ADDIS ABABA UNIVERSITY
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SYNTHETIC UNIT HYDROGRAPH DERIVATION USING GIUH AND GIUH BASED NASH IUH MODELS FOR UNGAUGED CATCHMENTS IN ABBAY BASIN

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ADDIS ABABA UNVERISITY

Addis Ababa Institute of Technology

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Thesis Submitted to the school of Graduate studies in Partial Fulfillment of the Requirements for the Degree of

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In

Civil Engineering

By

DEREJE GETACHEW MAMO

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SYNTHETIC UNIT HYDROGRAPH DERIVATION USING GIUH AND GIUH BASED NASH IUH MODELS FOR UNGAUGED CATCHMENTS IN ABBAY BASIN

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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ABSTRACT

Geomorphologic instantaneous unit hydrograph (GIUH) can be used as a transfer function for modeling the transformation of excess rainfall into surface runoff, in which excess rainfall is a production function to the hydrologic system. These models can be used to predict the temporal variation of the surface runoff at the outlet of ungauged catchment, which is useful in the hydrologic engineering applications. In this study the two types of geomorphologic approaches were studied, GIUH and GIUH based Nash IUH models, for unit hydrograph derivation from the observed data.

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 IUH model; and GIUH was developed from geomorphological characteristics of the catchments and probability density function of travel time of rainfall excess to the catchment outlet. These two models were developed in seven catchments in Abay basin (Fettam, Hoha, Neshi, Uke, Andassa, Azuari and Jedebe) with range of areas 161 to 573km^2.

In this study, velocity was set as a calibration parameter to calibrate the simulated unit hydrograph with the observed unit hydrograph in both models. A regionalized streamflow velocity equation was developed from the simulated data with R^2 equal to 0.89 and 0.96 for GIUH and GIUH based Nash IUH model respectively. Then, these equations were applied on chemoga catchment, which was considered as ungauged catchment, to observe the reliability of the models.

The performances of the calibrated models were evaluated using Nash-Sutcliffe efficiency (NSE), Error (ERR), percentage error in peak discharge (PEP) and percentage error in time to peak (PETP). From model evaluation results, GIUH model efficiency (EFF) varies from 81.95 to 97.9 percent while GIUH based Nash IUH model efficiency varies from 75.68 to 97.51 percent. GIUH model is efficient in peak discharge estimation (R^2=0.988) than GIUH based Nash IUH model (R^2=0.884). In general, the result of both GIUH and GIUH based Nash IUH models are almost comparable and their results are acceptable but the GIUH model is superior.

Moreover, a simple GIUH based regional triangular unit hydrograph equations were developed and compared with the SCS triangular unit hydrograph. The R^2 value in peak discharge estimation using SCS method is 0.48 while using GIUH method is 0.95. This implies that the derived GIUH based regional triangular unit hydrograph equation is quite different from SCS triangular unit hydrograph and GIUH has better efficiency.

Finally, this paper concludes that, the regionalized GIUH based equation for peak discharge, time to peak and base time of the triangular unit hydrograph can be used, within the errors specified in this research paper, for any ungauged catchment found in neighbor of the study area having similar character and varying in areas from 161 to 573km^2. Further work may in the future be required to explain any non linear relationship of rainfall runoff and homogeneity of the catchments.

KEY WORDS: Geomorphologic Instantaneous Unit Hydrograph (GIUH), Nash IUH Model, ILWIS (Integrated Land and Water Information System), Rainfall-Runoff modeling, Ungauged Catchment.

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Symbol	Meaning
DEM :	Digital Elevation Model
DRH :	Direct Runoff Hydrograph
GIS :	Geographical Information System
GIUH :	Geomorphologic Instantaneous Unit Hydrograph
GUH :	Geomorphologic Unit Hydrograph
ILWIS :	Integrated Land and Water Information System
IUH :	Instantaneous Unit Hydrograph
NSE :	Nash Sutcliff Efficiency
PEP :	Percentage Error of Peak Discharge
PETP :	Percentage Error of Time to Peak
SCS:	Soil Conservation Service
SUH :	Synthetic Unit Hydrograph
TUH :	Triangular Unit Hydrograph
UH :	Unit Hydrograph
K :	Storage coefficient (Nash parameter)
n :	Number of linear reservoir(Nash parameter)
R_a :	Stream area ratio
R_b :	Bifurcation ratio
R_L :	Stream length ratio
V :	Stream flow Velocity
V_o :	Hilly slope velocity
Ω :	Stream network order
L_Ω :	Length of highest order stream

1. INTRODUCTION

1.1. General

Rainfall-runoff process is accepted as one of the most complex and non linear real world phenomena in the field of water engineering. The process consists of the movement of rainfall through different media and its transformation to the runoff in channels, either natural or manmade. For appropriate design and management of hydraulic structures, engineers must be concerned with the peak discharge and the time to peak for large storm events. This type of information is needed for a wide variety of design applications, including dams, spillways, and culverts. Moreover, water resources planning, development and operation of various schemes requires accurate estimation of hydrologic response of the basin.

Unluckily, many streams in Ethiopia are ungauged and do not have flow records. Even when stream gauges are in place, the record is often too short to accurately predict extreme events. In many cases, runoff characteristics may be estimated using rainfall-runoff models. In surface hydrology, rainfall-runoff and soil erosion process are very important and needs good understanding. These hydrological processes are nonlinear and involve various climatic, topographic, soils, land use information. To develop a good understanding and hydrological model for these processes, a reliable and wide variety geophysical and hydro-meteorological data are required. These models may be broadly classified as empirical, conceptual, lumped, and physically based distributed models. Empirical models have their own limitations as they are site specific, whereas, conceptual models are flexible and based on the simplification/approximation of physical concepts of the processes. Therefore, conceptual model has a room for theoretical simulation of individual components of the process. For example, Nash based instantaneous unit hydrograph (IUH) model (Chow et al., 1988) is based on the concept of 'a cascade of linear reservoirs' in the watershed. The physically based distributed models on the other hand, need precise computational skill for solving the governing equations and therefore, require a significant quantum of watershed information along with event based rainfall and runoff data for the calibration of model parameters (Singh and Frevert, 2002). It is the fact that most of the Ethiopian as well the watersheds of developing countries do not have sufficient historical hydrological records and detailed watershed information needed for physically based distributed models. Therefore, a simplified representation of the rainfall-runoff process through the input-output linkage without a detailed physical description of the process can be used and this is the basis of the linear theory of hydrologic systems. For mathematical description of linear hydrologic system, the unit hydrograph (UH) or unit pulse response function can be employed for system identification in hydrologic analysis (Chow et al., 1988). Therefore, geomorphology based approach becomes a one of the most popular modeling tools for the computation of runoff hydrographs under these circumstances (Rodriguez-Iturbe and Valdes, 1979; Yen and Lee, 1997).

Geomorphology of a river basin describes the status of topographic features of the surfaces and streams, and its relationship with hydrology provides the geomorphological control on basin hydrology. Geomorphology reflects the topographic and geometric properties of the watershed and its drainage channel network. It controls the hydrologic processes from rainfall to runoff, and the subsequent flow routing through the drainage network (Jain and Sinha, 2003).

Therefore, the geomorphologic approach provides a more accurate model as it considers the runoff generation to be dependent on the input data and the physical properties of the watershed as well. The geomorphological instantaneous unit hydrograph model avoids updating of variation of hydrological data from time to time and can be used for an ungauged hydro-meteorological similar watershed. The theoretical coupling of quantitative geomorphology and hydrology of an area can also be generalized to incorporate the non-linear effects of rainfall runoff transformation. The IUH of a basin is equal to the probability density function (PDF) of the holding time of the water particles in the basin.

In this study, GIUH and GIUH based Nash IUH models approach have been studied for unit hydrograph derivation and finally a regional equation was developed for ungauged catchments found in neighbor of the study area.

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 or catchments for which available observed discharge time series is inadequate for the calibration. Conventional techniques of unit hydrograph derivation require historical rainfall-runoff data.

Earlier works have provided an understanding of basin geomorphology-hydrology relationship through empirical relationships. Snyder proposed synthetic unit hydrograph approach (SUH) for ungauged basin as a function of catchment area, basin shape, topography, channel slope, stream density and channel storage; and derived the basin coefficient by averaging out other parameters (Chow et al., 1988). These parameters are transferred to ungauged sites through regionalization.

In the process of regionalization, the parameters of unit hydrograph 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 is 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).

Furthermore, the hydrologic response is a function of climatic parameters, landuse, soil parameters and topography and therefore, for any physical based models requires time to time change in their parameters due to variation encountered with respect to the gradual climatic changes and landuse of the watersheds (Rodriguez-Iturbe et al., 1982). However, the geomorphological parameters are mostly time-invariant in nature and therefore, geomorphology based approach could be the most suitable technique for modeling the rainfall-runoff process for ungauged catchments. The rainfall pattern, in general is undergoing a change due to global changes in land-atmospheric-ocean conditions. Further, because of different activities in the watershed, its land use is also having a gradual changes and this has an impact on the characteristics of the runoff produced from the watersheds.

Therefore, the combination of the hydrologic characteristics of the catchment with the geomorphologic parameters can provide a good solution to the hydrologic behavior of the ungauged catchments in particular.

The geomorphological instantaneous unit hydrograph (GIUH) (Rodriguez-Iturbe and Valdes, 1979) and further simplified by Gupta et al. (1980) is a hydrological model that relates the geomorphological features of a basin to its response to rainfall. They can be applied to ungauged basins having scarce hydrologic data (Al-Wagdany et al., 1998). Thus, the GIUH based transfer function approach is applicable in such a situation where rainfall data is available but runoff data are not, and it is a more powerful technique for the flood estimation than the commonly used parametric Clark model and Nash’s cascade technique (Yen and Lee, 1997).

Another advantage of GIUH technique is its potential for deriving the unit hydrograph (UH) using the geomorphologic characteristics obtainable from topographic map / remote sensing, possibly linked with geographic information system (GIS) and digital elevation model (DEM) (Rodriguez-Iturbe and Valdes, 1979; Rosso, 1984; Kumar et al., 2007). However, the GIUH technique is applicable for the estimation of the direct runoff component of the stream flow and hence, can be used to generate the direct runoff hydrograph (DRH). Once the DRH is computed, the flood hydrograph can be simply obtained by adding the base flow component.

Tsehay Zeray (2009, Thesis) has attempted similar work in four selected catchments of upper Blue Nile and has found good results. However, these catchments are located around Lake Tana area and by no means represent the upper Blue Nile. This study focus on the same catchments used by Tsehay Zeray thesis and additional catchments distributed elsewhere in the Blue Nile basin. Finally, a regionalized equation was developed for those ungauged catchments found in neighbor the study area.

1.3. Objectives

1.3.1. General Objectives

This study was intended to address the problem of predictions of runoff characteristics in ungauged catchments using a set of some selected gauged catchments in Abbay 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).

1.3.2. Specific Objectives

- To derive a unit hydrographs, using Geomorphological Instantaneous Unit Hydrograph (GIUH) and Geomorphologic based Nash instantaneous unit hydrograph models.

- To evaluate the geomorphological characteristics of the catchments and Horton ratio's by employing ILWIS (Integrated Land and Water Information System).

- To compare the unit hydrographs developed by GIUH and GIUH based Nash IUH models together with the observed catchment average unit hydrographs using selected various performances criteria and to select the better model.

2. LITERATURE REVIEW

2.1. The Catchment Hydrologic Cycle

Catchment modeling requires a clear understanding of the hydrologic cycle at catchment scale. The catchment hydrologic cycle involves many processes. Many hydrologists investigated this cycle by a number of studies. A summary of the cycle is given by Chow et al (1988) or detail description of some processes can be found in the book of Kirby (1978). To summary the processes, a brief description is presented and is illustrated in figure 2.1 below.

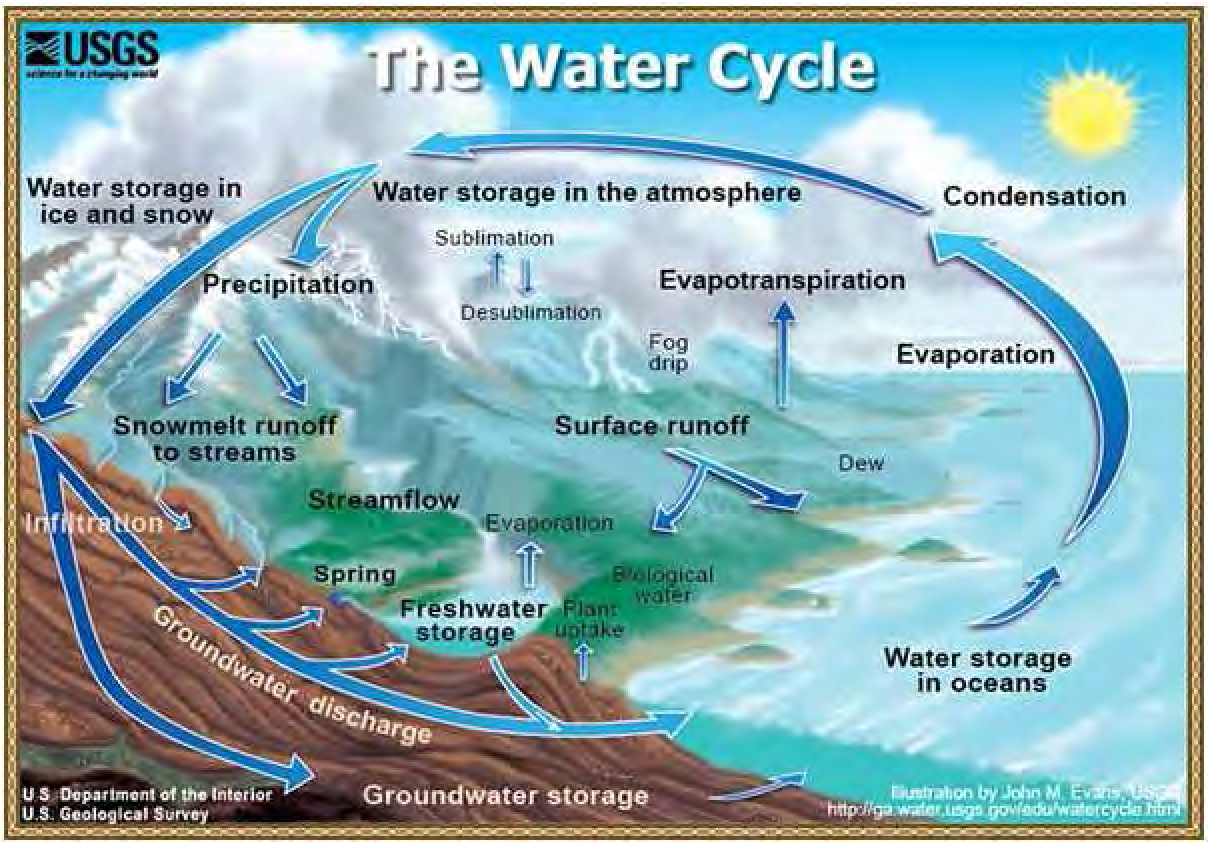


Figure 2.1: Physical processes involved in Runoff Generation

Precipitation is the most essential process for the generation of runoff at a catchment scale. The distribution of precipitation varies spatially and temporally by the nature. Precipitation can be in the form of snow, hail, dew, rain and rime. In this study precipitation is considered in the form of rain only. Rainfall travels in a catchment in different directions. Due to vegetation, part of rainfall is intercepted by vegetation canopy. Interception is known as a loss function to catchment runoff depending on vegetation type, vegetation density. The rest of rainfall moves down the vegetation as stream flow, drip off the leaves, or directly falls to the ground as through fall. Rainfall remains at the land surface as depression storage and either evaporates, infiltrates or is discharged as overland flow. By infiltration of rainwater, the water moves primarily in downward direction by unsaturated subsurface flow and recharges the saturated zone. This process is termed percolation or natural recharge and fills the aquifers of groundwater system. In some cases at the shallow subsurface layer where the lateral hydraulic conductivity is higher than the vertical one, the direct infiltration partly goes toward the channel through interflow or through flow. The groundwater pattern is influenced by the catchment characteristics, especially the topographic factors of the catchment, before being discharged to the channel network system. Aquifers of the groundwater system also can discharge groundwater across the catchment boundary. Evaporation and transpiration at the land surface cause the decrease of water storage in the subsurface. As a consequence unsaturated flow in upward direction is generated that is called capillary rise.

2.2. Catchment Runoff Generation

Basically, the runoff generation at a catchment scale in general or hillslope scale in particular includes two main components: (1) surface runoff, (2) subsurface runoff. There are a number of flow processes within each main component as illustrated in figure 2.2.

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 a saturation mechanism where the soil becomes saturated by the perennial groundwater rising to the surface or by lateral or vertical percolation above an impending horizon (Dunne, 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.

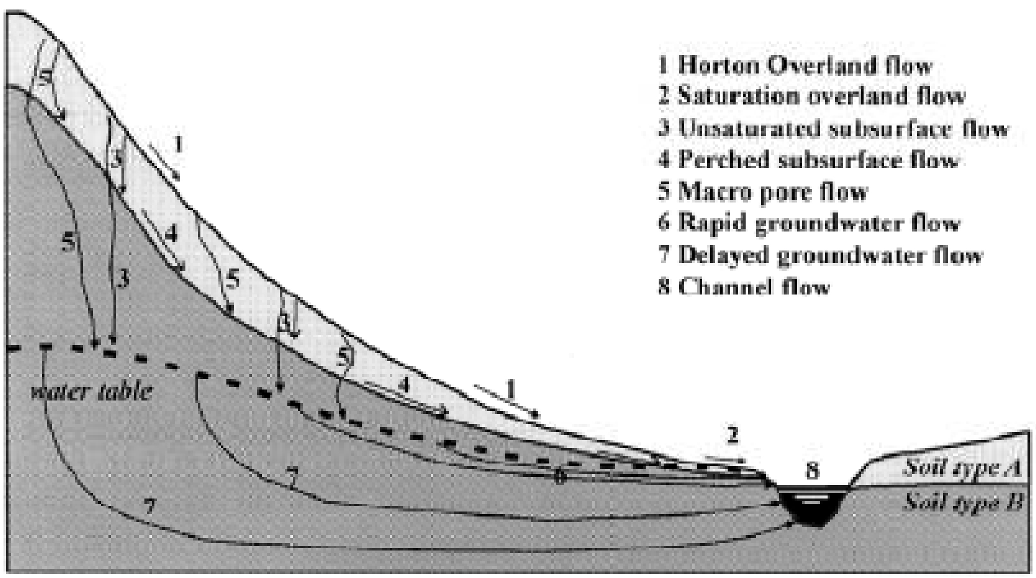


Figure 2.2: Cross sectional presentation of hillslope flow process (Rientjes, 2004)

2.3. 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 by Todini (1988) for his historical review of rainfall runoff modeling.

2.3.1. 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 2.3 provides a general overview of the hydrological models using the classification criteria randomness, spatial discretization and model structure.

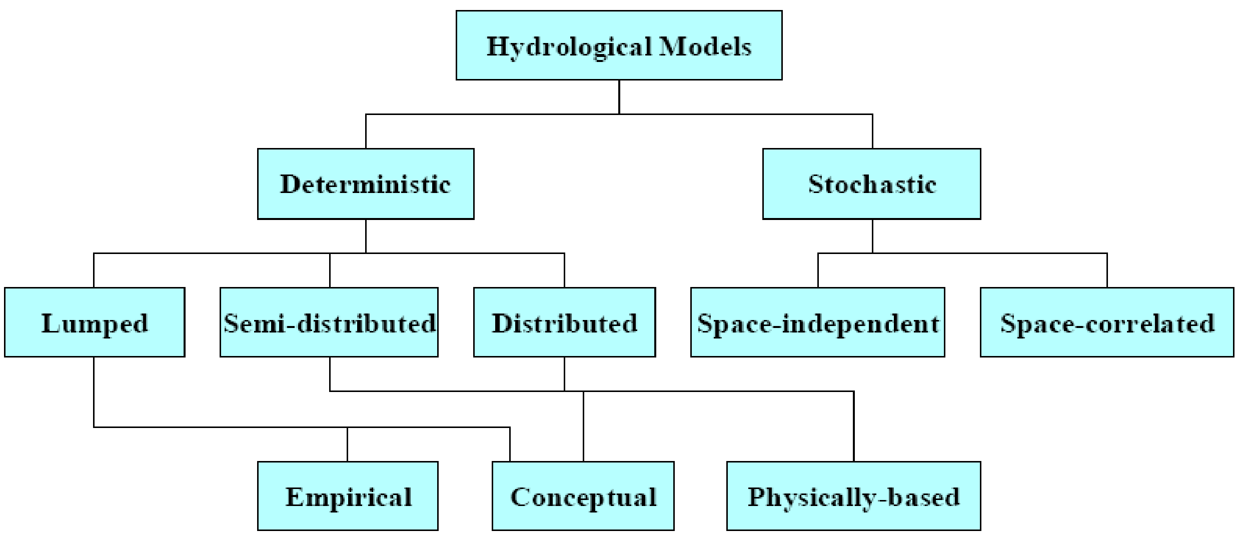


Figure 2.3: Classification of Hydrological models (Chow, 1998, modified)

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, 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: 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.

2.4. 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 in 1932 (Chow *et al*., 1988) 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 remain. 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 4. 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 are shown in Figure 2.4.

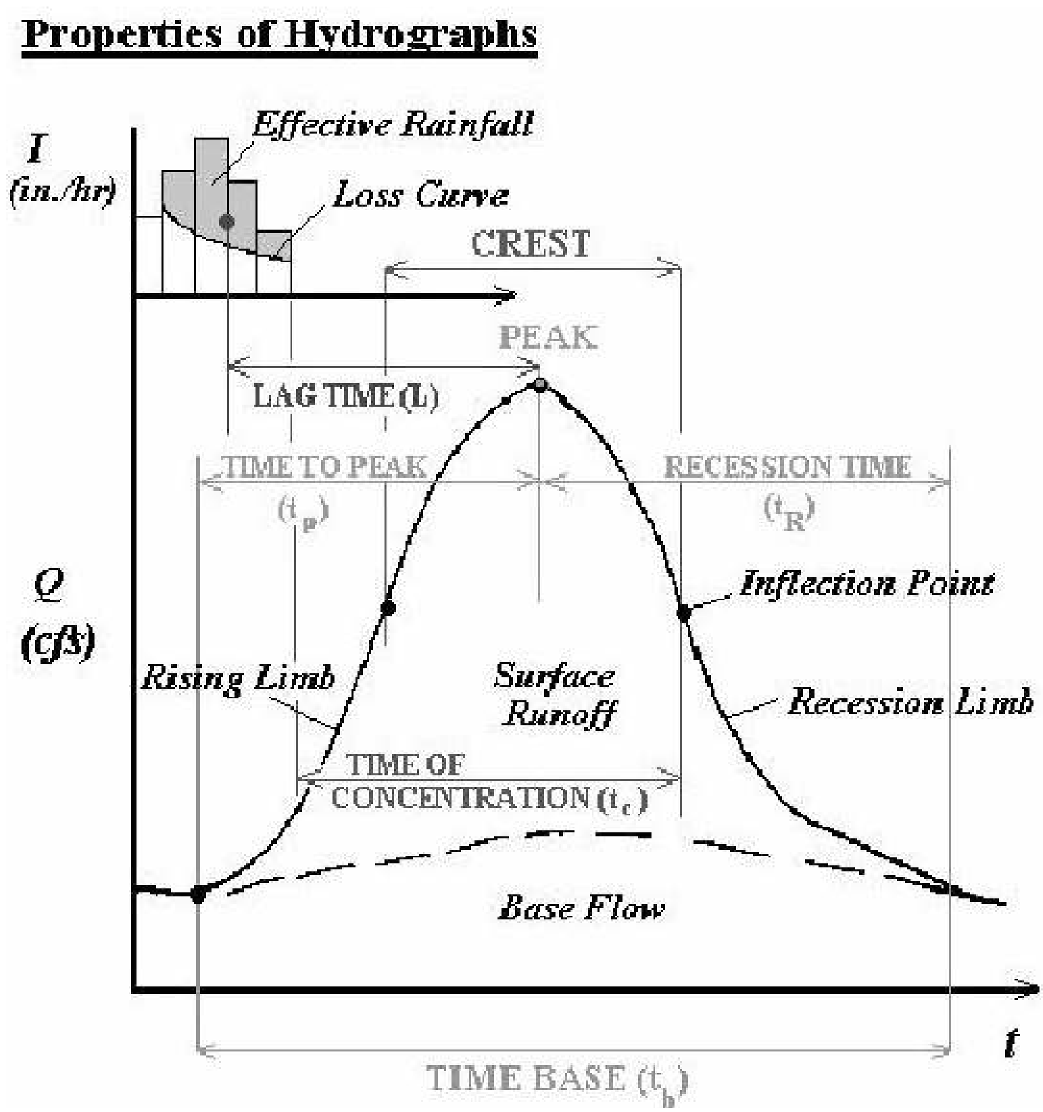


Figure 2.4: Properties of an idealized hydrograph shape. (Source: U.S Army Corps of Engineers)

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 used the unit hydrograph theory on watersheds ranging from 1300 km^2 to 8000 km^2. Linsley recommended that the unit hydrograph only be used on watersheds less than 5000 km^2, while Ponce suggested that it should only be applied on midsize catchments between 2.5 km^2 and 250 km^2. Shaw 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 (Chow *et al*., 1988).

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.

2.4.1. 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 (SCS unit hydrograph).
2. Those based on models of watershed storage (Clark unit hydrograph), and
3. Those relating hydrograph characteristics (peak flow rare, base time, etc.) to watershed characteristics (Snyder unit hydrograph).

2.4.2. Existing Methods for Flood Prediction in Ungauged Catchments

2.4.2.1. SCS Dimensionless Unit Hydrograph

The dimensionless unit hydrograph developed by the Soil Conservation Service in 1972 (Chow *et al*., 1988), has been obtained from the unit hydrographs for a great number of watersheds of different sizes and for many different locations. The SCS dimensionless Unit 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 Unit 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 qp and whose time base, t_b, is equal to 2.67 t_p. The time is usually expressed in hours and the discharge in m^3/s/cm. 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 $Q_p = CA/t_p$, 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 $t_l = 0.6tc$, where tc is the time of concentration of the watershed. The time to peak, t_p, is then equal to $t_r/2 + t_l$, where t_r is effective rainfall duration.

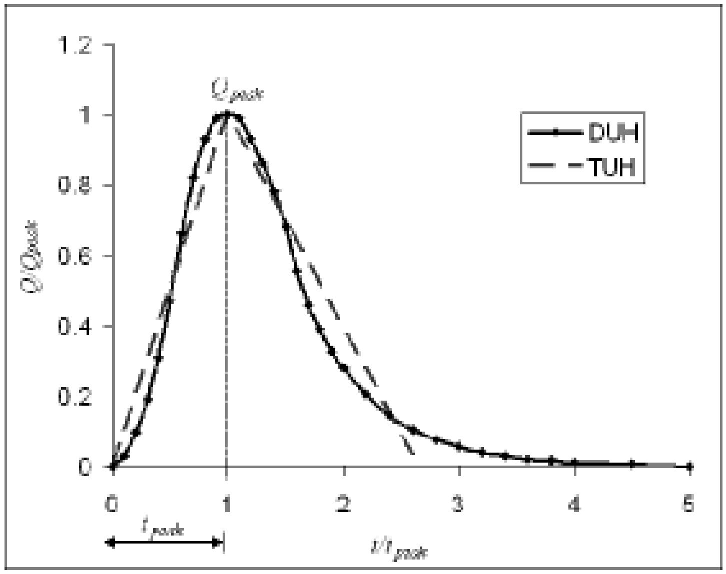


Figure 2.5: SCS dimensionless and triangular unit hydrograph

The limitations of SCS method are: 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. 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.

2.4.2.2. Clark Unit Hydrograph

Clark in 1945 developed his own synthetic unit hydrograph method that incorporated a parameter to model the watershed storage (R) and time of concentration (t_c). The Clark synthetic unit hydrograph method incorporates the processes of attenuation and translation of runoff through the use of the time-area curve. Clark 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 (but not limited to) are 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.

The limitations of Clark method are: 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.

2.4.2.3. Snyder Unit Hydrograph

The synthetic unit hydrograph of Snyder 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,

$$t_l = C_1 C_t (L L_c)^{0.3} \qquad 2.2$$

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

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, L_c is the distance in kilometers (miles) from the outlet to a point on the stream nearest the centroid of the watershed area, and C_1 = 0.75 (1.0 for English units). The product LL_c is a measure of watershed shape. C_t is a coefficient derived from gauged watersheds in the same region, and represents variations in watershed slopes and storage characteristics. The peak discharge of the standard unit hydrograph Equation 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 L_c 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 m^3/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 $5.5t_R$, 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.

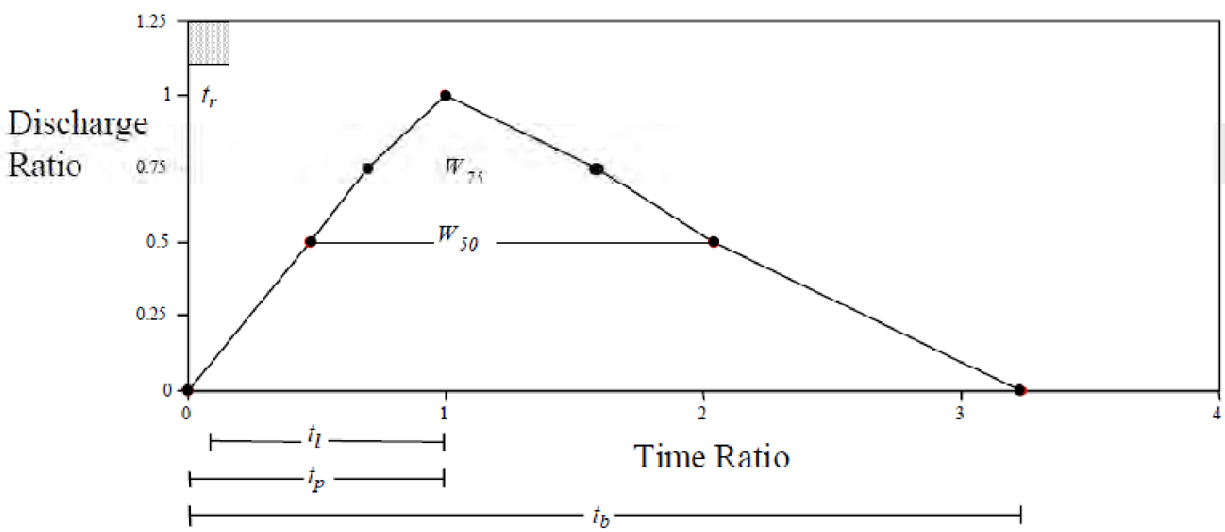


Figure 2.6: The form of the Snyder's unit hydrograph.

Limitations of Snyder method are: 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.

2.4.3. Unit Hydrographs for Different Rainfall Duration

When a unit hydrograph of a given excess-rainfall duration is available, the unit hydrographs of other durations can be derived. If other durations are integral multiples of the given duration, the new unit hydrograph can be easily computed by application of the principles of superposition and proportionality. However, a generation method of derivation applicable to unit hydrographs of any required duration may be used on the basis of the principle of superposition. This is the S-hydrograph method. The theoretical S-hydrograph is that resulting from a continuous excess rainfall as a constant rate of 1crn/hr (or 1in/hr) for an indefinite period. This is the unit step response function of a watershed system. The curve assumes a deformed S shape and its ordinates ultimately approach the rate of excess rainfall at a time of equilibrium (Chow *et al*., 1988).

