-Iturbe and Valdes (1979) linked the hydrologic response of watersheds in terms of the IUH or *u(t)* to their geomorphologic parameters, and to a dynamic parameter. They called this instantaneous unit hydrograph as Geomorphological Instantaneous Unit Hydrograph (GIUH). The GIUH is interpreted as the probability density function of the travel times to the out let of the rain drops, which is randomly and uniformly distributed over the catchment. The travel times on hillslope or along the streams are assumed exponential distributed and the initial and transitional probabilities are calculated based on Horton's morphomotric parameters. The parameters are area ratio (R_A), bifurcation ratio (R_B) and length ratio (R_L).

Using Strahler's stream ordering (Strahler, 1969). The Horton ratios can be expressed quantitatively as follows.

Bifurcation ratio (R_B)

$$R_B = \frac{N_i}{N_{i+1}}$$

Where: N_i and N_{i+1} are the number of streams in order *i* and *i+1*. Let Ω represent the highest stream order in the watershed, $i = 1, 2, \ldots, \Omega$

Length ratio (R_L)

$$R_L = \frac{\bar{L}_{i+1}}{\bar{L}_i}$$

Where:

$\bar{L}_i$ is average length of channels of order i is: $\bar{L}_i = \frac{1}{N_i}\sum_{j=1}^{N_i} L_{j,i}$

Area ratio (R_A)

$$R_A = \frac{\bar{A}_{i+1}}{\bar{A}_i}$$

Where:

$\bar{A}_i$ is the mean area of the contributing watershed to streams of order i, $\bar{A}_i = \frac{1}{N_i}\sum_{j=1}^{N_i} A_{j.i}$

A_{ij} represents the total area that drains into the j^{th} stream of ordr i

The R_A, R_B, and R_L values are vary normally between 3 and 5 for R_B, between 1.5 and 3.5 for R_L and between 3 and 6 for R_A (Rodríguez-Iturbe and Valdés, 1983).

The initial probability accounts for a drop falling any hillslope areas, neglecting rain falls on the channel network, in catchment of a given order and represented by θ_1, θ_2, θ_3 and θ_4.

The transition probability accounts for the changing state of a drop from lower order stream to the higher order stream and represented by P_{12}, P_{13}, P_{14}, P_{23}, P_{24} and P_{34}.

The initial probabilities as well as the transition probabilities can be defined as functions of only geomorphologic and geometric parameters. The initial probability of a drop falling in an area of order i is equal to the percent contributing area for the given order. The initial probabilities are thus related to the average ratio of the average area of sub-basins of a given order to the average area of sub-basins of the next higher order (*i.e.*, the area ratio, R_A). Similarly, the transition probabilities from state i (stream order) to state j, where j represents a stream of higher order, are functions of the number of streams of order i draining into streams of order *j*, divided by the total number of streams of order *i*. Thus, the transition probabilities are related to the average ratio of the number of streams of a given order to the number of streams of the next order (*i.e.*, bifurcation ratio, R_B).

The impulse response function denoted by *u(t)*, which is represented as GIUH in this case is expressed as

$$GIHU(t) = \frac{\partial}{\partial t} prob(T_B \le t) \text{ or} \qquad 4.6$$

$$GIHU(t) = \frac{\partial}{\partial t} (\sum_{Si} prob(T_{Si} \le t).prob(S_i))$$

Where

T_B : is the time of travel to the catchment outlet.

Tsi : the travel time in a particular path must be equal to the sum of travel times in the element of that path,

Prob (S_i) : the probability of a drop which will travel all possible paths S_i to the outlet.

Prob (T_{si}) : the probability density function of the total path travel time T_{is}

The number of possible paths is less than or equal to $2^{\Omega-1}$, for fourth order catchment the total number of possible paths S_i of water are equal to 8.

Path 1: $a_1 \rightarrow r_1 \rightarrow r_2 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet.

Path 2: $a_1 \rightarrow r_1 \rightarrow r_2 \rightarrow r_4 \rightarrow$ outlet.

Path 3: $a_1 \rightarrow r_1 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet.

Path 4: $a_1 \rightarrow r_1 \rightarrow r_4 \rightarrow$ outlet.

Path 5: $a_2 \rightarrow r_2 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet.

Path 6: $a_2 \rightarrow r_2 \rightarrow r_4 \rightarrow$ outlet.

Path 7: $a_3 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet

Path 8: $a_4 \rightarrow r_4 \rightarrow$ outlet

Where:

a_i : denoted when the rain drop is in hillslope state of order i, and

r_i : denoted when the rain drop is in channel state of order i.

The path probability of any path is calculated as,

$$prob(S_i) = \theta_j . P_{ij} . P_{jk} ... P_{l\Omega} \qquad 4.7$$

Where:

θ_j :The initial state probabilities and P_{ij} are the transition probabilities.

The transition probabilities quantify the relative frequency with which stream segments of order i flow into stream segments of order j, $j \geq i+1$. For a given drainage network, these probabilities can be estimated as the ratio of the number of streams of order i that flow into streams of order j to the total number of streams of order i . Under the assumption that natural drainage networks are topologically random, the transition probabilities are given by the following expression:

$$p_{i,j} := \begin{cases} \dfrac{2\,N_{i+1}}{N_i} + \dfrac{(N_i - 2\,N_{i+1})\,\mathrm{E}(j,\Omega)}{N_i \left(\displaystyle\sum_{k=i+1}^{\Omega} \mathrm{E}(k,\Omega) \right)} & j = i+1 \\[2ex] \dfrac{(N_i - 2\,N_{i+1})\,\mathrm{E}(j,\Omega)}{N_i \left(\displaystyle\sum_{k=i+1}^{\Omega} \mathrm{E}(k,\Omega) \right)} & otherwise \end{cases} \qquad 4.8$$

In this expression, *E(i, Ω)* represents the expected number of links of order *i* in a network of order Ω.

Under the random topological model assumption, it is given by,

$$E := (i,\Omega) \rightarrow N_i \left(\prod_{j=2}^{i} \frac{N_{j-1} - 1}{2\,N_j - 1} \right)$$

For the 4th order stream network, the above expressions were solved and The transition probabilities can be calculated using following formula.

$$P_{12} = \frac{2}{R_B} + \frac{(2R_B - 1)(R_B^2 - 2R_B)}{R_B^2(2R_B - 1) + R_B(R_B^2 - 1) + (R_B^2 - 1)(R_B - 1)}$$

$$P_{13} = \frac{(R_B^2 - 1)(R_B - 1)}{R_B^2(2R_B - 1) + R_B(R_B^2 - 1) + (R_B^2 - 1)(R_B - 1)}$$

$$P_{14} = \frac{(R_B^2 - 1)(R_B - 1)(R_B - 2)}{R_B^3(2R_B - 1) + R_B^2(R_B^2 - 1) + R_B(R_B^2 - 1)(R_B - 1)}$$

$$P_{23} = \frac{R_B - 2}{2R_B - 1} + \frac{2}{R_B}$$

$$P_{24} = \frac{(R_B - 1)(R_B - 2)}{R_B(2R_B - 1)}$$

$$P_{34} = 1$$

The initial state probability θ_j is equal to the ratio of the area draining directly into streams of order i to the total basin area. It is given by the following expression:

$$\theta_\omega := \begin{cases} \dfrac{N_1 A_1}{A_\Omega} & \omega = 1 \\ \dfrac{N_\omega \left(A_\omega - \left(\displaystyle\sum_{k=1}^{\omega - 1} \frac{p_{k,\omega} N_k A_k}{N_\omega} \right) \right)}{A_\Omega} & \text{otherwise} \end{cases} \qquad 4.9$$

For 4th order stream network, solving the above expressions, the initial state probabilities calculated using the following formula.

$$\theta_1 = \frac{N_1 \bar{A}_1}{A_4}$$

$$\theta_2 = \left(\frac{R_B}{R_A}\right)^2 - \left(\frac{R_B}{R_A}\right)^3 .P_{12}$$

$$\theta_3 = \left(\frac{R_B}{R_A}\right) - \left(\frac{R_B}{R_A}\right)^2 .P_{23} - \left(\frac{R_B}{R_A}\right)^3 .P_{13}$$

$$\theta_4 = [1 - \theta_1 - \theta_2 - \theta_3]$$

The travel times Ts; in particular path must be equal to the sum of travel times in the elements of that path

$$T_{si} = T_{ai} + T_{ri} + T_{ri+1} + \ldots + T_{r\Omega}$$

Where:

T_{ai} :the travel time on the hillslope and T_{ri} travel times in each stream segment of order i $(1 \le i \le \Omega, \Omega$ is the highest order, four in this case).

Assuming that these individual times of travel are independent variables such that f_{Ta} is the probability density function of T_{ai} and f_{Tri} is the probability density function of T_{ri}, the probability density function of the sum, T_{si}, is a multiple convolution integral of the form :

$$prob(T_{si} t) = \sum_{s=S} f_{si}(t)$$, or

$$prob(T_{si} t) = \sum_{s=S} f_{Tai}(t) * f_{Tri}(t) * f_{Tri+1}(t) * \ldots * f_{Tr\Omega} \qquad 4.10$$

Where:

$*$: Convolution operator

$f_{Tai}(t)$: Exponential probability density function corresponding to the travel time of a drop in a given hillslope

$f_{Tri}(t)$: Exponential probability density function corresponding to the travel time of a drop in a given channel

The exponential density functions has the following form

For hillslope

$$f_{Tai}(t) = \alpha_i e^{(-\alpha_i t)}$$

For channels

$$f_{Tri}(t) = \lambda_i e^{(-\lambda_i t)}$$

The parameters α_i and λ_i can be estimated from the following expression;

$$\lambda_i = \frac{V_s}{\bar{L}_i}, and, \alpha_i = \frac{V_0}{L_0}, where, L_o = \frac{1}{2D}$$

Where:

L_o : The average overland flow length, and

D : Drainage density

Then, α is constant for any given hillslope and λ is changed according to the average length of each given order stream,

Therefore the geomorphological instantaneous unit hydrograph can be computed as:

$$GIUH(t) = \sum_{s=S} f_{Tai}(t) * f_{Tri}(t) * f_{Tri+1} * ...prob(S) \tag{4.11}$$

The above equation can be solved as convolution of non identical exponential density function of a give path S_1

$$f_{Si} = \sum_{j=1}^{\Omega} \frac{\lambda_i ... \lambda_\Omega . \exp(-\lambda_j t)}{(\lambda_i - \lambda_j)...(\lambda_{j-1} - \lambda_j)(\lambda_{j+1} - \lambda_j)...(\lambda_\Omega - \lambda_j)}, j \neq i \tag{4.12}$$

The probability density function equation of each of the eight possible paths are computed and attached in Appendix-B.

The density functions were then multiplied with the corresponding path probabilities in discritized time step using excel spreadsheet and the GIUH is determined.

Finally, The IUH coordinates obtained from the ED-GIUH model were converted to the corresponding unit hydrographs for a given time interval Δt using the following equation

$$U_i = \frac{1}{N}(0.5u_{i-N} + u_{i-N+1} + ... + u_{i-1} + 0.5u_i), \qquad 4.13$$

Where:

U_i : The i^{th} ordinate of the unit hydrograph

N : The number of computational intervals

u_i : The i^{th} ordinate of the IUH

The general flow chart of ED-GIUH model development procedure is presented below (Figure 4.1).

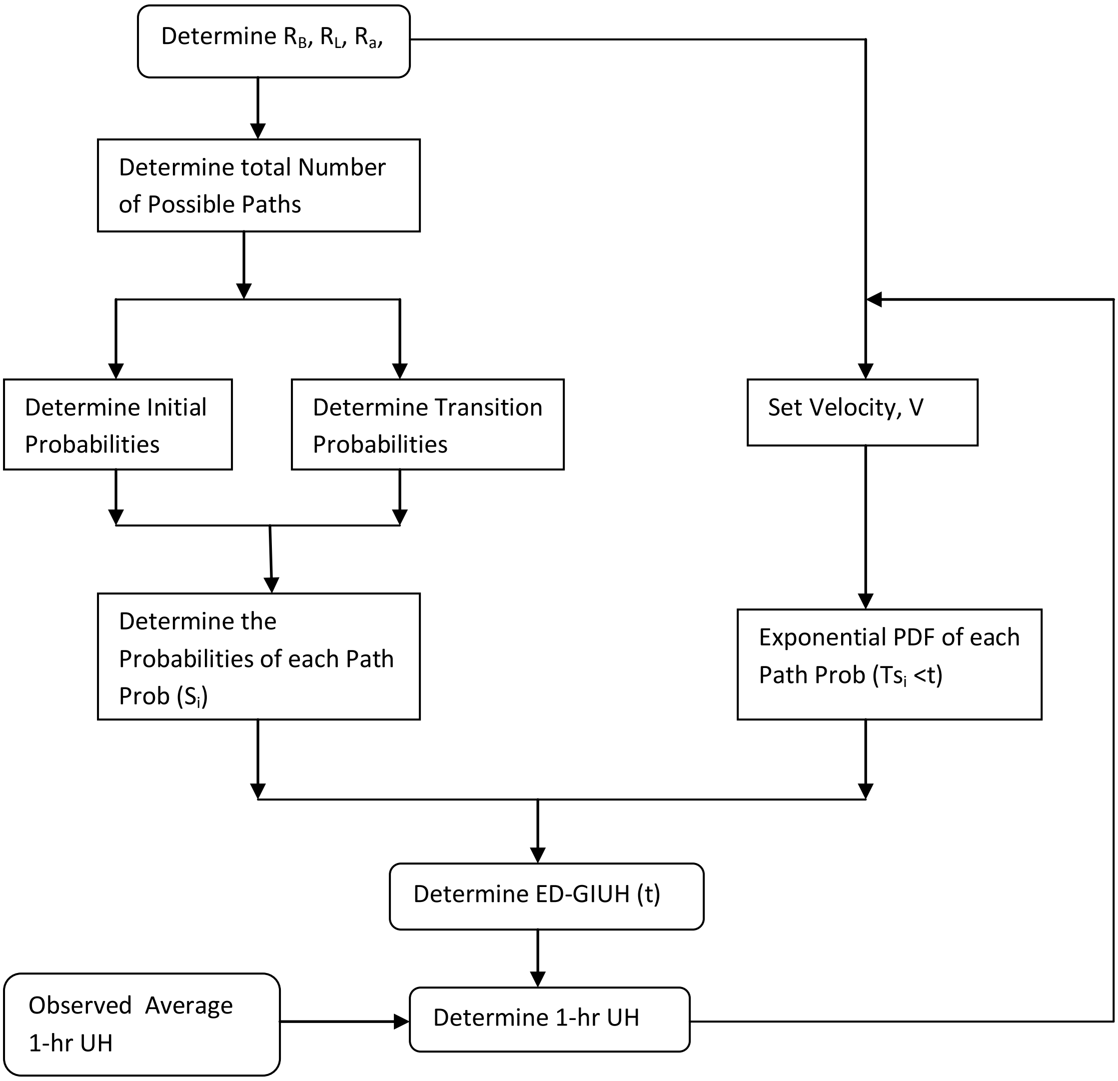


Figure 4-1: Flow Chart of ED-GIUH Model Development

4.3. GIUH Based Nash Model Development

In this model the shape parameter (n) and scale parameter (k) of the Nash's conceptual model of instantaneous unit hydrograph (IUH) were related with Horton's ratios, namely, bifurcation ratio (R_B) length ratio (R_L) and area ratio (R_A) Roso (1984).

Nash (1957) had proposed the concept of hypothetical linear reservoir cascade, which is based on the assumption that operations performed by catchment on 1unit effective rainfall are analogous to those performed by routing through a cascade of *n* identical linear reservoirs in serious to obtain the out flow from the n^{th} reservoir (Figure 4.2).

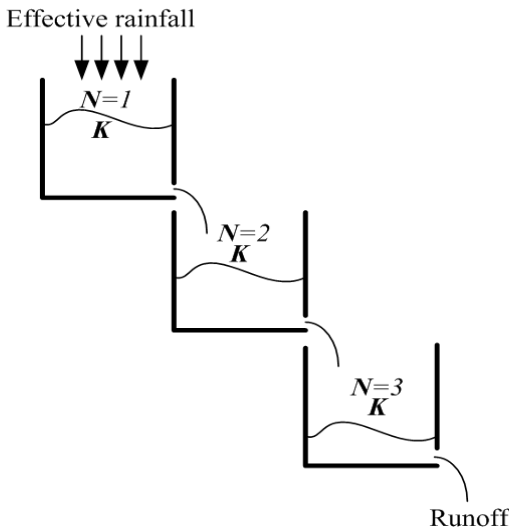


Figure 4. 2: Nash cascade model

For derivation of IUH, the Nash model uses two parameters viz. number of linear reservoirs (*n*), which is dimensionless and storage coefficient (*k*) in hour. The governing equation of the Nash IUH model is expressed as:

$$u(t) = \left[\frac{1}{K\Gamma(n)}\right]\left(\frac{t}{K}\right)^{n-1} e^{\frac{-t}{K}} \tag{4.14}$$

Where:

$u(t)$: The ordinate of IUH at time t,

K : The storage coefficient for the reservoirs, and

$\Gamma(n)$: The standard gamma function.

$$\Gamma(n) = \int_0^{\infty} m^{n-1} e^{-m} dm \qquad 4.15$$

Where:

m :integration or dummy variable.

The corresponding S-hydrograph for the series of n identical linear reservoirs is found by integrating Equation 4.14

$$g(t) = \int_0^{1} u(\tau) d\tau \qquad 4.16$$

Substituting Equation 4.14 and4.15 in the above (Equation 4.16) it can be rewritten as:

$$g(t) = \frac{\int_0^{\frac{t}{K}} m^{n-1} e^{-m} dm}{\int_0^{\infty} m^{n-1} e^{-m} dm} = U\left(n; \frac{t}{K}\right) \qquad 4.17$$

Where;

$U(*,*)$: The ratio of the incomplete gamma function to the gamma function, and can be calculated by the free statistic calculator (Daniel Soper).

The continuous S-hydrograph given in Equation 4.17 is explicitly defined for all values of $t > 0$. Hence, unit hydrographs of any specified duration can be obtained from the Nash IUH without interpolation hydrograph ordinates. It follows that the ordinates of a unit hydrograph of duration D are given by:

$$U(D,t) = \frac{I\left(n; \frac{t}{K}\right) - I\left(n; \frac{t-D}{K}\right)}{D} \qquad 4.18$$

To use this expression it is necessary only to estimate the Nash parameter pair (*n, K*)

Roso (1984) estimate the complete shape of the Nash IUH by linking the t_p and q_p of the Nash IUH with t_p and q_p of the GIUH suggested by Rodriguez-Iturbe & Valdes (1979) that it is adequate to assume a triangular IUH and only specify the time to peak and peak of the IUH. These characteristics have simple expressions obtained by regression of the peak and time to peak of the analytic solution of the ED-GIUH for a wide range of parameters Equation2.1 and Equation 2.2. The t_p and q_p of the Nash IUH can be estimated by equating the first derivative of Equation 4.14 with respect to time t to zero, t becomes the time to peak discharge, t_p

$$\frac{\partial}{\partial t}\ln[u(t)] = \left[-\frac{1}{K} + \frac{(n-1)}{t}\right] = 0,$$

$$t = t_p = K(n-1) \qquad 4.19$$

Substituting the value of t_p of Equation4.19 in Equation4.14 and simplifying, we get

$$q_p = \left[\frac{1}{K\Gamma(n)}\right] e^{-(n-1)} (n-1)^{(n-1)} \qquad 4.20$$

From Equation 4.19 and Equation 4.20, we get

$$q_p.t_p = \left[\frac{(n-1)}{\Gamma(n)}\right] e^{-(n-1)} (n-1)^{(n-1)} \qquad 4.21$$

In order to relate the parameters of Nash model and that of the GIUH of Rodriguez-Iturbe, Equating Equation 4.21 with the product of Equation 2.1 and 2.2 it could be written as;

$$\left[\frac{(n-1)}{\Gamma(n)}\right] e^{-(n-1)} (n-1)^{(n-1)} = 0.5764\left[\frac{R_B}{R_A}\right]^{0.55} \times R_L^{0.05} \qquad 4.22$$

All the terms in the right-hand side of Equation are known. The only unknown term is the Nash model parameter n, Rosso (1984), determined the value of n by using Newton-Raphson non-linear optimization technique and is expressed as,

$$n = 3.29\left(\frac{R_B}{R_A}\right)^{0.78} R_L^{0.07} \qquad 4.23$$

Equating equation 4.19 and 2.2, the relationship for obtaining the value of K for a given velocity V could mathematically be expressed as;

$$K = \frac{0.7}{V}\left(\frac{R_A}{R_B R_L}\right)^{0.48} L_\Omega \qquad 4.24$$

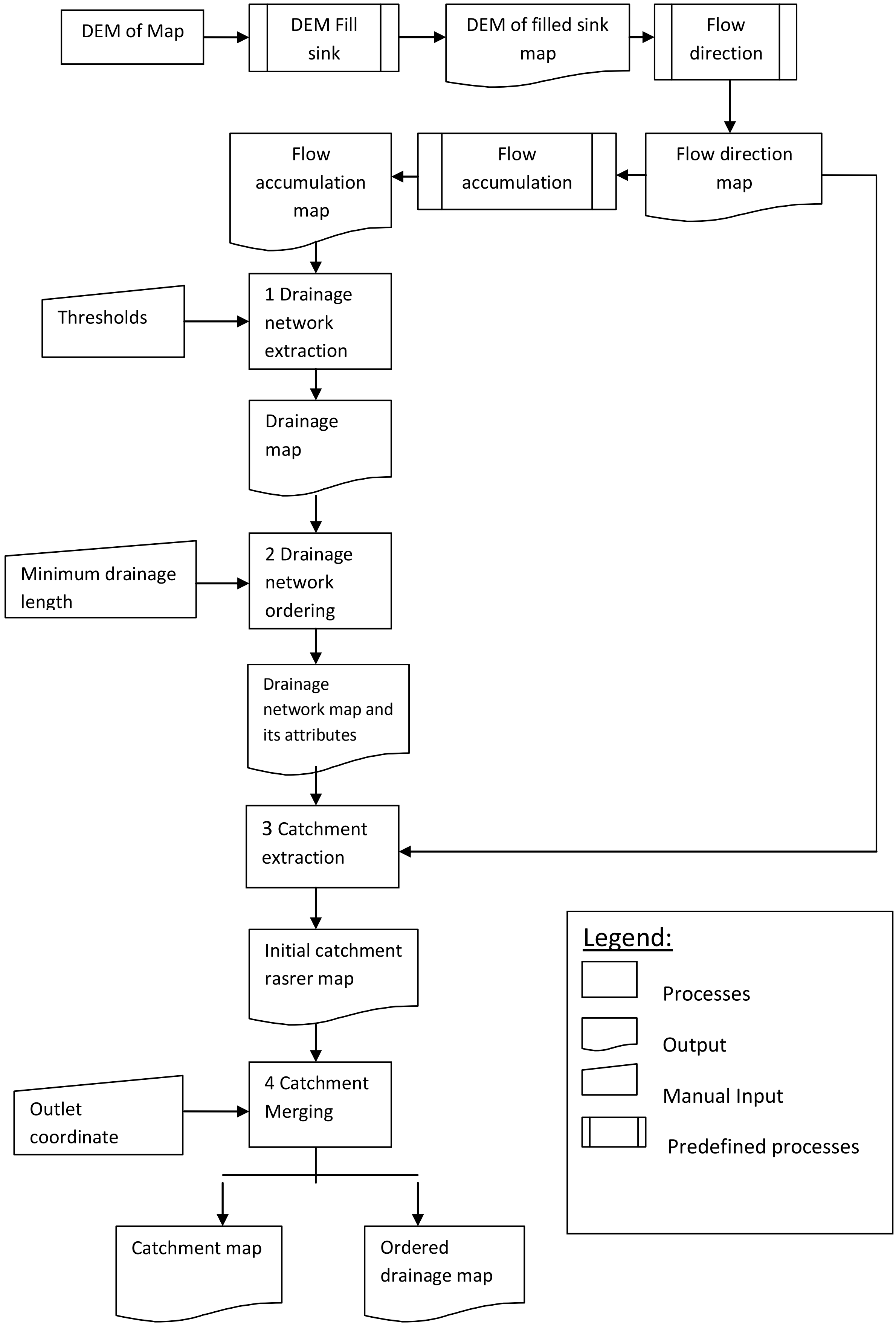
The scale parameter of the model is time variant and depends on both, the catchment geomorphology and the stream flow velocity along the stream network of the watershed. In this study, the velocity parameter was set as a calibrating parameter of the model. The derived values of *n* and *k* were used to determine the complete shape of GIUH based Nash model using Equation 4.14. Subsequently, the D-hour UH was obtained using the relationship between IUH and UH of D-hour as given by Equation 4.18.

To complete the computation of the IUHs of both models, extraction of the geomorphologic parameters from the Digital Elevation Model (DEM) of each catchment under the study is presented below.

4.4. Geomorphological Characteristics Extraction from Digital Elevation Model (DEM) Processing

The Digital Elevation Model is a digital topographic map, which contains the elevation of all the points located at the region. A 90 ×90 DEM data was obtained from Ethiopian Ministry of Water Resource GIS Department. ILWIS (Integrated Land and Water Information System) – GIS package developed at ITC, Netherlands, is used in this study to process the DEM to extract the study catchments as well as the topological drainage network. Newly implemented routine, called DEM hydro-processing module in ILWIS allows to extract ,through several steps (Figure 4.2) ,the Horton statistics such as the number of streams, the average length of streams, the average area of the catchments of a given strahler order as well as the Horton ratios R_B, R_L, R_A .

Figure 4. 3: DEM processing algorithm of ILWS



4.5. Model Evaluation Criteria

In order to evaluate how well the geomorphological unit hydrograph techniques predict an observed hydrograph, the following objective functions were used. The hydrographs were evaluated based on a visual comparison, an evaluation of the model Nash-Sutcliffe efficiency (NSE), percentage error in peak discharge (PEP), and percentage error in time to peak (PETP). Based on the results of these objective functions the predictive capabilities of the models were evaluated.

$$NSE = 1.0 - \frac{\sum_{i=1}^{n}(Q_{oi} - Q_{pi})^2}{\sum_{i=1}^{n}(Q_{oi} - \bar{Q})^2} \qquad 4.25$$

$$PEP = \frac{(Q_{op} - Q_{pp})}{Q_{op}} \times 100 \qquad 4.26$$

$$PETP = \frac{(T_{op} - T_{pp})}{T_{op}} \times 100 \qquad 4.27$$

Where:

Q_{oi} : Ordinate of observed discharge

Q_{pi} : Ordinate of predicted discharge

Q_{op} : Ordinate of observed peak discharge

Q_{pp} : Ordinate of predicted peak discharge

T_{op} : Time to peak discharge of observed hydrograph

T_{pp} : Time to peak discharge of predicted hydrograph

$\bar{Q}$: Average of the ordinates of observed discharge

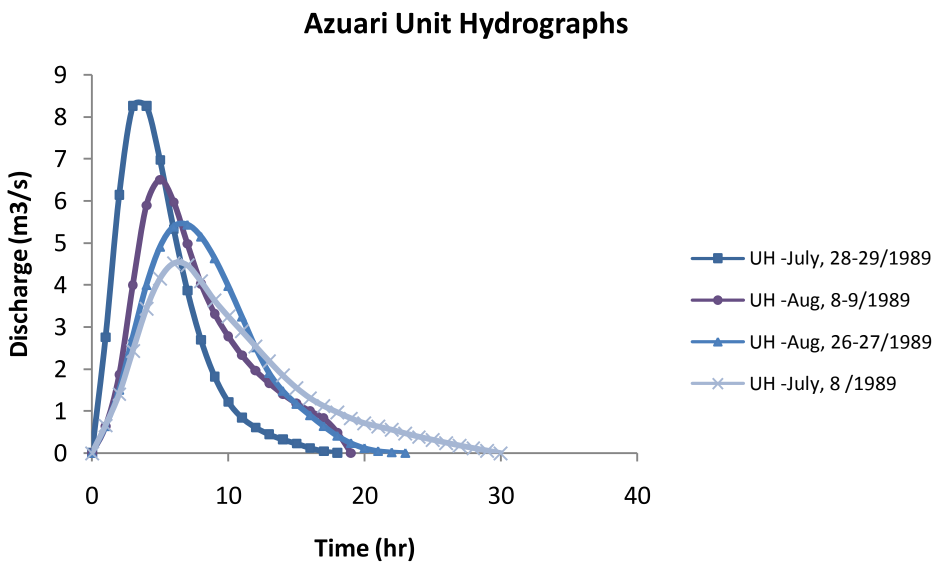
Nash and Sutcliffe (1970) first proposed the dimensionless coefficient of model efficiency (NSE). An NSE value of 1 indicates a perfect fit between the simulated and observed hydrographs.

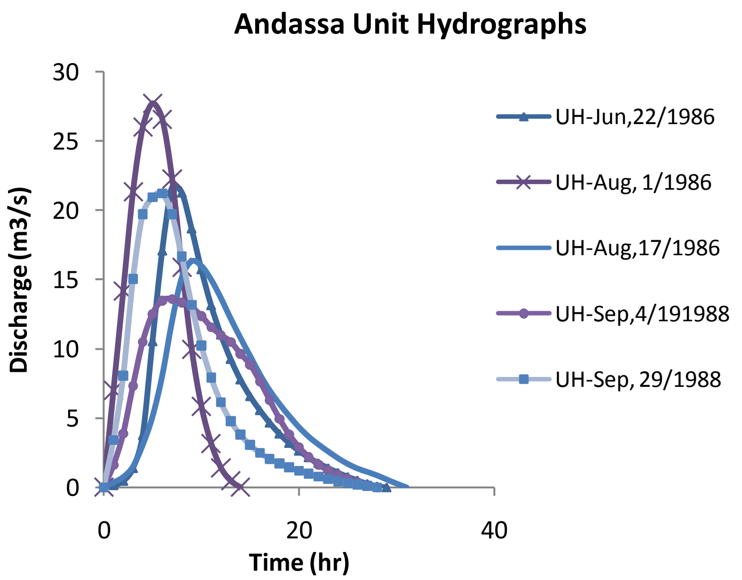
The geomorphological unit hydrograph techniques were compared based on visual comparison as well as these statistics. The results of these comparisons may be found in Section 5.

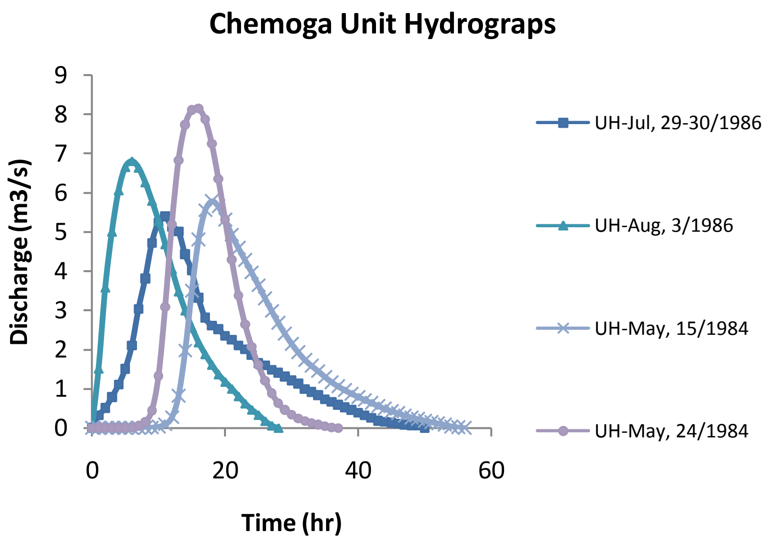
5. RESULTS AND DISCUSSION

5.1. Catchment Average Unit Hydrograph

Using the procedures outlined in part 4,1-hr unit hydrograph for all the selected concurrent rainfall-runoff events of all catchments were derived and shown in Figure 5.1.







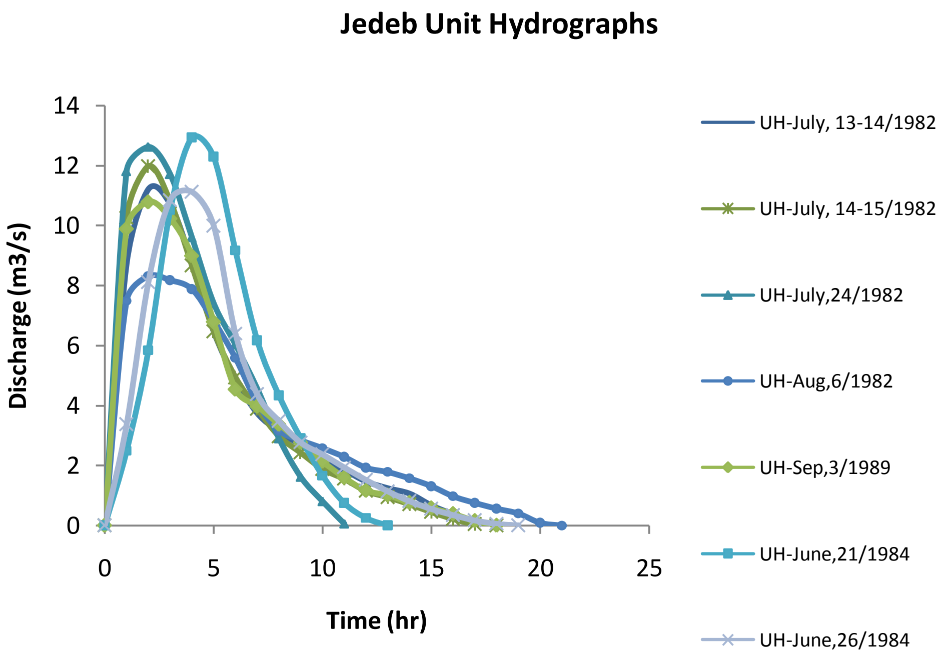
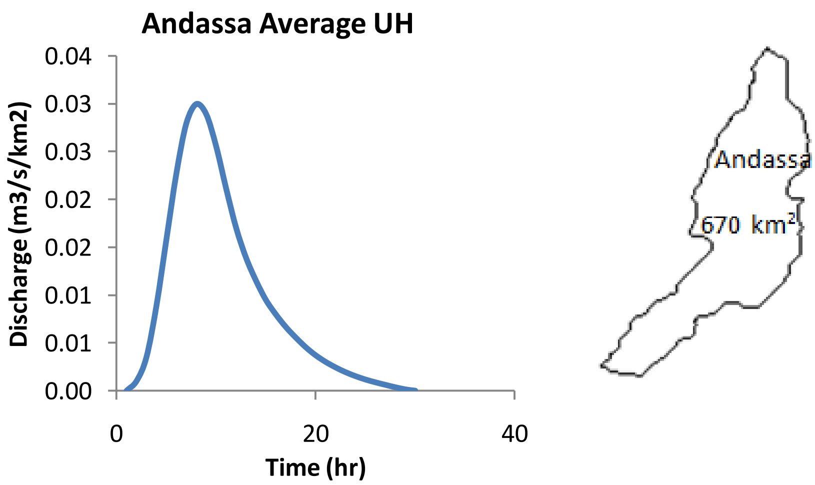
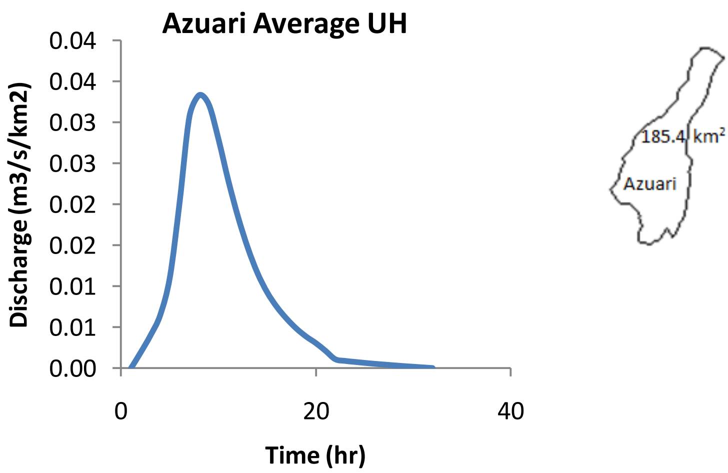


Figure 5. 1: 1-hr unit hydrographs of the catchments.

5.1.2 Averaging Unit Hydrographs

When all the singled peaked storms have been analyzed and a corresponding number of unit hydrographs obtained for a catchment, 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). The resulting unit hydrograph is then checked to ensure that the enclosed volume of runoff is equivalent to a unit of effective rainfall. Using the above approach, the average unit hydrograph were obtained for all four catchments used in this study, as shown in Figure 5.2.