2.4.4. Instantaneous Unit Hydrograph

If the excess rainfall is of unit amount and its duration is infinitesimally small, the resulting hydrograph is an impulse response function called the instantaneous unit hydrograph (IUH). For an IUH, the excess rainfall is applied to the drainage area in zero time. Of course, this is only a theoretical concept and cannot be realized in actual watersheds, but it is useful because the IUH characterizes the watershed's response to rainfall without reference to the rainfall duration. Therefore, the IUH can be related to watershed geomorphology (Rodriguez-lturbe and Valdes, 1979).

2.5. Review on the selected model approaches

For this study the two GIUH models are selected. These are:

i. Geomorphologic Instantaneous Unit Hydrograph (GIUH) Model
ii. Geomorphologic Based Nash Instantaneous Unit Hydrograph Model

2.5.1. Geomorphologic Instantaneous Unit Hydrograph (GIUH) Model

GIUH is 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 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 proposed a numbering system to perform the ordering of stream links. Strahler (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 2.7, 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.

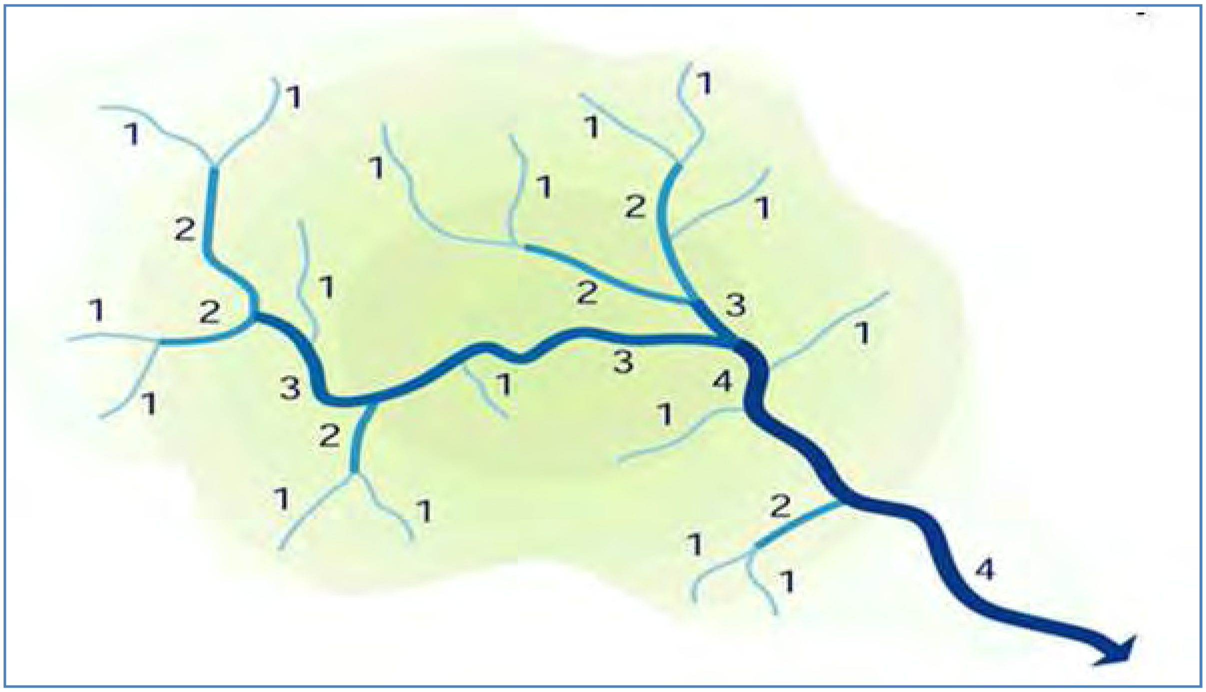


Figure 2.7: Surface Flow Paths of a fourth-Order Catchment

Drainage density(D) is defined as the length of drainage per unit area (Chow *et al*., 1988)

$$D = \frac{L}{A} \tag{2.8}$$

Where,

A is the area of the catchment, and

L is 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:

$$L_o = \frac{1}{2D} \tag{2.9}$$

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 to which 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 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:

$$q_p = 1.31 {R_l}^{0.43} \left(\frac{V}{L_\Omega}\right) (hour^{-1}) \qquad 2.10$$

$$t_p = 0.44 {R_l}^{-0.38} \left(\frac{R_B}{R_A}\right)^{0.55} \left(\frac{L_\Omega}{V}\right) (hour) \qquad 2.11$$

$$q_p t_p = 0.5764 \left(\frac{R_B}{R_A}\right)^{0.55} {R_l}^{0.05} \qquad 2.12$$

Where,

L_Ω is the length in kilometers of the highest order and

V is the expected peak velocity in m/s.

Since the first introduction of the 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). In this study, the GIUH model development was done mainly based on the reference of the works of Rodriguez-Iturbe & Valdes, (1979), and Bras and Rodriguez-Iturbe (1989); and the GIUH based Nash model development is based on the work of Roso, (1984). The detail of GIUH model development is briefly discussed in Section 4.

2.5.2. Geomorphologic Based Nash Instantaneous Unit Hydrograph Model

The Nash model is based on the concept that IUH can be derived by routing the instantaneous inflow through a cascade of linear reservoirs with equal storage coefficient. The outflow from the first reservoir is considered as inflow to the second reservoir, and so on. For derivation of IUH, the Nash model uses two parameters; 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:

$$u(t) = \left(\frac{1}{K\Gamma(n)}\right) * \left(\frac{t}{K}\right)^{n-1} * e^{\frac{-t}{K}} \qquad 2.13$$

where, u (t) denotes IUH ordinates in hour^{-1}, t is sampling time interval in hour, n and k are parameters of Nash IUH model as described above and Γ is γ function.

A UH of desired duration (D) may be derived using the following equation:

$$U(D,t) = \frac{1}{D}\left[I\left(n,\frac{t}{K}\right) - I\left(n,\frac{t-D}{K}\right)\right] \qquad 2.14$$

where, U (D,t) denotes ordinates of UH of D-hour duration in hour^{-1}, t is the sampling time interval in hour, I(n, t/k) is the incomplete γ function of order n at (t/ k) and D is the duration of UH in hour.

2.5.3. Derivation of UH using the GIUH based Nash model

The complete shape of the GIUH can be obtained by linking q_p and t_p of the GIUH with the scale (k) and shape (n) parameters of the Nash IUH model. By equating the first derivative (with respect to t) of Equation (2.13) to zero, t becomes the time to peak discharge, t_p. Thus, taking the natural logarithm of both sides of Equation (2.13), differentiating with respect to t and simplifying, we get,

$$\frac{\partial}{\partial t}\ln[u(t)] = \left[-\frac{1}{K} + \frac{n-1}{t}\right] \qquad 2.15$$

Equating (2.15) to zero results in the following expression, where t is equal to t_p

$$\left[-\frac{1}{K} + \frac{(n-1)}{t}\right] = 0 \qquad 2.16$$

and hence,

$$t = t_p = K(n-1) \qquad 2.17$$

Substituting the value of t_p from equation 2.17 in equation 2.13 and simplifying, we get

$$q_p = \left(\frac{1}{K\Gamma(n)}\right) * e^{-(n-1)*}(n-1)^{n-1} \qquad 2.18$$

From equations (2.17) and (2.18), we get

$$q_p t_p = \frac{(n-1)}{\Gamma(n)} * e^{-(n-1)} * (n-1)^{n-1} \qquad 2.19$$

From geomorphologic relationship, (from equation 2.12),

$$q_p t_p = 0.5764 \left(\frac{R_B}{R_A}\right)^{0.55} R_l^{\,0.05}$$

Equating Equation (2.19) with Equation (2.12), we get,

$$\frac{(n-1)}{\Gamma(n)} * e^{-(n-1)} * (n-1)^{n-1} = 0.5764 \left(\frac{R_B}{R_A}\right)^{0.55} R_l^{\,0.05} \qquad 2.20$$

All the terms in the right-hand side of Equation (2.20) are known. The only unknown term is the Nash model parameter n. The value of n was determined by solving Equation (2.20) using Newton-Raphson non linear optimization technique as was used by Rosso (1984) and is expressed as,

$$n = 3.29 \left(\frac{R_B}{R_A}\right)^{0.78} R_L^{\,0.007} \qquad 2.21$$

The relationship for obtaining the value of k could mathematically be expressed as,

$$K = \frac{0.7}{V} * \left(\frac{R_A}{R_B R_L}\right)^{0.48} L_\Omega \qquad 2.22$$

Where, L_Ω is length of the basin of order Ω; R_A and R_B are Horton's ratios; and v is the uniform stream flow velocity which is taken as constant. Thus, the scale parameter of the model is time variant and depends on both, the catchment geomorphology and the average 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 are used to determine the complete shape of GIUH based Nash model using Equation (2.13). Subsequently, the D-hour UH is obtained using the relationship between IUH and UH of D-hour as given by Equation (2.14). The DSRO hydrograph is estimated by convoluting the excess-rainfall hyetograph with the UH obtained above.

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

Geographic Information Systems (GIS) can be defined as computer based tools that display, store, analyze, retrieve, and process spatial data. GIS is being more and more involved in hydrology and water resources and showing promising results. GIS provides representations of the spatial features of the earth, while hydrological models are concerned with the flow of water and its constituents over the land surface and in the subsurface environment. GIS with its upcoming advanced technology has been a great advantage to hydrological modeling. Hydrological modeling using GIS has been great developed during the last decade when people realized the utility of incorporating GIS with hydrologic modeling. Berry and Sailor (1987) noted some of the advantages of GIS in hydrology and water resources. According to them, GIS provides a powerful tool for expressing complex spatial relationships. It provides an opportunity to fully incorporate spatial conditions into hydrologic inquiries. GIS are highly specialized database management systems for spatially distributed data.

GIS provides a digital representation of the catchment characterization used in hydrological modeling. Maidment (1996) summarized the different levels of hydrological modeling in association with GIS as follows: hydrologic assessment; hydrologic parameter determinations; hydrologic modeling inside GIS; and linking GIS and hydrologic models. With respect to GIS processing products, Digital Elevation Models (DEM) is more important in rainfall runoff modeling. The DEM have proved to be very efficient in extracting the hydrological data from the remote sensing data by analyzing different topographical attributes (elevation, slope, aspect, relief, curvatures) for modeling purposes. 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. He also introduced different algorithms to extract catchments from DEM. DEM is popularly processed in Arcgis, Arcview (with Hec-Geo-HMS, ILWIS (Maathuis, 2005), etc. to extract hydrologic parameters or physical characteristics of a catchment and can serve for model simulation. The reason of adopting GIS technology in hydrological models is because it allows the spatial information to be displaced in integrative ways that are readily comprehensible and visual. The spatial information collected is further subjected to continuous GIS analysis.

In this study a GIS package used was ILWIS (Integrated land and Water Information System). It has a potential to extract the catchment characteristics easily using their DEM and calculate the Horton ratios that are used for GIUH model.

2.7. Previous Studies

2.7.1. General

The concept of the Geomorphological unit hydrograph is introduced by Rodriguez-Iturbe and Valdes in 1979. GIUH approach has been applied by several engineers to predict runoff from rainfall for ungauged catchments. They have been proposed to estimate floods for ungauged streams by using the information obtainable from topographic maps or remote sensing possibly linked with the Geographic Information Systems (GIS) and Digital Elevation Models (DEM) (Jain et al., 2000).

Lee and Chang (2005) reviewed the development of GIUH approach and concluded that the significant advance in research on the topographic runoff approaches was the development of the geomorphologic instantaneous unit hydrograph model (GIUH) proposed by Rodriguez- Iturbe and Valdes (1979).

During the last two decades, the use of catchment geomorphologic characteristics in runoff simulations has received a great deal of attention from hydrologists (Lee and Yen, 1997).

2.7.2. On the Study Area

Different flow prediction models have been studied in Abay basin to develop runoff prediction model in ungauged catchments. However, based on unit hydrograph concepts, only a few attempts are made in Ethiopia. Recently, some runoff prediction methods based on the concept of synthetic unit hydrographs has been developed. Mulugeta Azeze (2004) developed regionalization of Snyder‘s unit hydrograph in upper Awash and upper Tekeze basins.

Godana Seyoum (2005) has also developed a regionalized equation using Snyder unit hydrograph in Abay basin and he has found a good results. But, the problems in his research is,

- To transfer C_t and C_p to ungauged catchment, studying the hydrologic similarity of the ungauged catchment with the catchments used in research work are required.

- Unit hydrograph parameters are developed from the relation of 7 equations (t_p, q_p, T_p, Q_p, T_B, W_{50} and W_{75}). The area under the UH is adjusted to unit depth without altering the vertex of the determined values. Thus, this is a difficult task to get a unit depth without altering the vertexes.
- His derived equation requires the values of manning coefficient (n) and value of impervious area (I) to determine the value of watershed conveyance factor (Φ). The manning value varies from time to time and also it changes with the magnitude of discharge or depth of flow.

But, the geomorphological parameters are mostly time-invariant in nature. Therefore, the geomorphology based approach could be the most suitable technique for modeling the rainfall-runoff process for ungauged catchments.

Tsehay Zeray(2009) studied on unit hydrograph model development based on the concept of GIUH on some catchments in Abbay basin and he has found a good results. But, the catchments he used were small in number, concentrated on lake Tana area, and also he has not developed a regional simplified triangular unit hydrograph equation.

3. STUDY AREA AND DATA COLLECTION

3.1. Study Area

The study area is in the Abay basin. Eight catchments were selected; those are Fettam, Hoha, Neshi, Uke, Andassa, Azuari, Jedeb and Chemoga. Fettam, Jedeb and Chemoga are found in South Gojam sub-basin. Andassa and Azuari are found in North Gojam sub-basin. Hoha is found in Dabus sub-basin. Neshi is found in Fincha sub-basin and Ukke is found in Anger sub-basin. Catchment selection criteria are described in section 3.3.2.

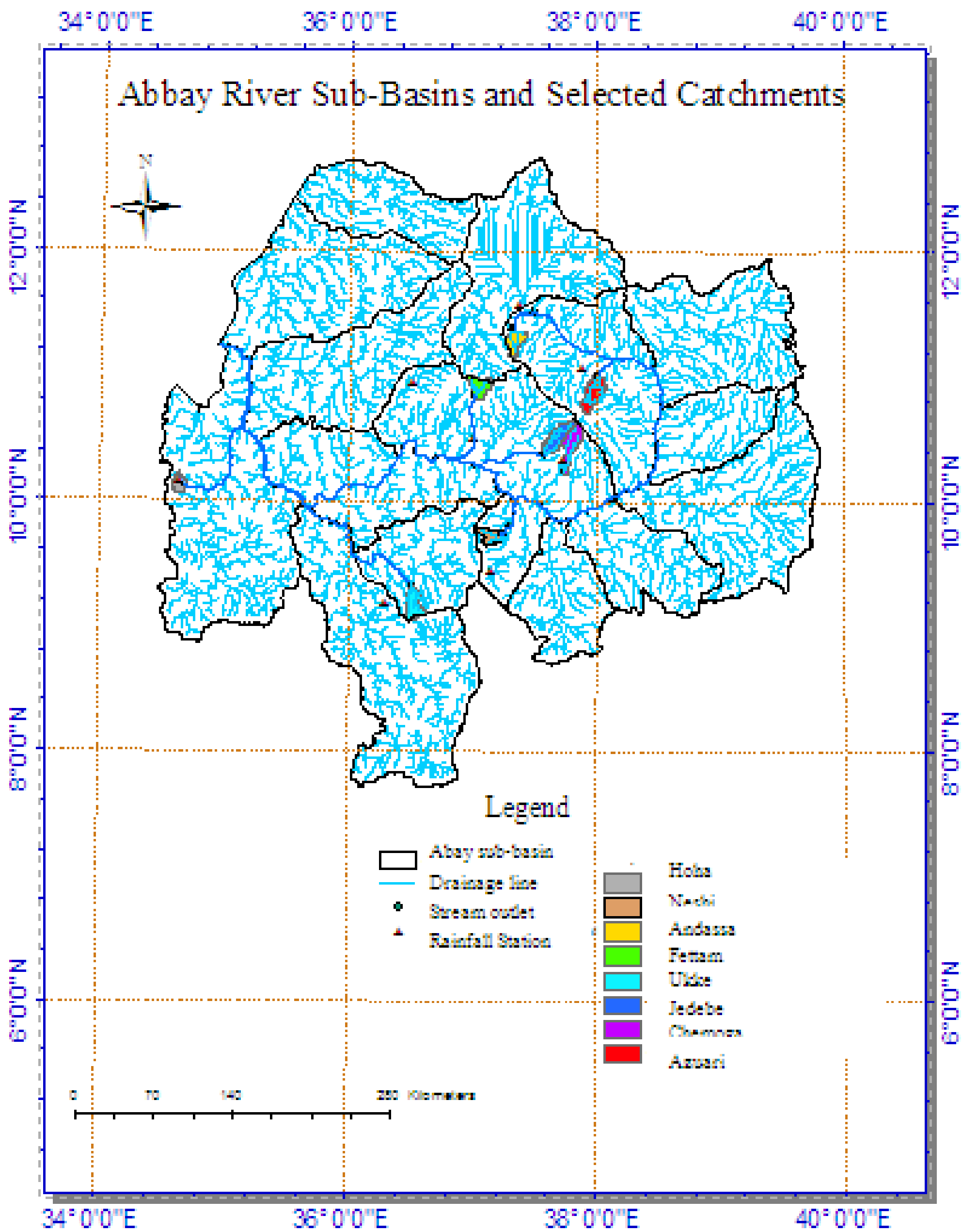


Figure 3.1: Location of the study area

3.2. Geomorphologic Parameters

Certain characteristics of the drainage watershed reflect hydrologic behavior and are therefore, useful, when quantified, in evaluating the hydrologic response of the watersheds. These characteristics relate to the physical characteristics of the drainage watershed as well as of the drainage network. Physical characteristics of the drainage watershed include drainage area, watershed shape, ground slope, and centroid (i.e. centre of gravity of the watershed). Channel characteristics include channel order, channel length, channel slope, and drainage density.

3.2.1. Watershed order and channel order

Drainage areas may be characterized in terms of the hierarchy of stream ordering. The order of the watershed is the order of its highest-order channel. A watershed is described as first, second, third, or higher order, depending upon the stream order at the outlet. All the selected catchments for this study has fourth order stream network.

3.2.2. Watershed area

Watershed area is defined as the area contained within the vertical projection of the drainage divide on a horizontal plane. Some areas in the drainage basin do not contribute to the runoff and are termed as closed drainage. These areas may be lakes, swamps etc. The watershed areas of the selected catchments are presented in Table 3.2. The selected watershed areas vary from 161 to 573 km^2.

3.2.3. Watershed shape

The watershed shape may influence the hydrograph shape, especially for small watersheds. For example, if the watershed is long and narrow, then it will take longer time for water to travel from the most extreme point to the outlet and the resulting hydrograph shape is flatter. For more compacted watershed, the runoff hydrograph is expected to be sharper with a greater peak and shorter duration. Numerous symmetrical and irregular forms of drainage areas are encountered in practice.

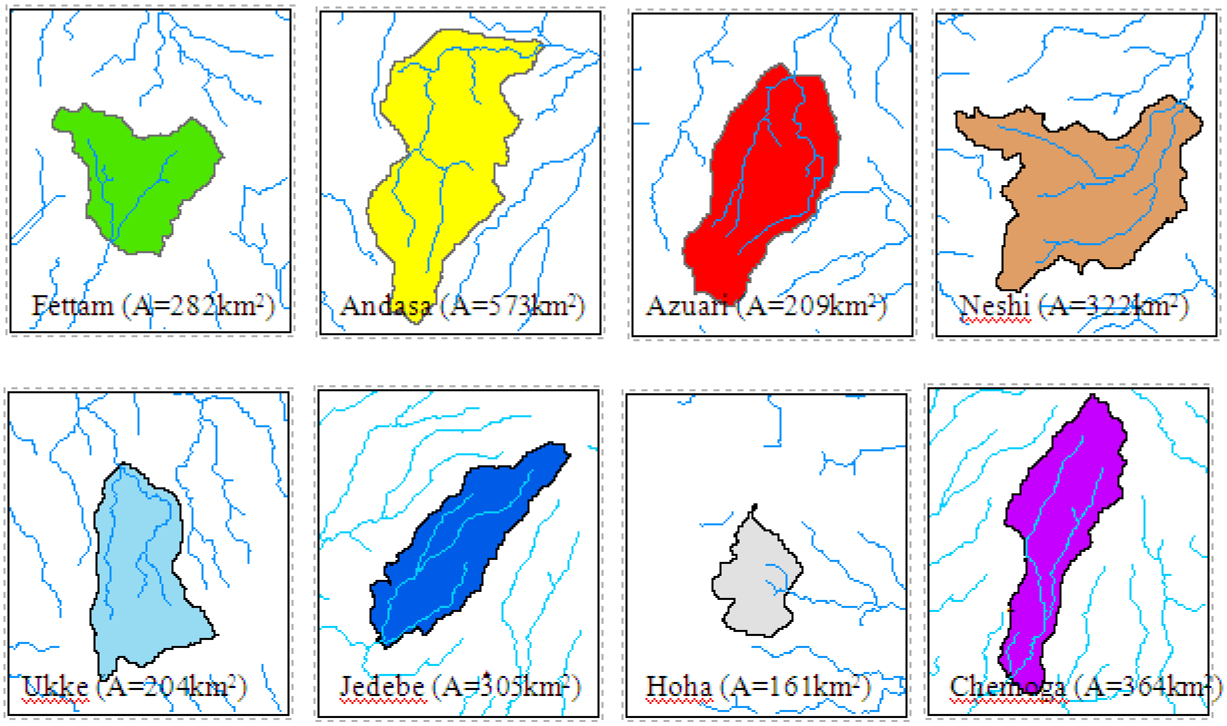


Figure 3.2: Shape and Area of the selected watersheds

Therefore, from the above figure, we can say that, Hoha and Fettam are round type of watersheds; while Uke, Jedeb and Chemoga are narrow elongated type of watershed whereas, Neshi, Andassa and Azuari are wider elongated type of watersheds.

3.2.4. Watershed slope

Watershed slope has a pronounced effect on the velocity of overland flow, watershed erosion potential, and local wind systems. Average basin slope is defined as (Singh, 1989)

$$S = h/L \qquad 3.1$$

Where S is the average basin slope (m/m), h is the fall (m) (i.e. difference in maximum and minimum elevations), and L is the horizontal distance (m) over which the fall occurs. The selected watershed slope varies from 0.94 to 4.37%.

3.2.5. Climate

The climate of the study area varies from humid to semiarid. In most of the catchment (particularly in the west), about 95% of the annual rainfall occurs in the wet season (May to October). In some years, short rains occur between April and May and in eastern areas, there is a bimodal rainfall pattern with the *belg* (short wet season from mid-February to mid-May) and the *kiremt* (main rainy season from June to September). The annual precipitation has an increasing trend from north-east to south-west. The mean annual temperature ranges from 5 to 30^{o}C depending on altitude (Yilma and Awulachew, 2009).

3.3. Data Collection

3.3.1. General

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. There are few reliable meteorological and hydrologic stations in the basin and most of the monitoring was started by NMSA and MOWR of Ethiopia in the early 1960s.

3.3.2. 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, 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, data's were collected adequately and used in the modeling work. Moreover, in this study catchments selected by Tsehay Zeray (2009) were revised for regional equation derivation since there was some area variation in his document from the actual. Stream flow recording stations with corresponding rainfall recording stations are summarized in Table 3.3 and Table 3.4.

Table 3.1: The Selected Catchments and their Locations

Hydrological Stations

Station No.	River Name	Site Name	Latitude	Longitude	Catchment Area(km^2)
113019	Fettam	Tilili	$10^0 51'$	$37^0 01'$	282
115003	Hoha	Nr Asossa	$10^0 09'$	$34^0 38'$	161
113026	Neshi	Nr. Shambo	$9^0 45'$	$37^0 15'$	322
114018	Uke	Nr. Nekemte	$9^0 19'$	$36^0 31'$	204
112018	Azuari	Nr. Mota	$10^0 58'$	$38^0 01'$	209
112004	Andassa	Nr. Bahirdar	$11^0 30'$	$37^0 29'$	573
113011	Jedeb	Nr. Amanuel	$10^0 24'$	$37^0 34'$	305
113008	Chemoga (as ungauged catchment)	Nr. Debremarkos	$10^0 18'$	$37^0 44'$	364

3.3.3. 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.

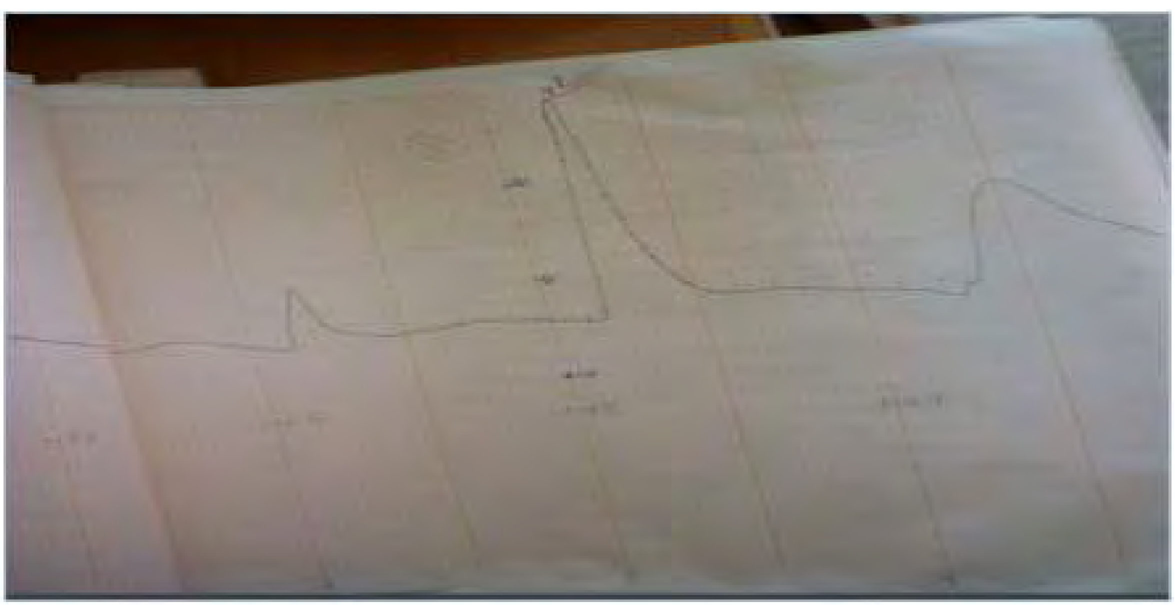


Figure 3.3: Sample chart of continuous stream flow hydrograph

3.3.4. 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.

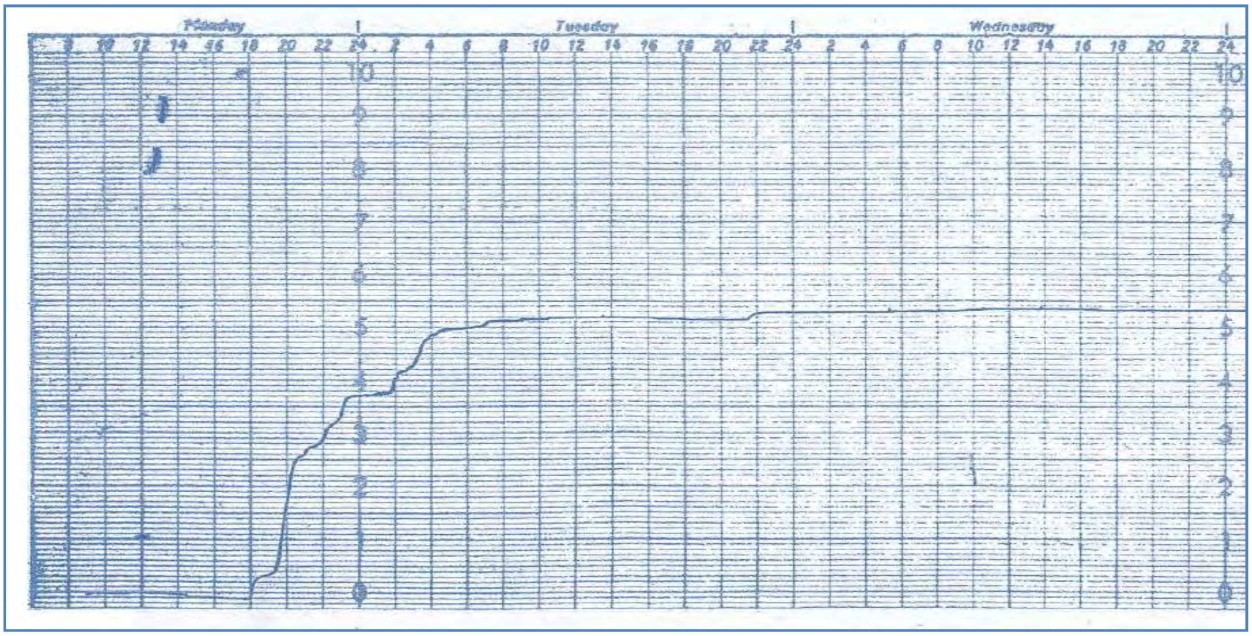


Figure 3.4: Sample chart of continuous rainfall intensity

Table 3.2: The Selected Catchments and their Corresponding Rainfall Stations

Station No.	River Name	Rainfall Station	Latitude	Longitude
113019	Fettam	Finoteselam	10^{0}51’	37^{0}01’
115003	Hoha	Hoha	10^{0}09’	34^{0}38’
113026	Neshi	Shambu	9^{0}45’	37^{0}15’
114018	Uke	Nekemte	9^{0}19’	36^{0}31’
112018	Azuari	Mota	11^{0}5’	37^{0}52’
112004	Andassa	Bahirdar	10^{0}20’	37^{0}40’
113011	Jedeb	Debremarkos	10^{0}20’	37^{0}40’
113008	Chemoga (as ungauged catchment)	Debremarkos	11^{0}36’	37^{0}45’

The data’s collected for the study areas are shown in table 3.3 below.

Table 3.3: The selected catchment and their corresponding collected data

River Name	Rainfall Station	Collected stream flow data	Collected rainfall data
Fettam	Finoteselam	July 3, 8, 17/1985, Oct 8/1985, July 27, 28/1986, Sept 11/1986, May 29/1987, July 24/1987, Nov 15/1988	July 1985, Oct 1985, July 1986, Aug 1986, Sept 1986, May 1987, July 1987, Nov 1988, July 1989
Hoha	Hoha	June 28/1985, July 1/1985, Aug 11/1985, Aug 12/1985, Aug 14/1985, Aug 25/1985	June 1985, July 1985, Aug 1985, Aug 1985, Aug 1985, Aug 1985
Neshi	Nr. Shambo	June 9/1987, July 3, 5/1987, Aug 4/1987, Sept 24/1987	June 1987, July 1987, Aug 1987, Sept 1987
Uke	Nr. Nekemte	Sept 6, 9, 17, 22/1988, Oct 3/1988, Oct 28/1989	Sept 1988, Oct 1988, Oct 1989
Azuari	Mota	July 8/1989, July 28-29/1989, Aug 8-9/1989, Aug 26-27/1989	July 1989, Aug 1989,
Andassa	Bahirdar	Jun 22/1986, Aug 1/1986, Aug 17/ 1986, Sep 4/ 1988, Sep 29/1988	Jun 1986, Aug 1986, Sep 1988
Jedeb	Debremarkos	July 13-14/ 1982, July 14-15/ 1982, July 24/ 1982, Aug 6/1982, Jun 21/1984, Jun 26/ 1984, Sep 3/ 1989	July 1982, Aug 1982, Jun 1984, Sep 1989
Chemoga (as ungauged catchment)	Debremarkos	July 29-30/ 1986, Aug 3/1986, May 15, 1984, May 24/ 1989	

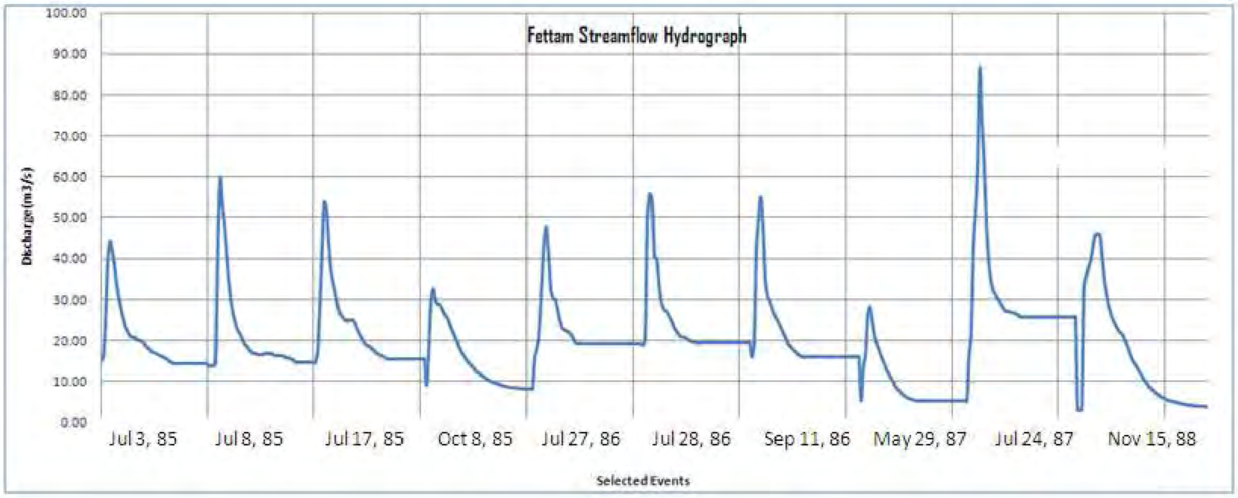


Figure 3.5: Fettam stream flow hydrograph for selected events

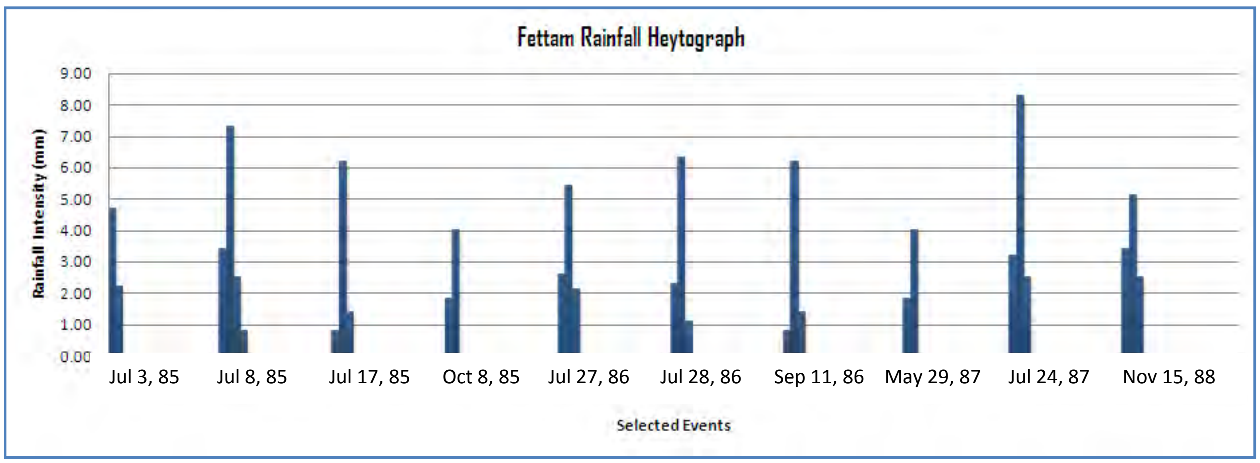


Figure 3.6: Fettam rainfall hyetographs for selected events

The rest catchments data's are attached in Appendix G and H

4. METHODOLOGY

4.1. Excess Rainfall and Direct Runoff Determination

The graph that plots the rainfall over the basin along a time scale is called the hyetograph. Excess rainfall, or effective rainfall, is that rainfall which is neither retained on the land surface nor infiltrated into the soil. Although a number of techniques are available for separating the losses from a rainfall hyetograph, the infiltration indices method is the simplest and the most popular techniques used for this purpose (C.S.P OJHA, 2008). In hydrological calculations involving floods, it is convenient to use a constant value of infiltration rate (including all losses from the total rainfall hyetograph) for the duration of the storm. For this study a Φ-index method was selected for infiltration rate computation.

The Φ-index is the constant rate of abstractions (mm/hr) that will yield an excess rainfall hyetograph (ERH) with a total depth equal to the depth of direct runoff (r_d) over the watershed(Chow et al., 1988).

- Procedures for determining excess rainfall hyetograph (ERH) from the observed rainfall hyetograph (RH) and stream flow hydrograph by Φ-index Method:

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 (V_d) and the equivalent depth of direct runoff r_d.

$$\text{Volume of direct runoff } (V_d) = \sum_{n=1}^{n} Q_n \, \Delta t \qquad 4.1$$

$$\text{Depth of direct runoff } r_d = \frac{Volume\ of\ Direct\ Runoff(Vd)}{Watershed\ Area(A)} \qquad 4.2$$

4. Estimate the rainfall abstraction by infiltration and surface storage in the watershed by Φ-index Method.

$$r_d = \sum_{m=1}^{M} (R_m - \Phi\Delta t) \qquad 4.3$$

Where:

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

Δt : time interval length (hr),

R_m : observed rainfall (mm) in time interval,

Φ*:* 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 Φ gives positive result and greater than the initial abstraction, the value of Φ is satisfactory.
- If the value of ϕ gives negative result this is not physically possible. To solve for a new trial value of ϕ, the value of the highest rainfall has taken by increasing the number of intervals, M.

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

4.1.1. Baseflow Separation Techniques

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 was 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.

4.2. Derivation of Unit Hydrograph

A unit hydrograph (UH) is a hydrograph of direct surface runoff from a unit excess rainfall occurring in unit time uniformly over the catchment area. A unit hydrograph can be interpreted as a multiplier that converts excess rainfall to direct surface runoff. The direct surface runoff is the stream flow hydrograph excluding base-flow contribution.

For known direct runoffs and storm characteristics including magnitude and duration, unit hydrograph can be derived. In this section the procedures to derive the unit hydrograph are listed below.

4.2.1. Conventional Method Using Single Peak Storm

When a storm is of short duration and fairly uniform intensity, the unit hydrograph can be derived by simply applying the single period storm technique.

The unit hydrograph for a gauging station is derived from a single period storm as follows:

i. Extract hydrographs for major floods from stream flow records using observed gauge-discharge or rating curves.
ii. Separate base-flow from total hydrograph to produce direct surface runoff hydrograph.
iii. Calculate the depth of direct runoff
iv. Examine the rainfall records available for the rain gauge stations in the catchment and compute average rainfall for the storm under investigation.
v. Derive the hydrograph and the total average rainfall for the time period which corresponds to the desired unit period of the unit hydrograph.
vi. Determine the loss rate and estimate the excess rainfall depth.
vii. Then, unit hydrograph can be 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.

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 (Chow *et al*., 1988)..

4.2.2. UH Derivation from Multi Period Storm

For a storm of long duration or a shorter one with variable intensity, the storm should be treated as consisting of a series of storms and the derivation of the unit hydrograph gets slightly more complicated. For this case 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.

$$Y_i = \sum_{k=1}^{i} X_{i-k+1} * U_k \qquad 4.4$$

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

$$U = (X^T * X)^{-1} * X^T * Y \qquad 4.5$$

This unit hydrograph derivation is applicable to a wide range of rain events and can be solved by Matlab (Matlab 7.0), in order to make the calculations easier.

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.

4.2.3. Change of Unit period of UH

Having derived a unit hydrograph for a particular unit period one may want to change the time period and derive a new hydrograph. The following two methods were used for this purpose:

i. Superimposition Method

   This method is suitable only when the new duration of the UH is an integer multiple of the given unit duration. For example, the UH of nT-duration, where n is an integer, can be derived by successive lagging of the T-duration unit hydrograph n times and then dividing the resulting hydrograph by n.

ii. S-Curve Method

   S-Curve of T-hr unit hydrograph is the graph that results from 1/T intensity of excess rainfall which occurs for an infinite period. This is a more general method than the method of superimposition. After having derived the S-curve of unit intensity from T-duration unit hydrograph, the unit hydrograph of T-duration can be obtained as follows:

   - Shift the S-curve by T hrs to obtain another S-curve,
   - Subtract another S-curve ordinate from the original S-curve,
   - The difference between the two S-curves represent the unit hydrograph for time T with a unit volume equal to iT = T mm (as i = 1mm/hr),
   - To derive 1mm UH of duration T divide the difference by T.

4.3. Derivation of an Average Unit Hydrograph

In Practice, unit hydrographs derived from different rainfall-runoff events of a particular catchment differ significantly from one another, even if the duration of effective rainfall is similar. So, 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 average basin representative unit hydrograph that may be applicable to the widest range of rainfall events and antecedent soil moisture conditions.

The graphical method (Wilson, 1974) of deriving an average UH can be applied by the following steps:

i) Plot the unit hydrographs derived from a number of individual events on a single plotting paper.

ii) Mark a point such that it corresponds to the average peak discharge ordinates and average time to peak.

iii) Draw an average unit hydrograph such that it passes through the average peak point, has unit volume, and generally conforms to the characteristics shapes of the individual unit hydrographs. Some trials for adjusting the ordinates of the derived unit hydrograph may be required for preserving the volume.

4.4. 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,

$$Q(t) = \int_0^t i(\tau) * u(t-\tau) d\tau \qquad 4.6$$

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 hill slope or along the streams are assumed exponential distributed and the initial and transitional probabilities are calculated based on Horton‘s parameters.

4.5. Determination of Horton’s ratio

Three of Horton’s ratios namely bifurcation ratio (R_b), stream-length ratio (R_l) and stream-area ratio (R_a) are unique representative parameters for a given watershed and are fixed values for a given watershed system.

4.5.1. Bifurcation Ratio (R_B)

The number of channels of a given order in a drainage basin is a function of the nature of the surface of that drainage basin. In general, the greater the infiltration of the soil material covering the basin, the fewer will be the number of channels required to carry the remaining runoff water. Moreover, larger the number of channels of a given order, the smaller is the area drained by each channel order. A dimensionless parameter based on the number of channels with respect to their order is termed as bifurcation ratio and is useful in defining the watershed response. The bifurcation ratio is given as follows.

$$R_B = \frac{N_i}{N_{i+1}} \qquad 4.7$$

Where R_B is the bifurcation ratio, N_i and N_{i+1} are the number of streams in order i and i+1 respectively, i = 1, 2, 3, … Ω and Ω is the highest stream order of the watershed.

4.5.2. Stream Length Ratio (R_L)

This refers to length of channels of each order. The average length of channels of each higher order increase as a geometric sequence, which can be further explained as: the first order channels are the shortest of all the channels and the length increase geometrically as the order increases. This relation is called Horton's law of channel length and can be formulated as follows.

$$R_L = \frac{\bar{L}_{i+1}}{\bar{L}_i} \qquad 4.8$$

Where $\bar{L}_i$, the average length of channels of order *i* is:

$$\bar{L}_i = \frac{1}{N_i}\sum_{j=1}^{N_i} L_{j,i} \qquad 4.9$$

The lengths of channels of a given order are determined largely by the type of soil covering the drainage basin. Generally, more pervious the soil, longer will be the channel length of a given order. Also, higher is the R_L more will be the imperviousness.

4.5.3. Stream Area Ratio (R_A)

The Channel area of order i, A_i is the area of the watershed that contributes to the channel segment of order i and all lower order channels. It can be quantified as:

$$R_A = \frac{\bar{A}_{i+1}}{\bar{A}_i} \tag{4.10}$$

Where: $\bar{A}_i$ is the mean area of the contributing watershed to streams of order i,

$$\bar{A}_i = \frac{1}{N_i}\sum_{j=1}^{N_i} A_{i,j} \tag{4.11}$$

$A_{i,i}$ represents the total area that drains into the j^th stream of order *i*.

The R_B, R_L and R_A values 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).

4.6. Initial and Transition State Probabilities

The initial probability accounts for a drop falling any hillslope areas, neglecting rain falls on the channel network, in catchment of a given order. The transition probability accounts for the changing state of a drop from lower order stream to the higher order stream. For four order stream, initial state probabilities can be represented by θ_1, θ_2, θ_3 and θ_4 while the transition probabilities can be 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,

$$GIUH(t) = \sum_{Si} prob(T_{si}) * prob(S_i) \tag{4.12}$$

Where:

T_{si} : 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 functions of the total path travel time T_{si}.

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(eight).

Path 1: a1 →r1→ r2 →r3 →r4 →outlet.

Path 2: a1 →r1 →r2 →r4 →outlet.

Path 3: a1→ r1 →r3 →r4 →outlet.

Path 4: a1 →r1 →r4 →outlet.

Path 5: a2 →r2 →r3 →r4 →outlet.

Path 6: a2 →r2 →r4 →outlet.

Path 7: a3 →r3→ r4 →outlet

Path 8: a4 →r4 →outlet

Where:

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

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

The path probability of any path is calculated as,

$$Prob(Si) = \theta_j * P_{ij} * P_{jk} \ldots\ldots P_{l\Omega} \qquad 4.13$$

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. 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:

$$P_{i,j}= \begin{cases} \frac{2N_{i+1}}{N_i} + \frac{(N_i-2N_{i+1})E(j,\Omega)}{N_i\left(\sum_{k=i+1}^{\Omega} E(k,\Omega)\right)} & j=i+1 \\ \\ \frac{(N_i-2N_{i+1})E(j,\Omega)}{N_i\left(\sum_{k=i+1}^{\Omega} E(k,\Omega)\right)} & Otherwise \end{cases} \qquad 4.14$$

In this expression, *E(i, Ω)* represents the expected number of links of orders i in a network of order Ω. Under the random topological model assumption, it is given by,

$$E(i,\Omega) \rightarrow N_i\left(\prod_{j=2}^{i} \frac{N_{j-1}-1}{2N_j-1}\right) \qquad 4.15$$

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

$$P_{12} = \frac{2}{R_B} + \frac{(2R_B-1)(R_B{}^2-2R_B)}{R_B{}^2(2R_B-1)+R_B(R_B{}^2-1)+(R_B{}^2-1)(R_B-1)} \qquad 4.16$$

$$P_{13} = \frac{(R_B{}^2-1)(R_B-1)}{R_B{}^2(2R_B-1)+R_B(R_B{}^2-1)+(R_B{}^2-1)(R_B-1)} \qquad 4.17$$

$$P_{14} = \frac{(R_B{}^2-1)(R_B-1)(R_B-2)}{R_B{}^3(2R_B-1)+R_B{}^2(R_B{}^2-1)+R_B(R_B{}^2-1)(R_B-1)} \qquad 4.18$$

$$P_{23} = \frac{R_B-2}{2R_B-1} + \frac{2}{R_B} \qquad 4.19$$

$$P_{24} = \frac{(R_B-1)(R_B-2)}{R_B(2R_B-1)} \qquad 4.20$$

$$P_{34} = 1 \qquad 4.21$$

The initial state probability 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:

$$\theta_\omega = \begin{cases} \frac{N_1 A_1}{A_\Omega} & \omega = 1 \\ \\ \frac{N_\omega \left(A_\omega - \sum_{k=1}^{\omega-1} \frac{P_{k,\omega} N_k A_k}{N_\omega} \right)}{A_\Omega} & Otherwise \end{cases} \qquad 4.22$$

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

$$\theta_1 = \frac{N_1 \bar{A}_1}{A_4} \qquad 4.23$$

$$\theta_2 = \left(\frac{R_B}{R_A}\right)^2 - \left(\frac{R_B}{R_A}\right)^3 * P_{12} \qquad 4.24$$

$$\theta_3 = \left(\frac{R_B}{R_A}\right) - \left(\frac{R_B}{R_A}\right)^2 * P_{23} - \left(\frac{R_B}{R_A}\right)^3 * P_{13} \qquad 4.25$$

$$\theta_4 = [1 - \theta_1 - \theta_2 - \theta_3] \qquad 4.26$$

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

$$T_{si} = T_{ai} + T_{ri} + T_{ri+1} + \cdots + T_{r\Omega} \qquad 4.27$$

Where:

T_{ai} is the travel time on the hilly slope and

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

Assuming that these individual times of travel are independent variables such that f T_{ai} is the probability density function of T_{ai} and f T_{ri} 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:

$$Prob(T_{si} t) = \sum_{s=S} f_{T_{si}}(t), \; or \qquad 4.28$$

$$Prob(T_{si}t) = \sum_{s=S} f_{T_{ai}}(t) * f_{T_{ri}}(t) * f_{T_{ri+1}}(t) * \dots * f_{T_{r\Omega}} \quad 4.29$$

Where:

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

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

The exponential density function has the following form.

For hillslope,

$$f_{T_{ai}}(t) = \alpha_i e^{(-\alpha_i t)} \quad 4.30$$

For channels,

$$f_{T_{ri}}(t) = \lambda_i e^{(-\lambda_i t)} \quad 4.31$$

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

$$\lambda_i = \frac{V_S}{\bar{L_i}}, and \quad 4.32$$

$$\alpha_i = \frac{V_0}{L_0}, \quad 4.33$$

$$where, L_0 = \frac{1}{2D} \quad 4.34$$

Where:

L_o: The average overland flow length, and

D: Drainage density (the ratio of total drainage length to drainage area).

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:

$$GIUH(t) = \sum_{s=S} f_{T_{ai}}(t) * f_{T_{ri}}(t) * f_{T_{ri+1}}(t) * \dots * f_{T_{r\Omega}} * Prob(S) \quad 4.35$$

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

$$f_{Si} = \sum_{j=1}^{\Omega} \frac{\lambda_i \ldots \lambda_\Omega * \exp(-\lambda_j t)}{(\lambda_i - \lambda_j)\ldots(\lambda_{j-1} - \lambda_j)(\lambda_{j+1} - \lambda_j)(\lambda_\Omega - \lambda_j)}, j \neq i \tag{4.36}$$

Then, using the above formula, the probability density function equations of each of the possible paths can be computed (Appendix C).

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 GIUH model were converted to the corresponding unit hydrographs for a given time interval Δt using the following equation:

$$U_i = \frac{1}{N}(0.5u_{i-N} + u_{i-N+1} + \cdots + u_{i-1} + 0.5u_i) \tag{4.37}$$

Where, U_i is the i[th] ordinate of the unit hydrograph of D-hour duration and computational interval Δt-hour, N is the number of computation intervals in D-hour and is equal to D/Δt, and u_i is the i[th] ordinate of the IUH (Rakesh Kumar, et al., 2007).

For example, a three order stream has four flow possible paths (S_1, S_2, S_3, and S_4).

1. First, find the probability density function of each path (fs1, fs2, fs3 and fs4).
2. Secondly, determine the corresponding path probabilities, prob(S_1), prob(S_2), prob(S_3) and prob(S_4).
3. Then, compute GIUH(t) using table below.

Table 4.1 GIUH(t) computation table

1	2	3	4	5	6
Time(hr)	fs_1*prob(S_1)	fs_2*prob(S_2)	fs_3*prob(S_3)	fs_4*prob(S_4)	GIUH(t)
1	-	-	-	-	Sum 2-5
2	-	-	-	-	Sum 2-5
3	-	-	-	-	Sum 2-5

The unit of the computed GIUH is in unit/hr. So it can be converted in to m^3/s by multiplying with excess rainfall and catchment area. In this method velocity was used to calibrate the simulated unit hydrograph with the observed unit hydrograph.

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

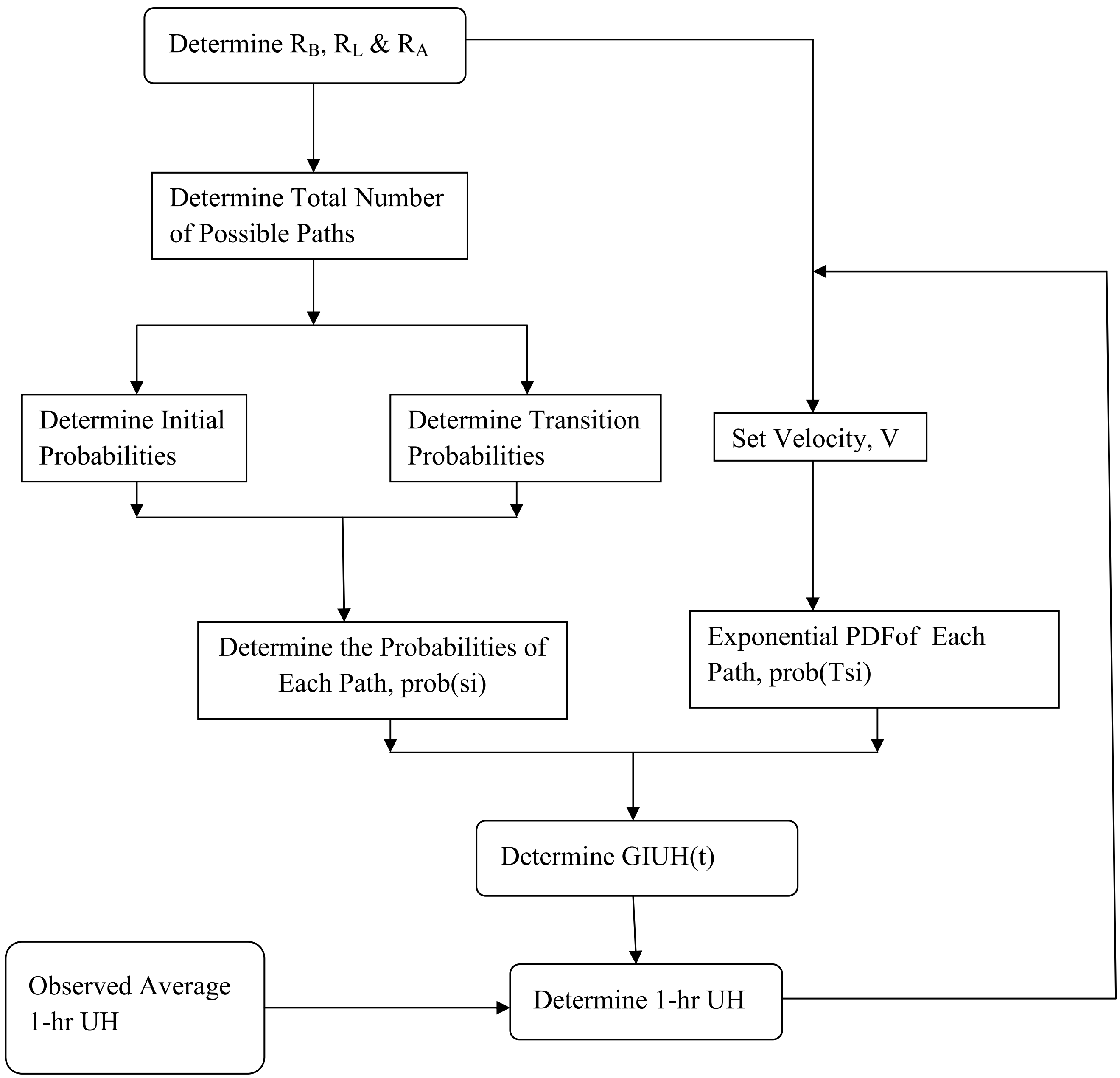


Figure 4.1: Flow Chart of GIUH Model Development

4.7. 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 1 unit 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 as shown in Figure 4.2.

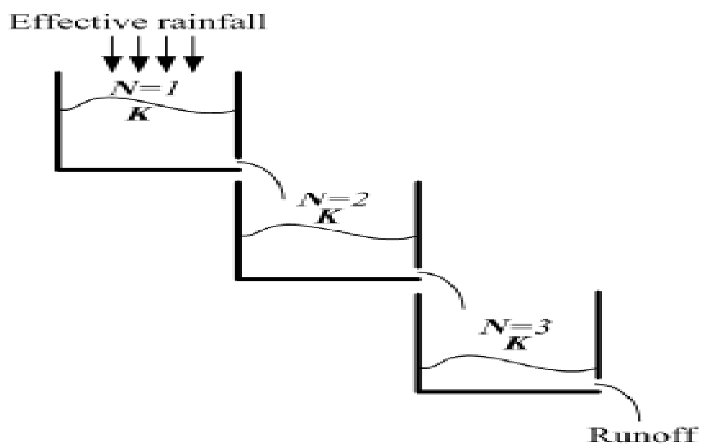


Figure 4.2: Nash cascade model

For derivation of IUH the Nash model uses two parameters; 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:

$$u(t) = \left[\frac{1}{K\Gamma(n)}\right]\left(\frac{t}{K}\right)^{n-1} e^{\frac{-t}{k}} \qquad 4.38$$

Where:

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

K : The storage coefficient for the reservoirs, and

Γ(n) : The standard gamma function.

Roso (1984) estimated 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 GIUH equation. The t_p and q_p of the Nash IUH can be estimated by equating the first derivative of Equation 4.37 with respect to time *t* to zero, *t* becomes the time to peak discharge, t_p

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

$$t = t_p = K(n-1) \qquad 4.39$$

Substituting the value of t_p of Equation 4.39 in Equation 4.38 and simplifying, we get

$$q_p = \left[\frac{1}{K\Gamma(n)}\right] e^{-(n-1)}(n-1)^{(n-1)} \qquad 4.40$$

From Equation 4.39 and Equation 4.40, we get

$$q_p * t_p = \left[\frac{(n-1)}{\Gamma(n)}\right] e^{-(n-1)}(n-1)^{(n-1)} \qquad 4.41$$

From GIUH formula,

$$q_p * t_p = 0.5764\left[\frac{R_B}{R_A}\right]^{0.55} * {R_L}^{0.05} \qquad 4.42$$

In order to relate the parameters of Nash model and that of the GIUH of Rodriguez-Iturbe, Equating Equation 4.41 with Equation 4.42 it could be written as,

$$\left[\frac{(n-1)}{\Gamma(n)}\right] e^{-(n-1)}(n-1)^{(n-1)} = 0.5764\left[\frac{R_B}{R_A}\right]^{0.55} * {R_L}^{0.05} \qquad 4.43$$

All the terms in the right hand side of equation 4.43 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,

$$n = 3.29\left(\frac{R_B}{R_A}\right)^{0.78} {R_L}^{0.07} \qquad 4.44$$

The relationship for obtaining the value of *K* for a given velocity V could mathematically be expressed as,

$$K = \frac{0.7}{V}\left(\frac{R_A}{R_B R_L}\right)^{0.48} * L_\Omega \qquad 4.45$$

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 4.38.

A unit hydrograph of desired duration (D) may be derived using the following equation:

$$U(D,t) = \frac{1}{D}\left(I\left(n,\frac{t}{k}\right) - I(n,(t-D)/K\right) \qquad 4.46$$

Where, U (D, t) denotes ordinates of UH of D-hour duration in hour^{-1}, t is the sampling time interval in hour, I (n, t/ k) is the incomplete γ function of order n at (t/ k) and D is the duration of UH in hour (Rakesh Kumar, et al., 2007).

4.8. 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) a 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 4.3), 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, and R_A.

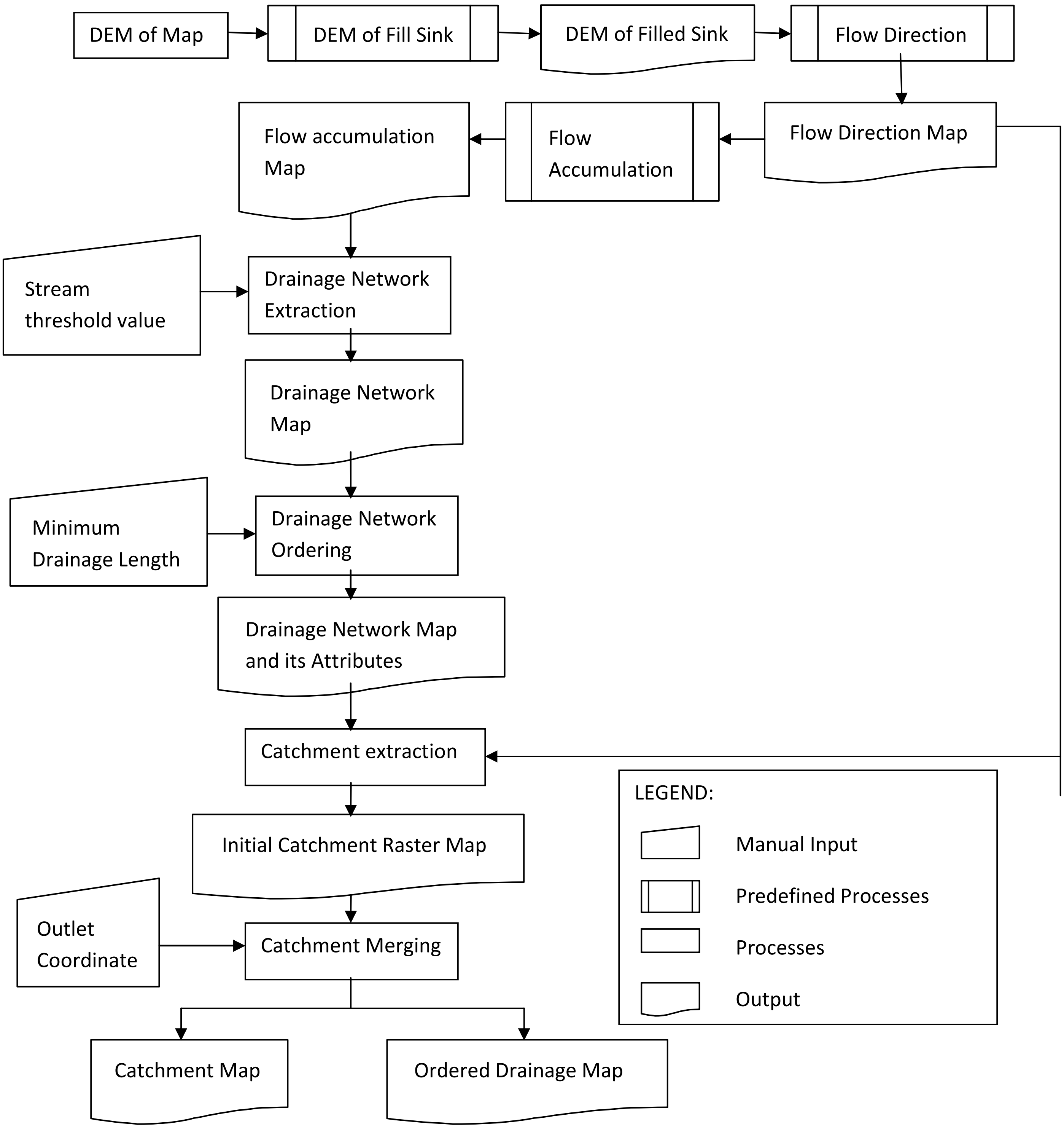


Figure 4.3: DEM processing algorithm of ILWS

4.9. Model Evaluation Criteria

The following objective functions were employed for evaluation of the unit hydrographs computed by the GIUH and GIUH based Nash models in comparison with the observed unit hydrographs.