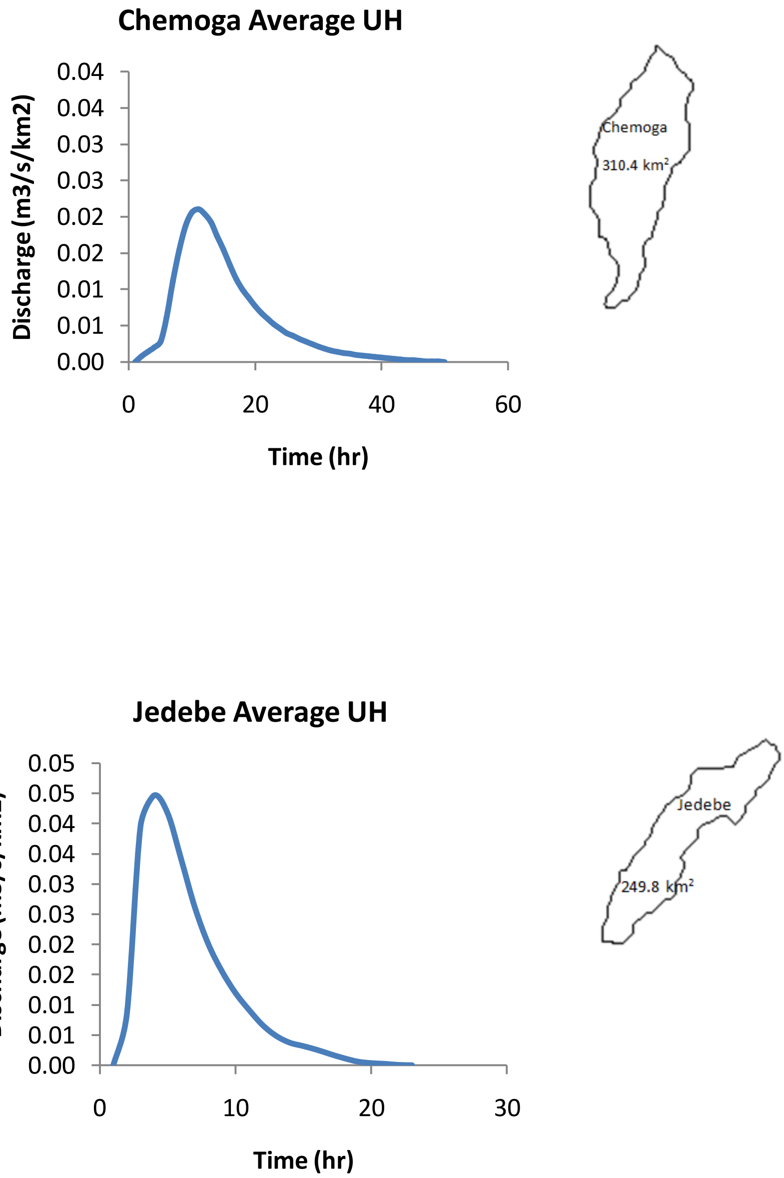


Figure 5. 2: Average unit hydrographs of the catchments

5.2. Geomorphologic Characteristics of the Catchments

The Horton Statistics operation calculates for all streams with stream order i, the number of streams, the average stream length (km), and the average area of catchments (km^2).The output is then stored in a table which can be used to construct so-called Horton plots. The Horton plots, which is obtained by calculating the expected values of the number of streams, the average stream length and the average area of the catchment by means of least square fit, enables to inspect the regularity of the extracted stream network based on the (Strahler) stream order numbers, and may serve as a quality control indicator for the entire stream network extraction process Matthus (2005). It is expected that:

- The number of streams show a relative decrease for subsequent Strahler order numbers,
- The length of streams and the catchment areas show a relative increase for subsequent Strahler order numbers

The Horton statistics output table of the catchments is shown in Table 5.1below and the Horton plots of the catchments attached in Appendix-A.

Therefore, from the Horton plots and the values of the Horton's ratios, it can be clearly seen that the drainage network is well extracted and the Horton's ratios are fall within the expected range.

Table 5. 1: Horton Statistics of the Catchments

Catchments	Stream Order	Number of Streams	Mean Stream Length (km)	Mean Area (km^2)	Horton's Ratio R_B	R_L	R_A	Drainage Density (m/km^2)	Longest flow path length (km)	Average Slope
Azuari	1	48	1.03	1.88	3.46	2.68	4.42	615.04	37.56	0.047
	2	9	4.34	14.32						
	3	4	10.57	42.39						
	4	1	20.41	185.55						
chemoga	1	72	1.13	1.91	4.28	3.33	5.66	559.37	44.59	0.030
	2	11	4.64	19.17						
	3	2	29.73	151.65						
	4	1	33.39	310.50						
Jedebe	1	57	1.01	1.68	3.86	3.66	5.44	538.98	43.86	0.038
	2	8	3.38	16.26						
	3	2	25.82	111.92						
	4	1	38.74	250.27						
Andassa	1	90	1.95	4.48	4.45	2.74	5.23	505.79	58.13	0.017
	2	21	6.96	27.23						
	3	5	11.69	123.00						
	4	1	47.06	671.06						
Muga (for validation)	1	128	1.19	1.91	4.62	2.52	5.53	538.98	66.15	0.027
	2	31	2.86	10.01						
	3	6	7.57	58.40						
	4	1	64.09	452.78						

5.3. ED-GIUH Model Results

The initial and transitional probabilities as well as the path probabilities for each possible water travel paths were calculated, from Horton ratios, and tabulated below in Table 5.2 to 5.4.

Table 5. 2: Initial State Probabilities for each catchments

Initial Probabilities	Values				
	Azuari	Jedebe	Chemoga	Andassa	Muga
Θ_1	0.48	0.36	0.43	0.62	0.58
Θ_2	0.23	0.23	0.25	0.27	0.28
Θ_3	0.18	0.24	0.23	0.17	0.19
Θ_4	0.10	0.18	0.03	-0.14	-0.05

Table 5. 3: Transition Probabilities for each catchments

Transition Probabilities	Values				
	Azuari	Jedebe	Chemoga	Andassa	Muga
P_{12}	0.80	0.77	0.74	0.73	0.72
P_{13}	0.20	0.21	0.21	0.21	0.21
P_{14}	0.13	0.16	0.19	0.19	0.20
P_{23}	0.82	0.79	0.77	0.76	0.75
P_{24}	0.18	0.21	0.23	0.24	0.25
P_{34}	1.00	1.00	1.00	1.00	1.00

Table 5. 4: Path Probabilities Prob(S) for each catchments

Path Probabilities	Values				
	Azuari	Jedebe	Chemoga	Andassa	Muga
Prob(S_1)	0.32	0.22	0.25	0.34	0.32
Prob(S_2)	0.07	0.06	0.07	0.11	0.11
Prob(S_3)	0.10	0.07	0.09	0.13	0.13
Prob(S_4)	0.06	0.06	0.08	0.12	0.12
Prob(S_5)	0.19	0.18	0.19	0.21	0.21
Prob(S_6)	0.04	0.05	0.06	0.07	0.07
Prob(S_7)	0.18	0.24	0.23	0.17	0.19
Prob(S_8)	0.10	0.18	0.03	-0.14	-0.05

The probability density function of travel time of the eight possible paths, obtained from the non identical convolution of the exponential probability density functions, were calculated as a function of the hillslope/stream flow velocities and time step using the equations explained in Appendix-B. Setting initial values for the hillslope/stream flow velocities, the calculation was done using Excel spreadsheet in discrete time step. The optimum velocity of the model was determined through calibration (Equation 5.1) as discussed below. After computing the ED-GIUH, 1-hr unit hydrograph was determined using Equation 4.13.

5.4. GIUH Based Nash Model Results

In order to determine the complete shape of Nash IUH it is enough to estimate the shape parameter n and the scale parameter K. The shape parameter n is only a function of the Horton ratios and this parameter was determined from the Horton ratio given in Horton statistics table, but the scale parameter of the model is time variant and depends on both, the catchment geomorphology and the stream flow velocity along the stream network of the watershed. Therefore, the scale parameter K was determined through calibration of the velocity parameters using Equation 5.1.

5.5. Calibration of the Models

All the parameters of the ED-GIUH and the GIUH based Nash models can be estimated from the geomorphological characteristics of the catchments except the dynamic parameter V, the channel velocity and the hillslope velocities in the case of ED-GIUH model and only the channel velocity in the case of GIUH based Nash model.

Al-Wagdany and Rao (1998) suggested that this non-measurable parameter is assumed to be calibrated through optimization. The optimum values of the velocities of the two models are those that produce simulated hydrograph similar to the observed hydrograph.

In this study, the error function defined by Lee (1972) was adopted, so as the time to peak and peak discharges of the estimated unit hydrograph is close to that of the corresponding values of the observed unit hydrograph.

$$ERR = \left[\left(\frac{Q_{po} - Q_{pc}}{Q_{pc}} \right)^2 + \left(\frac{T_{po} - T_{pc}}{T_{pc}} \right)^2 \right] \quad 5.1$$

Where:

Q_{po} : The peak discharge of the observed hydrograph

Q_{pc} : The peak discharge of the computed hydrograph

T_{po} : The time to peak of the observed hydrograph

T_{pc} : The time to peak of the computed hydrograph

Using the above error function, the channel velocities of the two models and the hillslope velocities of ED-GIUH model were calibrated simultaneously and manually so that the error becomes a minimum. The results of optimization of the models are shown below (Table 5.5).

Table 5. 5: Optimization result of the models

	Error %	
Catchment Name	ED-GIUH	GIUH based Nash
Azuari	4.00	6.96
Jedebe	0.00	1.82
Chemoga	18.36	8.33
Andassa	16.00	4.31

The models were able to simulate the unit hydrographs of the catchments. The high values of the error function in Chemoga and Andassa, in the case of ED-GIUH, were due to relatively higher lag of time to peak discharge of the observed and simulated unit hydrographs.

The calibrated hillslope velocity and channel velocity for ED-GIUH model is presented in Table 5.6. From this table it is clear that the values of the calibrated hillslope velocities are close to each other. For the sake of simplification an average velocity of the value of the hillslope velocities (0.0925m/s) was considered as a constant hillslope velocity for all catchments. Hence, the channel velocities of this model were re-calibrated to have the same error and the values are presented in Table 5.7.

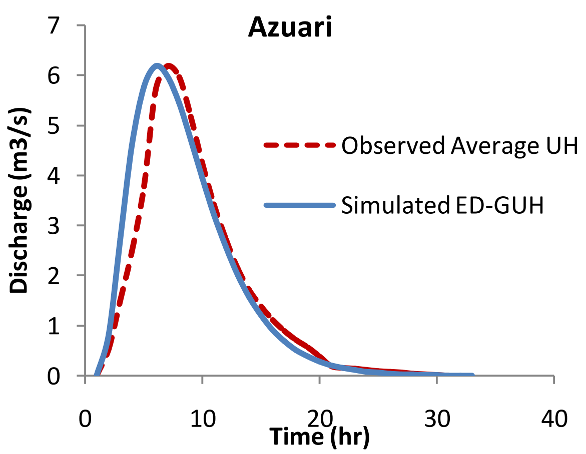
Table 5. 6: Calibrated velocity parameters of ED-GIUH

Catchment	Hillslope velocity Vo (m/s)	Channel velocity V (m/s)
Azuari	0.0920	1.920
Jedebe	0.0930	7.610
Chemoga	0.0935	2.150
Andassa	0.0915	3.863

Table 5. 7: Re-Calibrated velocity parameters of ED-GIUH

Catchments	Hillslope velocity Vo (m/s)	Channel velocity V (m/s)
Azuari	0.0925	1.915
Jedebe	0.0925	7.650
Chemoga	0.0925	2.165
Andassa	0.0925	3.814

The simulated unit hydrographs using the optimum velocity parameters (Table 5.7) of the ED-GIUH model together with the observed average unit hydrographs of the catchments are presented below (Figure 5.3).



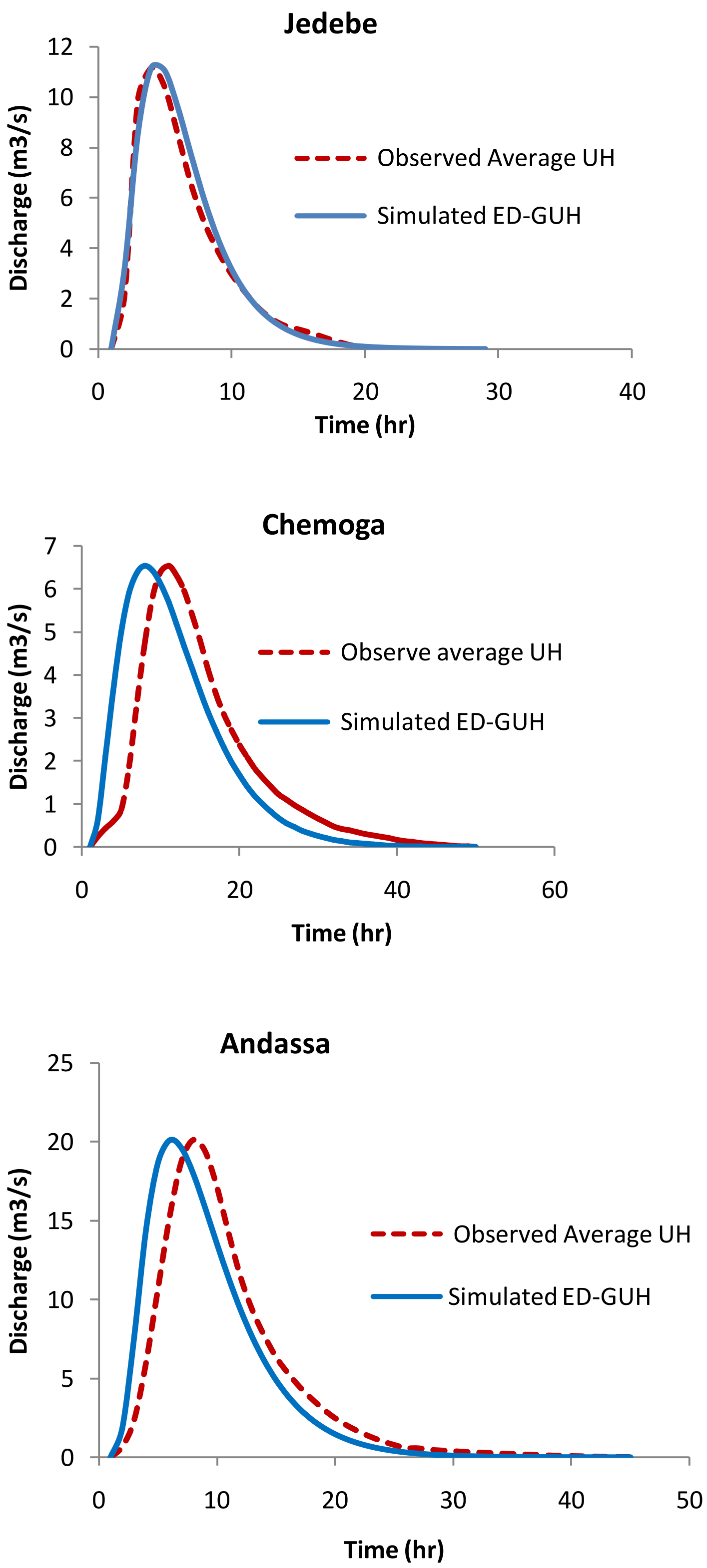


Figure 5. 3: Observed and Simulated unit hydrographs using ED-GIUH model

For GIUH based Nash model, since the scale parameter is a function of the channel velocity, this parameter was calibrated simultaneously and manually based on equation 5.1. The channel velocities and the Nash parameters after calibration are presented in Table 5.8 and Table 5.9 respectively.

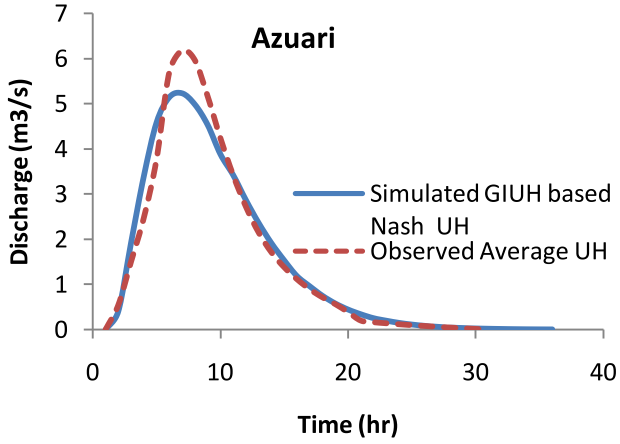
Table 5. 8: Calibrated velocity parameters of GIUH based Nash model

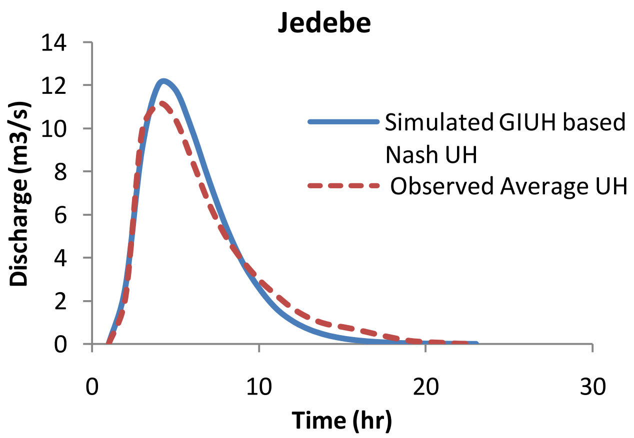
Catchment	Channel velocity V (m/s)
Azuari	2.90
Jedebe	8.80
Chemoga	2.60
Andassa	5.70

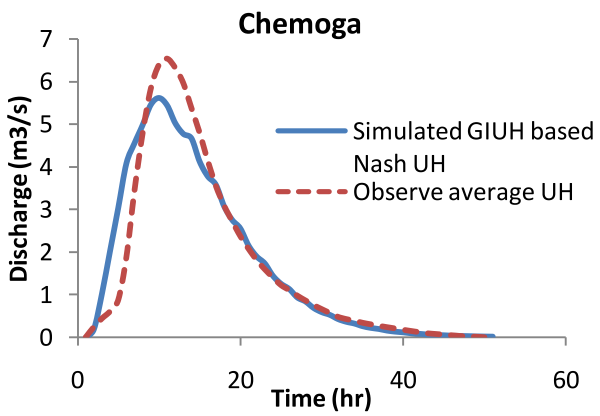
Table 5. 9: Derived Nash Parameters after Calibraition

Catchments	*n*	*K(hr)*
Azuari	2.91	2.72
Jedebe	2.76	1.61
Chemoga	2.88	4.47
Andassa	3.11	2.82

Having the values of the Nash parameters above (Table 5.9), the S-hydrographs developed by Equation 4.17, to determine the complete shape of IUHs of the Nash model, are attached in Appendix-C. Finally the 1hr unit hydrograph was obtained using the relationship between IUH and UH of D-hour as given by Equation 4.18. The simulated unit hydrographs using the optimum velocity parameters (Table 5.7) of the GIUH based Nash model together with the observed average unit hydrographs of the catchments are presented below (Figure 5.4).