(i) Efficiency (EFF);

$$EFF = \left(1 - \frac{\sum_{i=1}^{n}(Q_{oi}-Q_{ci})^2}{\sum_{i=1}^{n}(Q_{oi}-\bar{Q})^2}\right) * 100 \qquad 4.47$$

Where:

Q_{oi} : Ordinate of the observed discharge,

Q_{ci}: Ordinate of the computed discharge,

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

n : the number of ordinates.

(ii) Percentage error in peak (PEP),

$$PEP = \frac{(Q_{op}-Q_{cp})}{Q_{op}} * 100 \qquad 4.48$$

Where,

Q_{op} : Ordinate of observed peak discharge,

Q_{cp} : Ordinate of computed peak discharge.

(iii) Percentage error in time to peak (PETP).

$$PETP = \frac{(T_{op}-T_{cp})}{T_{op}} * 100 \qquad 4.49$$

Where,

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

T_{cp}: Time to peak of computed discharge.

(iv) Error Function, (Err)

$$Err = \left[\left(\frac{Q_{op} - Q_{cp}}{Q_{cp}} \right)^2 + \left(\frac{T_{op} - T_{cp}}{T_{cp}} \right)^2 \right] \qquad 4.50$$

Based on the results of these objective functions the predictive capabilities of the models were evaluated. Besides the objective function, the models were compared visually. A model will be accepted if its efficiency is greater than 80% and its error is small. The results of these comparisons found in Section 5.

5. RESULTS AND DISCUSSION

5.1. Unit Hydrograph Derivation

The rainfalls selected for this study were single peaked storms and the streamflow hydrographs were single peaked hydrographs. Therefore, the selected method for unit hydrograph derivation is direct or convolution method.

The necessary steps for unit hydrograph derivation from single peak storm are:

i. Select the single peaked hourly streamflow data and the corresponding hourly rainfall data.

ii. Separate the base-flow using straight line method from total streamflow hydrograph to produce direct surface runoff hydrograph.

iii. Calculate the volume and the depth of direct runoff using equation 4.1 and 4.2 respectively.

iv. Determine the loss rate using Φ-index method using equation 4.3 and estimate the excess rainfall depth.

v. Then, derive the unit hydrograph 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.

Table 5.1: Observed Stream flow data

Date	Time (hr)	Gauge Height(m)	Obs. Streamflow (m^3/s)
3/7/1985	6PM	1.36	15.82
	7.00	2.02	39.01
	8.00	2.10	42.18
	9.00	1.90	33.96
	10.00	1.74	27.81
	11.00	1.62	23.63
	12.00	1.55	21.36
4/7/1985	1AM	1.53	20.74
	2.00	1.53	20.61
	3.00	1.51	20.12
	4.00	1.50	19.91
	5.00	1.50	19.67
	6.00	1.48	19.07
	7.00	1.46	18.48
	8.00	1.44	17.96
	9.00	1.42	17.47
	10.00	1.41	17.05
	11.00	1.40	16.78
	12.00	1.39	16.50

Table 5.2: Observed Rainfall Data

Station: Finoteselam Region: Gojam
Element: Rainfall Intensity Month: July Year: 1985

Date/Hrs	1	2	3	4	5	6	7	8	9	10
0-1										
1-2										
2-3										
3-4										
4-5										
5-6										
6-7		3.5								
7-8										
8-9										
9-10		1.2								
10-11										
11-12										
12-13										
13-14		1.0								
14-15								1.2		
15-16								4.9	5.3	
16-17			1.8				5.5	1.9		

	1PM	1.38	16.25
	2.00	1.37	15.98
	3.00	1.36	15.82
	4.00	1.35	15.42
	5.00	1.33	15.03
	6.00	1.32	14.66
	7.00	1.31	14.38

17-18	12.4		4.0				0.9			
18-19			2.1	0.7						1.5
19-20										2.5
20-21				1.0						
21-22										
22-23					9.8					
23-24	2					3.5				

Table 5.3: Unit Hydrograph Computition

1	2	3	4	5 = 3-4	6 = 5/rd
Date	Time(hr)	Streamflow (m3/s)	Baseflow (m^3/s)	Direct Runoff (m^3/s)	Unit Hydrograph (m^3/s)
3/7/1985	6PM	15.82	15.82	0.00	0.00
	7.00	39.01	15.76	23.25	12.70
	8.00	42.18	15.70	26.48	14.47
	9.00	33.96	15.65	18.31	10.01
	10.00	27.81	15.59	12.22	6.68
	11.00	23.63	15.53	8.10	4.43
	12.00	21.36	15.47	5.89	3.22
4/7/1985	1AM	20.74	15.42	5.32	2.91
	2.00	20.61	15.36	5.25	2.87
	3.00	20.12	15.30	4.82	2.63
	4.00	19.91	15.24	4.67	2.55
	5.00	19.67	15.19	4.48	2.45
	6.00	19.07	15.13	3.94	2.15
	7.00	18.48	15.07	3.41	1.86
	8.00	17.96	15.01	2.95	1.61
	9.00	17.47	14.96	2.51	1.37
	10.00	17.05	14.90	2.15	1.18
	11.00	16.78	14.84	1.94	1.06
	12.00	16.50	14.78	1.72	0.94
	1PM	16.25	14.73	1.52	0.83
	2.00	15.98	14.67	1.31	0.72
	3.00	15.82	14.61	1.21	0.66
	4.00	15.42	14.55	0.87	0.47
	5.00	15.03	14.50	0.53	0.29
	6.00	14.66	14.44	0.22	0.12
	7.00	14.38	14.38	0.00	0.00
			Volume of DRO(m^3/s)	**143.07**	**78.18**
			Excess RF depth, rd(mm)	**1.83**	**1.00**

Observed Hourly rainfall data (refer table 5.2)

Rainfall	Value
P_1	1.80
P_2	4.00
P_3	2.10

Table 5.4: Φ-index Computation table

r_d	R_m	Δt	Φ	Result
1.83	4.00	1.00	2.17	ok
1.83	6.10	2.00	2.14	no
1.83	7.90	3.00	2.02	no

Therefore, the only rainfall that has an impact on runoff generation is p_2=4mm, the rest are losses. The excess rainfall is 1.83mm and the abstraction is 2.17mm.

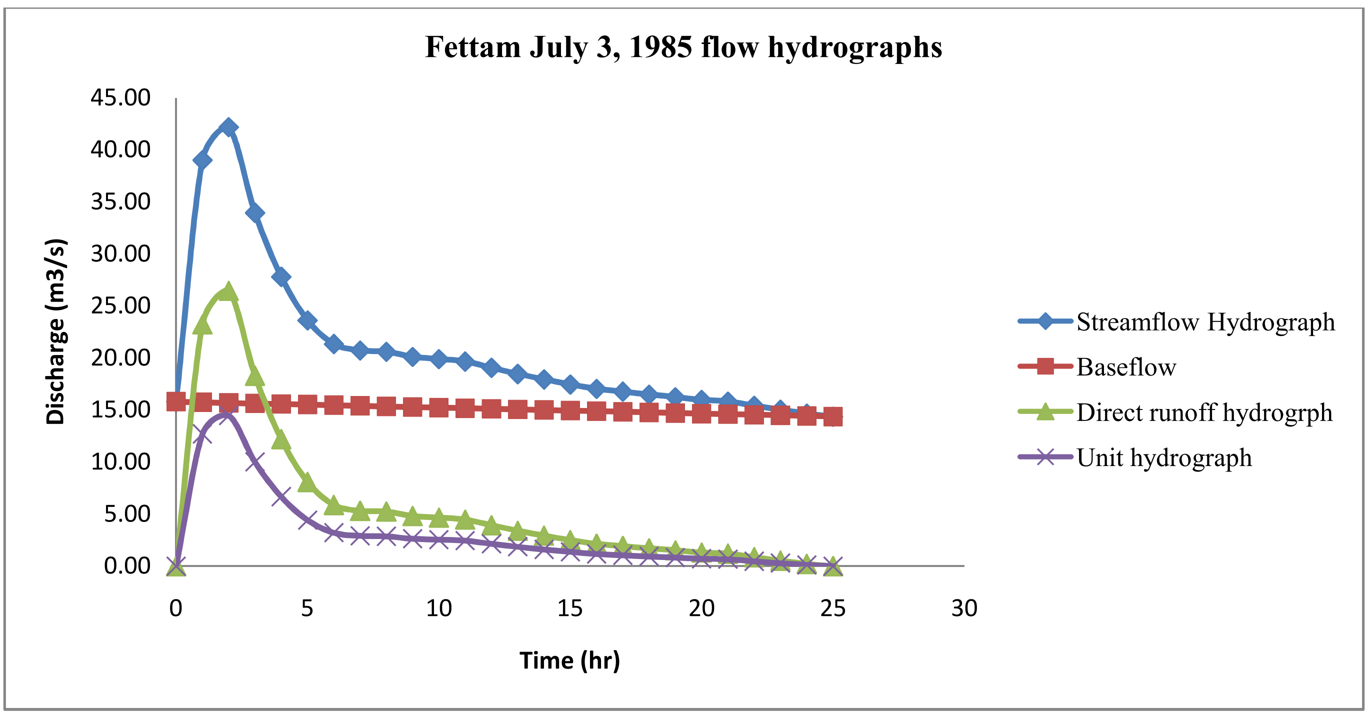
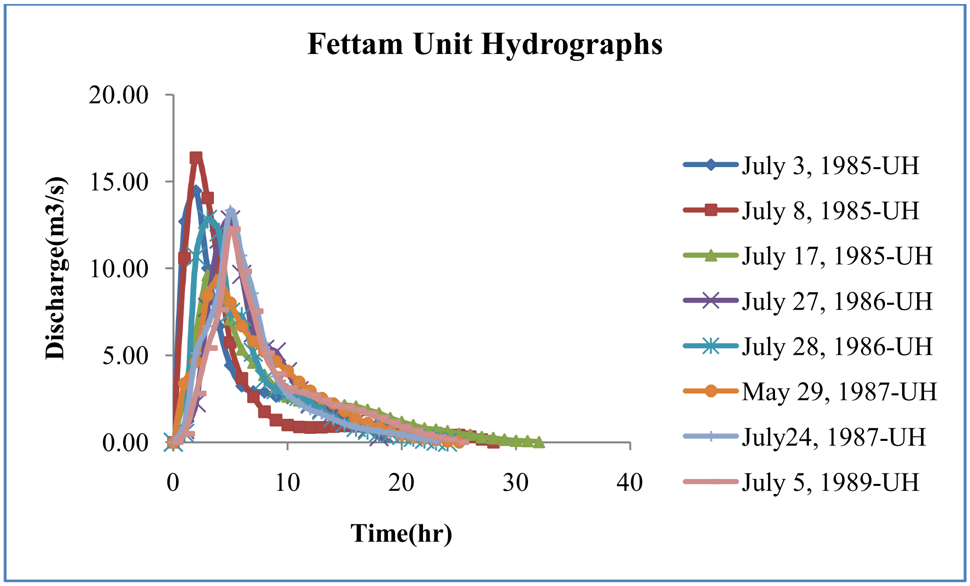


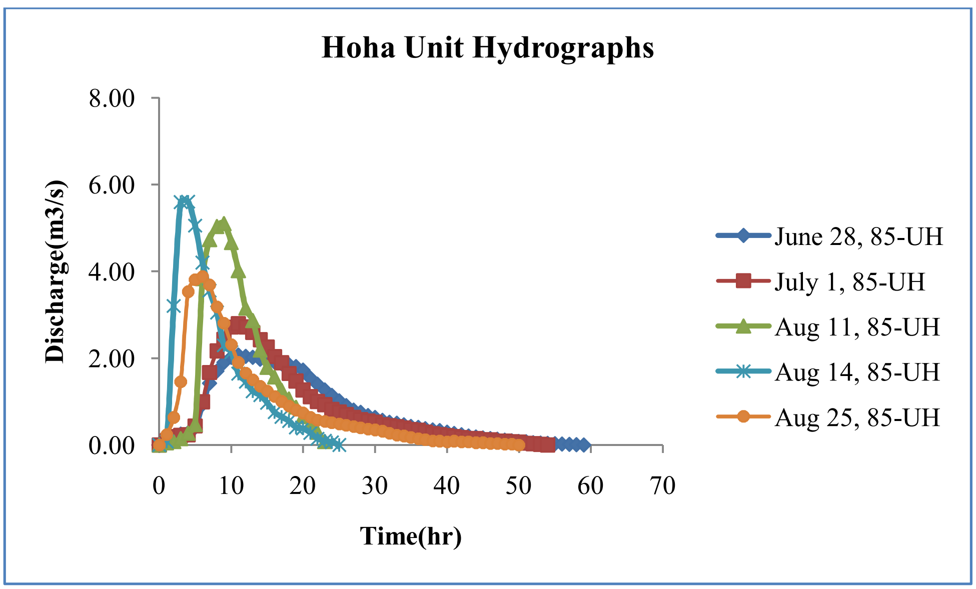
Figure 5.1: Fettam streamflow, direct runoff, base-flow and unit hydrographs

Similarly, the unit hydrographs were derived for all selected catchments using the above method and their results are attached in Appendix D

5.2. Catchment Average Unit Hydrograph

Using the procedures described in chapter 4, 1-hr unit hydrograph for all the selected concurrent rainfall-runoff events of all catchments were derived and shown in Figure 5.2.





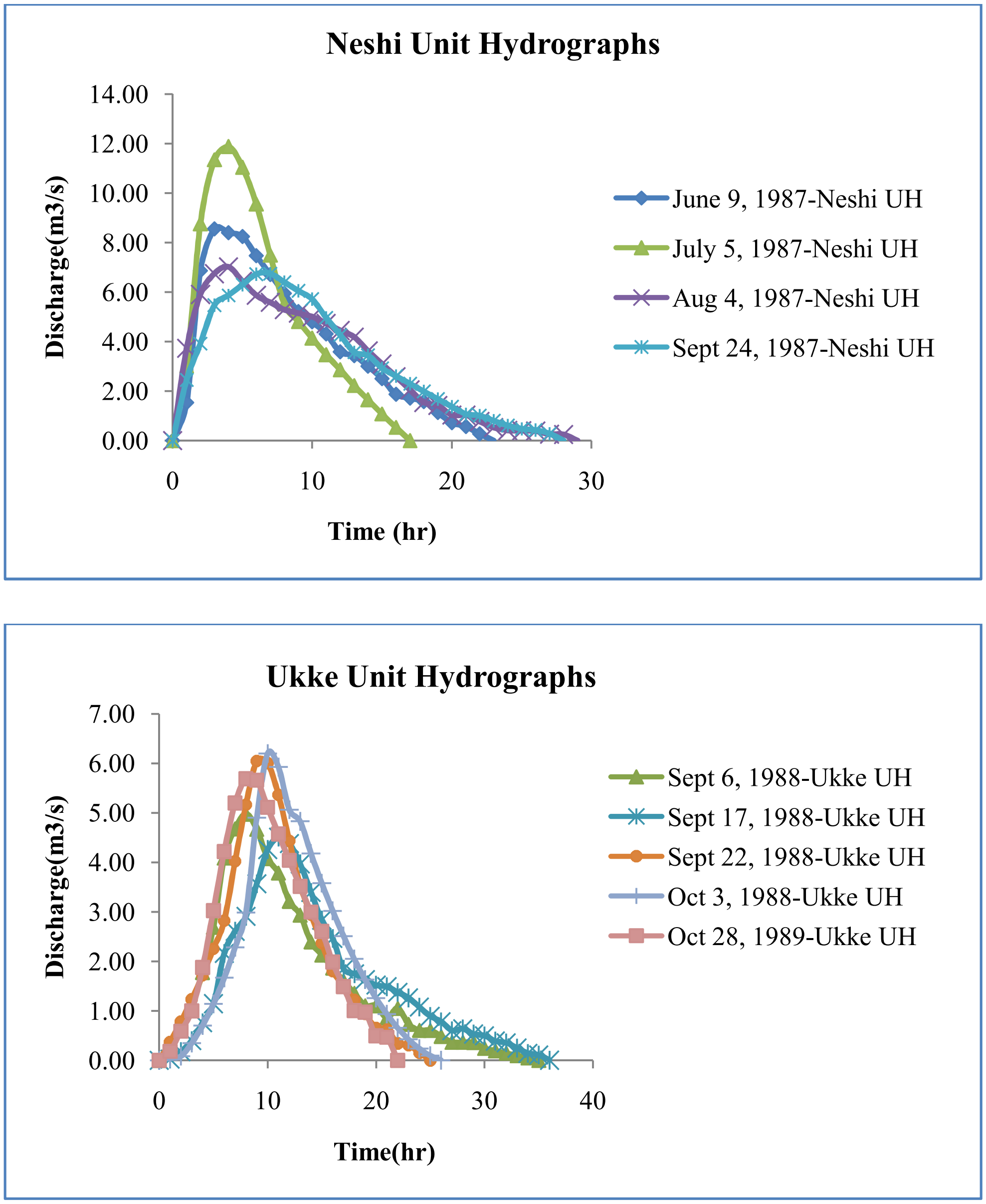


Figure 5.2: 1-hr unit hydrographs of the selected catchments.

5.2.1. Derivation of an Average Unit Hydrographs

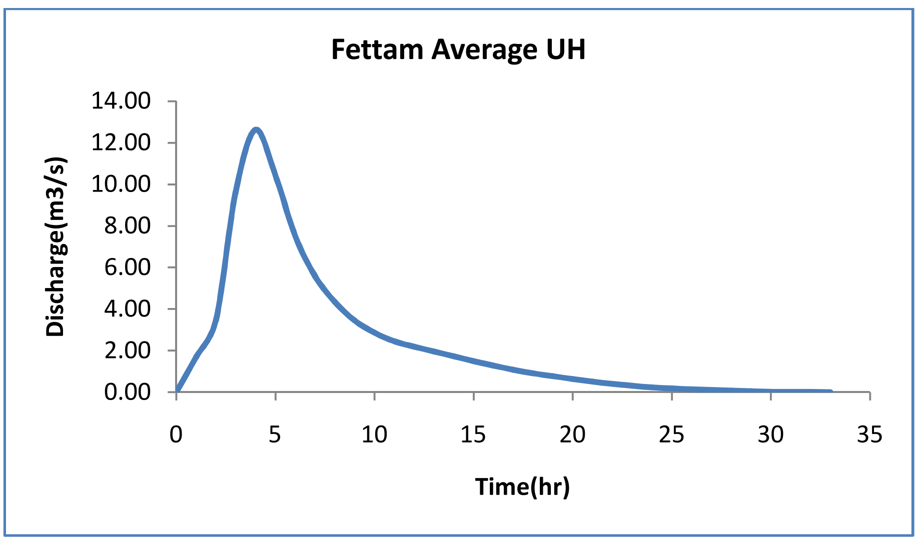
In Practice, unit hydrographs derived from different rainfall-runoff events of a particular catchment differ significantly from one another. These unit hydrographs are averaged for computing a basin representative average unit hydrograph (Ojha etal. 2008). The graphical method (Wilson, 1974) of deriving an average UH can be applied by the following steps:

i. Plot the unit hydrographs derived from a number of individual events on a single plotting paper.

ii. Mark a point such that it corresponds to the average peak discharge ordinates and average time to peak.

iii. Draw an average unit hydrograph such that it passes through the average peak point, has unit volume, and generally conforms to the characteristics shapes of the individual unit hydrographs. Some trials for adjusting the ordinates of the derived unit hydrograph may be required for preserving the volume.

Using the above approach, the average unit hydrograph were obtained for all catchments used in this study, as shown in Figure 5.3. The ordinates of the observed unit hydrographs are attached in Appendix D.



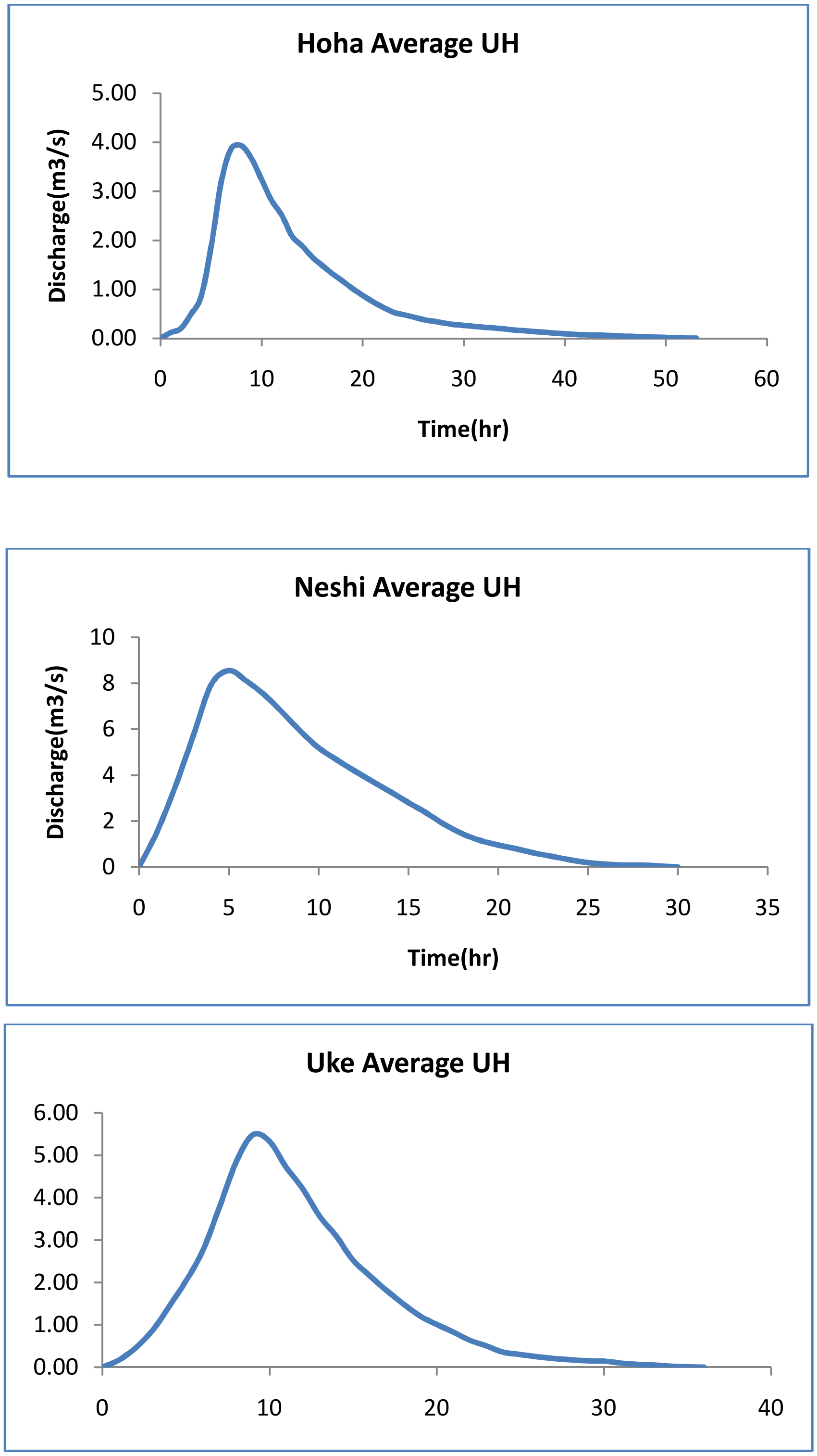


Figure 5.3: Average unit hydrographs

5.3. Geomorphologic Characteristics of the Catchments

By DEM processing and integrating in ILWIS the catchment characteristics were extracted and stored in output table.

The Horton Statistics operation calculated 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 Mathius (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 outputs and the Horton plots are shown in table 5.5 and figure 5.5 respectively.

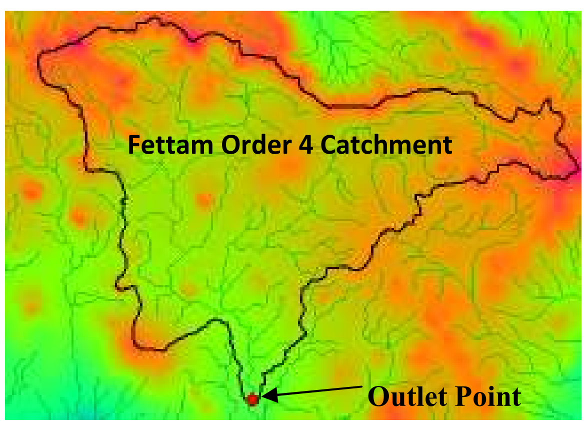


Figure 5.4: ILWIS extracted Fettam catchment with their drainage networks (the rest is attached in Appendix A)

Table 5.5: Horton Statistics of the Catchments

SNo.	Catchments	Stream Order	Number of Streams	Mean Stream Length (Km)	Mean Area (Km^2)	Horton's Ratios		
						R_B	R_L	R_A
		1	47	2.35	2.67			
		2	8	6.51	19.88			
		3	2	20.07	94.55			
1	Fettam	4	1	26.15	195.08	3.65	2.31	4.23
		1	34	2.41	3.05			
		2	9	5.23	14.76			
		3	2	16.16	81.68			
2	Hoha	4	1	17.19	165.85	3.35	2.02	3.93
		1	43	3.04	4.27			
		2	9	7.24	26.95			
		3	2	25.79	151.45			
3	Neshi	4	1	36.86	327.71	3.59	2.4	4.37
		1	35	3.03	3.12			
		2	11	5.85	12.82			
		3	2	17.37	86.23			
4	Uke	4	1	36.08	202.36	3.45	2.34	4.23
		1	41	2.42	2.81			
		2	8	7.58	20.44			
		3	4	10.85	44.24			
5	Azuari	4	1	23.28	196.03	3.27	2.04	3.86
		1	44	5.16	8.87			
		2	9	12.47	56.55			
		3	3	19.84	181.21			
6	Andassa	4	1	56.91	603.69	3.47	2.15	3.99
		1	36	3.30	4.01			
		2	6	14.40	44.59			
		3	2	21.76	145.60			
7	Jedeb	4	1	24.85	297.39	3.27	1.91	4.10
		1	33	3.40	4.72			
		2	7	9.38	34.45			
		3	2	33.73	169.32			
8	Chemoga	4	1	44.68	364.41	3.24	2.46	4.32

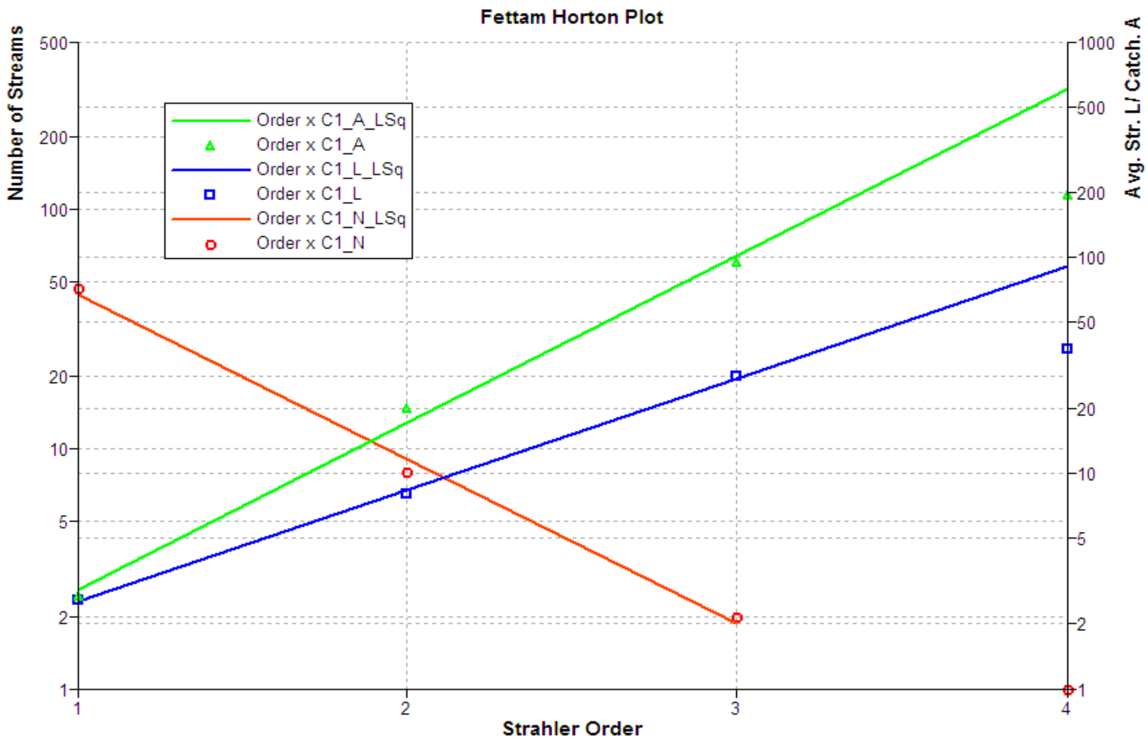


Figure 5.5: Horton plot of Fettam catchment (the rest is attached in Appendix B)

In the Horton plot the meaning of the legends are:

- C1_N is the number of streams,
- C1_Lis the average stream length (km),
- C1_A is the average area of catchments (km^2),
- C1_N_LSq is the expected values for C1_N by means of a least squares fit through C1_N,
- C1_L_LSq is the expected values for C1_L by means of a least squares fit through C1_L,
- C1_A_LSq is the expected values for C1_A by means of a least squares fit through C1_A.

Table 5.6: ILWIS based catchments result

Catch. Name	Catch. Area (km^2)	Total Drainage Length (m)	Drainage Density (m/km^2)	Longest Flow Path Length(m)	LFP U/S Elevation (m)	Longest Drainage Length(m)	LDP U/S Elevation(m)	Outlet Elevation(m)	Slope (%)
Fettam	194.51	177139.1	910.71	32885.31	2860	31987.1	2721	2430	1.31
Hoha	164.48	130223.1	791.72	21316.16	1708	20551.5	1667	1433	1.29
Neshi	326.30	216687.1	664.07	53240.42	2711	52210	2634	2212	0.94
Uke	201.26	178940.5	889.11	40068.87	2218	38771.4	2154	1370	2.12
Azuari	195.79	165990.10	847.82	31334.74	3107	29727.20	2966	1739	4.37
Andassa	602.07	352123.60	585.00	68708.04	3066	67475.70	2839	1717	1.96
Jedeb	295.83	203287.7	687.00	48378.03	3906	47309.60	3780	2176	3.58
Chemoga	361.05	181430.80	591.00	50521.54	3862	49522.60	3727	2342	3.01

Where: LFP is Longest Flow Path; and LDP is Longest Drainage Path.

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. R_B, R_L and R_A lies between 3 to 5, 1.5 to 3.5 and 3 to 6 respectively.

5.4. Model Development

5.4.1. GIUH Model

Since the method to derive the unit hydrographs is described in chapter 4, this section provides the results of every step.

The important steps one can be followed are:

1. Calculate Horton statistics R_B, R_L, and R_A by DEM processing and integrating in ILWIS(results are shown in table 5.5)
2. Estimate the hillslope/channel velocity and calculate coefficient of the exponential density function α, β_1, β_2, and β_3.
3. Compute the initial state probabilities, the transition probabilities and then the probability at a given path S_i. (Refer table 5.7, 5.8 and 5.9)
4. Compute the probability density function of travel time at each paths $f_l(S_1)$, $f_l(S_2)$, $f_l(S_3)$, $f_l(S_4)$, … are calculated in time steps using Excel spreadsheet.
5. Convolute the GIUH and derive the UH using 1mm of excess rainfall

Table 5.7: Initial State Probabilities

Catchments	Initial State Probabilities Values			
	θ_1	θ_2	θ_3	θ_4
Fettam	0.64	0.24	0.13	-0.01
Hoha	0.62	0.23	0.12	0.03
Neshi	0.55	0.24	0.16	0.05
Uke	0.54	0.23	0.16	0.07
Azuari	0.61	0.22	0.13	0.04
Andassa	0.66	0.23	0.12	-0.01
Jedeb	0.48	0.22	0.17	0.13

Table 5.8: Transition State Probabilities

Catchments	Transition State Probabilities Values					
	P_{12}	P_{13}	P_{14}	P_{23}	P_{24}	P_{34}
Fettam	0.78	0.20	0.09	0.81	0.19	1
Hoha	0.81	0.2 0	0.08	0.83	0.17	1
Neshi	0.79	0.20	0.09	0.81	0.19	1
Uke	0.80	0.20	0.08	0.83	0.17	1
Azuari	0.82	0.19	0.08	0.84	0.16	1
Andassa	0.80	0.20	0.08	0.82	0.18	1
Jedeb	0.82	0.19	0.07	0.85	0.15	1

Table 5.9: Path Probabilities Prob(S_i)

Catchments	Path Probabilities Values							
	Prob(S_1)	Prob(S_2)	Prob(S_3)	Prob(S_4)	Prob(S_5)	Prob(S_6)	Prob(S_7)	Prob(S_8)
Fettam	0.41	0.10	0.13	0.06	0.20	0.05	0.13	0.01
Hoha	0.42	0.08	0.12	0.05	0.19	0.04	0.12	0.03
Neshi	0.36	0.08	0.11	0.05	0.19	0.04	0.16	0.05
Uke	0.36	0.08	0.11	0.05	0.19	0.04	0.16	0.07
Azuari	0.42	0.08	0.12	0.05	0.19	0.04	0.13	0.04
Andassa	0.43	0.09	0.13	0.06	0.19	0.04	0.12	-0.01
Jedeb	0.33	0.06	0.09	0.04	0.19	0.03	0.17	0.13

Table 5.10: Values of travel time parameters

Catchments	α	λ_1	λ_2	λ_3	λ_4
Fettam	0.590	6.311	2.732	1.183	0.512
Hoha	0.513	1.539	0.762	0.377	0.187
Neshi	0.430	3.280	1.367	0.570	0.237
Uke	0.576	2.246	0.960	0.410	0.175
Azuari	0.451	2.410	1.181	0.579	0.284
Andassa	0.379	3.195	1.486	0.691	0.322
Jedeb	0.445	3.877	2.030	1.063	0.556

5.4.2. GIUH Based Nash Model

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 4.44.

5.5. Model Sensitivity Analysis

The most sensitive model parameters identified in literature are the velocity of the hillslope and stream flow (Al-Wagdany and Rao, 1998; Kirshen and Bras, 1983). Therefore, in this application, the Horton's ratios were also kept constant during calibration. The hillslope velocity and stream velocity were calibrated manually, simultaneously based on manual calibration.

5.6. Model Calibration

Except the dynamic parameter *V,* the channel velocity and the hillslope velocities, all the parameters of the GIUH and the GIUH based Nash models can be estimated from the geomorphological characteristics of the catchments. 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 with minimum error.

In this study, the error function defined by Lee (1972) (Equation 4.50) 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. The optimized results of the model are shown in table 5.11 below.

Table 5.11: Calibrated velocity Parameter of GIUH Model

Catchment Name	Hillslope velocity, v_o (m/s)	Channel Velocity, V(m/s)
Fettam	0.09	4.12
Hoha	0.09	1.03
Neshi	0.09	2.77
Uke	0.09	1.89
Azuari	0.09	1.62
Andassa	0.09	4.58
Jedeb	0.09	2.10

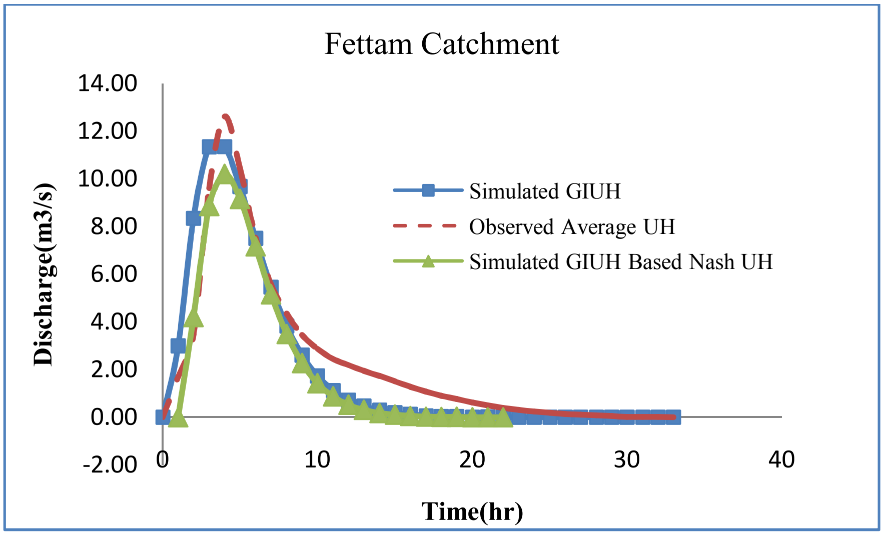
Table 5.12: Calibrated Velocity Parameters of GIUH based Nash Model

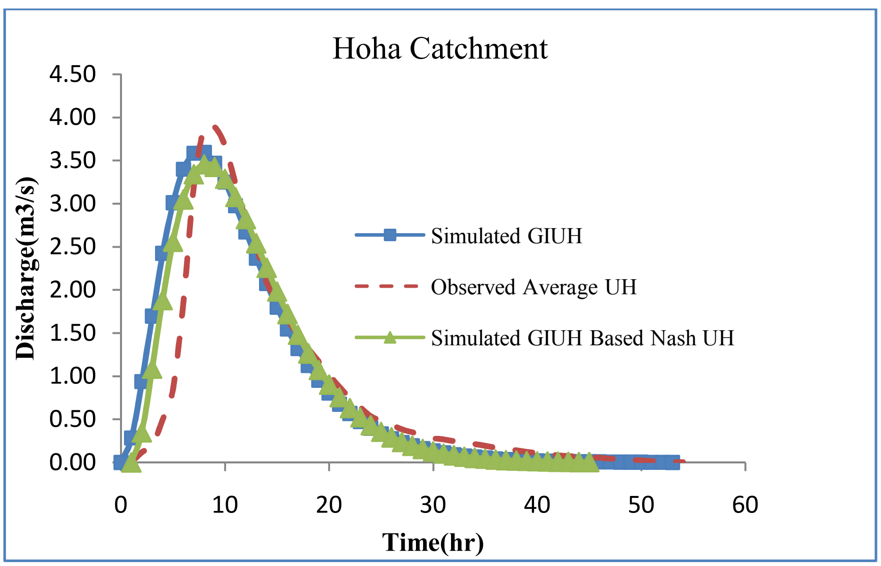
Catchment Name	Channel Velocity, V(m/s)
Fettam	9.9
Hoha	3.1
Neshi	7.9
Uke	5.6
Azuari	4.1
Andassa	9.8
Jedeb	4.3

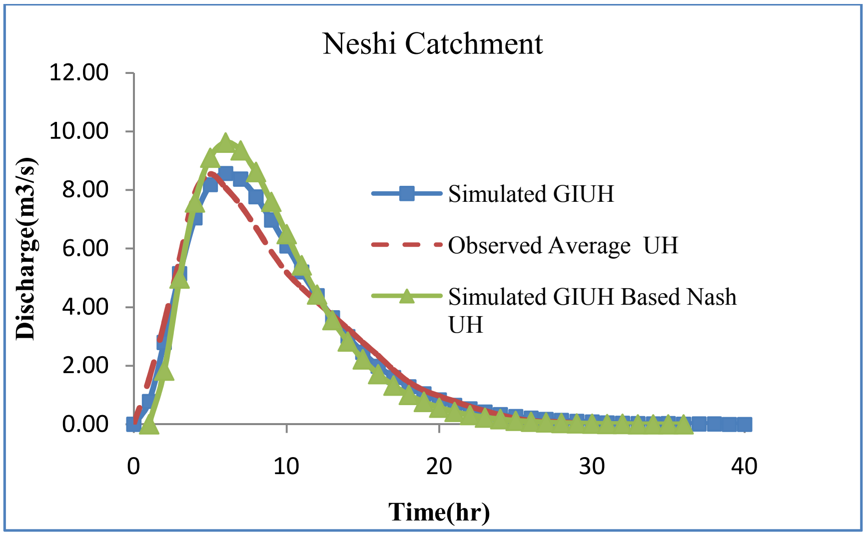
Table 5.13: Derived Nash parameters after calibration

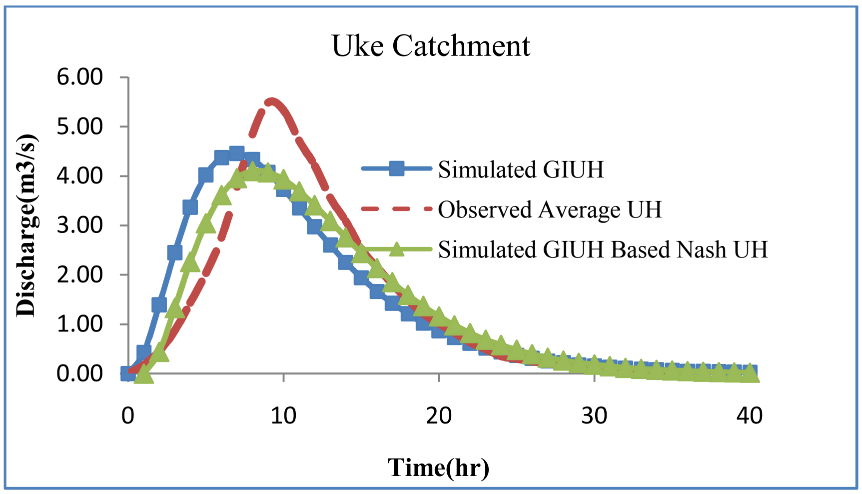
Catchment Name	Shape Parameter (n)	Scale Parameter K(hr)
Fettam	3.11	1.47
Hoha	3.05	3.45
Neshi	3.00	2.69
Uke	2.98	3.56
Azuari	3.04	2.70
Andassa	3.11	2.71
Jedeb	2.84	1.82

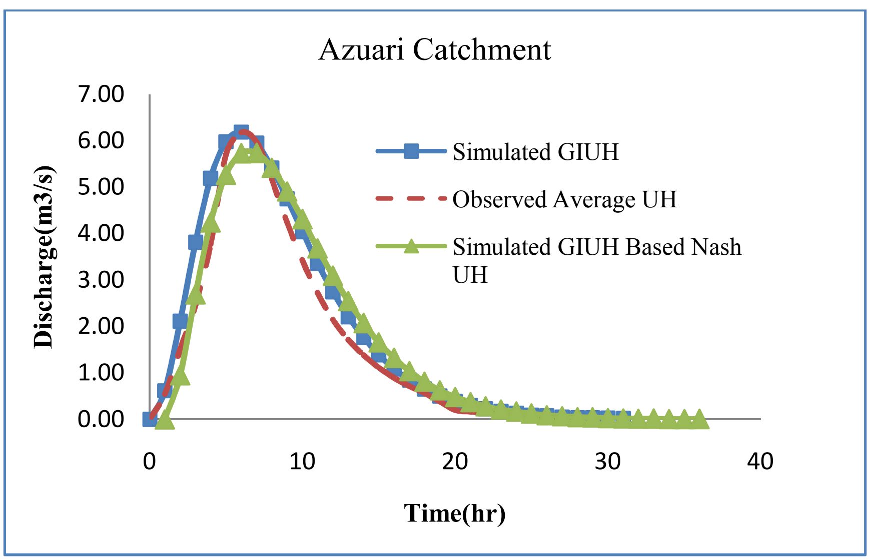
Using the calibrated (optimum) velocity of the GIUH and GIUH based Nash model, the observed and the simulated unit hydrographs of the catchments are presented in Figure 5.4 below.

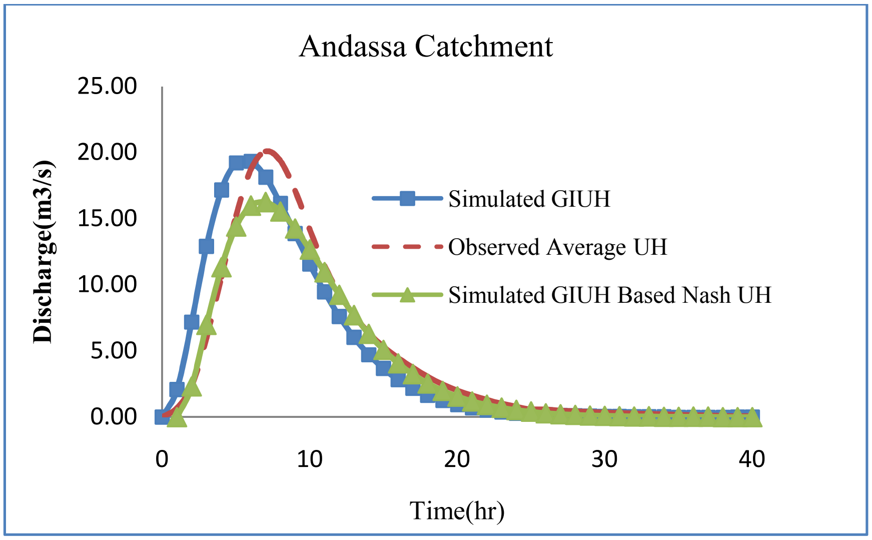





Neshi Catchment
12.00
10.00
8.00
6.00
4.00
2.00
0.00
Discharge(m3/s)
0
10
20
30
40
Time(hr)
Simulated GIUH
Observed Average UH
Simulated GIUH Based Nash UH


Uke Catchment
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Discharge(m3/s)
0
10
20
30
40
Time(hr)
Simulated GIUH
Observed Average UH
Simulated GIUH Based Nash UH


Azuari Catchment
7.00
6.00
5.00
4.00
3.00
2.00
1.00
0.00
Discharge(m3/s)
0
10
20
30
40
Time(hr)
Simulated GIUH
Observed Average UH
Simulated GIUH Based Nash UH



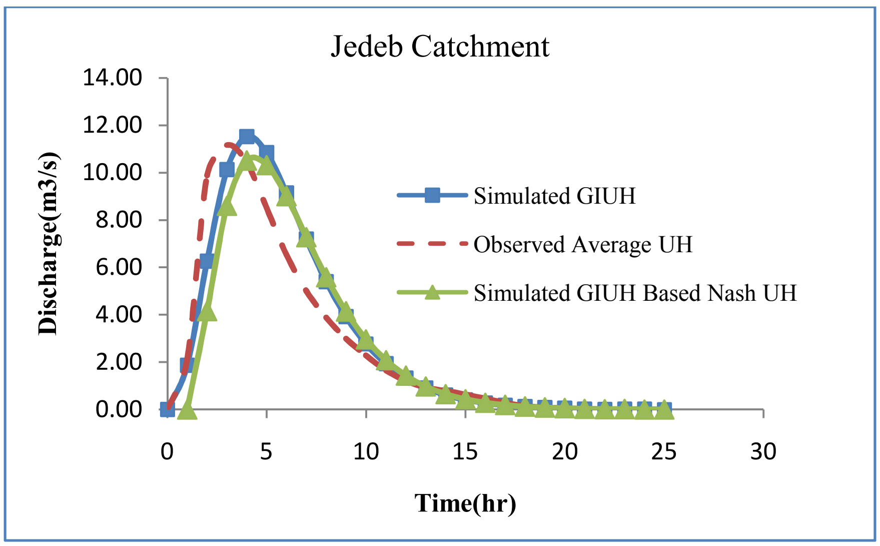


Figure 5.6: Observed and Simulated unit hydrographs using GIUH and GIUH based Nash model

The ordinates of the simulated unit hydrographs are attached in Appendix D.

Table 5.14: Peak discharge and time to peak of observed and simulated unit hydrographs for various catchments

Catchments	**Observed**		**GIUH**		**GIUH (Nash)**	
	Q_{op}(m^3/s)	**T_{op}(hr)**	**Q_{cp}(m^3/s)**	**T_{cp}(hr)**	**Q_{cp}(m^3/s)**	**T_{cp}(hr)**
Fettam	12.62	4	11.36	4	9.70	4
Hoha	3.89	8	3.59	8	3.54	7
Neshi	8.55	5	8.58	6	9.12	5
Uke	5.48	9	4.45	7	4.27	8
Azuari	6.18	6	6.19	6	5.76	6
Andassa	20.10	7	19.36	6	16.27	6
Jedeb	11.17	3	11.52	4	12.66	3
R^2 Value			**0.988**	**0.54**	**0.884**	**0.946**

From the above table we can conclude that GIUH model is efficient in peak discharge estimation than GIUH based Nash model, whereas GIUH based Nash model is more efficient than GIUH model in time to peak estimation.

5.6.1. Objective functions used for evaluation of the computed unit hydrographs

The following objective functions were employed for evaluation of the unit hydrographs computed by the GIUH and GIUH based Nash models in comparison with the observed unit hydrographs: (i) Efficiency (EFF); (ii) Percentage Error in Peak (PEP); (iii) Percentage Error in Time to Peak (PETP) and (iv) Error Function (ERR). The formulas of the objective functions were discussed in chapter four, here only their results are displayed.

Table 5.15: Objective functions result for observed and simulated unit hydrographs

Methods	**Error Functions**			
	EFF(%)	**PEP(%)**	**PETP(%)**	**ERR(%)**
Fettam Catchment				
GIUH	86.38	10.00	0.00	1.20
GIUH (Nash)	75.68	23.14	0.00	10.92
Hoha Catchment				
GIUH	90.14	7.60	0.00	0.70
GIUH (Nash)	85.87	9.00	12.50	2.65
Neshi Catchment				
GIUH	97.00	-0.30	-20.00	2.80
GIUH (Nash)	93.38	-6.67	0.00	0.89

Ukke Catchment				
GIUH	81.95	18.80	22.00	13.5
GIUH (Nash)	84.73	22.08	11.11	15.72
Azuari Catchment				
GIUH	95.34	-0.20	0.00	0.00
GIUH (Nash)	94.43	6.80	0.00	1.32
Andassa Catchment				
GIUH	85.05	3.70	14.30	2.90
GIUH (Nash)	89.79	19.05	14.29	10.44
Jedeb Catchment				
GIUH	88.53	-3.17	-33.30	6.30
GIUH (Nash)	97.53	13.34	0 .00	2.55

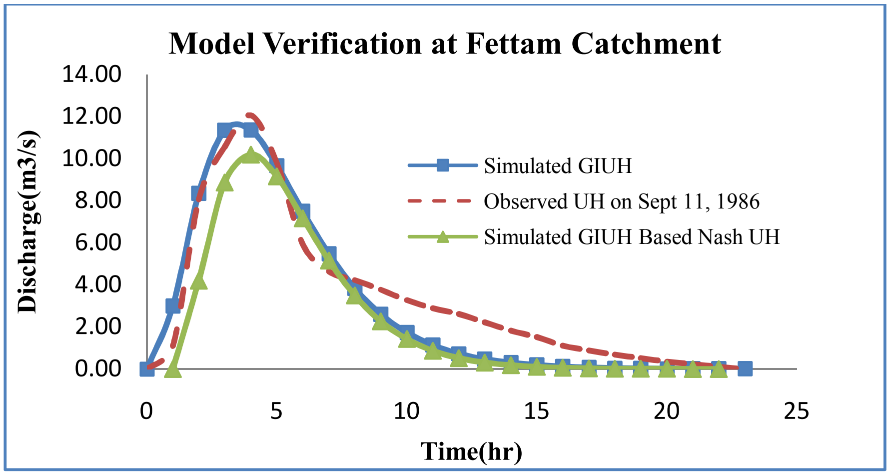
5.6.2. Comparison of objective functions used for evaluation of the computed unit hydrographs

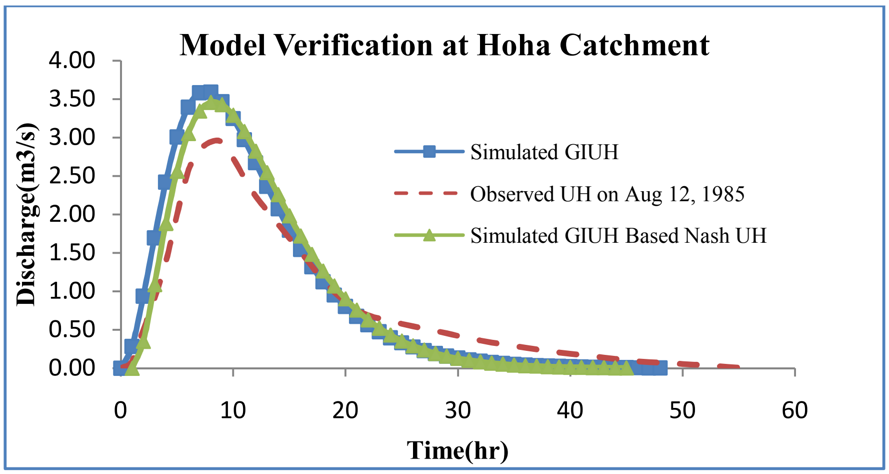
The objective functions based on comparison of the observed and computed unit hydrographs for the selected two methods, presented in Table 5.15, shows that the values of EFF for GIUH method are higher than the values of GIUH based Nash methods except for Uke, Andassa and Jedeb catchments. The values of PEP of GIUH method are smaller for all catchments than GIUH based Nash method. When we saw the overall error function, GIUH model has minimum error than GIUH based Nash model except for Neshi and Jedeb Catchments this is due to high error in time to peak.

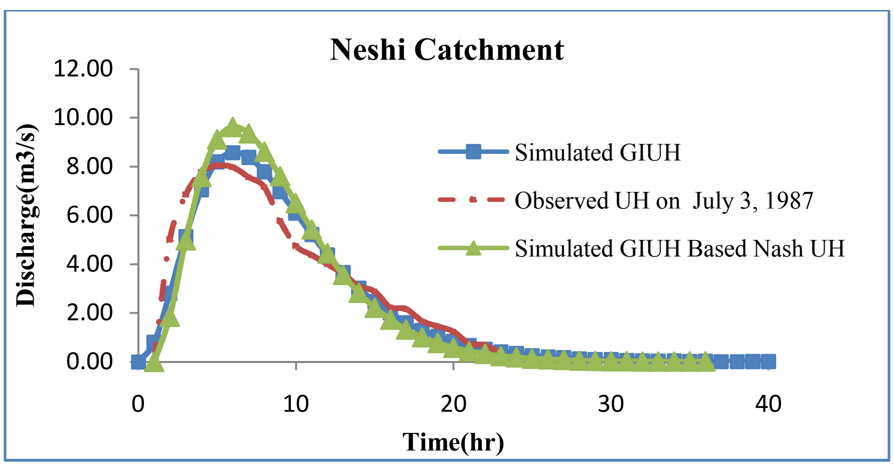
Therefore, from the above result, we can conclude that GIUH model has higher efficiency with minimum error than GIUH based Nash model.

5.7. 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. In this study one unit hydrograph was remained from averaging from each catchment for verification purpose. Figure 5.7 shows the comparison of the observed and predicted unit hydrographs of GIUH and GIUH based Nash model.


Model Verification at Fettam Catchment
Discharge(m3/s)
Time(hr)
Simulated GIUH
Observed UH on Sept 11, 1986
Simulated GIUH Based Nash UH
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
5
10
15
20
25


Model Verification at Hoha Catchment
Discharge(m3/s)
Time(hr)
Simulated GIUH
Observed UH on Aug 12, 1985
Simulated GIUH Based Nash UH
4.00
3.50
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0
10
20
30
40
50
60


Neshi Catchment
Discharge(m3/s)
Time(hr)
Simulated GIUH
Observed UH on July 3, 1987
Simulated GIUH Based Nash UH
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
10
20
30
40

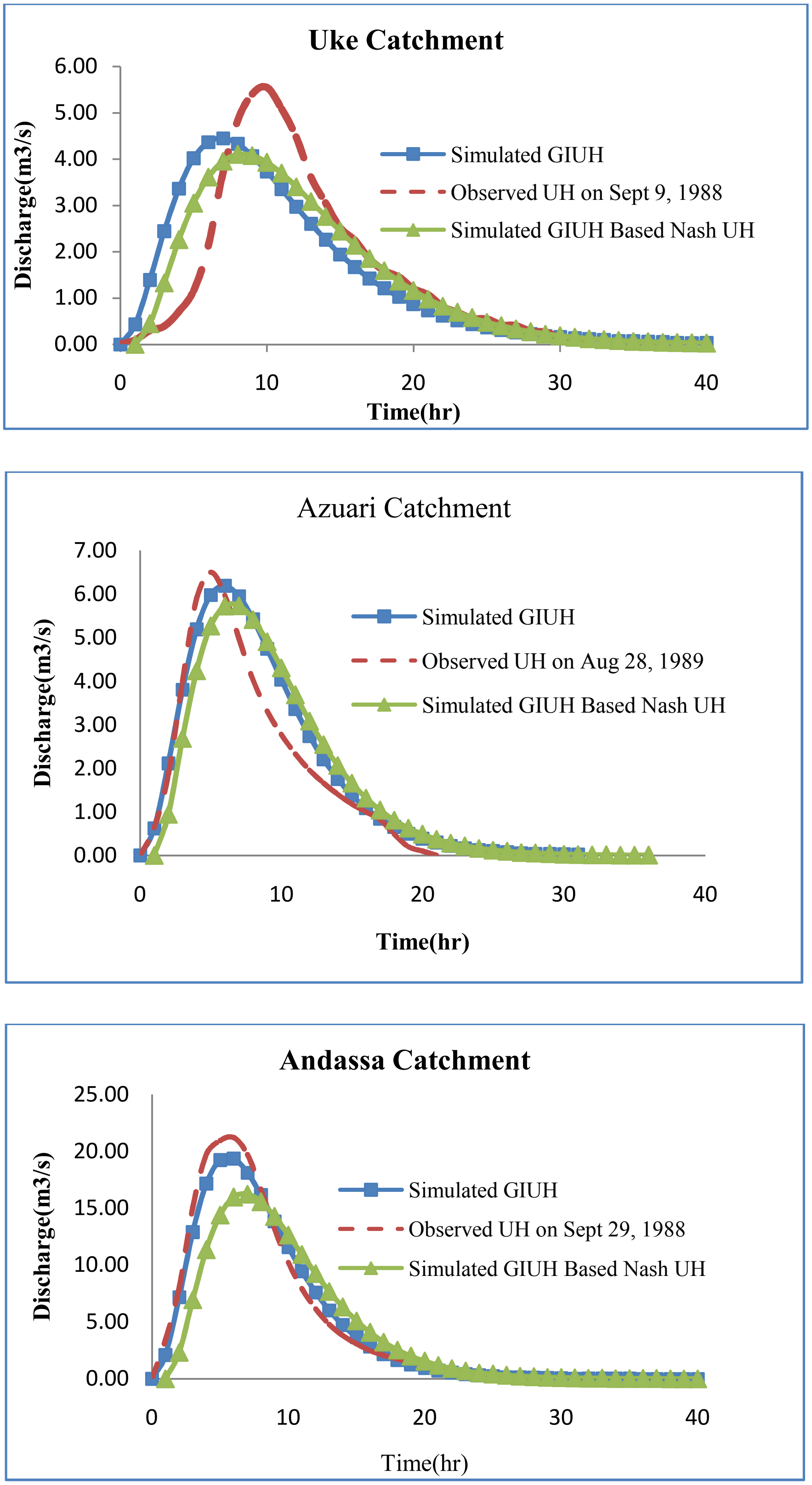


Figure 5.7: Verification of calibrated GIUH and GIUH based Nash model

Table 5.16: Error function results for model verification

Methods	Error Functions			
	EFF(%)	PEP(%)	PETP(%)	ERR(%)
Fettam Catchment				
GIUH	90.36	5.80	0.00	0.40
GIUH (Nash)	81.8	19.70	25.00	17.10
Hoha Catchment				
GIUH	84.12	-22.20	0.00	3.30
GIUH (Nash)	80.57	-20.40	12.50	4.90
Neshi Catchment				
GIUH	86.45	-6.60	-50.00	11.50
GIUH (Nash)	88.63	-12.90	-25.00	5.30
Ukke Catchment				
GIUH	93.00	10.50	12.50	3.40
GIUH (Nash)	93.17	23.30	0.00	9.20
Azuari Catchment				
GIUH	92.70	4.80	-20.00	3.00
GIUH (Nash)	96.15	11.8	0.00	1.80
Andassa Catchment				
GIUH	97.90	8.80	0.00	0.90
GIUH (Nash)	91.88	23.50	0.00	9.40

From the above table, despite the fact that, both models result have not shown great variation in model efficiency, GIUH model has smaller PEP and minimum ERR than GIUH based Nash model. Therefore, GIUH model is the preferable model. But when we see model simplicity GIUH based Nash model is very simple.

5.8. Relating the Velocity Parameter with the Catchment Characteristics

In the study area, there eight order four catchments. All order four catchments, except Hoha, are found very closely. Seven order four catchments were selected for regional equation derivation. Among the eight catchments, chemoga was left from regional equation derivation analysis by considering it as ungauged catchments. At the end, it used for validation test.

During regional equation derivation, the velocity parameter of GIUH model were related to the catchment characteristics of the selected study areas to develop a regional equation, using regression analysis, in order to give a solution to predict a surface runoff in ungauged catchments.

Table 5.17: Geomorphological characteristics of the selected catchments

Catch. Name	Area(km^2)	Longest Flow Path Length(Km), L	Slope, S	R_B	R_L	R_A	Vo	Vs
Fettam	194.51	32.89	0.01	3.65	2.31	4.23	0.09	4.12
Hoha	164.48	21.32	0.01	3.35	2.02	3.93	0.09	1.03
Neshi	326.30	53.24	0.01	3.59	2.40	4.37	0.09	2.77
Uke	201.26	40.07	0.02	3.45	2.34	4.23	0.09	1.89
Azuari	195.79	31.33	0.04	3.27	2.04	3.86	0.09	1.62
Andassa	602.07	68.71	0.02	3.47	2.15	3.99	0.09	4.58
Jedeb	295.83	48.38	0.04	3.27	1.91	4.10	0.09	2.10
Chemoga	361.05	50.52	0.03	3.24	2.46	4.32		

5.8.1. Selection of predictors (Sensitivity Analysis)

The different catchment characteristics considered for regression analysis were area (A), length of the longest flow path (L), average slope of the catchments (S), bifurcation ratio (R_B), length ratio (RL) and area ratio (R_A) as described in Table 5.13. 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.

SPSS(Statistical Package for Social Science) software were used to see the correlation results as shown in table 5.14.

Table5.18: Correlations of the catchment characteristics

Correlations							
		Vs	Area(km2)	Longest Flow Path Length(Km), L	Slope, S	RB	RL
Pearson Correlation	Vs	1.000	0.622	0.591	-0.361	0.674	0.233

As we can see in table 5.18 there is no a special catchment characteristics that predict the stream velocity highly. Therefore, all catchment characteristics were selected for 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). For this study, multiple linear regression method gave the best results than the others.