Azuari
Discharge (m3/s)
Time (hr)
Simulated GIUH based Nash UH
Observed Average UH
0
1
2
3
4
5
6
7
0
10
20
30
40


Jedebe
Discharge (m3/s)
Time (hr)
Simulated GIUH based Nash UH
Observed Average UH
0
2
4
6
8
10
12
14
0
10
20
30


Chemoga
Discharge (m3/s)
Time (hr)
Simulated GIUH based Nash UH
Observe average UH
0
1
2
3
4
5
6
7
0
20
40
60

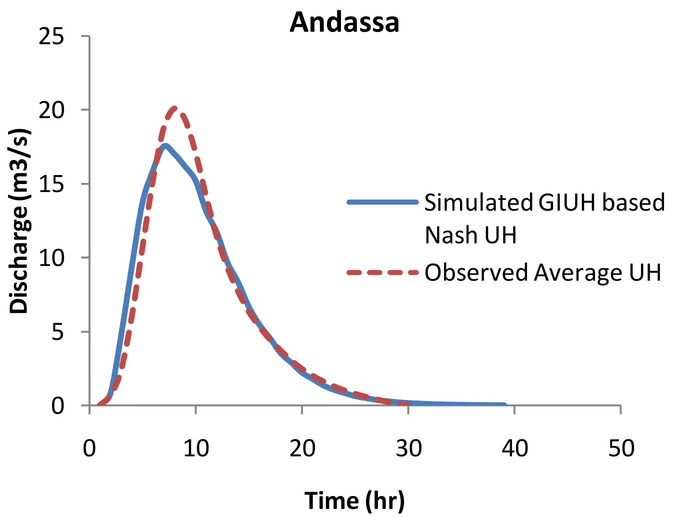


Figure 5. 4: Observed and Simulated unit hydrographs using GIUH based Nash model.

During optimization when the stream velocity increases, the magnitude of peak flow of the simulated unit hydrograph increases, while the time to peak decreases. Therefore, during calibration the error is minimized by increasing the velocity staring from low velocity values.

From the calibration result (Tables 5.7 and 5.8,) it is clearly seen that the optimized stream velocity parameter of Jedebe was obtained extremely very high. This is due to; the observed unit hydrograph of Jedebe has very short time to peak and relatively high peak discharge. Hence, to simulate this hydrograph the velocity gets very high.

The catchment characteristics as well as the rainfall charactristcis of this catchment are not uniquely different from other catchments.

In addition, in an attempt to relate the velocity parameters of the catchments with their geomorphological characteristics, the velocity parameter of Jedebe makes the statistical analysis invalid.

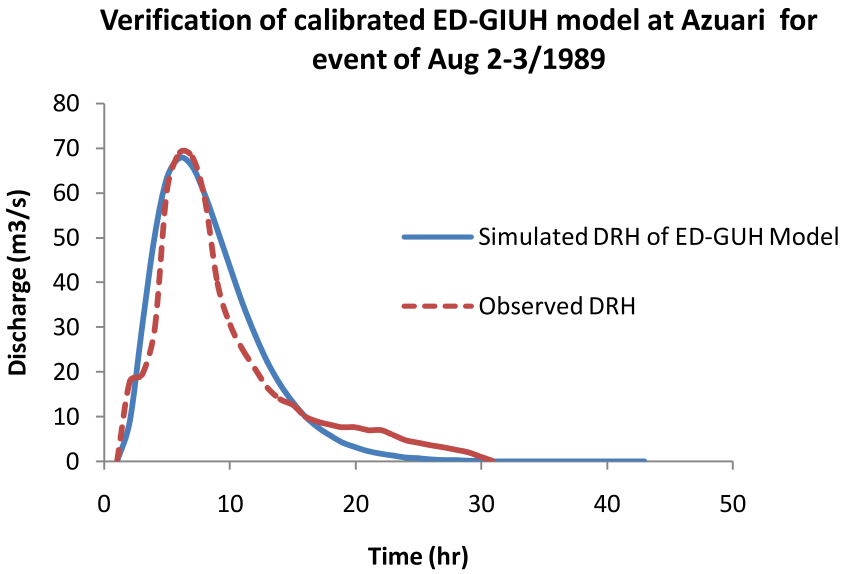
Therefore, the stream flow data as well as rainfall data need to be analyzed to identify the problem with this catchment. In this study, due to the above reasons, this catchment was not used in further analysis afterwards.

5.6. Verification of the Models

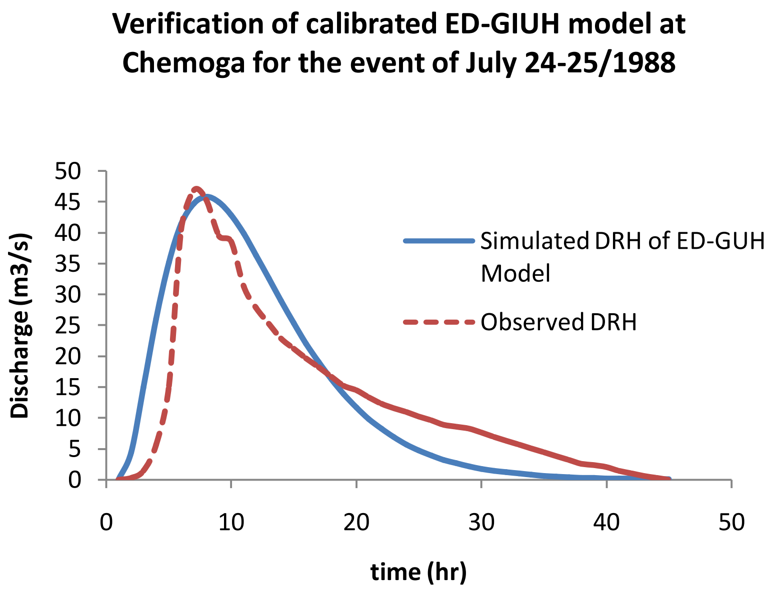
Verification refers to the testing of calibrated values, generally with data not used for calibration. It enables assessment of the reliability of the calibrated model.

After separating the base flow and calculating the excess rainfall by the same procedure as presented in Section 3, the observed direct runoff hydrographs were determined in the three catchments. The observed direct runoff hydrographs were then compared with the direct runoff hydrographs derived from the ED-GIUH and GIUH base Nash model using the 1-hr unit hydrograph and excess rainfall hyetographs of the same storm used in the derivation of the observed direct runoff hydrographs.

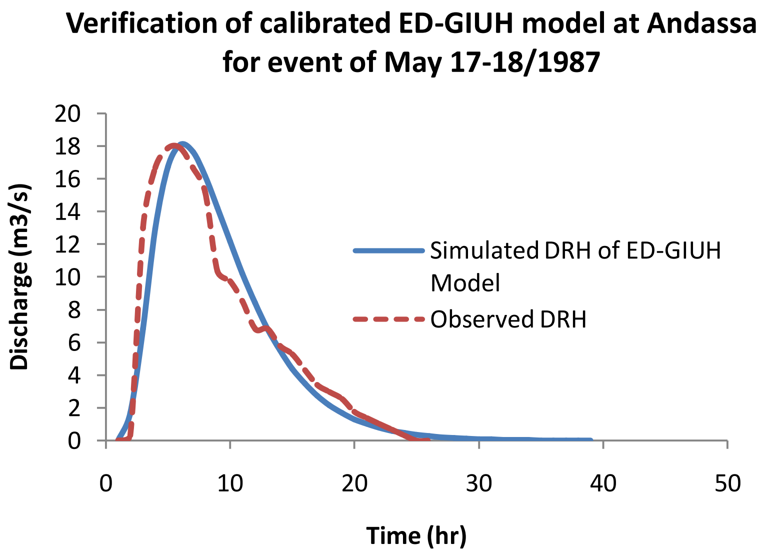
Figure 5.5a-c shows the comparison of the observed and predicted direct runoff hydrographs of ED-GIUH model. The plots show that the model can also adequately predict the peak discharge of the surface runoff.



(a)



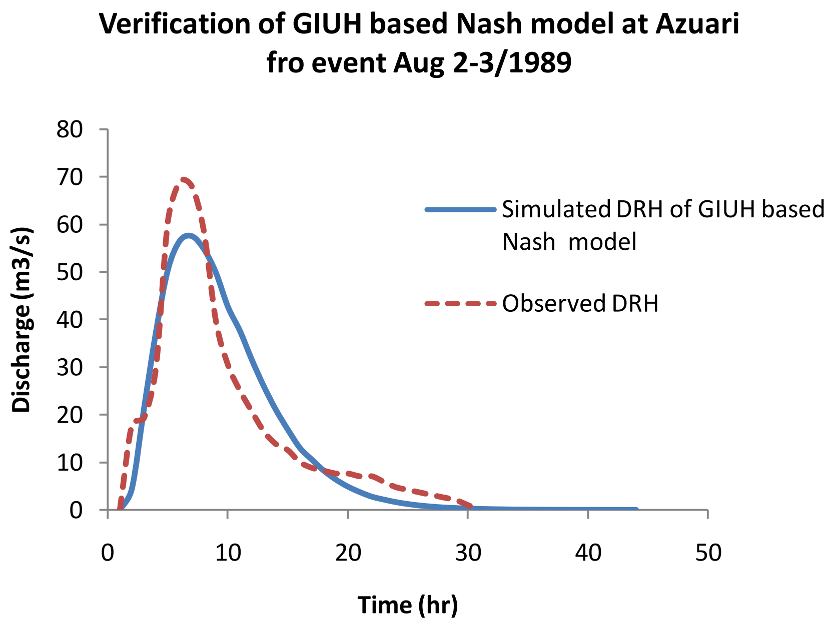
(b)



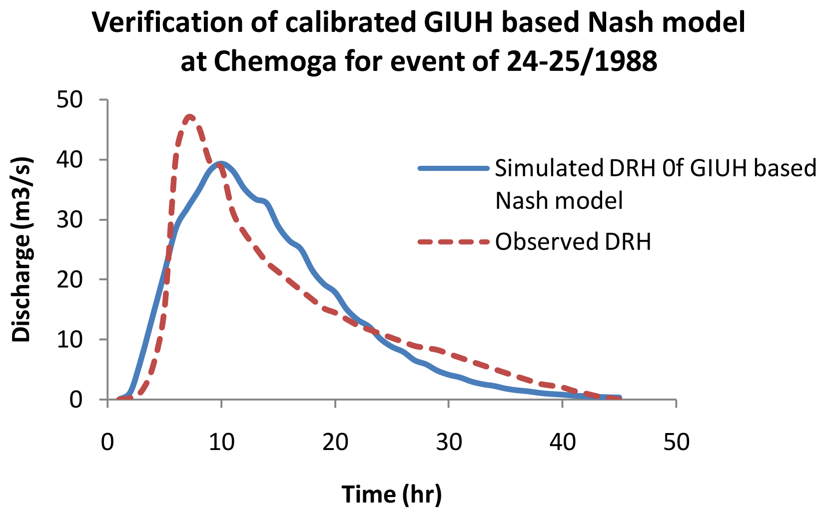
(c)

Figure 5. 5: Verification result of the calibrated ED-GIUH model.

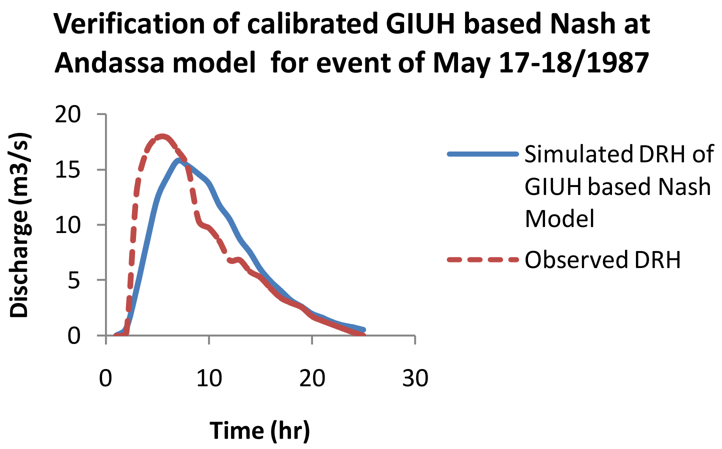
Figure 5.6a-c shows the comparison of the observed and predicted direct runoff hydrographs of GIUH based Nash model.


Verification of GIUH based Nash model at Azuari
fro event Aug 2-3/1989
Discharge (m3/s)
80
70
60
50
40
30
20
10
0
0
10
20
30
40
50
Time (hr)
Simulated DRH of GIUH based Nash model
Observed DRH

(a)


Verification of calibrated GIUH based Nash model
at Chemoga for event of 24-25/1988
Discharge (m3/s)
50
40
30
20
10
0
0
10
20
30
40
50
Time (hr)
Simulated DRH 0f GIUH based Nash model
Observed DRH

(b)



(c)

Figure 5. 6: Verification result of the calibrated GIUH based Nash model model.

After calibration, peak discharge and time to peak of the unit hydrographs of each catchment are presented in the following tables.

Table 5. 10: Peak Discharge of the unit hydrographs

Catchments	Observed Average UH	ED-GUH	GIUH based Nash
Azuari	6.18	6.18	5.240
Jedebe	11.17	11.17	12.06
Chemoga	6.54	6.54	5.610
Andassa	20.10	20.10	17.56

Table 5. 11: Time to peak discharge of the unit hydrographs

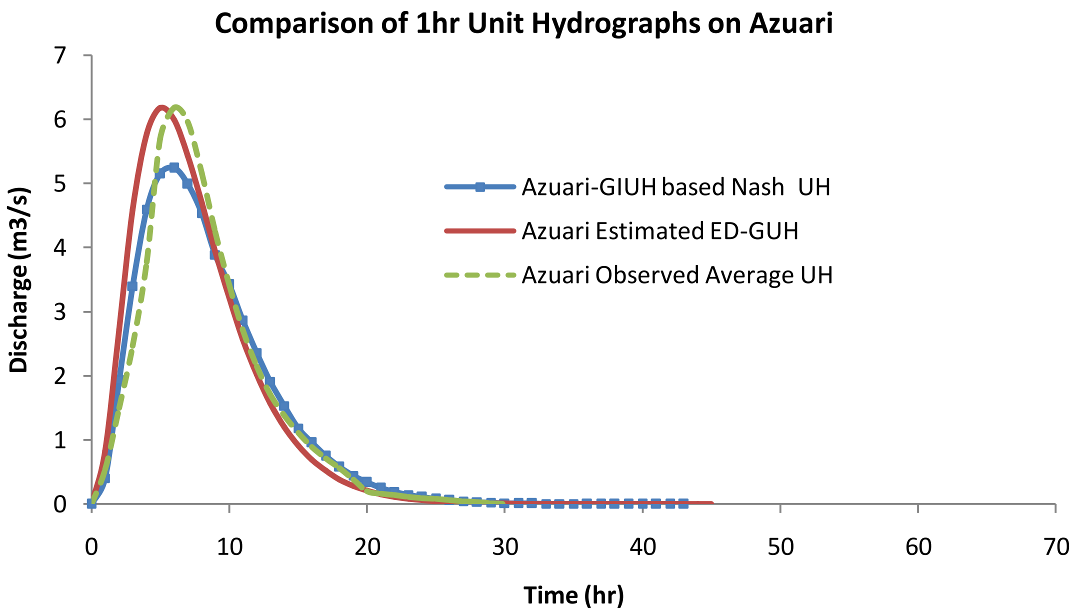
Catchments	Observed average UH	ED-GUH	GIUH based Nash
Azuari	7	5	6
Jedebe	3	3	3
Chemoga	10	7	9
Andassa	7	5	6

5.7. Comparison of Models

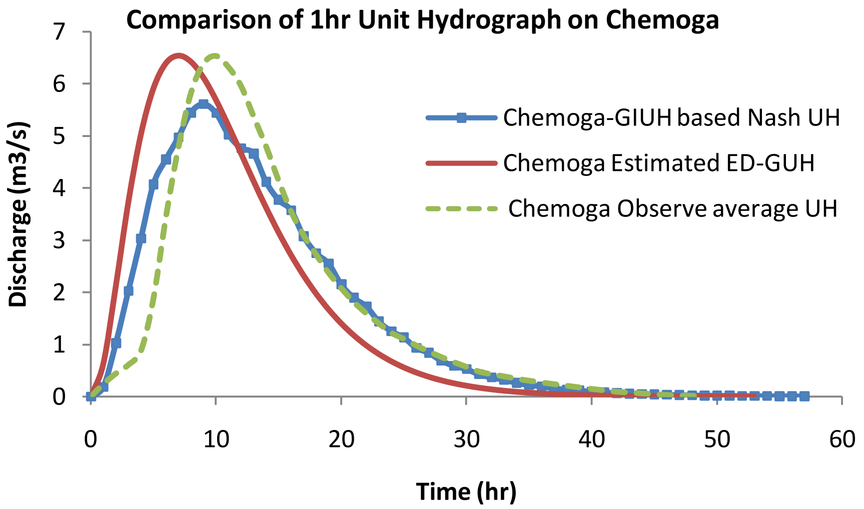
5.7.1. Visual Comparison

An ASCE task committee (1993) has recommended that both visual and statistical comparisons be made between the observed and predicted hydrographs whenever possible. Visual comparisons of simulated and observed hydrographs can provide a fast and comprehensive way to evaluate the overall accuracy of the model output.

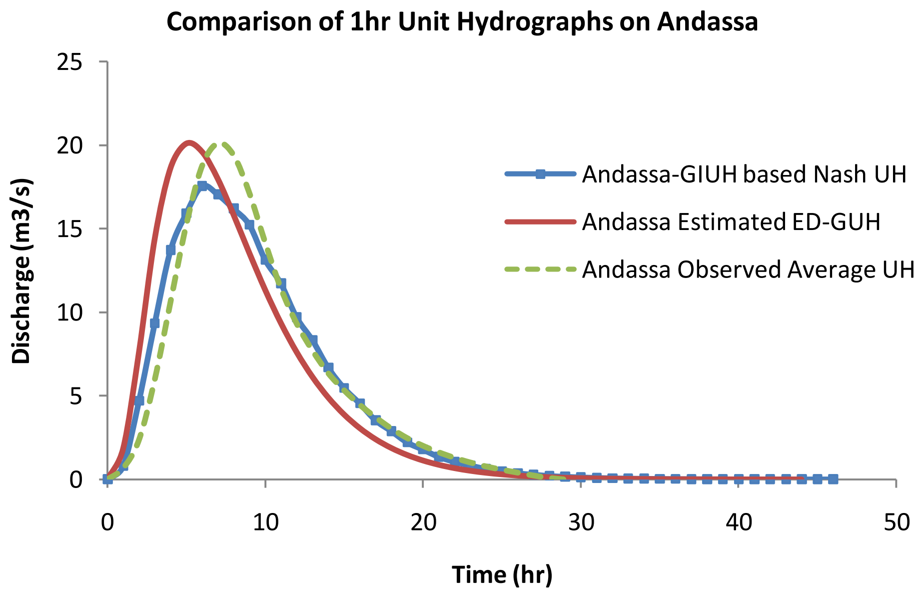
Figure 5.7a-c shows that the ED-GIUH model did the best job in predicting the peak discharge of the unit hydrographs where as the GIUH based Nash models under predict the peaks of the unit hydrographs of the catchments.



(a)



(b)



(c)

Figure 5. 7: Visual Comparison of the models

5.7.2. Comparisons of Error Functions

The results of the objective functions explained in Section 4 are presented in the following part.

Peak Discharge

The peak discharge for both unit hydrograph models and observed peak discharge are summarized in Table 5.10, the percentage error in peak discharge can be found in Table 5.12. The calibrated ED-GIUH model can capture the peak discharge of the average unit hydrographs of all catchments.