Multiple regression formula can be expressed as:

$$V_s = B + \alpha_1 X_1 + \alpha_2 X_2 + \alpha_3 X_3 + \cdots + \alpha_n X_n \qquad 5.1$$

Where:

Vs = Stream velocity,

B = constant value,

$\alpha_1, \alpha_2, \alpha_3 \ldots \alpha_n$, are coefficients of the respected catchment characteristics,

$X_1, X_2, X_3, \ldots Xn$, are values of the catchment characteristics.

5.8.3. SPSS Software Analysis

SPSS statistics 17.0 software was used to enter all the catchment characteristics and run stepwise regression procedures for selection of best multiple regression models. 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. Moreover, the variable with smallest correlation value would be removed through the analysis and also be out of regression equation. The SPSS software analysis results are attached in Appendix F.

SUMMARY OUTPUT	
Regression Statistics	
Multiple R	0.898
R Square	0.807
Adjusted R Square	0.730

Coefficients	
Intercept	-16.853
A	0.005
R_B	5.233

Therefore, the derived multiple linear regression formula for two variables is:

$Vs = -16.853 + 0.005A + 5.233R_B$ 5.2

Table 5.19: Summary of Residuals

Case Number	Observed Vs	Predicted Value	Residual
1	4.120	3.22	0.90
2	1.030	1.50	-0.47
3	2.770	3.56	-0.79
4	1.890	2.21	-0.32
5	1.620	1.24	0.38
6	4.580	4.32	0.26
7	2.100	1.74	0.36

Using Excel Regression

1. ***Combination of two variables***

The number of combination of n things taken k at a time is:

$$\binom{n}{k} = \frac{n!}{k!(n-k)!} \quad 5.3$$

The number of ways to choose 2 variables from the set of 6 variables is:

$$\binom{6}{2} = \frac{6!}{2!\,(6-2)!} = 15$$

There are 15 two variable combinations those are used to see their sensitivity with velocity.

Table 5.20: Sorted output with their largest adjusted R^2 value

Sno.	Relations	Multiple R	R Square	Adjusted R Square
1	A.RB	0.90	0.81	0.73
2	L.RB	0.85	0.73	0.62
3	S.RB	0.73	0.53	0.34
4	RL.RA	0.23	0.05	0.32
5	A.S	0.71	0.50	0.30
6	L.S	0.69	0.47	0.26
7	RB.RA	0.68	0.47	0.25
8	RB.RL	0.68	0.46	0.24
9	S.RA	0.36	0.13	0.22
10	S.RL	0.37	0.14	0.21
11	A.RL	0.64	0.41	0.18
12	A.RA	0.64	0.41	0.17
13	A.L	0.62	0.39	0.14
14	L.RL	0.60	0.36	0.10
15	L.RA	0.59	0.35	0.09

From the above table the relation between A and R_B gave the largest adjusted R^2 value, which means velocity is more sensitive to these variables. The linear regression coefficients are:

Coefficients	
Intercept	-16.853
A	0.005
R_B	5.233

General formula

$$Vs = -16.853 + 0.005A + 5.233R_B \quad 5.4$$

Summary of Residuals

Observation	Observed Vs	Predicted Vs	Residuals
1	4.12	3.22	0.90
2	1.03	1.50	-0.47
3	2.77	3.56	-0.79
4	1.89	2.21	-0.32
5	1.62	1.24	0.38
6	4.58	4.32	0.26
7	2.10	1.74	0.36

These results are similar to SPSS result.

4. Combination of three variables

The number of ways to choose 3 variables from the set of 6 variables is:

$$\binom{6}{3} = \frac{6!}{3!\,(6-3)!} = 20$$

Among the 20 relations developed the relation between A, S and RB gave the largest adjusted R^2 value, which means these variables predict the observed velocity more nearly than the others variables combinations.

SUMMARY OUTPUT	
Regression Statistics	
Multiple R	0.94
R Square	0.89
Adjusted R Square	0.81

The multiple linear regression coefficients are:

	Coefficients
Intercept	-28.084
A	0.005
S	47.576
R_B	8.197

$V_s = -28.084 + 0.005A + 47.576S + 8.197R_B$ 5.5

Summary of Residuals

Observation	Observed Vs	Predicted Vs	Residuals
1	4.12	3.43	0.65
2	1.03	0.81	0.19
3	2.77	3.42	-0.71
4	1.89	2.21	-0.36
5	1.62	1.78	-0.19
6	4.58	4.30	0.17
7	2.10	1.91	0.13

Table 5.21: Comparison results of two and three variable combination

Observation	Observed Vs	Predicted Vs(two Variables)	Predicted Vs(three Variables)
1	4.12	3.25	3.43
2	1.03	1.52	0.81
3	2.77	3.61	3.42
4	1.89	2.23	2.21
5	1.62	1.26	1.78
6	4.58	4.4	4.30
7	2.1	1.78	1.91
Adjusted R^2		**0.73**	**0.81**

The R square and adjusted R square value of three variable combinations is greater than the two variable combinations. Therefore, the three variables formula was selected.

5.8.4. SPSS Analysis for GIUH Based Nash IUH Model

For this model the sensitive variables to velocity are Area(A), Slope(S), and Biffurcation Ratio(R_B). The detail analysis is attached in the appendix F.

SUMMARY OUTPUT	
Regression Statistics	
Multiple R	0.979
R Square	0.958
Adjusted R Square	0.926

The multiple linear regression coefficients are:

Coefficients	
Intercept	-62.985
A	0.009
S	80.446
RB	18.973

The derived GIUH based three variable formula is

$$\mathbf{Vs = -62.985 + 0.009A + 80.446S + 18.973R_B} \qquad 5.6$$

Table 5.22: Stream Velocity results of GIUH based Nash IUH Model

Observations	GIUH Based Nash Model	
	Observed Vs	Predicted Vs
1	9.9	9.07
2	3.1	3.09
3	7.9	8.82
4	5.6	5.99
5	4.1	4.33
6	9.8	9.85
7	4.3	4.62

As we can see from the above tables, GIUH based Nash model has larger adjusted R square value than GIUH model. This implies that GIUH based Nash model predicts superior than GIUH model.

5.9. Validation of the GIUH Models

The purpose of this section is to check the reliability of the regional equation of the velocities of GIUH and GIUH based Nash models. The regional equation developed for predicting the velocity parameter of the GIUH model was used to estimate the direct runoff hydrographs of four events of the nearby catchment of chemoga, found in South Gojam sub basin, which was not used in the calibration of the models. The different geomorphological characteristics of this catchment were extracted from DEM using ILWS in the same method as those catchments used in the calibration of the model. The stream velocity of GIUH model was estimated using equation 5.5 and the hillslope velocity was set to be 0.09m/s. Then, using the value of stream and hilly slope velocity, the simulated unit hydrograph can be derived using the GIUH model.

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.

Similarly validation test for GIUH based Nash model also done using equation 5.6 for stream velocity calculation.

The results of the validation tests of the regional velocity equation for the selected events are shown in Figure 5.8.

Steps for calculating GIUH(t) after getting the value of Vs:

1. Estimate the parameters λi and αi using equation 4.32 and 4.33,
2. Compute the probability density function of each possible paths (fsi), using equation 4.36,
3. Compute the corresponding path probability, prob(Si), using equation 4.13,
4. Then, derive GIUH(t), using equation 4.35.

Steps for calculating GIUH based Nash IUH Model after getting the value of Vs:

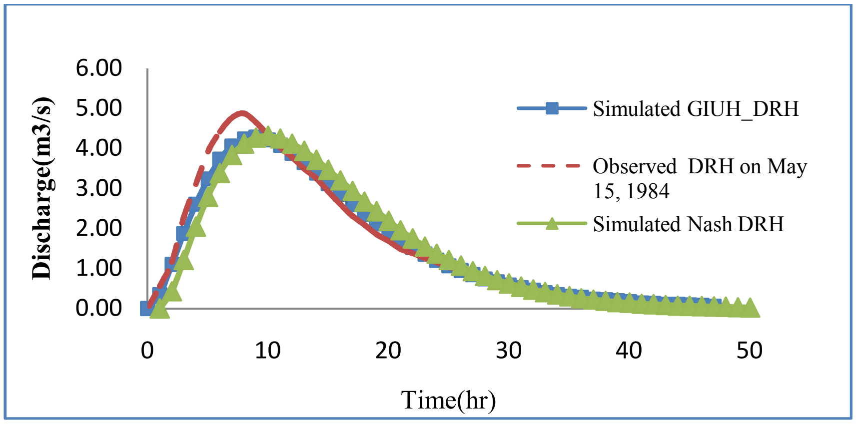
1. Determine the shape(n) and scale(K) parameter using equation 4.44 and 4.45 respectively.
2. Then, derive Nash based GIUH(t), using equation 4.38.

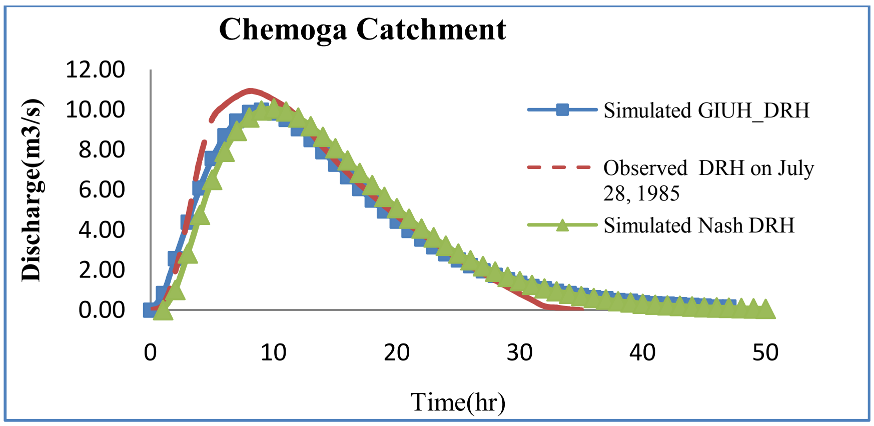
Table 5.23: Velocities result of chemoga catchment using GIUH model regional equation

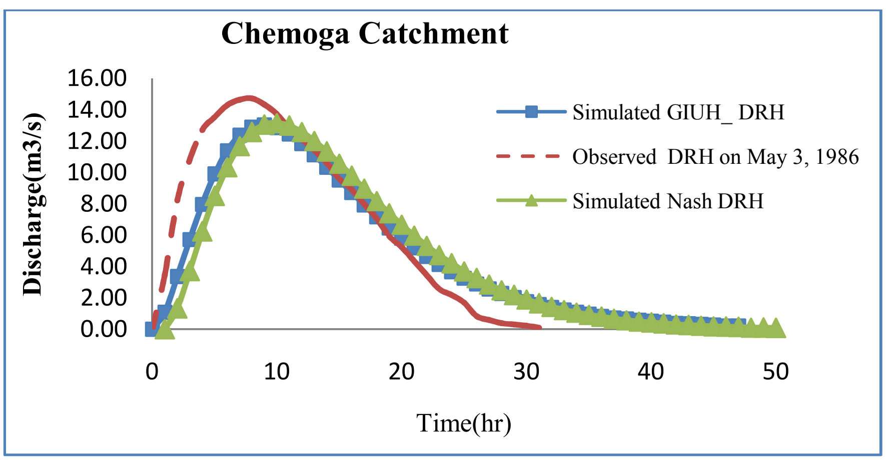
Catch. Name	Area(km^2)	Longest Flow Path Length(Km), L	Slope, S	R_B	R_L	R_A	Vo	Vs
Chemoga	361.05	50.52	0.03	3.24	2.46	4.32	**0.09**	**1.71`**

Table 5.24: Velocities result of chemoga catchment using GIUH based Nash IUH model regional equation

Catch. Name	Area(km^2)	Longest Flow Path Length(Km), L	Slope, S	R_B	R_L	R_A	Vs	n	K
Chemoga	361.05	50.52	0.03	3.24	2.46	4.32	**4.15**	**2.77**	**4.99**


Discharge(m3/s)
6.00
5.00
4.00
3.00
2.00
1.00
0.00
0
10
20
30
40
50
Time(hr)
Simulated GIUH_DRH
Observed DRH on May 15, 1984
Simulated Nash DRH


Chemoga Catchment
Discharge(m3/s)
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
10
20
30
40
50
Time(hr)
Simulated GIUH_DRH
Observed DRH on July 28, 1985
Simulated Nash DRH


Chemoga Catchment
Discharge(m3/s)
16.00
14.00
12.00
10.00
8.00
6.00
4.00
2.00
0.00
0
10
20
30
40
50
Time(hr)
Simulated GIUH_ DRH
Observed DRH on May 3, 1986
Simulated Nash DRH

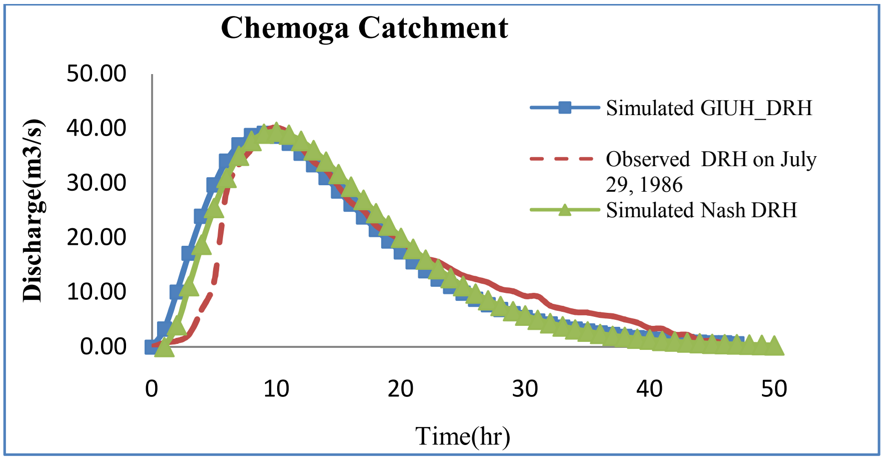


Figure 5.8: Direct Runoff hydrographs of selected events and simulated GIUH-DRH and GIUH based Nash model DRH

The value of DRH ordinates are attached in Appendix E.

Table 5.25: Error function result for validation test

Methods	Error Functions			
	EFF(%)	PEP(%)	PETP(%)	ERR(%)
Event-1(May 15, 1984)				
GIUH	97.08	1.45	-12.50	1.30
GIUH (Nash)	96.83	0.69	-12.5	1.24
Event-2(July 28, 1985)				
GIUH	96.59	8.73	-12.50	2.20
GIUH (Nash)	97.51	8.00	-12.50	1.99
Event-3(May 3, 1986)				
GIUH	84.84	11.32	-12.50	2.90
GIUH (Nash)	87.15	10.56	-12.50	2.63
Event-4(July 29, 1986)				
GIUH	83.84	2.63	10.00	1.30
GIUH (Nash)	81.76	1.27	10.00	1.27

The above plots and tables show that the observed and simulated Direct Runoff Hydrographs are fairly similar, and hence the derived regional equation gives satisfactory result for ungauged catchments and any point upstream from the outlet of the gauged catchments for any order stream.

When we evaluate the models, both gives nearly similar and good accuracy results, but the GIUH model is more complex. Therefore, it is better to use the GIUH based Nash model according to its simplicity and prediction reliability.

5.10. Derivation of Triangular Unit Hydrograph from GIUH Model

Equation 4.35 yields full analytical, but complicated, expressions to use GIUH model. However, for design of hydraulic structures it is adequate to assume a triangular UH and only specify the peak discharge, the time to peak and the base time of UH. These characteristics have simple expressions obtained by regression of stream flow velocity, catchment area, channel slope, peak discharge, time to peak, and the base time of the analytic solution to Equation 4.35 for a wide range of parameters.

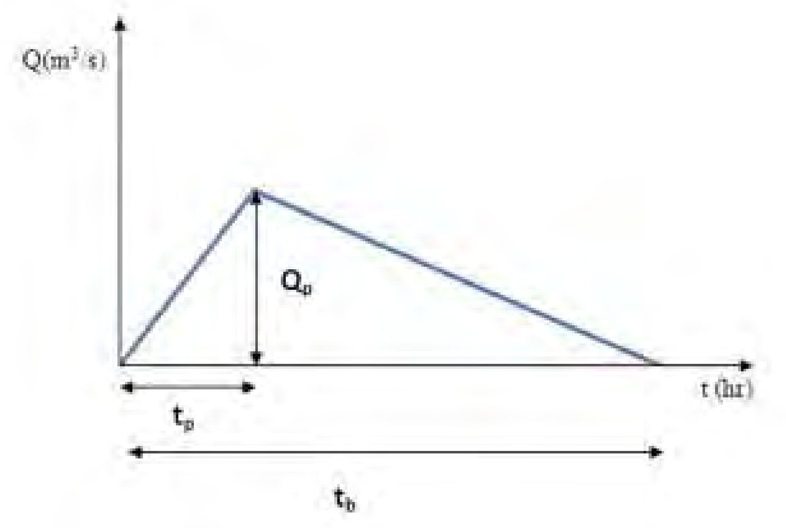


Figure 5.9: Triangular Unit Hydrograph

Table 5.26: Important parameters for unit hydrograph derivation

Catchment Name	Q_{cp} (m^3/s)	T_{cp} (hr)	Base time (t_b)-hr	Area (km^2)	Slope, S(m/m)	Channel Velocity, V(m/s)
Fettam	11.36	4.00	23.00	194.51	0.01	4.12
Hoha	3.59	8.00	48.00	164.48	0.01	1.03
Neshi	8.58	6.00	47.00	326.30	0.01	2.77
Uke	4.45	7.00	50.00	201.26	0.02	1.89
Azuari	6.19	6.00	36.00	195.79	0.04	1.62
Andassa	19.36	6.00	38.00	602.07	0.02	4.58
Jedeb	11.52	4.00	26.00	295.83	0.04	2.10

Using the results obtained from the analysis, a relationship between the peak flow rate, stream flow velocity, time to peak flow and base time of the unit hydrograph have been derived for catchments and are expressed as follows:

Steps for triangular unit hydrograph derivation

1. *Calculate Vs from,*

   $$V_s = -28.084 + 0.005*A + 47.576*S + 8.197*R_B \quad (R^2=0.89), \text{ from equation 5.5}$$

2. *Calculate Qp from,*

   $$Q_p = -4.034 + 0.017*A + 90.087*S + 2.508*Vs \qquad (R^2=0.94) \qquad 5.7$$

3. *Calculate tp from,*

   $$T_p = 6.422 - 0.486*Qp + 0.014*A \qquad (R^2=0.76) \qquad 5.8$$

4. *Calculate tb from,*

   $$T_b = 40.991 - 3.959*Qp + 0.120*A \qquad (R^2=0.91) \qquad 5.9$$

Where, Vs is in m/s, A is in Km^2, S is in m/m, R_B is dimensionless, Q_p is in m^3/s, T_p and T_b are in hr.

Table 5.27: Triangular unit hydrograph formula results and their percentage of errors

Catchment Name	Observed			Derived GIUH Triangular Method Results			Errors		
	Q_{op} (m^3/s)	T_{op} (hr)	T_{ob} (hr)	Qp-Estimated (m^3/s)	Tp-Estimated (hr)	Tb-Estimated (hr)	PEP (%)	PETP (%)	PETB (%)
Fettam	12.62	4	23	10.8	3.62	21.58	14.42	9.50	6.20
Hoha	3.89	8	48	2.55	6.97	50.63	34.45	12.88	-5.49
Neshi	8.55	5	47	9.33	6.61	43.21	-9.12	-32.20	8.06
Uke	5.48	9	50	6.05	7.07	41.19	-10.40	21.44	17.62
Azuari	6.18	6	36	7.31	6.15	35.55	-18.28	-2.50	1.26
Andassa	20.1	7	38	19.5	5.42	36.04	2.99	22.57	5.16
Jedeb	11.17	3	26	9.53	4.46	38.76	14.68	-48.67	-49.08

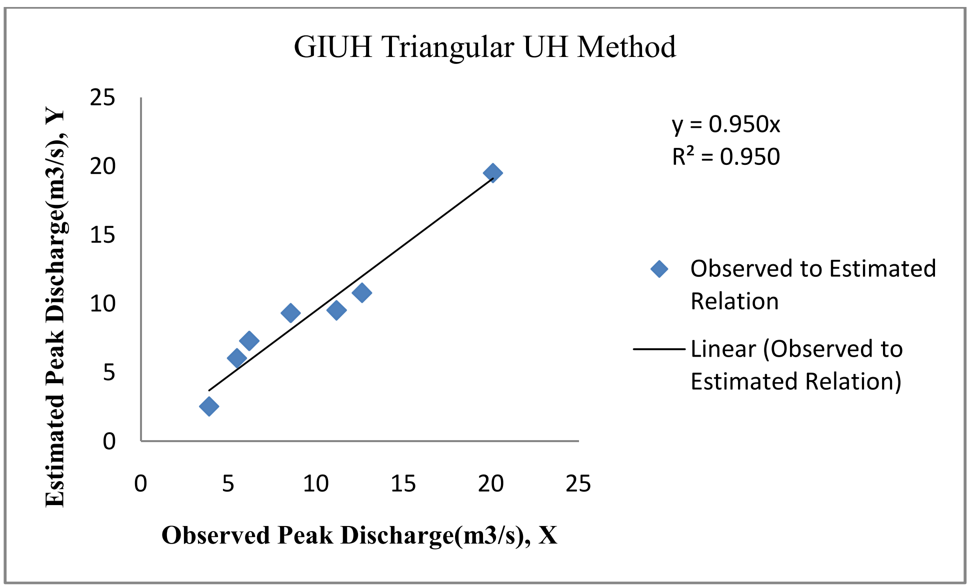


Figure 5.10: Relation of observed and estimated peak discharge using GIUH triangular UH method

For verification of the percentage error in peak discharge and time-to-peak discharge Timothy D. Straub (2000) rules were used.

- If the percentage error in peak discharge and time-to-peak discharge estimation is less than 15 percent, then the simulated result is in ***good agreement*** with the measured values.

- If it is between 15 and 35 percent, then the simulated result is in ***fair agreement*** with the measured values.
- If it is greater than 35 percent, then the simulated result is in ***poor agreement*** with the measured values.

For percentage error in peak discharge estimation, from table 5.23, we can see that except Azuari and Hoha, the rest five catchments (Fettam, Neshi, Uke, Andassa and Jedeb) are in good agreement range(less than 15 percent). Whereas, Azuari and Hoha are in fair agreement range (between 15 and 35 percent).
The r-square value of the observed and simulated peak discharge is 0.95(figure 5.8).

For percentage error in time to peak discharge estimation, Fettam, Azuari and Hoha are in good agreement range(less than 15 percent). Whereas, Neshi, Ukke and Andassa are in fair agreement range (between 15 and 35 percent). But, Jedebe is in poor agreement range. This is because of the small value of observed time to peak.
In general, most of the simulated values are in good agreement with the observed values.

5.11. Steps for SCS Triangular Unit Hydrograph Derivation

1. Obtain catchment area (A), longest flow path length (L), slope (S) and elevation difference (H) from ILWIS result (Refer table 5.2).
2. Determine the time of concentration(t_c) using (Sorell and Hamilton, 1991),

   i) Kirpich formula

   $$T_c = 0.0078L^{0.77}S^{-0.385} \qquad 5.10$$

   Where, T_c is in minute, L is in feet and S is feet/feet.

   ii) California Culvert Practice (CCP)

   $$T_c = 60(11.9L^3/H)^{0.385} \qquad 5.11$$

   Where: T_c is in minute, L is in mile and H is in feet.

   iii) SCS Lag equation

   $$T_c = \frac{100L^{0.8}\left(\left(\frac{1000}{CN}\right)-9\right)^{0.7}}{1900\ S^{0.5}} \qquad 5.12$$

Where, Tc is in minute, L is in feet and slope(S) is in %.

This formula requires the curve number (CN) value. However, curve number is depending on land use and hydrological soil group. It requires time to time change due to variation encountered with respect to the gradual climatic changes.

Therefore, for Tc computation the average of kirpitch and California Culvert Practice was taken.

3. Determine the time lag (t_l) using,

   $t_l = 0.6t_c$ 5.13

4. Determine time to peak(t_p) using,

   $t_p = t_r/2 + t_l$ 5.14

   Where, t_r is effective rainfall duration.

5. Determine the base time(t_b) using,

   $t_b = 2.67t_p$ 5.15

6. Determine the peak discharge(Q_p) using,

   $Q_p = CA/t_p$ 5.16

   Where, C = 2.08 and A is the drainage area in square kilometers.

7. Finally, compare the estimated values of SCS Triangular UH with the observed unit hydrograph using PEP, PETP and PETB criteria.

Table 5.28: SCS Triangular unit hydrograph formula results and their percentage of errors

Catchment Name	Observed			SCS Triangular Method Results			Errors		
	Q_{op} (m^3/s)	T_{op} (hr)	T_{ob} (hr)	Qp-Estimated (m^3/s)	Tp-Estimated (hr)	Tb-Estimated (hr)	PEP (%)	PETP (%)	PETB (%)
Fettam	12.62	4	23	11.21	3.61	9.64	11.19	9.75	58.09
Hoha	3.89	8	48	12.49	2.74	7.31	-221.12	65.76	84.77
Neshi	8.55	5	47	12.07	5.62	15.01	-41.17	-12.46	68.06
Uke	5.48	9	50	11.93	3.51	9.37	-117.75	61.02	81.27
Azuari	6.18	6	36	17.08	2.38	6.36	-176.44	60.27	82.32
Andassa	20.1	7	38	24.13	5.19	13.86	-20.04	25.85	63.53
Jedeb	11.17	3	26	18.41	3.34	8.92	-64.83	-11.40	65.68

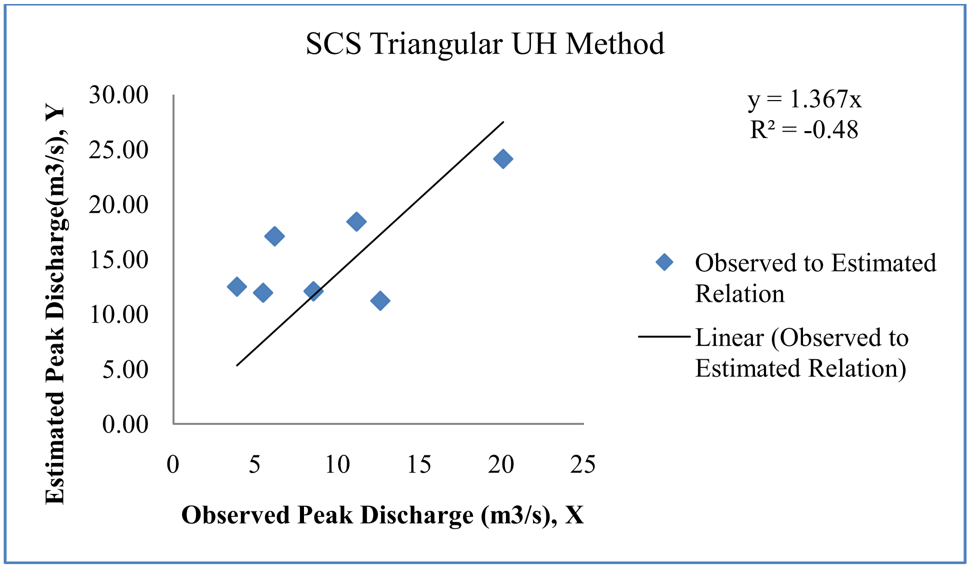


Figure 5.11: Relation of observed and estimated peak discharge using SCS triangular UH method

As we can see in table 5.24, Percentage error in Peak discharge and time to peak discharge estimation are less than 15 for Fettam, and between 15 and 35 for Andassa. But, for the rest catchments it is greater than 35(poor agreement range). The r-square value of Percentage error in Peak discharge estimation is 0.48, which is a small value. Therefore, SCS triangular unit hydrograph method is not recommended for unit hydrograph derivation for those catchment except Fettam and Andassa.

5.12. Comparision of Simulated GIUH Based Triangular Unit Hydrograph and Godana's Snyder Unit Hydrograph Method

Table 5.29: Parameters of observed and simulated unit hydrographs

Catchments	Parameters of Observed Unit Hydrograph			Parameters of Simulated Unit Hydrograph					
				Godana's Snyder UH Method			Triangular GIUH Method		
	$Q_p(m^3/s)$	t_p(hr)	T_B(hr)	$Q_p(m^3/s)$	t_p(hr)	T_B(hr)	$Q_p(m^3/s)$	t_p(hr)	T_B(hr)
Andassa	20.10	7.00	38.00	22.21	5.05	26.82	19.50	5.42	36.04
Azuari	6.18	6.00	36.00	8.17	3.00	17.50	7.31	6.15	35.55
Chemoga	5.71	9.00	58.00	8.50	4.75	28.00	9.08	7.06	48.34
Hoha	3.89	8.00	48.00	3.54	5.50	37.00	2.55	6.97	50.63
Jedeb	11.17	3.00	26.00	19.80	2.25	12.25	9.53	4.46	38.76
Neshi	8.55	5.00	47.00	6.40	5.10	37.00	9.33	6.61	43.21
Ukke	5.48	9.00	50.00	2.69	5.51	34.60	6.05	7.07	41.19

Table 5.30: Percentage of errors of Godana's Snyder unit hydrograph and GIUH triangular unit hydrograph method

Catchments	Percentage of Errors					
	Godana's Snyder UH Method			Triangular GIUH Method		
	PEP(%)	PETP(%)	PETB(%)	PEP(%)	PETP(%)	PETB(%)
Andassa	-10.50	27.86	29.42	2.99	22.57	5.16
Azuari	-32.20	50.00	51.39	-18.28	-2.50	1.26
Chemoga	-48.86	47.22	51.72	-59.02	21.56	16.66
Hoha	9.00	31.25	22.92	34.45	12.88	-5.49
Jedeb	-77.26	25.00	52.88	14.68	-48.67	-49.08
Neshi	25.15	-2.00	21.28	-9.12	-32.20	8.06
Ukke	50.91	38.78	30.80	-10.40	21.44	17.62

In general, as we can see from the above table 5.30, the triangular based GIUH method gave a better result than Godana's derived Snyder unit hydrograph method. Moreover, GIUH method gives the complete shape of the unit hydrograph easily and does not depend on climatic parameters.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions

Geomorphological parameters and rainfall-runoff records were used in this study to apply GIUH and GIUH based Nash IUH models on the selected catchments (Fettam, Hoha, Neshi, Ukke, Azuari Andassa, Jedebe and Chemoga).

The geomorphological parameters required for derivation of the GIUH and GIUH based Nash IUH models, such as, length of main stream, catchment area, bifurcation ratio, length ratio and area ratio (Horton's Parameters) have been evaluated using the GIS package called ILWIS (Integrated Land and water Information System). Using Horton's parameters and the calibrated velocity, for each catchment, the complete shape of GIUH and GIUH based Nash IUH models were developed.

From model evaluation results, GIUH model efficiency (EFF) varies from 81.95 to 97.9 percent while GIUH based Nash IUH model efficiency varies from 75.68 to 97.51 percent. GIUH model has an error (ERR) in estimation varying from 0 to 13.5 percent whereas GIUH based Nash IUH model varies from 0.89 to 17.1 percent. Moreover, GIUH model varies in percentage error in peak discharge (PEP) estimation from 0.2 to 22.2 percent while GIUH based Nash IUH model varies from 0.69 to 23.14 percent. In addition, GIUH model is efficient in peak discharge estimation (R^2=0.988) than GIUH based Nash IUH model (R^2=0.884).

In general, the result of both GIUH and GIUH based Nash IUH modes are almost comparable and their results are acceptable but the GIUH model is superior. However, the method of GIUH derivation is difficult as compared to GIUH based Nash IUH model. Therefore, a simple GIUH based regional triangular unit hydrograph equations are developed (refer equations 5.5 to 5.9).

For GIUH model stream flow velocity (V_s) and hilly slope velocity (V_o) are set as calibrating parameter. For the selected catchment V_o is constant, equal to 0.09m/s while V_s is variable. It can be derived using equation 5.5 (R^2 = 0.89). For GIUH based Nash IUH model the calibrating parameter is only stream flow velocity (V_s) and it can be derived using equation 5.6 (R^2 = 0.96).

Finally, the developed GIUH based regional triangular unit hydrograph was compared with the SCS triangular unit hydrograph. The R^2 value in peak discharge estimation using SCS triangular unit hydrograph method is 0.48 while using GIUH based triangular unit hydrograph equation is 0.95. This implies that the derived GIUH based regional triangular unit hydrograph equation is quite different from SCS triangular unit hydrograph and GIUH is better in efficiency.

Therefore, the regionalized equation of GIUH triangular UH can be used as ready reference to the field Engineers for design of hydraulic structures, within the errors specified in this research paper, for any ungauged catchment found in neighbor of the study area having similar character and varying in areas from 161 to 573km^2, since the research is conducted within this area range.

6.2. Recommendations

- In this study for baseflow separation, straight line method was selected and for excess rainfall calculation, phi-index method was selected. These had a significant impact on the model predictions. Therefore, the best fit method should be selected based on the site condition.

- For each of the selected catchments, the catchment average unit hydrographs were derived from four to eight 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. 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 the exact value of a stream threshold for the digital elevation data operation. When the value of the stream threshold decreases, the channel network becomes dense, smaller drainage networks would be included. Therefore, to obtain the optimum stream threshold value, it is required to know the site condition and the minimum drainage length.

- Derivation of the regression velocity equation was done by fitting the channel velocities of the GIUH and GIUH based Nash IUH model in only seven 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. Further, the triangular unit hydrograph formula also improved.

- 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 homogenous regions.

- Moreover, further work in the future may be required to see the variation of the regionalized equation using homogeneity of the catchments and non linear relationship with the paramete

REFERENCES

Abbott, M.B., 1986a: An Introduction to the European Hydrological System – Système Hydrologique Europèen, *SHE*–1: History and Philosophy of a Physically-Based Distributed Modeling System. Journal of Hydrology 87, 45-59.

Al-Wagdany AS, Rao AA., 1998: Correlation of the velocity parameter of three geomorphological instantaneous unit hydrograph methods. Hydrological Processes 12, 651– 659.

BCEOM, 1999: *Abay River Basin Integrated Master Plan, Main Report.* Ministry of Water Resources, Addis Ababa.

Bedient P.B., Huber W.C., 1992: Hydrology and Floodplain analysis. 2nd edition, New York, Addison–Wesley Publishing Company.

Bhaskar, N.R., Parida, B.P. and Nayak, A.K., 1997: Flood estimation for Ungauged catchment using the GIUH. Journal of water resources planning and management, 123(4), 228-238.

Bras, R.L. and Rodriguez-Iturbe, I., 1989: A review of the search for a quantitative link between hydrologic response and fluvial geomorphology, New directions for surface water modeling. IAHS Publ.no.181, 149-163.

Chow, V. T., D. R. Maidment, and L. W. Mays. 1988: *Applied Hydrology*. New York: McGraw-Hill Book Company.

Dooge, J.C.I., 1977: Problems and methods of rainfall - Runoff modelling. In: T. A.Ciriani, U. Maione and J. R.Wallis (Editors), Mathematical models for surface water hydrology. John Wiley & Sons, pp. 71-108.

Dune, T., 1982: Relation of field studies and modeling in the prediction of storm runoff. Journal of Hydrology, 65, 25-48.

Godana Seyoum, 2005: Development of Regional Unit Hydrograph model for watersheds in the Abay Basin. Final Thesis, Addis Ababa University.

Gupta V.K., Waymire E., Wang C.T., 1980: A representation of an instantaneous unit hydrograph from geomorphology, Water Resource Research, 16(5), pp 855-862.

Horton, R.E., 1932: Drainage-basin characteristics. EOS Transactions of AGU, 13: 350-361.

Horton, R.E., 1945: Erosional development of streams and their drainage basins: Hydrophysica approach to quantitative morphology. Geological Society of America Bulletin, 56, 275 - 370.

Jain, S.K., Singh, R.D., and Seth, S.M., 2000: Design Flood Estimation Using GIS Supported GIUH Approach, Water Resources Management, Vol. 14, P. 369-376.

Jain, V., and Sinha, R. 2003: Derivation of u V.nit hydrograph from GIUH analysis for a Himalayan river. *Water Resour. Manage.*, 17, 355-375.

Jakeman A.J., Littlewood I.G., Whitehead P.G., 1990: Computation of the instantaneous unit hydrograph and identifiable component flows with application to two small upland catchments, Journal of Hydrology, 117, pp 275-300.

Kirby, M.J. (Editor), 1978: Hillslope hydrology. Landscape systems, a series in geomorphology. John Wiley and Sons, 389 pp.

Kildore L. J., 1997: Developpement and evaluation of a GIS-based spatially distributed unit hydrograph model, Thesis submitted to the Faculty of the Virginia Polytechnic Institute and State University

Kirshen, D.M. and Bras, R.L., 1983: The linear channel and its effect on the geomorphologic IUH. Journal of Hydrology. Vol. 65, P.175–208.

Kumar, R. Chatterjee, C. Singh, R.D. Lohani1, A.K and Sanjay Kumar, 2007: Runoff estimation for ungauged catchments using GIUH, Hydrol. Process. 21, 1829–1840.

Lee K.T., 1998: Generating Design Hydrographs by DEM Assisted Geomorphic Runoff Simulation: A Case Study. J. the American Water Resources Association, Vol.34, No. 2.

Lee, K.T., and Chang, C.H., 2005: Incorporating Subsurface Flow Mechanism into Geomorphology-Based IUH Modeling. Department of River and Harbor Engineering, National Taiwan Ocean University, 2, Bee-Ning Road, Keelung, Taiwan 202, ROC.

Maathuis, B., 2006: DEM based Hydro – processing- Introduction to the tools developed, tutorial with exercise (Version 1). International Institute for Geo-information Science and Earth Observation (ITC), Enschede, 71 pp.

Maidment, D.R. and Djokic, D. (Editors), 2000: Hydrologic and Hydraulic Modeling Support with Geographic Information Systems. ESRI, 232 pp.

Maidment, D. R. 1993a: GIS and hydrologic modeling. In Environmental Modeling with GIS. eds. M. F. Goodchild, B. O. Parks, L. Steyaert, New York: Oxford University Press.

Maidment, D.R., 1996: GIS and Hydrologic Modeling. An Assessment of Progress, Presented at the Third International Conference on GIS and Environmental Modeling, Santa Fe, New Mexico, pp. 22-26.

Maidment, D. R., 1992: Handbook of Hydrology, McGraw Hill Book Co.

McCuen Richard H, 1989: Hydrology Design and Analysis. Prentice Hall Englewood Cliff, New Jersey.

Moore, I.R., Grayson, R.B. and Ladson, A.R., 1992: Digital terrain modelling: A review of Hydrological, Geomorphological and Biological applications. In: K. Beven and I.R. Moore (Editors), Terrain analysis and distributed modelling in hydrology. John Wiley and Sons, pp. 7-34.

Moore, I.R., 1996: Hydrologic modelling and GIS. In: F.M. Goodchild et al. (Editors), GIS and environmental modeling. GIS World, Inc, Collins, pp. 143-148.

Mulugeta Azeze, 2004: Development of Synthetic Unit Hydrographs for Watersheds in the Upper Awash and Upper Tekeze Basins. Final Thesis, Addis Ababa University, Ethiopia.

Nash, J. E. and Sutcliffe, J. V., 1970: River Now Fore- casting through Conceptual Models I: discussion of principles, Journal of Hydrology, Vol. 10, 282-290.

Raghunath H. M., 1985: *Hydrology, Principles, Analysis, Design*. New Age International Publishers.

Rakesh Kumar, et al., 2007: Runoff Generation for Ungauged Catchments using Geomorphlogic Instantaneous Unit Hydrograph Models, Hydrol. Process 21, 1829-1840.

Rodríguez-Iturbe, I. and A. Rinaldo, 1997: Fractal River Basins: Chance and Self-Organization. Cambridge University Press, New York.

Rodriguez-Iturbe I, Gonzalez-Sanabria M, Bras RL. 1982a: The geomorphoclimatic theory of the instantaneous unit hydrograph. *Water Resources Research* 18(4): 877–886.

Rodriguez-Iturbe I, Gonzalez-Sanabria M, Caamano G. 1982b: On the climatic dependence of the IUH: a rainfall-runoff analysis of the Nash model and the geomorphoclimatic theory. *Water Resources Research* 18(4): 887–903.

Rodriguez-Iturbe, I., and Valdes, J.B., 1979: The geomorphologic structure of hydrologic response. Water Resour. Res, 15(6): 1409-14-20.

Rosso R. 1984. Nash model relation to Horton order ratios. Water Resources Research 20(7): 914–920.

Schumann, A.H., Funke, R. and Schultz, G.A., 2000: Application of a geographic information system for conceptual rainfall-runoff modeling. Journal of Hydrology, 240: 45-61.

Singh, V.P., 1988: Hydrologic systems. Volume 1: Rainfall - runoff modeling. Prentice Hall, 480 pp.

Singh, V.P. and Woolhiser, D.A., 2002: Mathematical modeling of Watershed hydrology. Hydrologic Engineering, 7(4): 270 -292

Sorell and Hamilton, 1991: Unit Hydrograph Technical Manual-NOHRSC.

Soil Conservation Service, Hydrology, 1972: Sec. 4 of National Engineering Handbook,

Strahler, A.N., 1957: Quantitative analysis of watershed geomorphology. EOS Transactions of AGU, 38: 912-920.

Tarboton, D.G., Bras, R.L. and Rodriguez-Iturbe, I., 1991: On the extraction of channel networks from digital elevation data. Hydrological processes, 5: 81-100.

Timothy D. Straub, et al, 2000: Equations for Estimating Clark Unit Hydrograph Parameters for Small Rural Watersheds in Illinois, Water Resource Investigation Report 00-4184.

Todini, E., 1988: Rainfall-runoff modeling—Past, present and future, Journal of Hydrology, 100(1-3): 341-352

Tsehay Zeray, 2009: Application of the Geomorphologic Instantaneous Unit Hydrograph Concept for Runoff Prediction in Ungauged Catchments. Final Thesis, Addis Ababa University, Ethiopia.

U.S Army Corps of Engineers, 1994: Engineering and design Flood-runoff analysis Manual. Engineer Manual 1119-2-1417 Washington, DC, USA.

Wilson, E.M., 1974: Engineering Hydrology, Second Edition, The Macmillan Press ltd., London Basingstoke Companies.

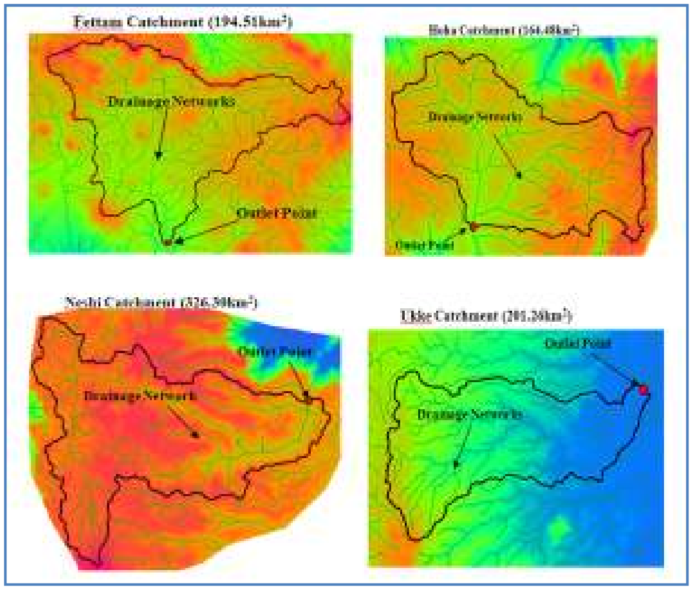
Yen, B.C., and Lee, K.T., 1997: Unit Hydrograph Derivation for Ungauged Watersheds by Stream-Order Laws, Journal of Hydrologic Engineering, ASCE, Vol. 2, No. 1, P. 1-9.

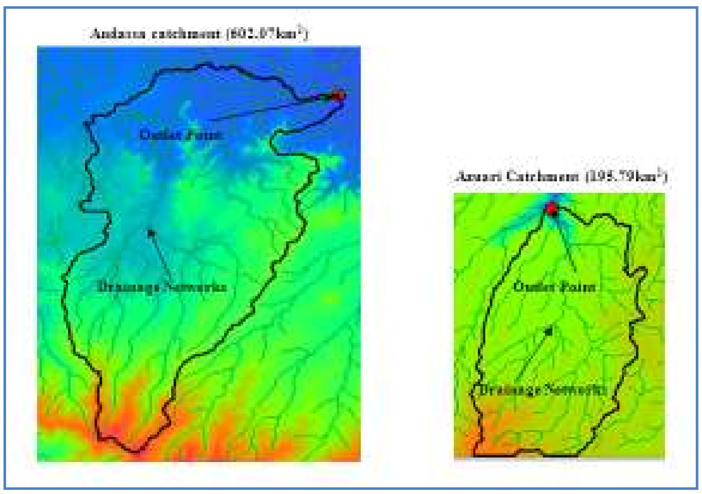
Yilma, A.D.; Awulachew, S.B., 2009: *Blue Nile characterisation and geospatial atlas*. Addis Ababa, Ethiopia: IWMI.

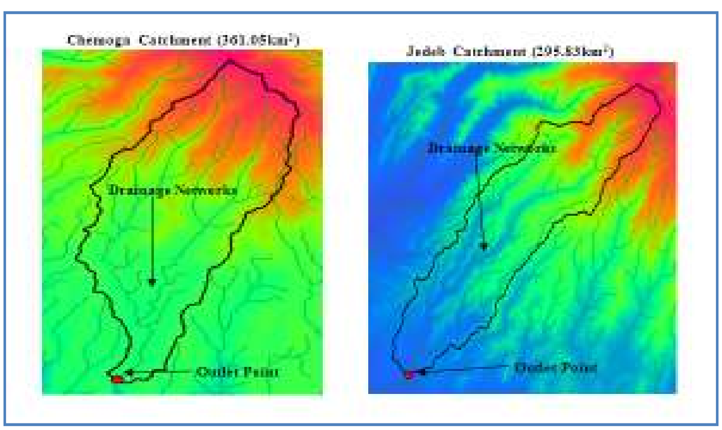
Zelealem Fisseha, 2007: Derivation of Unit Hydrograph from Geomorphological Instantaneous Unit Hydrograph Analysis for Ungauged Catchments of Ghiba Basin. Final Thesis, Arbaminch University, Ethiopia.

APPENDIXES

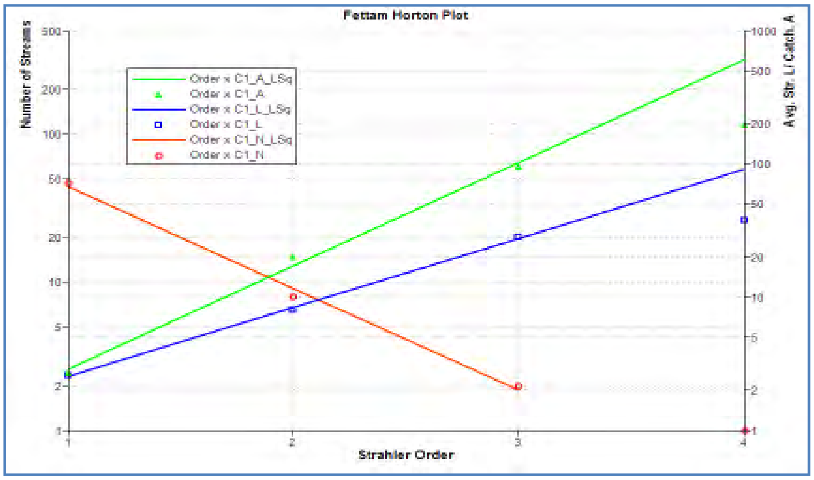
APPENDIX-A: ILWIS Extracted Drainage Catchments

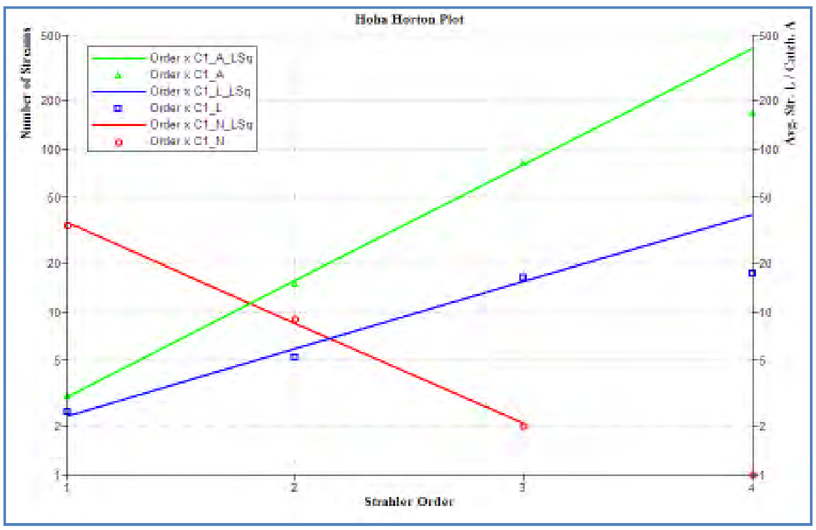


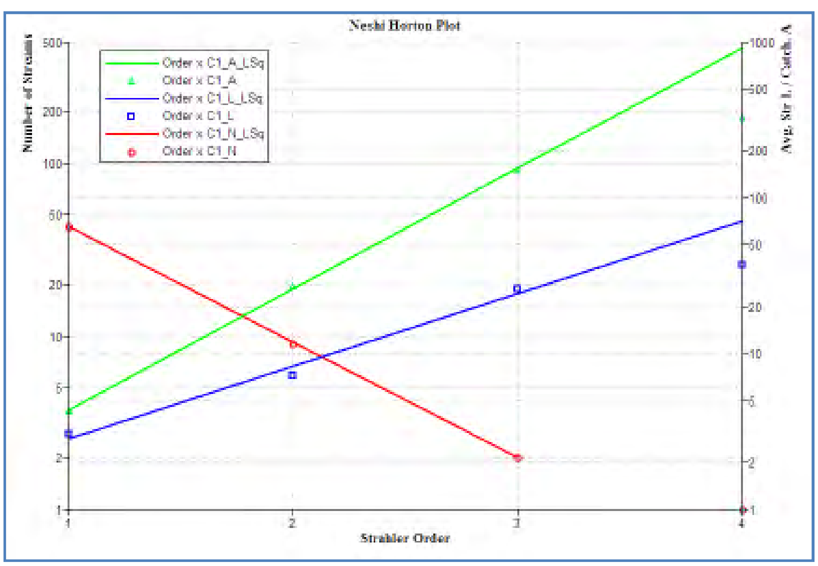


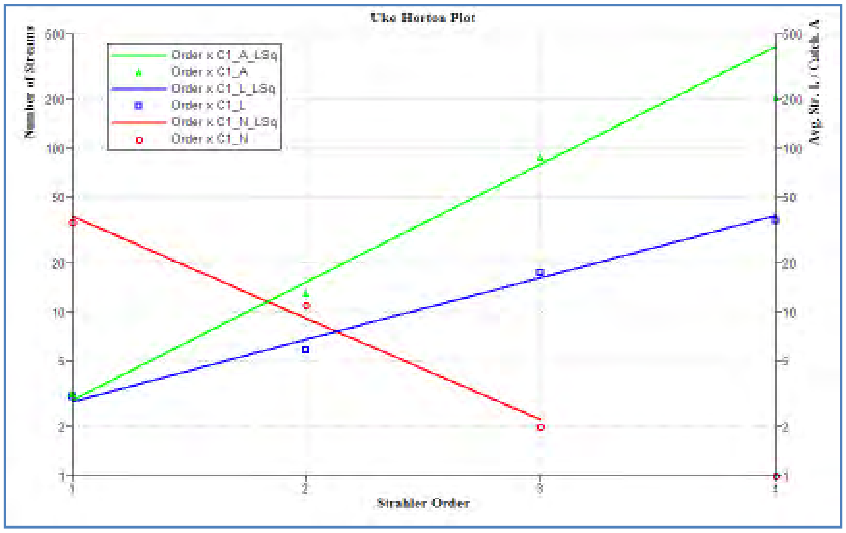


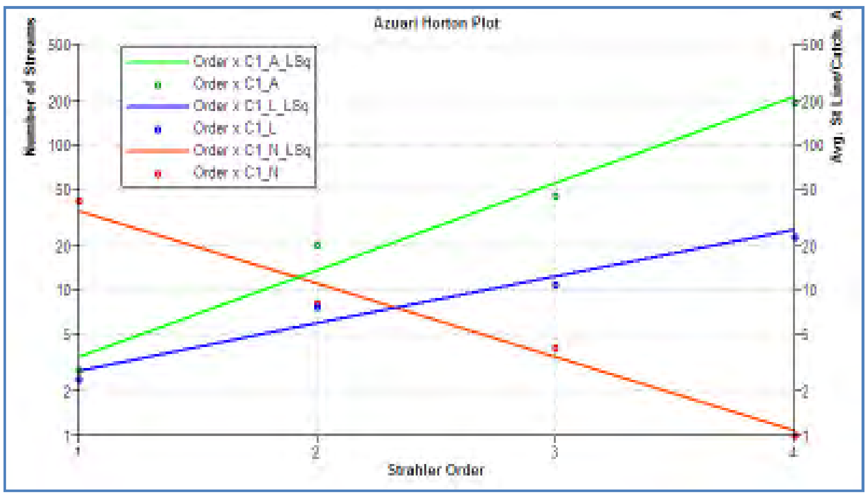
APPENDIX-B: Horton Plots

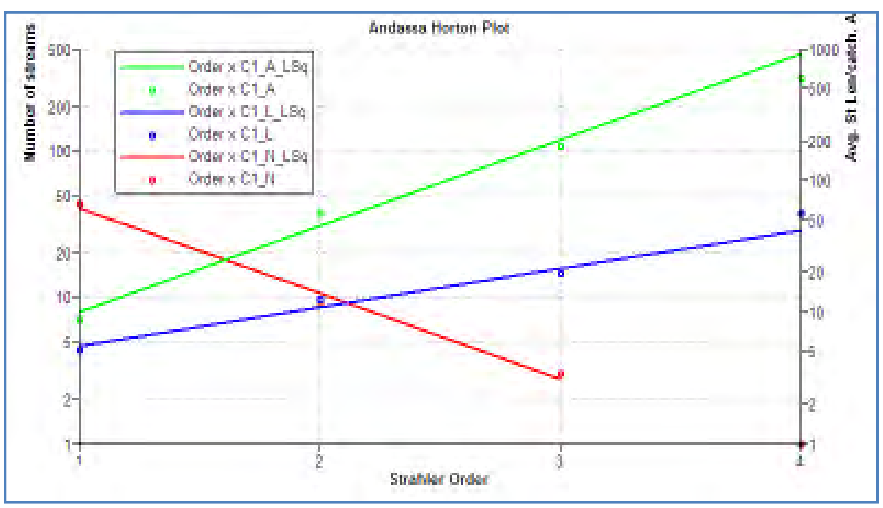


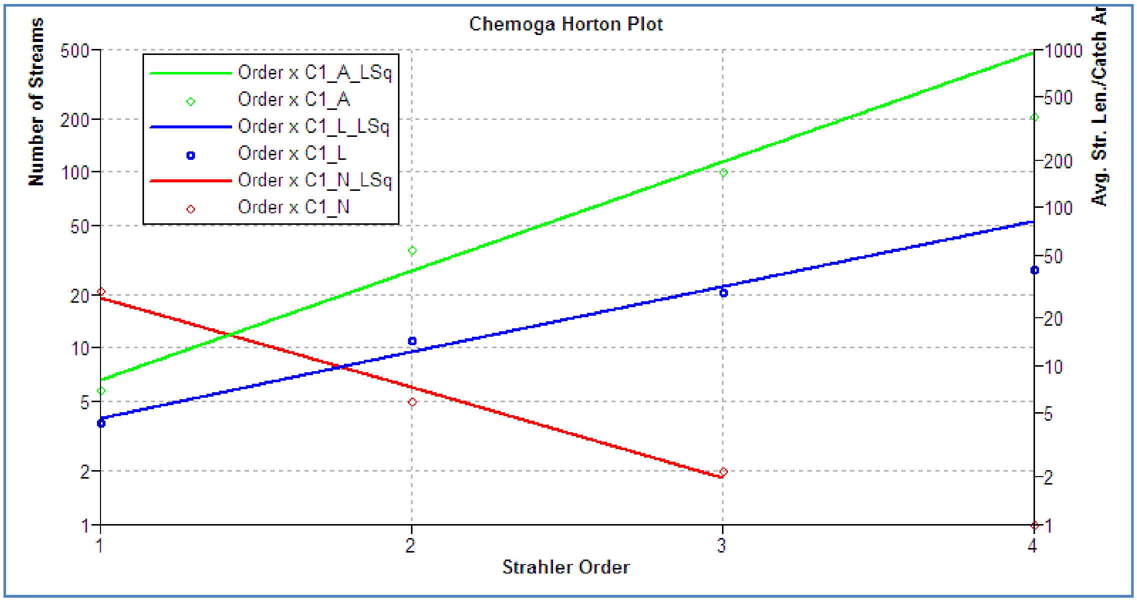


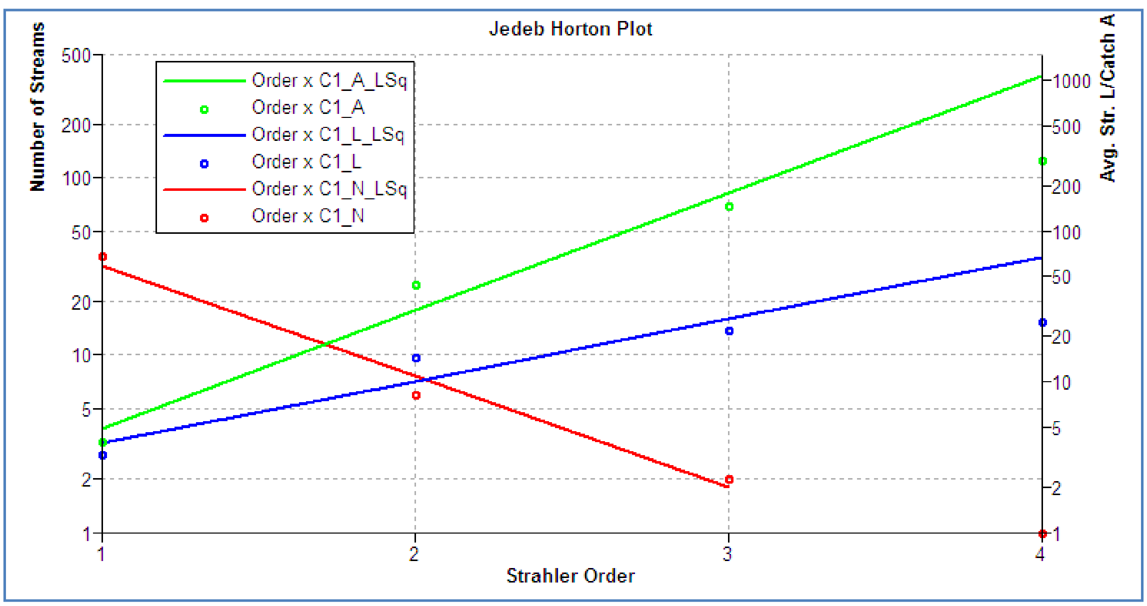



Uke Horton Plot
Order x C1_A_LSq
Order x C1_A
Order x C1_L_LSq
Order x C1_L
Order x C1_N_LSq
Order x C1_N
Number of Streams
Avg. Str. L / Catch. A
Strahler Order


Azuari Horton Plot
Order x C1_A_LSq
Order x C1_A
Order x C1_L_LSq
Order x C1_L
Order x C1_N_LSq
Order x C1_N
Number of Streams
Avg. St Line/Catch. A
Strahler Order


Andassa Horton Plot
Order x C1_A_LSq
Order x C1_A
Order x C1_L_LSq
Order x C1_L
Order x C1_N_LSq
Order x C1_N
Number of streams
Avg. St Len/catch. A
Strahler Order


Chemoga Horton Plot
Number of Streams
Avg. Str. Len./Catch Ar
Order x C1_A_LSq
Order x C1_A
Order x C1_L_LSq
Order x C1_L
Order x C1_N_LSq
Order x C1_N
Strahler Order


Jedeb Horton Plot
Number of Streams
Avg. Str. L/Catch A
Order x C1_A_LSq
Order x C1_A
Order x C1_L_LSq
Order x C1_L
Order x C1_N_LSq
Order x C1_N
Strahler Order

APPENDIX-C: Probability Density Equation of Each Possible Path

The convolution equations of the probability density functions of each possible path can be computed as follows:

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

For four order stream there are 8 possible paths:

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

$$f_{s1} = \alpha\lambda_1\lambda_2\lambda_3\lambda_4 \left[\frac{e^{-\alpha t}}{(\lambda_1-\alpha)(\lambda_2-\alpha)(\lambda_3-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_2-\lambda_1)(\lambda_3-\lambda_1)(\lambda_4-\lambda_1)} + \frac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_1-\lambda_2)(\lambda_3-\lambda_2)(\lambda_4-\lambda_2)} + \frac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_1-\lambda_3)(\lambda_2-\lambda_3)(\lambda_4-\lambda_3)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)(\lambda_2-\lambda_4)(\lambda_3-\lambda_4)}\right]$$

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

$$f_{s2} = \alpha\lambda_1\lambda_2\lambda_4 \left[\frac{e^{-\alpha t}}{(\lambda_1-\alpha)(\lambda_2-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_2-\lambda_1)(\lambda_4-\lambda_1)} + \frac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_1-\lambda_2)(\lambda_4-\lambda_2)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)(\lambda_2-\lambda_4)}\right]$$

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

$$f_{s3} = \alpha\lambda_1\lambda_3\lambda_4 \left[\frac{e^{-\alpha t}}{(\lambda_1-\alpha)(\lambda_3-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_3-\lambda_1)(\lambda_4-\lambda_1)} + \frac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_1-\lambda_3)(\lambda_4-\lambda_3)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)(\lambda_3-\lambda_4)}\right]$$

Path 4: $a_1 \longrightarrow r_1 \longrightarrow r_4 \longrightarrow$ outlet

$$f_{s4} = \alpha\lambda_1\lambda_4\left[\frac{e^{-\alpha t}}{(\lambda_1-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_1 t}}{(\alpha-\lambda_1)(\lambda_4-\lambda_1)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_1-\lambda_4)}\right]$$