The GIUH based Nash model under predict the peak discharge of the unit hydrographs of the catchments.

Time to Peak Discharge

Time to peak discharge for each unit hydrographs; the observed and predicted times to peak are summarized in Table 5.11 despite the fact that the 1-hr time step is not very precise for the analysis of the time to peak discharge comparison of unit hydrographs, the percentage error in GIUH based Nash model was resulted lower than that of ED-GIUH model in all catchments.

Table 5. 12: Comparison of simulated peak discharge and time to peak with the observed values

Hydrograph	Percentage error in peak discharge		Percentage error in time to peak	
	ED-GUH	GIUH based Nash	ED-GUH	GIUH based Nash
Azuari	-0.01	15.23	28.57	14.29
Chemoga	0.02	14.19	30.00	10.00
Andassa	-0.02	12.65	28.57	14.29

Model Efficiency

The model efficiency based on NSE of both models in all catchments is summarized in Table 5.13. Generally, the GIUH base Nash model gives higher values of NSE than that of ED-GIUH model in all catchments with minimum value of NSE equal to 0.89. ED-GIUH model also gives reasonably higher value except the value of NSE = 0.65 at chemoga.

Table 5. 13: Comparison of Model Efficiency Using Nash-Sutcliffe Efficiency (NSE)

Hydrograph	ED-GUH	GIUH based Nash
Azuari	0.92	0.96
Chemoga	0.65	0.89
Andassa	0.81	0.95

5.8. Relating the Velocity Parameter with the Catchment Characteristics

From the above discussions it is clear that ED-GIUH model was superior in predicting the peak surface runoff than GIUH based Nash model. Hence, the velocity parameter of this model were related to the catchment characteristics of the study areas to develop a regional equation, using regression analysis, in order to give a solution to prediction of surface runoff in ungauged catchments.

Table 5. 14: Geomorphological characteristics of the catchments

Catchements	V (ED-GIUH)	S	L_Ω (km)	RL	RA	RB	A(km^2)	L(km)
Azuari	1.92	0.047	20.41	2.68	4.42	3.46	185.4	37.56
Chemoga	2.17	0.030	33.39	3.33	5.66	4.28	310.4	44.59
Andassa	3.81	0.017	47.06	2.74	5.23	4.45	670	58.13
Muga (Validation)	-	0.027	64.09	2.52	5.53	4.62	452	66.15

5.8.1. Selection of predictors

The different catchment characteristics considered for regression analysis were area (A), average slope of the catchments (S), average stream length of the highest order (L_Ω), length ratio (RL), area ratio (R_A), bifurcation ratio (R_B), and length of the longest flow path (L) (Table 5.14). Out of all these catchment characteristics, those having high correlation coefficient with the velocity parameter were considered for further analysis. Correlation analysis is a commonly used procedure for selection of potential predictors to predict a response variable under consideration. Table5.15 shows correlations of the relevant catchment characteristics.

Table 5. 15: Correlation matrix between the velocity parameter and catchment characteristics

	S	L_Ω (km)	RL	RA	RB	A(km^2)	L(km)	V (ED-GIUH)
V (ED-GIUH)	-0.88	0.93	-0.31	0.29	0.72	0.99	0.98	1

It was found that area (A), average slope of the catchments (S), average stream length of the highest order (L_Ω), and length of the longest flow path (L) have high correlation with the velocity parameter(V). The rest catchment characteristics were dropped from further analysis.

5.8.2. Development of regional regression equation

Regional regression equations are useful for estimating parameters at ungauged sites and relatively straight forward for using information from gauged sites for equation development.

Several regression models were developed (linear, power, exponential and logarithmic) and those best models were investigated to verify the velocity used for the model development in addition to the error criteria (R^2). The four regression models investigated were:

1. Linear Model

$$V = G + \alpha_1 X_1 + \alpha_2 X_2 + \ldots + \alpha_n X_n \qquad 5.2$$

2. Power Model

$$V = G.X_1^{\alpha_1} X_2^{\alpha_2} \ldots x_n^{\alpha_n} \qquad 5.3$$

3. Exponential Model

$$V = G.e^{\alpha_1 X_1} e^{\alpha_2 X_2} ... e^{\alpha_n X_n} \qquad 5.4$$

4. Logarithmic Model

$$V = G + \alpha_1 \log X_1 + \alpha_2 \log X_2 + ... + \alpha_n \log X_n \qquad 5.5$$

Where:

V : the dependent variable (velocity parameter)

X : the independent variables (catchment characteristics)

G : unknown constant

α : unknown coefficients

SPSS statistical analysis software package was used to run a back ward stepwise regression procedures for selection of best multiple regression model. In this procedure all variables are entered into the equation and then sequentially removed. The variable with the smallest partial correlation with the dependent variable is considered first for removal. If it meets the criterion for elimination, it is removed. After the first variable is removed, the variable remaining in the equation with the smallest partial correlation is considered next. The procedure stops when there are no variables in the equation that satisfy the removal criteria.

Of the four regression models the linear regression model performs best in this region with the resulting linear regression equation of Equation5.6. A summary of the regression results are located in Appendix- E.

$$V = 40.619S + 0.151L - 5.787 \qquad R^2 \approx 1 \qquad 5.6$$

Where:

V : the velocity parameter (m/s).

S : average slope of the catchment (m/m).

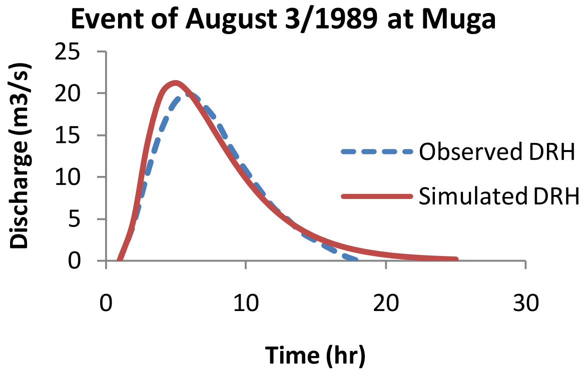
L : the longest flow path length of the catchments (km).

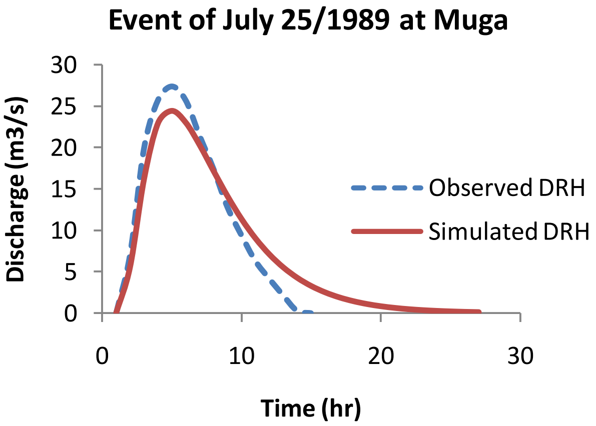
5.9. Validation of the ED-GIUH Model

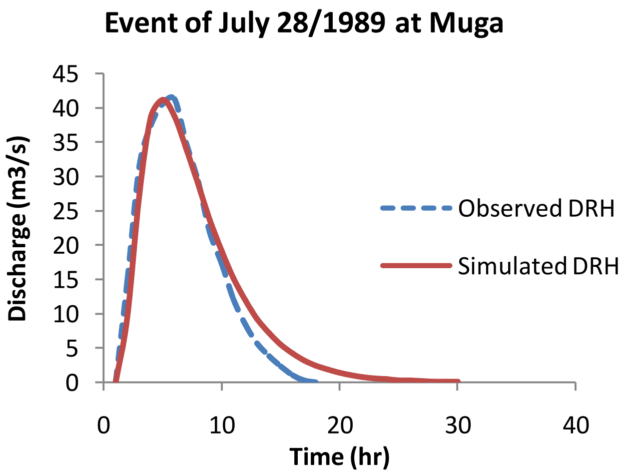
The velocity relation developed for predicting the velocity parameter of the ED-GIHU model was used to estimate the direct runoff hydrographs of four events of the nearby catchment of Muga, which was not used in the calibration of the models and found adjacent to Azuari and Chemoga catchments.

The different geomorphological characteristics of this catchment were extracted from DEM using ILWS in the same fashion as those catchments used in the calibration of the models. The channel velocity of ED-GIUH model was estimated using Equation 5.6. The hillslope velocity was set to be 0.0925 which is supposed to be very near to the actual value of the hillslope velocity of the catchment and identical in the region.
Since the purpose of this section is to check the reliability of the regional equation of the channel velocity of ED-GIUH model, four selected storm events were used to see how well the model simulate these events.

For selected storm events an observed direct runoff hydrographs were prepared after subtracting the base flow using straight line method. The corresponding estimated direct runoff hydrographs were prepared by the principle of convolution. Here, the excess rainfall was determined in same way as stated in Section 4. Convoluting the excess rainfall with the estimated 1-hr unit hydrograph of this catchment, estimated direct runoff hydrographs were determined. The results of the validation test of the regional velocity equation for the selected events are shown in plots of Figure 5.8.


Event of August 3/1989 at Muga
Discharge (m3/s)
Time (hr)
Observed DRH
Simulated DRH


Event of July 25/1989 at Muga
Discharge (m3/s)
Time (hr)
Observed DRH
Simulated DRH


Event of July 28/1989 at Muga
Discharge (m3/s)
Time (hr)
Observed DRH
Simulated DRH

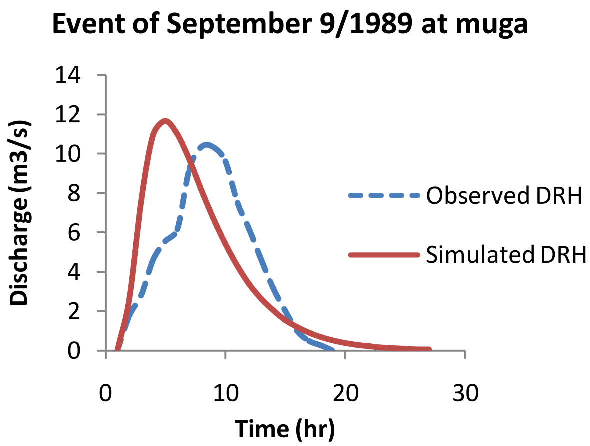


Figure 5. 8: Validation test of the regional velocity equation of ED-GIUH model at Muga catchment.

The above plots show that the observed and simulated direct runoff hydrographs are fairly similar, and hence the validation process of the velocity equation is deemed adequate.

6. CONCLUSIONS AND RECOMMENDATIONS

Conclusions

- The geomorphological characteristics required for the derivation of ED-GIUH and GIUH based Nash model for the catchments under the study were easily and accurately extract from DEM using GIS package called ILWS. The parameters of these models, in terms of Horton ratios, were estimated from the geomorphological characteristics with reasonable accuracy.

- The GIUH models were tasted in four catchments of North and South Gojam sub-basins of Abay basin, whose areas are in range of 185-670 km^2.

- The calibrated ED-GIUH model was successful in simulation of the peak discharge of the unit hydrographs during calibration of the model. The verification part also shows the reliability of the model in prediction of the peak discharge of the runoff hydrographs of the selected events.

- The calibrated GIUH based Nash model can predict the time to peak discharge of the unit hydrographs better than that of ED-GIUH model. Both models adequately predict the shape of the unit hydrographs but GIUH based Nash model did better job in predicting the shape of the hydrographs. Thus, despite the relative simplicity of the model in determination of IUH, from the calibration and verification results it can be conclude that this model was not adequately successful in simulation of the peak discharge of the hydrographs and usually under predict the peak discharge of the UHs or the DRHs.

- From the calibration and verification results of ED-GIUH model, it can be conclude that this model can give promising results in simulating the peak discharge of the runoff hydrographs and is a potential model in runoff prediction in an ungauged catchment.

- The velocity parameter is the input parameter in both models, which is a debating parameter actively developing. Many scientists suggest different methods of estimating this parameter, but none of the published methods has solved the problem of the velocity parameter unambiguously and satisfactorily. Suggested values include the average flow velocity or the velocity at the instant of peak discharge Rodriguez-Iturbe and Valdes (1979), the velocity of the flood wave Kirshen and Bras (1983). Recently, Franchini and O'Connell (1996) develop a prediction equation for U in terms of the Horton length ratio and the length of the highest-order stream of the catchment, along with a time of concentration derived from the time base of the IUH, via an empirical formula Al-Wagdany and Rao (1997) and Al-Wagdany and Rao (1998) propose that this velocity should be considered as a purely calibrating parameter to be determined through optimization.

- In this study the velocity parameter was very difficult to be measured and set as a calibrating parameter to enable the models to simulate the observed catchment average unit hydrographs. The optimum velocity was then determined through optimization.

- Further, an attempt was made to determine the velocity parameter of the ED-GIUH model as a function of the geomorphologic characteristics of the catchments through a regional regression equation. A linear regression model was found to be best in estimating the velocity parameter as a function of the longest flow path of the catchments and the average slope of the catchments and a validation test was made in a nearby catchment of Muga which was not used in the calibration of the model. The results of the validation were found adequate in simulating the observed direct runoff hydrographs of Muga.

Recommendations

- The separation of rainfall excess from total rainfall and base flow separation from observed stream flow data has significant impact on the model predictions, Accuracy would certainly be improved by integrating precise methods.

- For each of the four catchments, the catchment average unit hydrographs were derived from four to seven rainfall runoff events; this is due to scarcity of 1-hr rainfall as well as runoff data sets or non concurrent of these data sets, if available. A better understanding of the application of the catchment unit hydrographs will certainly be obtained if a wider range of events could be studied.

- Channel network extraction requires definition of a threshold value for the digital elevation data operation. When the value of the threshold area is decreased, the channel network becomes dense. Lee (1998) recommends that for 40m×40m resolution DEM data a threshold area of 280 cells can be used as a criterion for net work extraction.

- Derivation of the regression velocity equation was done by fitting the channel velocities of the ED-GIUH model in only three catchments. The velocity relation equation will be improved and more representative of the region if more number of catchments include in the calibration and derivation processes.

- During the derivation of the velocity equation in the region as well as in the validation test in an adjacent catchment, a homogenous test was not conducted. It was assumed fairly homogenous. The performance may be improved if such analysis were made in homogeonous regions.

- Finally, the ED-GIUH models can be adopted for peak flow predictions in ungauged catchments.

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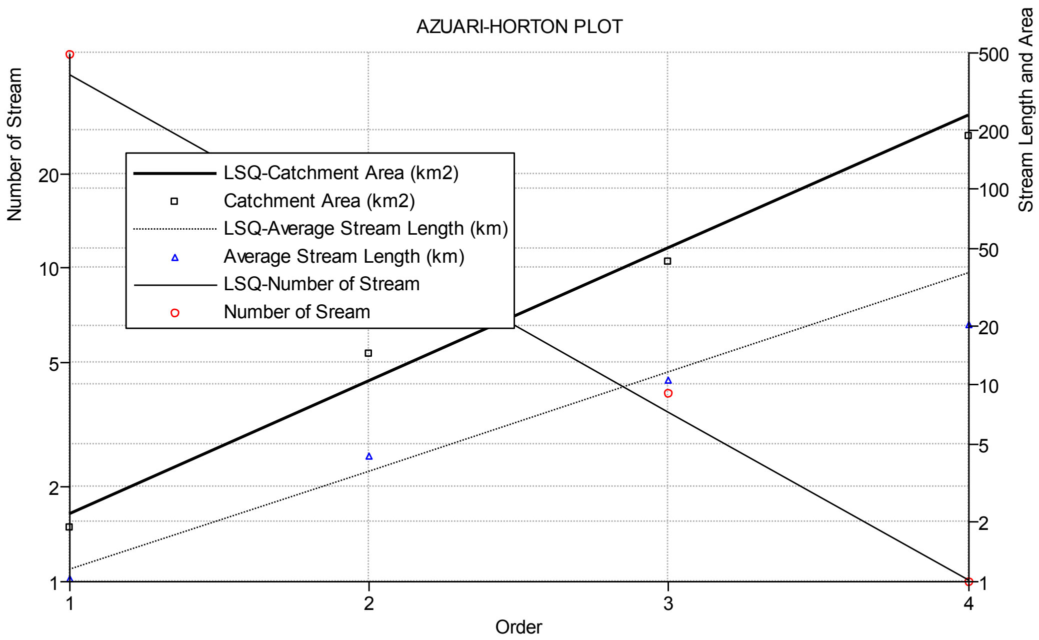
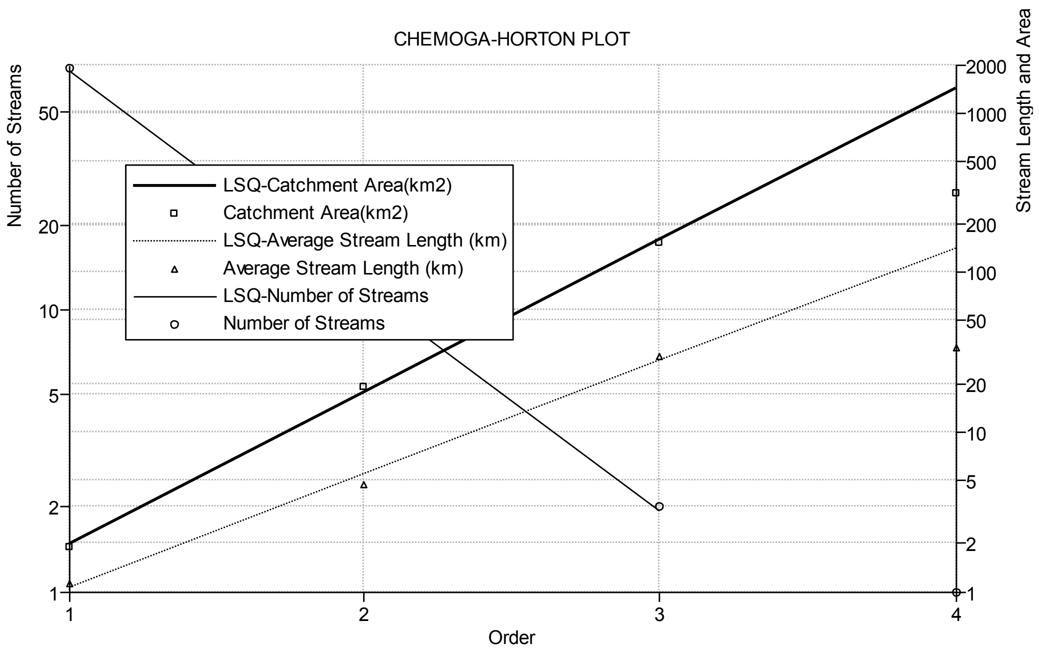
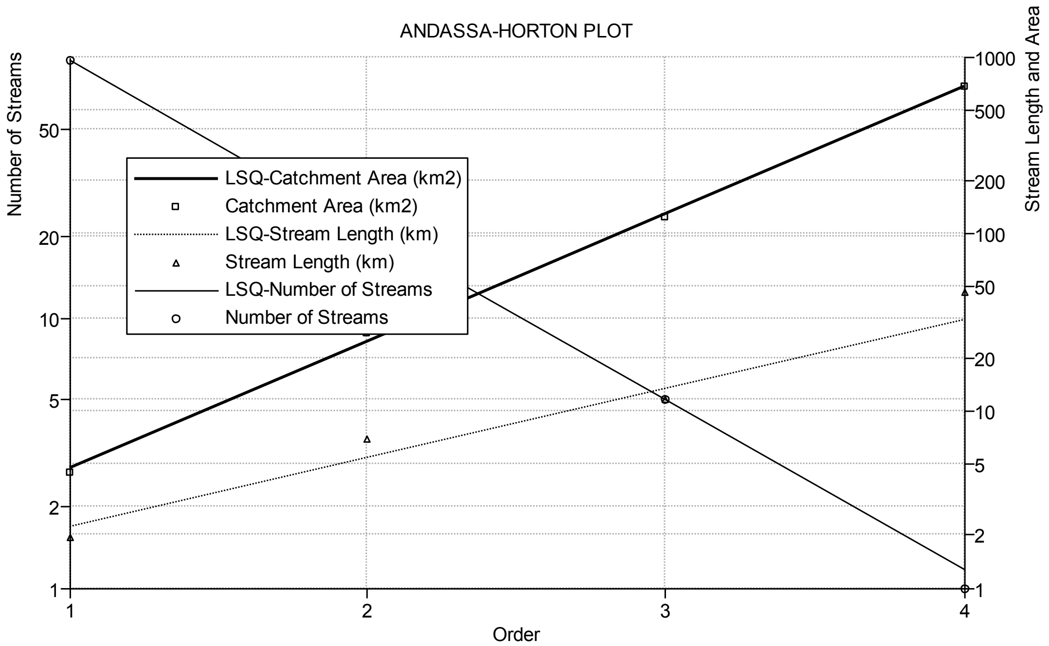
APPENDICIES

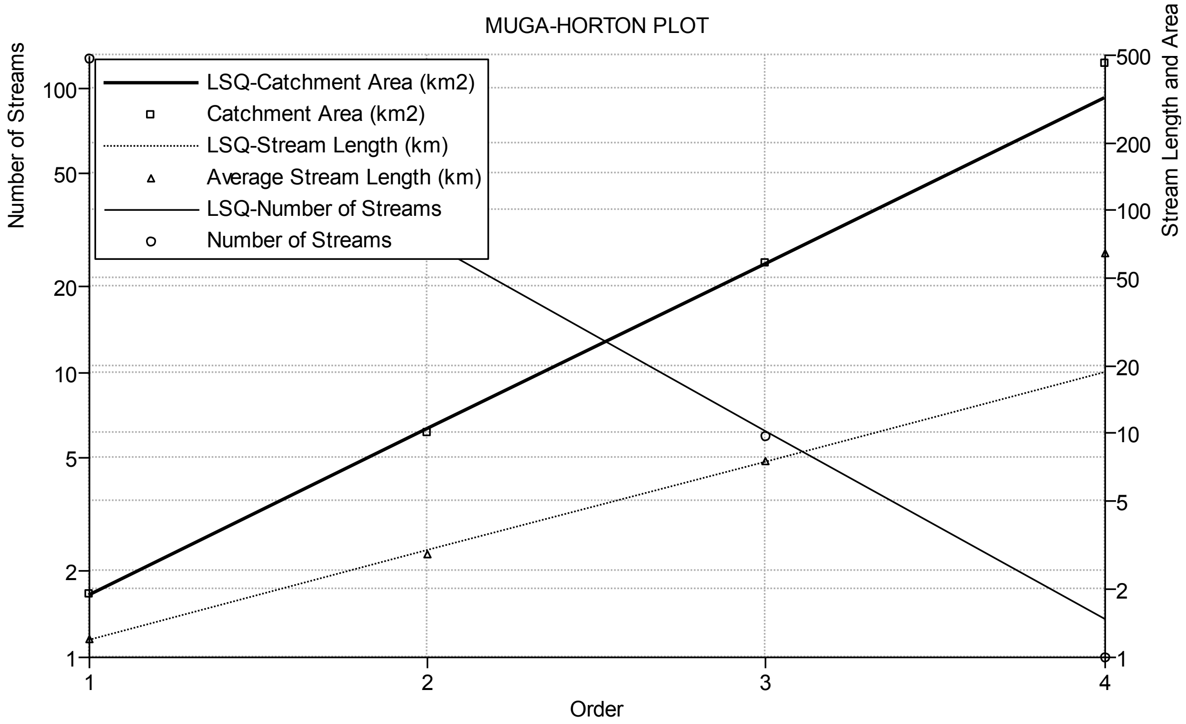
APPENDIX-A: **Horton Plots**






JEDEBE-HORTON PLOT
Number of Streams
Stream Length and Area
Order
LSQ-Catchment Area (km2)
Catchment Area (km2)
LSQ-Average Stream Length (km)
Average Stream Length (km)
LSQ-Number of Streams
Number of Streams
1
2
3
4
1
2
5
10
20
50
1
2
5
10
20
50
100
200
500
1000


ANDASSA-HORTON PLOT
Number of Streams
Stream Length and Area
Order
LSQ-Catchment Area (km2)
Catchment Area (km2)
LSQ-Stream Length (km)
Stream Length (km)
LSQ-Number of Streams
Number of Streams
1
2
3
4
1
2
5
10
20
50
1
2
5
10
20
50
100
200
500
1000



MUGA-HORTON PLOT
Number of Streams
Stream Length and Area
Order
LSQ-Catchment Area (km2)
Catchment Area (km2)
LSQ-Stream Length (km)
Average Stream Length (km)
LSQ-Number of Streams
Number of Streams
1
2
5
10
20
50
100
500
200
1
2
3
4

APPENDIX-B**: Probability Density Equations of Each Possible Path**

The convolution equations of the probability density functions of each possible path are therefore computed as follows.

Path 1: $a_1 \rightarrow r_1 \rightarrow r_2 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet.

$$f_{S1} = \alpha\lambda_1\lambda_2\lambda_3\lambda_4 \left[\begin{array}{l} \dfrac{e^{-\alpha_1 t}}{(\lambda_1-\alpha)(\lambda_2-\alpha)(\lambda_3-\alpha)(\lambda_4-\alpha)} + \dfrac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_2-\lambda_1)(\lambda_3-\lambda_1)(\lambda_4-\lambda_1)} + \dfrac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_1-\lambda_2)(\lambda_3-\lambda_2)(\lambda_4-\lambda_2)} + \\ \dfrac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_1-\lambda_3)(\lambda_2-\lambda_3)(\lambda_4-\lambda_3)} + \dfrac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)(\lambda_2-\lambda_4)(\lambda_3-\lambda_4)} \end{array} \right]$$

Path 2: $a_1 \rightarrow r_1 \rightarrow r_2 \rightarrow r_4 \rightarrow$ outlet.

$$f_{S2} = \alpha\lambda_1\lambda_2\lambda_4 \left[\frac{e^{-\alpha_1 t}}{(\lambda_1-\alpha)(\lambda_2-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_2-\lambda_1)(\lambda_4-\lambda_1)} + \frac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_1-\lambda_2)(\lambda_4-\lambda_2)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)(\lambda_2-\lambda_4)} \right]$$

Path 3: $a_1 \rightarrow r_1 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet.

$$f_{S3} = \alpha\lambda_1\lambda_3\lambda_4 \left[\frac{e^{-\alpha_1 t}}{(\lambda_1-\alpha)(\lambda_3-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_3-\lambda_1)(\lambda_4-\lambda_1)} + \frac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_1-\lambda_3)(\lambda_4-\lambda_3)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)(\lambda_3-\lambda_4)} \right]$$

Path 4: $a_1 \rightarrow r_1 \rightarrow r_4 \rightarrow$ outlet.

$$f_{S4} = \alpha\lambda_1\lambda_4 \left[\frac{e^{-\alpha_1 t}}{(\lambda_1-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_4-\lambda_1)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)} \right]$$

Path 5: $a_2 \rightarrow r_2 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet.

$$f_{S5} = \alpha\lambda_2\lambda_3\lambda_4 \left[\frac{e^{-\alpha_1 t}}{(\lambda_2-\alpha)(\lambda_3-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_3-\lambda_2)(\lambda_4-\lambda_2)} + \frac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_2-\lambda_3)(\lambda_4-\lambda_3)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_2-\lambda_4)(\lambda_3-\lambda_4)} \right]$$

Path 6: $a_2 \rightarrow r_2 \rightarrow r_4 \rightarrow$ outlet.

$$f_{S6} = \alpha\lambda_2\lambda_4 \left[\frac{e^{-\alpha_1 t}}{(\lambda_2-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_4-\lambda_2)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_2-\lambda_4)} \right]$$

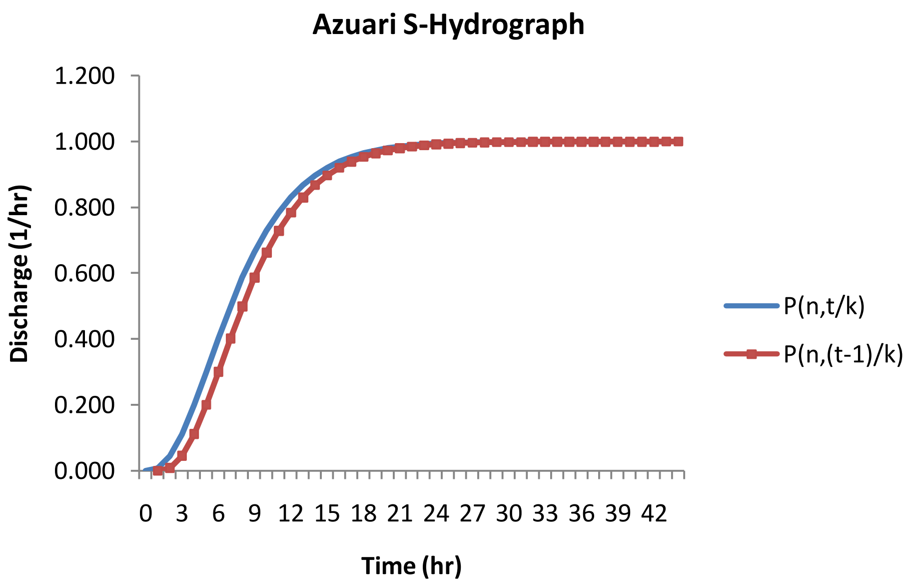
Path 7: $a_3 \rightarrow r_3 \rightarrow r_4 \rightarrow$ outlet

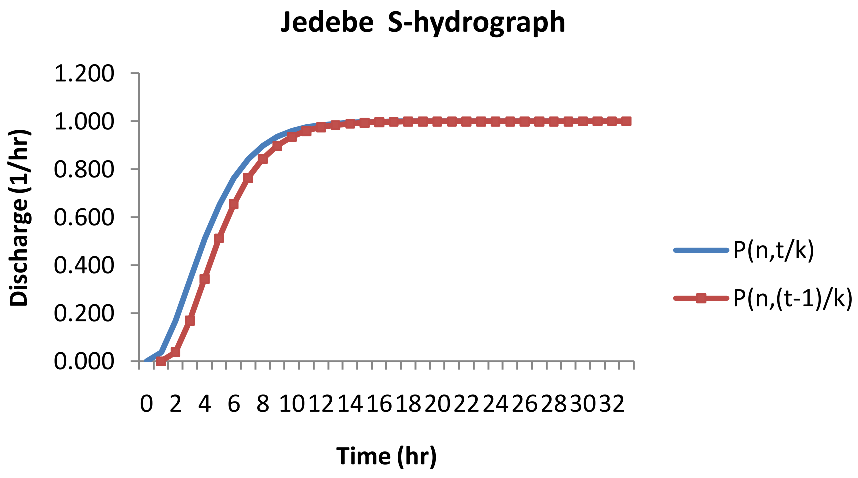
$$f_{S7} = \alpha\lambda_3\lambda_4\left[\frac{e^{-\alpha_1 t}}{(\lambda_3-\alpha)(\lambda_4-\alpha)}+\frac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_4-\lambda_3)}+\frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_3-\lambda_4)}\right]$$

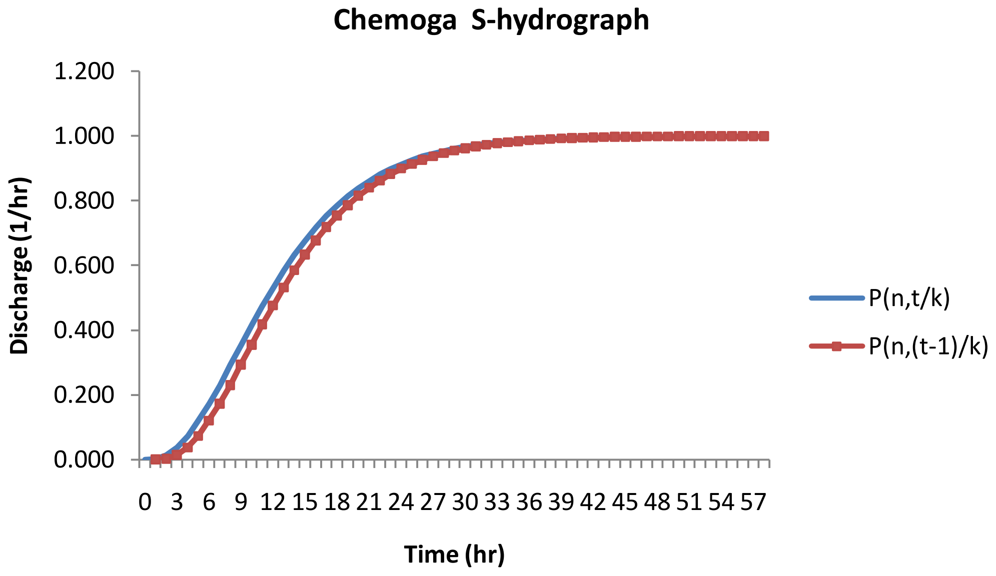
Path 8: $a_4 \rightarrow r_4 \rightarrow$ outlet

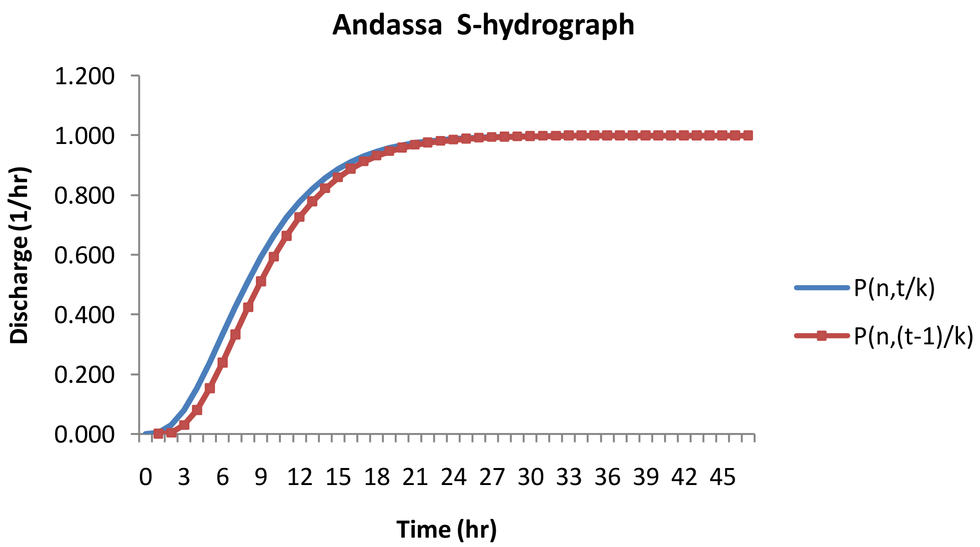
$$f_{S8} = \alpha\lambda_4\left[\frac{e^{-\alpha_1 t}}{(\lambda_4-\alpha)}+\frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)}\right]$$

APPENDIX-C: **The S-Hydrographs for Unit Hydrograph Derivation of GIUH Based Nash Model**






Chemoga S-hydrograph
Discharge (1/hr)
1.200
1.000
0.800
0.600
0.400
0.200
0.000
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45 48 51 54 57
Time (hr)
P(n,t/k)
P(n,(t-1)/k)


Andassa S-hydrograph
Discharge (1/hr)
1.200
1.000
0.800
0.600
0.400
0.200
0.000
0 3 6 9 12 15 18 21 24 27 30 33 36 39 42 45
Time (hr)
P(n,t/k)
P(n,(t-1)/k)

APPEDNIX-D: **Ordinates of Observed and Simulated Unit Hydrographs**

Azuari			Chemoga			Jedebe			Andassa		
	simulated			simulated			simulated			simulated	
Observed UH	ED-GUH	Nash-GUH	Observed UH	ED-GUH	Nash-GUH	Observed UH	ED-GUH	Nash-GUH	Observed UH	ED-GUH	Nash-GUH
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.54	0.83	0.41	0.24	0.63	0.18	2.14	3.16	2.66	0.68	1.87	0.79
1.50	2.73	1.93	0.44	2.12	1.02	9.89	8.52	9.05	2.37	7.63	4.70
2.47	4.60	3.40	0.61	3.73	2.03	11.17	11.21	12.06	5.97	14.41	9.34
3.77	5.77	4.60	0.88	5.03	3.03	10.42	11.13	11.80	10.62	18.64	13.71
5.70	6.18	5.15	1.88	5.91	4.07	8.51	9.64	9.92	15.23	20.10	15.92
6.18	5.99	5.24	3.44	6.40	4.55	6.53	7.73	7.62	18.74	19.61	17.56
5.97	5.44	4.99	4.79	6.54	4.98	5.01	5.92	5.51	20.10	17.98	17.05
5.17	4.71	4.54	5.83	6.43	5.45	3.88	4.39	3.82	19.38	15.83	16.20
4.24	3.94	3.89	6.39	6.12	5.61	2.99	3.20	2.60	17.12	13.54	15.23
3.42	3.21	3.43	6.54	5.69	5.44	2.28	2.30	1.67	14.17	11.34	13.15
2.72	2.57	2.87	6.32	5.20	5.02	1.66	1.64	1.07	11.42	9.35	11.72
2.15	2.02	2.36	5.96	4.67	4.77	1.21	1.16	0.68	9.32	7.61	9.69
1.71	1.57	1.91	5.38	4.14	4.66	0.93	0.81	0.42	7.71	6.13	8.34
1.38	1.20	1.53	4.80	3.63	4.12	0.79	0.57	0.26	6.35	4.90	6.69
1.12	0.92	1.19	4.15	3.15	3.77	0.64	0.40	0.16	5.34	3.89	5.47
0.89	0.69	0.96	3.55	2.71	3.57	0.45	0.28	0.10	4.46	3.07	4.55
0.71	0.52	0.75	3.09	2.32	3.07	0.29	0.20	0.06	3.72	2.41	3.54
0.56	0.39	0.58	2.72	1.97	2.75	0.15	0.14	0.03	3.04	1.88	2.89
0.39	0.29	0.44	2.37	1.67	2.55	0.09	0.10	0.02	2.48	1.46	2.22
0.20	0.21	0.34	2.08	1.41	2.15	0.06	0.07	0.01	2.01	1.14	1.79
0.16	0.16	0.25	1.82	1.18	1.89	0.01	0.05	0.01	1.63	0.88	1.35
0.14	0.12	0.20	1.60	0.98	1.73	0.00	0.03	0.00	1.30	0.68	1.05
0.12	0.08	0.15	1.40	0.82	1.44	0.00	0.02	0.00	1.02	0.52	0.84
0.10	0.06	0.11	1.22	0.68	1.25	0.00	0.02	0.00	0.78	0.40	0.63
0.08	0.04	0.08	1.11	0.56	1.13	0.00	0.01	0.00	0.58	0.31	0.49
0.06	0.03	0.06	0.98	0.46	0.93	0.00	0.01	0.00	0.41	0.24	0.37
0.05	0.02	0.05	0.87	0.38	0.84	0.00	0.01	0.00	0.24	0.18	0.29
0.03	0.02	0.03	0.76	0.31	0.69	0.00	0.00	0.00	0.09	0.14	0.21
0.02	0.01	0.03	0.66	0.26	0.59	0.00	0.00	0.00	0.00	0.10	0.16
0.00	0.01	0.02	0.57	0.21	0.52	0.00	0.00	0.00	0.00	0.08	0.12
0.00	0.01	0.01	0.48	0.17	0.43	0.00	0.00	0.00	0.00	0.06	0.09
0.00	0.00	0.01	0.43	0.14	0.36	0.00	0.00	0.00	0.00	0.05	0.07
0.00	0.00	0.01	0.39	0.11	0.32	0.00	0.00	0.00	0.00	0.03	0.05

0.00	0.00	0.01	0.34	0.09	0.26	0.00	0.00	0.00	0.00	0.03	0.04
0.00	0.00	0.00	0.31	0.07	0.22	0.00	0.00	0.00	0.00	0.02	0.03
0.00	0.00	0.00	0.27	0.06	0.19	0.00	0.00	0.00	0.00	0.02	0.02
0.00	0.00	0.00	0.24	0.05	0.16	0.00	0.00	0.00	0.00	0.01	0.02
0.00	0.00	0.00	0.21	0.04	0.13	0.00	0.00	0.00	0.00	0.01	0.01
0.00	0.00	0.00	0.18	0.03	0.12	0.00	0.00	0.00	0.00	0.01	0.01
0.00	0.00	0.00	0.15	0.03	0.09	0.00	0.00	0.00	0.00	0.00	0.01
0.00	0.00	0.00	0.12	0.02	0.08	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.09	0.02	0.07	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.08	0.01	0.05	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.06	0.01	0.05	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.05	0.01	0.04	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.03	0.01	0.03	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.02	0.01	0.03	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.01	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.02	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.01	0.00	0.00	0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

APPENDIX-E: **Results of Regression Analysis**

Variables Entered/Removed[b]

Coefficients[a]

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-5.787	.000		.	.
	S	40.619	.000	.604	.	.
	L	.151	.000	1.539	.	.

a. Dependent Variable: V

Model	Variables Entered	Variables Removed	Method
1	L, S[a]	.	Enter

a. Tolerance = .000 limits reached.

b. Dependent Variable: V

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	1.000[a]	1.000	.	.

a. Predictors: (Constant), L, S

ANOVA[b]

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	2.108	2	1.054	.	.[a]
	Residual	.000	0	.		
	Total	2.108	2			

a. Predictors: (Constant), L, S

b. Dependent Variable: V

APPENDIX-F: **Stream Flow Data**

AZUARI

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
July,8/1989	5PM	0.72	3.73
	6	0.73	3.91
	7	0.76	4.5
	8	0.8	5.35
	9	0.9	7.83
	10	0.965	9.73
	11	1	10.85
	12	1.02	11.52
July,9/1989	1:20AM	1.02	11.52
	2	0.98	10.2
	3	0.95	9.27
	4	0.94	8.97
	5	0.92	8.39
	6	0.9	7.83
	7	0.88	7.29
	8	0.855	6.65
	9	0.84	6.28
	10	0.82	5.8
	11	0.81	5.57
	12	0.805	5.46
	1PM	0.79	5.13
	2	0.78	4.91
	3	0.78	4.91
	4	0.775	4.81
	5	0.765	4.6
	6	0.76	4.5
	7	0.755	4.4
	8	0.75	4.3
	9	0.743	4.16
	10	0.74	4.1
	11	0.735	4.01
	12	0.73	3.91
July,10/1989	1AM	0.725	3.82
	2	0.72	3.73

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
July,28/1989	8:20PM	0.80	5.35
	9	1.54	37.15
	10	1.61	41.52
	10:40	1.61	41.52
	11	1.52	35.86
	12	1.38	27.52
July,29/1989	1AM	1.26	21.34
	2	1.16	16.84
	3	1.07	13.29
	4	0.98	10.2
	5	0.92	8.39
	6	0.90	7.83
	7	0.87	7.03
	8	0.86	6.65
	9	0.84	
	10	0.83	5.92
	11	0.82	5.8
	12	0.81	5.48
	1PM	0.80	5.35

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)	Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
Aug,27/1989	1pm	0.8	5.35	Aug,7/1989	6pm	0.8	5.57
	2	0.8	4.91		7	0.8	4.91
	3	0.8	4.70		8	0.7	4.30
	4	0.7	4.10		9	0.8	5.13
	5	0.7	4.30		10	0.8	5.13
	6pm	0.7	4.10		11	0.9	7.83
	7	0.8	5.80		12am	0.8	5.35
	8	0.9	6.78	Aug,8/1989	1	1.1	14.81
	9	0.9	8.39		2	1.2	21.82
	10	1.2	17.70		3	1.2	21.82
	11	1.2	21.34		4	1.2	19.02
	12am	1.3	26.43		5	1.1	15.61
	1	1.3	27.53		6am	1.1	13.29
	2	1.3	27.53		7	1.0	11.86
	3	1.3	25.90		8	1.0	11.18
	4	1.3	23.81		9	1.0	9.89
	5	1.2	20.86		10	0.9	9.27
	6am	1.2	18.13		11	0.9	8.68
	7	1.1	14.81		12am	0.9	8.11
	8	1.0	11.86		1	0.9	7.56
	9	1.0	11.18		2	0.9	7.29
	10	1.0	9.89		3	0.9	7.03
	11	0.9	9.27		4	0.9	6.52
	12pm	0.9	8.11		5	0.9	6.52
Aug 27/1989		0.9	7.03		6pm	0.9	7.56
	2	0.9	6.52		7	1.0	11.18
	3	0.8	6.04		8	1.0	11.86
	4	0.8	5.57		9	1.0	11.18
	5	0.8	5.13		10	0.9	9.27
					11	1.0	9.89
					12am	0.9	9.27

CHEMOGA

	May,15/1984		May,16/1984		May,17/1984		May,18/1984	
Time (hrs)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)
1	0.37	0.56	0.61	2.17	0.48	1.14	0.48	1.10
2	0.37	0.56	0.59	1.96	0.48	1.13	0.48	1.10
3	0.37	0.56	0.58	1.87	0.48	1.10	0.48	1.10
4	0.37	0.56	0.57	1.80	0.48	1.10	0.48	1.14
5	0.37	0.56	0.56	1.73	0.48	1.10	0.48	1.14
6am	0.76	4.01	0.56	1.68	0.48	1.09	0.48	1.14
7	0.84	5.33	0.55	1.61	0.47	1.07		
8	0.88	5.90	0.54	1.56	0.47	1.05		
9	0.88	6.02	0.54	1.52	0.47	1.03		
10	0.87	5.72	0.53	1.45	0.46	1.01		
11	0.84	5.33	0.52	1.40	0.46	0.99		
12am	0.82	4.86	0.52	1.37	0.46	0.99		
1	0.81	4.68	0.51	1.34	0.46	0.97		
2	0.79	4.42	0.51	1.30	0.46	0.97		
3	0.77	4.17	0.50	1.26	0.46	0.97		
4	0.75	3.90	0.50	1.26	0.46	0.97		
5	0.73	3.62	0.50	1.23	0.46	0.97		
6pm	0.71	3.32	0.50	1.22	0.45	0.96		
7	0.69	3.08	0.49	1.20	0.45	0.96		
8	0.67	2.84	0.49	1.18	0.45	0.96		
9	0.65	2.62	0.49	1.16	0.45	0.96		
10	0.64	2.46	0.48	1.14	0.45	0.96		
11	0.62	2.29	0.48	1.14	0.45	0.96		
12am	0.61	2.17	0.48	1.14	0.47	1.03		

Time (hrs)	May,24/1984		May,25/1984		May,26/1984		May,27/1984	
	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)
1	0.47	1.05	0.57	1.80	0.62	2.28	0.60	2.05
2	0.48	1.10	0.57	1.80	0.62	2.25	0.60	2.05
3	0.55	1.61	0.73	3.54	0.62	2.25	0.60	2.05
4	0.79	4.42	0.85	5.42	0.62	2.23	0.60	2.05
5	0.86	5.52	0.90	6.43	0.62	2.23	0.60	2.05
6am	0.88	6.02	0.92	6.80	0.62	2.23	0.60	2.05
7	0.89	6.14	0.93	6.87	0.62	2.23	0.60	2.05
8	0.89	6.08	0.93	6.94	0.62	2.23		
9	0.86	5.56	0.92	6.83	0.61	2.19		
10	0.82	4.95	0.90	6.43	0.61	2.16		
11	0.78	4.34	0.87	5.81	0.61	2.16		
12am	0.75	3.77	0.83	5.14	0.61	2.14		
1	0.71	3.30	0.80	4.51	0.61	2.14		
2	0.68	2.98	0.76	3.93	0.61	2.14		
3	0.66	2.66	0.73	3.54	0.61	2.14		
4	0.64	2.46	0.71	3.25	0.61	2.14		
5	0.63	2.38	0.69	3.02	0.61	2.14		
6pm	0.63	2.32	0.67	2.78	0.61	2.14		
7	0.61	2.21	0.65	2.62	0.61	2.14		
8	0.60	2.12	0.64	2.46	0.61	2.14		
9	0.59	2.01	0.63	2.40	0.61	2.14		
10	0.59	1.94	0.63	2.35	0.60	2.09		
11	0.58	1.90	0.62	2.31	0.60	2.05		
12am	0.58	1.87	0.62	2.28	0.60	2.05		

Time (hrs)	Aug,3/1986		Aug,4/1986		Aug,5/1986		Aug,6/1986		Aug,7/1986	
	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)	Stage Height (m)	Discharge (m3/s)
1	1.52	27.00	1.76	40.56	1.63	32.81	1.57	29.56	1.35	19.31
2	1.51	26.51	1.82	44.47	1.62	32.26	1.56	29.04	1.34	18.91
3	1.51	26.51	1.83	45.14	1.61	31.71	1.54	28.01	1.34	18.91
4	1.52	26.76	1.82	44.47	1.60	31.16	1.53	27.51	1.33	18.50
5	1.52	27.00	1.82	44.47	1.60	31.16	1.52	27.00	1.32	18.11
6am	1.58	30.09	1.83	45.14	1.59	30.62	1.51	26.51	1.31	17.72
7	1.66	34.52	1.84	45.82	1.58	30.09	1.50	26.02	1.31	17.72
8	1.76	40.56	1.84	45.82	1.58	30.09	1.49	25.54	1.29	17.33
9	1.82	44.47	1.84	45.82	1.57	29.56	1.47	24.82	1.29	17.33
10	1.87	47.89	1.83	45.14	1.57	29.56	1.47	24.58	1.31	17.72
11	1.89	49.30	1.82	44.47	1.57	29.56	1.46	24.11	1.32	18.11
12am	1.89	49.30	1.81	43.80	1.58	30.09	1.44	23.19		
1	1.88	48.59	1.79	42.49	1.60	31.16	1.44	23.19		
2	1.86	47.19	1.78	41.84	1.62	32.26	1.43	22.74		
3	1.84	45.82	1.76	40.56	1.64	33.38	1.42	22.29		
4	1.82	44.47	1.74	39.30	1.64	33.38	1.41	21.85		
5	1.80	43.14	1.72	38.07	1.64	33.38	1.40	21.42		
6pm	1.76	40.56	1.70	36.86	1.64	33.38	1.40	21.42		
7	1.74	39.30	1.68	35.68	1.64	33.38	1.38	20.56		
8	1.72	38.07	1.68	35.68	1.63	32.81	1.37	20.35		
9	1.70	36.86	1.67	35.09	1.63	32.81	1.37	20.14		
10	1.68	35.68	1.66	34.52	1.62	32.26	1.36	19.72		
11	1.70	36.86	1.64	33.38	1.60	31.16	1.36	19.72		
12am	1.70	36.86	1.64	33.38	1.58	30.09	1.35	19.31		

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)	Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
July,29/86	9:25PM	1.43	22.74		10	1.72	38.07
	10	1.51	26.51		11	1.7	36.86
	11	1.65	33.94		12	1.685	35.97
	12	1.85	46.5	July,31/86	1AM	1.67	35.09
July,30/86	1AM	1.95	53.67		2	1.65	33.94
	2	1.985	56.33		3	1.64	33.37
	2:30	1.985	56.33		4	1.63	32.81
	3	1.97	55.18		5	1.615	31.98
	4	1.95	53.67		6	1.605	31.43
	5	1.92	51.46		7	1.59	30.62
	6	1.9	50		8	1.58	30.09
	7	1.88	48.59		9	1.57	29.56
	8	1.88	48.59		10	1.555	28.62
	9	1.93	52.19		11	1.55	28.52
	10	2	57.49		12	1.54	28.01
	11	2.02	59.06		1PM	1.53	27.5
	12	2	57.49		2	1.52	27
	1PM	1.97	55.18		3	1.51	26.51
	2	1.93	52.19		4	1.5	26.02
	3	1.91	50.73		5	1.49	25.53
	4	1.88	48.59		6	1.47	24.58
	5	1.85	46.5		7	1.47	24.58
	6	1.81	43.86		8	1.455	23.88
	7	1.79	42.48		9	1.45	23.65
	8	1.76	40.56		10	1.44	23.19
	9	1.74	39.3		11	1.43	22.74
	10	1.72	38.07				

JEDEBE

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)	Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
Aug,5/82	12PM	1.92	27.46	June,21/84	4:30PM	1.12	6
Aug,6/82	1AM	2.94	75.85		5	1.24	7.72
	1:30	3	79.46		5:20	1.22	7.42
	2	2.82	68.88		6	2.24	32.13
	2:30	2.73	63.88		6:30	2.365	36.54
	3	2.66	60.12		7	2.27	33.16
	4	2.5	51.91		8	1.88	21.18
	5	2.4	47.18		9	1.575	13.84
	6	2.27	41.3		9:15	1.6	14.37
	7	2.2	38.3		10	1.42	10.76
	7:30	2.175	37.26		11	1.28	8.35
	8:30	2.175	37.26		12	1.18	6.83
	9	2.16	36.64		1AM	1.12	6
	10	2.14	35.82	July,24/82	8PM	1.6	17.3
	11	2.1	34.22		9	2.7	62.25
	12	2.06	32.65		10	2.26	40.86
	1PM	2.04	31.88		11	2.2	38.3
	2	2.01	30.37		12	2.1	34.22
	3	1.98	29.63		1AM	1.96	28.9
	4	1.96	28.9		2	1.82	24.03
	5	1.94	28.17		3	1.74	21.45
	6	1.92	27.46		4	1.66	19.03
					5	1.63	18.16
					6	1.6	17.3

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
June,26	3:20AM	1.5	12.3
	3:30	1.66	15.71
	4	1.88	21.18
	5	2.48	40.88
	6	2.32	34.92
	7	2.08	26.95
	8	1.9	21.72
	9	1.76	18.08
	9:30	1.7	16.64
	10	1.72	17.11
	11	1.69	16.4
	12	1.65	15.48
	1PM	1.62	14.81
	2	1.59	14.16
	3	1.56	13.52
	4	1.54	13.11
	5	1.53	12.9
	6	1.51	12.5
	7	1.5	12.3

ANDASSA

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)	Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
June,22/86	5AM	0.8	2.86	Aug,1/86	Aug,1/87	1.6	24.79
	6	0.95	5.21		1:30	1.64	26.52
	7	1.65	26.97		2	1.64	26.52
	7:30	1.735	30.9		2:30	1.62	25.65
	8	1.71	29.72		3	1.84	36.18
	9	1.61	25.22		4	2.52	81.6
	10	1.52	21.51		5	2.94	119.85
	11	1.43	18.12		6	3.1	136.54
	12	1.36	15.7		6:30	3.14	140.9
	1PM	1.29	13.4		7	3.14	140.9
	2	1.23	11.7		8	2.96	121.86
	3	1.17	10.07		9	2.52	81.6
	4	1.13	9.06		10	2.16	55.08
	5	1.08	7.87		11	1.96	42.74
	6	1.045	7.1		12	1.8	34.11
	7	1.01	6.36		1PM	1.7	29.25
	8	0.98	5.77		2	1.62	25.65
	9	0.95	5.21		3	1.57	23.53
	10	0.93	4.85		4	1.52	21.51
	11	0.9	4.34		5	1.49	20.34
	12	0.88	4.02		6	1.46	19.21
June,23/86	1AM	0.87	3.86		7	1.44	18.48
	2	0.85	3.56		7:30	1.43	18.12
	3	0.84	3.42				
	4	0.83	3.27				
	5	0.82	3.13				
	6	0.81	3				
	7	0.8	2.86				

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)	Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
Sep,4/88	0:30AM	1.47	19.58	Sep,29/88	10PM	1.22	11.42
	1	1.51	21.11		11	1.27	12.86
	2	1.57	23.53		12	1.56	23.11
	3	1.93	41.06	Sep,30/88	1AM	2.04	47.48
	4	2.08	49.94		2	2.2	57.74
	5	2.15	54.42		2:30	2.24	60.48
	6	2.18	56.4		3:30	2.24	60.48
	7	2.18	56.4		4	2.22	59.1
	8	2.13	53.12		5	2.04	47.48
	9	2.11	51.84		6	1.88	38.3
	10	2.09	50.57		7	1.76	32.12
	11	2.07	49.32		8	1.66	27.42
	12	2.05	48.08		9	1.58	23.94
	1PM	2.03	46.88		10	1.5	20.72
	2	2.01	45.68		11	1.46	19.21
	3	1.96	42.76		12	1.42	17.76
	4	1.91	39.94		1PM	1.38	16.37
	5	1.83	35.65		2	1.36	15.7
	6	1.75	31.63		3	1.34	15.04
	7	1.69	28.78		4	1.32	14.4
	8	1.64	26.52		5	1.3	14.03
	9	1.6	24.79		6	1.29	13.47
	10	1.57	23.53		7	1.28	13.16
	11	1.54	22.3		8	1.26	12.57
	12	1.53	21.9		9	1.25	12.28
Sep,5/88	1AM	1.51	21.11		10	1.24	11.99
	2	1.49	20.34		11	1.235	11.84
	3	1.48	19.96		12	1.23	11.7
	4	1.47	19.58	Oct,1/88	1AM	1.22	11.42

Date	Time (hr)		Stage Height (m)	Discharge (m3/s)
Aug,17/1986		2	1.10	8.81
		3	1.10	8.81
		4	1.10	8.69
		5	1.08	8.34
	6am		1.08	8.34
		7	1.07	8.22
		8	1.07	8.10
		9	1.12	9.31
		10	1.78	33.11
		11	1.81	34.62
	12am		1.76	32.12
		1	1.67	27.42
		2	1.59	23.94
		3	1.50	20.73
		4	1.42	17.76
		5	1.36	15.70
	6pm		1.32	14.40
		7	1.28	13.16
		8	1.25	12.57
		9	1.21	11.42
		10	1.19	10.87
		11	1.17	10.33
Aug, 18/1986, 1	12am	1	1.15	9.81
			1.12	9.31
		2	1.11	9.06
		3	1.10	8.81

MUGA

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)	Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
Aug,3/89	2AM	0.81	4.8	July,25/89	5PM	1.12	11.44
	3	0.83	5.13		6	1.13	11.71
	4	0.85	5.47		7	1.14	11.99
	5	0.92	6.78		8	1.15	12.27
	6	1.11	11.17		9	1.16	12.55
	7	1.31	17.25		10	1.19	13.42
	8	1.45	22.45		11	1.4	20.5
	9	1.53	25.78		12	1.69	33.29
	10	1.55	26.66	July,26/89	1AM	1.8	39.12
	11	1.52	25.35		2	1.83	40.8
	12	1.47	23.26		3	1.8	39.12
	1PM	1.39	20.12		4	1.72	34.83
	2	1.32	17.59		5	1.64	30.82
	3	1.25	15.26		6	1.54	26.22
	4	1.18	13.12		7	1.46	22.85
	5	1.13	11.71		8	1.38	19.75
	6	1.07	10.14		9	1.32	17.59
	7	1.03	9.17		10	1.26	15.58
	8	0.99	8.25		11	1.2	13.72
	9	0.95	7.39		12	1.16	12.55
	10	0.91	6.58		1PM	1.12	11.44
	11	0.88	6.01				
	12	0.85	5.47				
	1AM	0.83	5.13				
	1:30	0.81	4.8				

Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)	Date	Time (hrs)	Stage Height (m)	Discharge (m3/s)
July,28/89	1AM	1.06	9.89	Sep,9/89	5PM	1.04	9.4
	2	1.5	24.5		6	1.11	11.17
	3	1.84	41.37		7	1.15	12.27
	4	1.94	47.34		8	1.21	14.02
	5	1.99	50.5		9	1.24	14.95
	6	2	51.15		10	1.26	15.58
	7	1.9	44.89		11	1.35	18.65
	8	1.8	39.12		12	1.38	19.75
	9	1.66	31.8	Sep,10/89	0:30	1.38	19.75
	10	1.56	27.12		1	1.36	19.01
	11	1.44	22.05		2	1.3	16.91
	12	1.34	18.29		3	1.26	15.58
	1PM	1.26	15.58		4	1.21	14.02
	2	1.2	13.72		5	1.16	12.55
	3	1.15	12.27		6	1.12	11.44
	4	1.1	10.91		7	1.08	10.39
	5	1.07	10.14		8	1.06	9.89
					9	1.05	9.65
					10	1.04	9.4

APPENDIX-G: **Rainfall Data**

MOTA

JULY, 1989

Time	Date														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0-1					1.9										1.4
1-2					1.3					7.5					7.8
2-3					1.4					0.8					0.2
3-4							1.8								0.5
4-5				1.7				0.3							0.4
5-6											11.6				
6-7				0.4							3.8				
7-8															
8-9															
9-10															
10-11															
11-12															
12-13															
13-14															
14-15															
15-16					1.3		4.6							1.9	
16-17							0.8	0.6							
17-18			3.8												
18-19				0.3				1.4							
19-20				0.9	0.5	0.8						0.8	12.8		5.8
20-21				3.4			0.3	14					0.7		9.0
21-22				0.3		0.3	0.8	1.9				1.9			3.5
22-23							0.8								2.9
23-24				0.5	5.5										0.1
Max. 1hr			3.8	3.4	5.5	0.8	4.6	14		7.5	11.6	1.9	12.8	1.9	9

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1	11.2						1.0		2.3							
1-2	1.2					1.0	1.3									
2-3																
3-4							1.2									
4-5			1.0				0.8									
5-6			1.9			4.4										
6-7						3.0										
7-8						1.5										
8-9						0.2										
9-10																
10-11																
11-12																
12-13		1.7														
13-14		1.5														
14-15		1.6														
15-16																
16-17																
17-18										0.6			0.6			
18-19					6.4	19										
19-20						1.1	14		4.4				2.3			
20-21	1.0					1.7	2.1		2.0		0.2	0.8	0.1			0.5
21-22	4.5								0.7		3.0			0.4		0.2
22-23	0.3										0.2	0.8				
23-24	7.0						1.5									
Max. 1hr	11.2	1.7	1.9		6.4	19	14		4.4	0.6	3.0	0.8	2.3	0.4		0.5

MOTA

August 1989

Time	Date														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0-1															7.0
1-2															1.5
2-3															1.2
3-4				0.4											
4-5				0.8											
5-6			2.3	0.4											
6-7			4.0	0.5									0.1	3.6	
7-8			2.0	0.4										1.5	
8-9														0.4	
9-10															
10-11															
11-12															
12-13															
13-14						0.6									
14-15															
15-16								0.1						1.2	
16-17														0.2	
17-18													0.7	1.6	
18-19									0.2			0.5	0.1	6.0	
19-20	0.8				2.0			0.4	2.4					0.4	
20-21	0.6								0.8		9.4				
21-22	0.1	0.4		6.5			6.0			1.4	0.9				
22-23				0.4	2.2		0.9			0.4					
23-24					0.4										
Max. 1hr	0.8	0.4	4.0	6.5	2.2	0.6	0.4	0.4	2.4	1.4	9.4	0.5	0.7	6.0	7.0

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1	7.4	1.5				6.8	0.6		3.0						0.5	
1-2	2.2								1.2							
2-3																
3-4			0.8	0.9				2.8								
4-5				0.3												7.6
5-6				1.8		5.8	0.1	2.8						5.6		0.6
6-7				0.4	11.6	0.2										
7-8					1.1											
8-9																
9-10																
10-11																
11-12																
12-13																
13-14																
14-15														0.6		
15-16																
16-17																
17-18										0.8						
18-19				12.4						2.6	0.2					
19-20					0.1		1.9						2.1			
20-21			2.2										0.9	0.4		
21-22			3.0		3.0						7.7	1.1	0.3			4.4
22-23		0.6	3.0	0.1	2.8	13.1					8.7		0.8		7.8	1.4
23-24						4.0			3.5		0.8					
Max. 1hr	7.4	1.5	3.0	12.4	11.6	13.1	1.9	2.8	3.5	2.6	7.7	1.1	2.1	5.6	7.8	7.6

DEBERE MARKOS

MAY, 198

Time	Date														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0-1						1.3									
1-2						1.0									
2-3						0.8									
3-4															
4-5															
5-6															
6-7															
7-8															
8-9															
9-10															
10-11															
11-12															
12-13													9.3		
13-14							0.3						0.2		
14-15															
15-16						4.5								2.8	
16-17														3.0	
17-18														2.0	
18-19															
19-20															
20-21															
21-22														0.7	
22-23						0.4	0.2						0.3		
23-24						1.3									
Max. 1hr						4.5							9.3	3.0	

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1	0.8							0.4								
1-2		0.3						0.6								
2-3		0.4						1.0								
3-4								1.0								
4-5								0.2								
5-6																
6-7																
7-8																
8-9																
9-10																
10-11																
11-12															0.2	
12-13																
13-14			1.1													
14-15									1.2							
15-16									1.3							
16-17													7.4			
17-18																
18-19																3.2
19-20																
20-21													0.9		3.0	0.2
21-22		7.6														0.1
22-23		0.4														
23-24	0.4													0.2		
Max. 1hr	0.8	7.6	1.1					1.0					7.4	0.2	3.0	3.2

JUNE, 1984

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1										0.7						
1-2																
2-3			0.2													
3-4																
4-5																
5-6																
6-7																
7-8																
8-9																
9-10	0.1			1.8												
10-11	0.2															
11-12	1.1								8.4						1.6	
12-13	2.7													0.8	0.2	
13-14			5.3										4.3			
14-15			0.1													
15-16																
16-17													0.3			
17-18			0.8										2.6		0.6	
18-19			1.0								4.3		0.6			
19-20											9.2					
20-21									1.3	1.0			1.2	7.3		
21-22			0.6							1.3			0.3			
22-23			0.8							1.0						
23-24										1.0						
Max. 1hr	2.7		5.3	1.8					8.4		9.2		4.3	7.3	1.6	

JULY, 1986

Time	Date														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0-1											1.9				1.8
1-2												0.3			
2-3															
3-4		8.1													
4-5															
5-6															
6-7															
7-8															
8-9															
9-10								1.2			0.7				0.2
10-11								3.1		1.6	0.8				
11-12		0.8								8.0	0.4	1.0		1.0	1.1
12-13							0.2		6.1			0.9			
13-14						4.6									
14-15			1.0		4.1										
15-16		1.6													
16-17		0.8							0.2		2.7				
17-18		0.9										0.3			
18-19												2.5			0.3
19-20												1.6		0.5	
20-21									0.2			0.2		0.6	
21-22															
22-23															2.6
23-24											5.6				3.5
Max. 1hr		8.1	1.0		4.1	4.6	0.2	3.1	6.1	8.0	2.7	2.5		1.0	3.5

AUGUST, 1986

Time	Date														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0-1	0.2										1.4				
1-2		6.2													
2-3															
3-4		1.5													
4-5		1.0													
5-6		1.0	0.3												
6-7		0.3	6.0												
7-8		0.4	3.0	4.3							0.3				
8-9															
9-10															
10-11															
11-12													2.7		
12-13															
13-14		0.6													2.0
14-15	7.2		19.8												
15-16															0.4
16-17	0.3			1.1											
17-18				0.7											
18-19				0.6											
19-20				0.8									0.3		
20-21				0.5									0.4		
21-22				0.5											
22-23			0.2	0.8											
23-24											3.1				
Max. 1hr	7.2	6.2	19.8	4.3							3.1		2.7		2.0

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1																
1-2	2.1															
2-3																
3-4																
4-5																
5-6																
6-7																
7-8	4.5															
8-9																
9-10																
10-11																
11-12																
12-13																
13-14	0.3															
14-15																
15-16	3.0															
16-17	3.2															
17-18																
18-19																
19-20																
20-21																
21-22																
22-23																
23-24																
Max. 1hr	4.5															

JULY, 1986

Time	Date															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
0-1	2.3	1.7								12.8						
1-2	1.1	1.3														
2-3	0.6	0.5						4.3								
3-4																
4-5								0.5				0.2				
5-6																
6-7																
7-8																
8-9																
9-10																
10-11																
11-12														2.4		
12-13		7.7		0.8		12.7		0.7						3.2	0.2	
13-14	1.6		4.3			2.1		0.6								
14-15	6.4	0.4						1.2						1.8	0.5	
15-16						1.8								1.0		
16-17																
17-18									0.6		0.8					
18-19		0.8										1.2				
19-20		1.0			3.6						0.2	0.8				
20-21		1.4			1.7											
21-22		6.0														
22-23		2.0														
23-24		1.8														
Max. 1hr	6.4	7.7	4.3	0.8	3.6	12.7		4.3	0.6	12.8	0.8	1.2		3.2	0.5	

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1	0.7													1.3		
1-2								0.2						0.4		
2-3								0.6								
3-4								0.6						3.6		
4-5										0.8			0.4			
5-6							0.1	3.0		0.7				6.4		
6-7									1.4					1.4		
7-8						1.5	0.3		1.0					2.3		
8-9						0.8			0.4							
9-10						7.0										
10-11															0.3	
11-12			1.3												0.4	
12-13			2.1											0.6		
13-14				1.2	10.1											
14-15				1.1	7.8	0.2								8.0		
15-16								0.1								
16-17						2.4			6.0				15.6			
17-18								0.2	0.4	5.5					0.5	
18-19										2.1			1.6			
19-20				2.6	0.2											
20-21				2.4	0.3								1.2			
21-22		1.0			1.4			5.2						8.8		0.3
22-23		0.6						7.5						1.0		
23-24							4.9	3.2		0.4						
Max. 1hr	0.7	1.0	2.1	2.6	10.1	7.0	4.9									

BAHIR DAR

JUNE, 1986

Time	Date														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
0-1			1.4								1.6					
1-2			0.5					0.2								
2-3								0.2								
3-4																
4-5																
5-6																
6-7								0.5								
7-8																
8-9																
9-10																
10-11																
11-12																
12-13																
13-14						0.1										
14-15																
15-16									0.2							
16-17																
17-18																
18-19										3.3	2.5					
19-20									1.2	0.2						
20-21									0.5					11.7		
21-22				2.6										6.0		
22-23			0.2			0.1								4.6		
23-24			2.8											3.1		
Max. 1hr			2.8	2.6		0.1			1.2	3.3	2.5			11.7		

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1		3.9	0.8			0.2	3.9					0.4		0.2		
1-2		0.8	0.5		0.8		0.4					11.4				
2-3			1.5		4.2		0.6			1.6	0.2	0.4	0.8	0.5		
3-4					4.8		0.2			0.6		16.4	1.0	0.2		
4-5					3.4						0.3	11.2	6.0	0.4		
5-6					0.4					5.0	0.8		7.1			
6-7						0.3				9.3			2.8			
7-8										3.8		0.1				
8-9										3.0						
9-10			0.4					0.3		2.2						
10-11										0.2						
11-12																
12-13									0.2							
13-14																
14-15																
15-16																
16-17																0.3
17-18																1.4
18-19																
19-20	4.8					0.6			0.9		9.1	0.1				
20-21	4.0					1.6					8.9					
21-22	1.4	1.4				0.9			0.2	1.0	0.7					
22-23		4.5				0.1								0.8		
23-24						0.1			0.9					1.4		
Max. 1hr	4.8	4.5	10.2		4.8	1.6	3.9		0.9	9.3	9.1	16.4	7.1			

JULY, 1986

Time	Date															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
0-1																
1-2																
2-3																
3-4																
4-5							0.1									
5-6																
6-7								1.3								
7-8																
8-9																
9-10																
10-11																
11-12																
12-13																
13-14		0.9														
14-15												1.4		0.3		
15-16												28.6			21.4	
16-17								0.3				6.7			3.5	
17-18		1.9		0.4								3.0				
18-19	3.0			4.6			0.1	1.4		3.4		0.9				
19-20	1.0		7.3	1.6						1.9		0.4		3.6		
20-21	2.5		1.3			4.4		0.8	2.0	2.8		0.2		4.5		
21-22	5.2	6.0	0.4				0.7			2.0		0.3		0.2		
22-23	1.0	0.8	0.3		6.9		2.0									
23-24	0.2		1.5		1.3		2.1									
Max. 1hr	5.2	6.0	7.3	4.6	6.9	4.4	2.1	1.4	2.0	3.4		28.8		4.5	21.4	

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1								0.3		0.5	1.4					
1-2						1.3				0.4	0.6			10.1		
2-3						0.4				0.2	1.5	13.2	0.4	5.6		
3-4										0.9	1.0	0.2		6.5	0.2	
4-5									0.6	1.0	0.2			2.8		
5-6						0.4				1.5				0.3		
6-7						0.3				0.2			1.1			
7-8																
8-9																
9-10																
10-11																
11-12																
12-13																
13-14																
14-15																
15-16											0.2			5.6		
16-17							0.2				0.2					
17-18				1.7			2.2								11.2	
18-19					0.6	15.9	20.7	0.5	15.0		0.3					0.4
19-20		7.0			0.6		2.4	5.5		2.1	1.9					
20-21					0.1	1.2	1.8	6.8	28.4	4.9	0.6					
21-22			0.5	0.5		4.6	4.7	1.1	5.7	3.0	1.5					
22-23				24.6		0.6	1.6			1.5	2.4					
23-24				4.2						2.5	1.5					
Max. 1hr		7.0	0.5	24.6	0.6	15.9	20.7	6.8	28.4	4.9	2.4	13.2	1.1	10.1	11.2	0.4

AUGUST, 1988

Time	Date														
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0-1															
1-2													0.2		
2-3															
3-4															
4-5															
5-6				6.8											
6-7			1.2												
7-8															
8-9															
9-10															
10-11															
11-12															
12-13															
13-14															
14-15											0.6	0.3			
15-16	0.2					15					1.0				
16-17														0.8	
17-18										3.1					
18-19		0.4	8.8		0.4					1.0					0.3
19-20		0.9	19.6		0.8		8.6							3.4	
20-21		0.8	6.2				6.6							2.8	
21-22		3.6	4.8	0.8					0.8					0.6	
22-23		0.5	1.6						0.1						0.4
23-24															
Max.1 hr	0.20	3.60	19.6	6.8	0.8	15	8.6		0.8	3.1	1.0	0.3	0.2	3.4	0.4

Time	Date															
	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1						2.0	1.6									
1-2						0.6										
2-3						1.0	0.6									
3-4																
4-5																
5-6																
6-7																
7-8																
8-9																
9-10																
10-11																
11-12								0.8								
12-13																
13-14													0.6			
14-15																
15-16											0.7					
16-17																
17-18						1.6										
18-19						6.1										
19-20						1.0										
20-21	1.5					1.8			0.6		0.9					
21-22	0.8					4.7										
22-23	0.8	2.4				4.8				1.2	0.6					
23-24	1.5					1.2										
Max. 1hr	1.5	2.4				6.10										

BAHIR DAR

SEPTEMBER, 1988

Time	Date															
	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	
0-1						0.6										
1-2						1.6	0.4									
2-3																
3-4																
4-5																
5-6		0.4														
6-7	1.2															
7-8																
8-9																
9-10																
10-11																
11-12																
12-13			3.4													
13-14		1.5	3.0							7.6						
14-15		0.2	0.9							0.4						
15-16								0.8					3.4			
16-17		1.3								2.4						
17-18		0.2	0.8						0.1	5.6						
18-19					0.3	3.9	1.1		0.5		0.4		0.8			
19-20				3.5				6.0	3.8		11.0		0.6			
20-21				1.0				2.1	1.4			0.3				
21-22				3.0						4.8	0.1					
22-23				0.1					2.0							
23-24			0.3						2.0							
Max. 1hr	1.2	1.5	3.4	3.5	0.3	3.9	1.1	6.0	3.8	7.6	11.0	0.3	3.4			

Time is local standard time.

Rainfall in (mm)