Path 5: $a_2 \longrightarrow r_2 \longrightarrow r_3 \longrightarrow r_4 \longrightarrow$ outlet

$$f_{s5} = \alpha\lambda_2\lambda_3\lambda_4\left[\frac{e^{-\alpha t}}{(\lambda_2-\alpha)(\lambda_3-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_3-\lambda_2)(\lambda_4-\lambda_2)} \right.$$
$$\left. + \frac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_2-\lambda_3)(\lambda_4-\lambda_3)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_2-\lambda_4)(\lambda_3-\lambda_4)}\right]$$

Path 6: $a_2 \longrightarrow r_2 \longrightarrow r_4 \longrightarrow$ outlet

$$f_{s6} = \alpha\lambda_2\lambda_4\left[\frac{e^{-\alpha t}}{(\lambda_2-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_2 t}}{(\alpha-\lambda_2)(\lambda_4-\lambda_2)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_2-\lambda_4)}\right]$$

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

$$f_{s7} = \alpha\lambda_3\lambda_4\left[\frac{e^{-\alpha t}}{(\lambda_3-\alpha)(\lambda_4-\alpha)} + \frac{e^{-\lambda_3 t}}{(\alpha-\lambda_3)(\lambda_4-\lambda_3)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)(\lambda_3-\lambda_4)}\right]$$

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

$$f_{s1} = \alpha\lambda_4\left[\frac{e^{-\alpha t}}{(\lambda_4-\alpha)} + \frac{e^{-\lambda_4 t}}{(\alpha-\lambda_4)}\right]$$

APPENDIX-D: Ordinates of Observed and Simulated Unit Hydrographs

1. Fettam Catchment

Time(hr)	Observed (m^3/s)	Simulated(m^3/s)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	1.69	2.99	4.20
2	3.47	8.35	8.86
3	9.61	11.34	10.20
4	12.62	11.36	9.15
5	10.45	9.68	7.17
6	7.53	7.50	5.15
7	5.59	5.47	3.49
8	4.35	3.83	2.26
9	3.45	2.60	1.42
10	2.87	1.73	0.87
11	2.45	1.13	0.52
12	2.19	0.73	0.30
13	1.95	0.47	0.18
14	1.73	0.30	0.10
15	1.50	0.19	0.06
16	1.28	0.12	0.03
17	1.08	0.07	0.02
18	0.90	0.05	0.01
19	0.77	0.03	0.01
20	0.62	0.02	0.00
21	0.50	0.01	0.00
22	0.39	0.01	0.00
23	0.31	0.00	0.00
24	0.24	0.00	0.00
25	0.18	0.00	0.00
26	0.13	0.00	0.00
27	0.10	0.00	0.00
28	0.07	0.00	0.00
29	0.05	0.00	0.00
30	0.02	0.00	0.00
31	0.01	0.00	0.00
32	0.01	0.00	0.00
33	0.00	0.00	0.00

2. Azuari

Time(hr)	Observed (m^3/s)	Simulated(m^3/s)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	0.54	0.62	0.95
2	1.50	2.11	2.69
3	2.47	3.81	4.24
4	3.77	5.19	5.26
5	5.70	5.98	5.73
6	6.18	6.19	5.73
7	5.97	5.95	5.42
8	5.17	5.42	4.91
9	4.24	4.75	4.31
10	3.42	4.04	3.69
11	2.72	3.36	3.09
12	2.15	2.74	2.55
13	1.71	2.21	2.07
14	1.38	1.76	1.66
15	1.12	1.39	1.32
16	0.89	1.09	1.04
17	0.71	0.84	0.81
18	0.56	0.65	0.63
19	0.39	0.50	0.48
20	0.20	0.39	0.37
21	0.16	0.30	0.28
22	0.14	0.23	0.21
23	0.12	0.17	0.16
24	0.10	0.13	0.12
25	0.08	0.10	0.09
26	0.06	0.08	0.07
27	0.05	0.06	0.05
28	0.03	0.04	0.04
29	0.02	0.03	0.03
30	0.00	0.02	0.02
31	0.00	0.02	0.02
32	0.00	0.01	0.01
33	0.00	0.01	0.01
34	0.00	0.01	0.01
35	0.00	0.01	0.00
36	0.00	0.00	0.00

3. Neshi Catchment			
Time(hr)	Observed (m^3/s)	Simulated(m^3/s)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	1.55	2.16	1.84
2	3.48	4.62	4.98
3	5.65	6.70	7.58
4	7.92	8.01	9.11
5	8.55	8.54	9.62
6	8.08	8.44	9.36
7	7.48	7.93	8.61
8	6.72	7.19	7.60
9	5.90	6.36	6.50
10	5.19	5.52	5.42
11	4.67	4.73	4.44
12	4.19	4.01	3.57
13	3.73	3.38	2.83
14	3.27	2.83	2.22
15	2.80	2.36	1.72
16	2.35	1.96	1.32
17	1.86	1.63	1.01
18	1.45	1.35	0.76
19	1.16	1.11	0.58
20	0.96	0.92	0.43
21	0.79	0.76	0.32
22	0.61	0.63	0.24
23	0.46	0.52	0.18
24	0.32	0.43	0.13
25	0.20	0.35	0.09
26	0.13	0.29	0.07
27	0.09	0.24	0.05
28	0.08	0.20	0.04
29	0.04	0.16	0.03
30	0.00	0.13	0.02
31	0.00	0.11	0.01
32	0.00	0.09	0.01
33	0.00	0.07	0.01
34	0.00	0.06	0.01
35	0.00	0.05	0.00
36	0.00	0.04	0.00
37	0.00	0.03	0.00
38	0.00	0.03	0.00
39	0.00	0.02	0.00
40	0.00	0.02	0.00
41	0.00	0.02	0.00
42	0.00	0.01	0.00
43	0.00	0.01	0.00
44	0.00	0.01	0.00
45	0.00	0.01	0.00
46	0.00	0.01	0.00
47	0.00	0.00	0.00

4. Andassa Catchment			
Time(hr)	Observed (m^3/s)	Simulated(m^3/s)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	0.68	2.08	2.33
2	2.37	7.20	6.96
3	5.97	12.90	11.34
4	10.62	17.16	14.41
5	15.23	19.23	15.97
6	18.74	19.36	16.24
7	20.10	18.13	15.56
8	19.38	16.15	14.27
9	17.12	13.87	12.66
10	14.17	11.58	10.94
11	11.42	9.46	9.25
12	9.32	7.61	7.69
13	7.71	6.03	6.30
14	6.35	4.73	5.10
15	5.34	3.67	4.08
16	4.46	2.83	3.23
17	3.72	2.17	2.54
18	3.04	1.65	1.98
19	2.48	1.25	1.54
20	2.01	0.95	1.19
21	1.63	0.71	0.91
22	1.30	0.53	0.69
23	1.02	0.40	0.53
24	0.78	0.30	0.40
25	0.58	0.22	0.30
26	0.57	0.16	0.23
27	0.48	0.12	0.17
28	0.43	0.09	0.13
29	0.39	0.07	0.09
30	0.34	0.05	0.07
31	0.31	0.04	0.05
32	0.27	0.03	0.04
33	0.24	0.02	0.03
34	0.21	0.01	0.02
35	0.18	0.01	0.02
36	0.15	0.01	0.01
37	0.12	0.01	0.01
38	0.09	0.00	0.01
39	0.08	0.00	0.00
40	0.06	0.00	0.00
41	0.05	0.00	0.00
42	0.03	0.00	0.00
43	0.02	0.00	0.00
44	0.01	0.00	0.00
45	0.00	0.00	0.00

5. Hoha catchment			
Time(hr)	Observed (m^3/s)	Simulated(m^3/s)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	0.12	0.28	0.35
2	0.20	0.94	1.09
3	0.49	1.70	1.88
4	0.85	2.42	2.56
5	1.87	3.01	3.05
6	3.19	3.40	3.34
7	3.88	3.58	3.46
8	3.89	3.59	3.43
9	3.67	3.47	3.29
10	3.24	3.25	3.08
11	2.81	2.97	2.82
12	2.51	2.67	2.54
13	2.09	2.37	2.26
14	1.89	2.07	1.98
15	1.66	1.80	1.72
16	1.49	1.55	1.48
17	1.32	1.32	1.26
18	1.17	1.13	1.07
19	1.01	0.95	0.90
20	0.88	0.80	0.76
21	0.75	0.68	0.63
22	0.64	0.57	0.52
23	0.54	0.48	0.43
24	0.49	0.40	0.35
25	0.44	0.33	0.29
26	0.38	0.28	0.24
27	0.35	0.23	0.19
28	0.32	0.19	0.16
29	0.28	0.16	0.13
30	0.27	0.13	0.10
31	0.25	0.11	0.08
32	0.23	0.09	0.07
33	0.21	0.08	0.05
34	0.19	0.06	0.04
35	0.17	0.05	0.03
36	0.16	0.04	0.03
37	0.14	0.04	0.02
38	0.12	0.03	0.02
39	0.11	0.03	0.01
40	0.09	0.02	0.01
41	0.08	0.02	0.01
42	0.07	0.01	0.01
43	0.07	0.01	0.01
44	0.06	0.01	0.00
45	0.05	0.01	0.00
46	0.04	0.01	0.00
47	0.04	0.01	0.00
48	0.03	0.00	0.00
49	0.02	0.00	0.00
50	0.02	0.00	0.00
51	0.01	0.00	0.00

6. Ukke Catchment			
Time(hr)	Observed (m^3/s)	Simulated(m^3/s)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	0.17	0.43	0.44
2	0.46	1.39	1.33
3	0.87	2.44	2.26
4	1.44	3.37	3.05
5	2.04	4.02	3.61
6	2.75	4.37	3.96
7	3.78	4.45	4.10
8	4.84	4.33	4.08
9	5.48	4.07	3.93
10	5.33	3.73	3.69
11	4.72	3.35	3.41
12	4.20	2.97	3.09
13	3.56	2.60	2.76
14	3.09	2.26	2.44
15	2.53	1.94	2.14
16	2.16	1.67	1.85
17	1.81	1.42	1.59
18	1.49	1.21	1.36
19	1.21	1.02	1.16
20	1.01	0.87	0.98
21	0.83	0.73	0.82
22	0.63	0.62	0.69
23	0.50	0.52	0.57
24	0.35	0.44	0.48
25	0.30	0.37	0.39
26	0.25	0.31	0.33
27	0.20	0.26	0.27
28	0.17	0.22	0.22
29	0.15	0.18	0.18
30	0.14	0.15	0.15
31	0.10	0.13	0.12
32	0.07	0.11	0.10
33	0.05	0.09	0.08
34	0.02	0.08	0.06
35	0.01	0.06	0.05
36	0.00	0.05	0.04
37	0.00	0.05	0.03
38	0.00	0.04	0.03
39	0.00	0.03	0.02
40	0.00	0.03	0.02
41	0.00	0.02	0.01
42	0.00	0.02	0.01
43	0.00	0.02	0.01
44	0.00	0.01	0.01
45	0.00	0.01	0.01
46	0.00	0.01	0.00
47	0.00	0.01	0.00
48	0.00	0.01	0.00
49	0.00	0.01	0.00
50	0.00	0.00	0.00

7. Jedeb			
		Simulated(m^3/s)	
Time(hr)	Observed(m^3/s)	GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	2.14	1.86	4.15
2	9.89	6.26	8.61
3	11.17	10.13	10.51
4	10.42	11.52	10.32
5	8.51	10.85	9.00
6	6.53	9.14	7.28
7	5.01	7.19	5.59
8	3.88	5.40	4.14
9	2.99	3.92	2.97
10	2.28	2.78	2.08
11	1.66	1.93	1.44
12	1.21	1.33	0.97
13	0.93	0.90	0.65
14	0.79	0.61	0.43
15	0.64	0.41	0.28
16	0.45	0.27	0.18
17	0.29	0.18	0.12
18	0.15	0.12	0.08
19	0.09	0.08	0.05
20	0.06	0.05	0.03
21	0.01	0.03	0.02
22	0.00	0.02	0.01
23	0.00	0.01	0.01
24	0.00	0.01	0.00
25	0.00	0.01	0.00
26	0.00	0.00	0.00

APPENDIX-E: Ordinates of Observed and Simulated Direct Runoff Hydrographs (DRH) for Ungauged Chemoga Catchment

Event 1. May, 15 1984					Event 2. July, 28 1985			
Time(hr)	Observed-DRH (m3/s)	Simulated-DRH(m^3/S) GIUH	Simulated-DRH(m^3/S) GIUH(Nash)		Time(hr)	Observed-DRH (m3/s)	Simulated-DRH(m^3/S) GIUH	Simulated-DRH(m^3/S) GIUH(Nash)
0	0.00	0.00	0.00		0	0.00	0.00	0.00
1	0.56	0.36	0.43		1	0.29	0.83	1.01
2	1.17	1.10	1.22		2	1.93	2.58	2.84
3	2.28	1.88	2.04		3	4.59	4.39	4.77
4	3.11	2.62	2.79		4	7.54	6.10	6.50
5	3.92	3.25	3.39		5	9.54	7.57	7.90
6	4.42	3.73	3.83		6	10.24	8.70	8.94
7	4.77	4.06	4.12		7	10.68	9.46	9.62
8	4.87	4.23	4.27		8	10.94	9.88	9.97
9	4.65	4.28	4.31		9	10.82	9.98	10.06
10	4.33	4.22	4.25		10	10.50	9.85	9.92
11	4.02	4.08	4.12		11	10.13	9.52	9.62
12	3.78	3.88	3.94		12	9.63	9.06	9.19
13	3.49	3.65	3.71		13	8.86	8.51	8.66
14	3.25	3.39	3.47		14	8.15	7.91	8.09
15	2.93	3.12	3.21		15	7.48	7.29	7.48
16	2.60	2.86	2.94		16	6.86	6.67	6.86
17	2.31	2.60	2.68		17	6.28	6.07	6.25
18	2.09	2.35	2.43		18	5.73	5.49	5.66
19	1.85	2.12	2.19		19	5.21	4.95	5.10
20	1.69	1.90	1.96		20	4.72	4.44	4.57
21	1.48	1.71	1.75		21	4.25	3.98	4.08
22	1.38	1.52	1.55		22	3.80	3.55	3.63
23	1.26	1.36	1.38		23	3.38	3.17	3.21
24	1.15	1.21	1.21		24	2.97	2.82	2.83
25	1.05	1.07	1.07		25	2.57	2.51	2.49
26	0.96	0.95	0.94		26	2.20	2.22	2.19
27	0.85	0.84	0.82		27	1.84	1.97	1.91
28	0.78	0.75	0.72		28	1.49	1.75	1.67
29	0.70	0.66	0.62		29	1.15	1.55	1.45
30	0.61	0.59	0.54		30	0.82	1.37	1.26
31	0.52	0.52	0.47		31	0.51	1.21	1.10
32	0.46	0.46	0.41		32	0.20	1.07	0.95
33	0.41	0.40	0.35		33	0.13	0.94	0.82
34	0.34	0.36	0.30		34	0.05	0.83	0.71

Event 1. May, 15 1984 Cont.					Event 2. July, 28 1985 Cont.			
Time(hr)	Observed DRH	GIUH	GIUH(Nash)		Time(hr)	Observed DRH	GIUH	GIUH(Nash)
35	0.27	0.32	0.26		35	0.00	0.74	0.61
36	0.26	0.28	0.22		36	0.00	0.65	0.52
37	0.20	0.25	0.19		37	0.00	0.57	0.45
38	0.17	0.22	0.17		38	0.00	0.51	0.39
39	0.13	0.19	0.14		39	0.00	0.45	0.33
40	0.09	0.17	0.12		40	0.00	0.39	0.28
41	0.05	0.15	0.10		41	0.00	0.35	0.24
42	0.00	0.13	0.09		42	0.00	0.31	0.21
43	0.00	0.12	0.08		43	0.00	0.27	0.18
44	0.00	0.10	0.06		44	0.00	0.24	0.15
45	0.00	0.09	0.05		45	0.00	0.21	0.13
46	0.00	0.08	0.05		46	0.00	0.19	0.11
47	0.00	0.07	0.04		47	0.00	0.16	0.09
48	0.00	0.06	0.03		48	0.00	0.14	0.08
49	0.00	0.05	0.03		49	0.00	0.13	0.07
50	0.00	0.05	0.02		50	0.00	0.11	0.06
51	0.00	0.04	0.02		51	0.00	0.10	0.05
52	0.00	0.04	0.02		52	0.00	0.09	0.04
53	0.00	0.03	0.01		53	0.00	0.08	0.03
54	0.00	0.03	0.01		54	0.00	0.07	0.03
55	0.00	0.03	0.01		55	0.00	0.06	0.02
56	0.00	0.02	0.01		56	0.00	0.05	0.02
57	0.00	0.02	0.01		57	0.00	0.05	0.02
58	0.00	0.02	0.01		58	0.00	0.04	0.01
59	0.00	0.01	0.00		59	0.00	0.03	0.01
60	0.00	0.01	0.00		60	0.00	0.03	0.00
61	0.00	0.01	0.00		61	0.00	0.02	0.00
62	0.00	0.01	0.00		62	0.00	0.01	0.00
63	0.00	0.00	0.00		63	0.00	0.01	0.00
64	0.00	0.00	0.00		64	0.00	0.00	0.00

Event 3. May, 3 1986			
Time(hr)	Observed-DRH (m3/s)	Simulated-DRH(m^3/S)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	3.47	1.09	1.33
2	8.22	3.37	3.71
3	10.83	5.74	6.24
4	12.72	7.99	8.51
5	13.54	9.91	10.34
6	14.29	11.39	11.70
7	14.64	12.38	12.59
8	14.73	12.92	13.05
9	14.34	13.06	13.16
10	13.74	12.88	12.99
11	12.81	12.46	12.59
12	12.11	11.85	12.02
13	11.42	11.14	11.34
14	10.62	10.35	10.58
15	9.62	9.54	9.79
16	8.92	8.73	8.98
17	7.99	7.94	8.18
18	7.06	7.18	7.41
19	5.94	6.48	6.68
20	5.24	5.82	5.98
21	4.35	5.21	5.34
22	3.46	4.65	4.75
23	2.58	4.15	4.20
24	2.20	3.69	3.71
25	1.71	3.28	3.26
26	0.85	2.91	2.86
27	0.61	2.58	2.50
28	0.40	2.28	2.19
29	0.32	2.02	1.90
30	0.24	1.79	1.65
31	0.12	1.58	1.44
32	0.00	1.40	1.24
33	0.00	1.23	1.07
34	0.00	1.09	0.93
35	0.00	0.96	0.80
36	0.00	0.85	0.69
37	0.00	0.75	0.59

Event 4. July, 29 1986			
Time(hr)	Observed-DRH (m3/s)	Simulated-DRH(m^3/S)	
		GIUH	GIUH(Nash)
0	0.00	0.00	0.00
1	0.74	3.27	3.97
2	1.07	10.10	11.12
3	2.27	17.21	18.69
4	6.80	23.93	25.48
5	11.86	29.68	30.99
6	28.50	34.11	35.04
7	33.42	37.10	37.70
8	36.21	38.71	39.10
9	39.31	39.14	39.44
10	40.20	38.60	38.91
11	39.15	37.32	37.70
12	36.53	35.51	36.01
13	33.93	33.36	33.96
14	32.35	31.01	31.70
15	29.36	28.58	29.32
16	26.46	26.15	26.90
17	25.01	23.78	24.52
18	22.25	21.52	22.20
19	20.87	19.40	20.00
20	19.51	17.42	17.93
21	17.56	15.60	16.00
22	16.26	13.93	14.22
23	15.58	12.42	12.59
24	14.31	11.05	11.11
25	13.07	9.82	9.77
26	12.42	8.72	8.57
27	11.77	7.73	7.50
28	10.57	6.84	6.55
29	10.21	6.06	5.70
30	9.31	5.36	4.96
31	9.24	4.74	4.30
32	7.56	4.19	3.72
33	6.96	3.70	3.22
34	6.36	3.27	2.78
35	6.28	2.88	2.39
36	5.82	2.55	2.06
37	5.61	2.25	1.77

Event 3. May, 3 1986			
Time(hr)	Observed DRH	GIUH	GIUH(Nash)
38	0.00	0.66	0.51
39	0.00	0.58	0.43
40	0.00	0.51	0.37
41	0.00	0.45	0.32
42	0.00	0.40	0.27
43	0.00	0.35	0.23
44	0.00	0.31	0.20
45	0.00	0.27	0.17
46	0.00	0.24	0.14
47	0.00	0.21	0.12
48	0.00	0.19	0.10
49	0.00	0.17	0.09
50	0.00	0.15	0.07
51	0.00	0.13	0.06
52	0.00	0.11	0.05
53	0.00	0.10	0.05
54	0.00	0.09	0.04
55	0.00	0.08	0.03
56	0.00	0.07	0.03
57	0.00	0.06	0.02
58	0.00	0.05	0.01
59	0.00	0.04	0.00
60	0.00	0.04	0.00
61	0.00	0.03	0.00
62	0.00	0.02	0.00
63	0.00	0.01	0.00
64	0.00	0.00	0.00

Event 4. July, 29 1986			
Time(hr)	Observed DRH	GIUH	GIUH(Nash)
38	5.03	1.98	1.52
39	4.45	1.75	1.30
40	3.38	1.54	1.11
41	3.30	1.36	0.95
42	2.26	1.20	0.81
43	2.18	1.06	0.69
44	1.16	0.93	0.59
45	1.08	0.82	0.50
46	0.54	0.73	0.43
47	0.00	0.64	0.36
48	0.00	0.56	0.31
49	0.00	0.50	0.26
50	0.00	0.44	0.22
51	0.00	0.39	0.19
52	0.00	0.34	0.16
53	0.00	0.30	0.14
54	0.00	0.27	0.11
55	0.00	0.23	0.10
56	0.00	0.21	0.08
57	0.00	0.18	0.07
58	0.00	0.16	0.05
59	0.00	0.13	0.04
60	0.00	0.10	0.03
61	0.00	0.08	0.01
62	0.00	0.05	0.00
63	0.00	0.03	0.00
64	0.00	0.00	0.00

APPENDIX-F: Results of Regression Analysis

SPSS Analysis for Simulated GIUH Velocity Regionalization

Correlations

		Vs	Area(km2)	Longest Flow Path Length(Km), L	Slope, S	RB	RL
Pearson Correlation	Vs	1.000	.622	.591	-.361	.674	.233
	Area(km2)	.622	1.000	.927	-.046	.042	.114
	Longest Flow Path Length(Km), L	.591	.927	1.000	-.017	.108	.256
	Slope, S	-.361	-.046	-.017	1.000	-.788	-.426
	RB	.674	.042	.108	-.788	1.000	.435
	RL	.233	.114	.256	-.426	.435	1.000

Variables Entered/Removed[b]

Model	Variables Entered	Variables Removed	Method
1	RL, Area(km2), Slope, S, RB, Longest Flow Path Length(Km), L[a]	.	Enter
2	.	RL	Stepwise (Criteria: Probability-of-F-to-enter <= .050, Probability-of-F-to-remove >= .100).
3	.	Longest Flow Path Length(Km), L	Stepwise (Criteria: Probability-of-F-to-enter <= .050, Probability-of-F-to-remove >= .100).
4	.	Slope, S	Stepwise (Criteria: Probability-of-F-to-enter <= .050, Probability-of-F-to-remove >= .100).

a. All requested variables entered.

b. Dependent Variable: Vs

Model Summary[e]										
					Change Statistics					
Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	R Square Change	F Change	df1	df2	Sig. F Change	Durbin-Watson
1	.963[a]	.927	.746	.628035	.927	5.102	5	2	.172	
2	.963[b]	.927	.830	.512790	.000	.000	1	2	.997	
3	.943[c]	.890	.807	.546881	-.038	1.550	1	3	.302	
4	.898[d]	.807	.730	.647294	-.083	3.005	1	4	.158	1.408

a. Predictors: (Constant), RL, Area(km2), Slope, S, RB, Longest Flow Path Length(Km), L

b. Predictors: (Constant), Area(km2), Slope, S, RB, Longest Flow Path Length(Km), L

c. Predictors: (Constant), Area(km2), Slope, S, RB

d. Predictors: (Constant), Area(km2), RB

e. Dependent Variable: Vs

Coefficients[a]								
		Unstandardized Coefficients		Standardized Coefficients			95.0% Confidence Interval for B	
Model		B	Std. Error	Beta	t	Sig.	Lower Bound	Upper Bound
1	(Constant)	-31.245	10.156		-3.077	.091	-74.943	12.453
	Area(km2)	.010	.005	1.121	1.876	.201	-.013	.032
	Longest Flow Path Length(Km), L	-.047	.052	-.562	-.904	.461	-.269	.176
	Slope, S	59.498	35.509	.584	1.676	.236	-93.285	212.281
	RB	9.253	2.703	1.148	3.424	.076	-2.375	20.882
	RL	-.006	1.494	.000	-.004	.997	-6.432	6.421
2	(Constant)	-31.257	7.863		-3.975	.028	-56.280	-6.235
	Area(km2)	.010	.004	1.122	2.509	.087	-.003	.022
	Longest Flow Path Length(Km), L	-.047	.038	-.563	-1.245	.302	-.167	.073

	Slope, S	59.540	27.472	.585	2.167	.119	-27.888	146.968
	RB	9.253	2.207	1.148	4.193	.025	2.231	16.276
3	(Constant)	-28.084	7.933		-3.540	.024	-50.109	-6.060
	Area(km2)	.005	.001	.600	3.610	.023	.001	.009
	Slope, S	47.576	27.447	.467	1.733	.158	-28.628	123.781
	RB	8.197	2.172	1.017	3.773	.020	2.166	14.229
4	(Constant)	-16.853	5.417		-3.111	.027	-30.777	-2.929
	Area(km2)	.005	.002	.594	3.021	.029	.001	.010
	RB	5.233	1.586	.649	3.300	.021	1.156	9.310
a. Dependent Variable: Vs								

Vs = -16.853 + 0.005A + 5.233RB

Casewise Diagnostics[a]

Case Number	Std. Residual	Vs	Predicted Value	Residual
1	1.349	4.120	3.24674	.873261
2	-.761	1.030	1.52261	-.492612
3	-1.297	2.770	3.60939	-.839388
4	-.533	1.890	2.23476	-.344764
5	.549	1.620	1.26469	.355313
6	.282	4.580	4.39727	.182728
7	.497	2.100	1.77834	.321661
a. Dependent Variable: Vs				

APPENDIX-G: Stream Flow Data

Fettam Catchment Stream flow Data

Site: Tililie

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
3/7/1985	4PM	1.26	13.26
	5	1.24	17.78
	5:30	1.33	14.9
	6	1.36	15.82
	6:30	1.64	24.3
	7	2.02	39.01
	7:30	2.14	44.13
	8	2.1	42.18
	8:30	2.01	38.57
	9	1.9	33.96
	9:30	1.81	30.42
	10	1.74	27.81
	10:30	1.67	25.33
	11	1.62	23.63
	11:30	1.58	22.32
	12	1.55	21.36
4/7/1985	12:30AM	1.54	20.89
	1	1.53	20.74
	2	1.53	20.61
	3	1.51	20.12
	4	1.5	19.91
	5	1.5	19.67
	6	1.48	19.07
	7	1.46	18.48
	8	1.44	17.96
	9	1.42	17.47
	10	1.41	17.05
	11	1.4	16.78
	12	1.39	16.5
	1PM	1.38	16.25
	2	1.37	15.98
	3	1.36	15.82
	4	1.35	15.42
	5	1.33	15.03
	6	1.32	14.66
	7	1.31	14.38
	8	1.31	14.38
	9	1.31	14.46
	12	1.31	14.46

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
8/7/1985	3PM	1.28	13.75
	4.00	1.28	13.75
	4:20	1.28	13.75
	4:30	1.32	14.77
	5.00	2.12	43.50
	5:30	2.44	59.68
	6.00	2.32	53.29
	6:30	2.22	48.26
	7.00	2.08	41.67
	7:30	1.92	34.77
	8.00	1.80	30.04
	8:30	1.71	26.74
	9.00	1.64	24.30
	9:30	1.59	22.64
	10.00	1.55	21.36
	10:30	1.51	20.12
	11.00	1.47	18.92
	11:30	1.45	18.34
	12.00	1.43	17.62
	0:30AM	1.41	17.19
	1.00	1.40	16.91
	0.06	1.39	16.72
	2.00	1.39	16.64
	3.00	1.39	16.55
	4.00	1.39	16.64
	5.00	1.40	16.78
	6.00	1.40	16.91
	7.00	1.40	16.91
	8.00	1.40	16.78
	9.00	1.39	16.55
	10.00	1.38	16.36
	11.00	1.38	16.36
	12.00	1.38	16.23
	1PM	1.37	16.14
	2.00	1.37	16.09
	3.00	1.36	15.82
	4.00	1.36	15.77
	5.00	1.35	15.55
	6.00	1.34	15.21
	7.00	1.32	14.77
	7:30	1.34	15.29

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
27/7/86	6PM	1.42	17.47
	7.00	1.36	15.82
	7:30	1.36	15.82
	8.00	1.43	17.76
	8:30	1.51	20.12
	9.00	1.56	21.68
	9:30	1.69	26.03
	10.00	1.93	35.19
	10:30	2.06	40.77
	11.00	2.15	44.90
	11:30	2.21	47.77
	12.00	2.21	47.77
28/7/86	0:30AM	2.15	44.90
	1.00	2.05	40.33
	1:30	1.94	35.60
	2.00	1.86	32.36
	2:30	1.81	30.42
	3.00	1.81	30.23
	3:30	1.81	30.42
	4.00	1.80	30.04
	4:30	1.77	28.92
	5.00	1.73	27.45
	5:30	1.69	26.03
	6.00	1.66	24.99
	6:30	1.64	24.30
	7.00	1.61	23.30
	8.00	1.60	22.81
	9.00	1.59	22.48
	10.00	1.58	22.16
	11.00	1.56	21.52
	12.00	1.53	20.74
	1PM	1.50	19.82
	2.00	1.48	19.22

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
28/7/86	2:30PM	1.47	18.92
	3.00	1.52	20.43
	3:30	1.99	37.71
	4.00	2.25	49.74
	5.00	2.37	55.90
	5:30	2.49	62.18
	6.00	2.33	53.81
	6:30	2.13	43.96
	7.00	2.06	40.55
	7:30	2.06	40.77
	8.00	2.04	39.88
	8:30	1.98	37.28
	9.00	1.90	33.96
	9:30	1.83	31.19
	10.00	1.79	29.47
	10:30	1.76	28.54
	11.00	1.74	27.81
	12.00	1.73	27.27
29/7/86	1AM	1.71	26.74
	2.00	1.68	25.68
	3.00	1.65	24.47
	4.00	1.61	23.30
	5.00	1.58	22.32
	6.00	1.56	21.68
	7.00	1.55	21.21
	8.00	1.54	20.89
	9.00	1.53	20.74
	10.00	1.52	20.43
	11.00	1.51	20.12
	12.00	1.50	19.82
	1PM	1.49	19.52
	1:30	1.49	19.37
	2.00	1.49	19.37
	2:30	1.79	29.66

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
17/7/85	1PM	1.42	17.47
	2.00	1.41	17.19
	2:30	1.61	23.30
	3.00	1.74	27.81
	3:30	1.96	36.44
	4.00	2.07	41.22
	4:30	2.44	59.68
	5.00	2.33	53.81
	5:30	2.32	53.03
	6.00	2.30	52.01
	6:30	2.24	49.25
	7.00	2.13	43.96
	7:30	2.04	39.88
	8.00	1.98	37.28
	8:30	1.94	35.60
	9.00	1.91	34.37
	9:30	1.88	33.16

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
5/7/1989	1PM	1.49	19.52
	2.00	1.49	19.37
	3.00	1.50	19.82
	3:30	2.01	38.57
	4.00	2.51	63.59
	4:30	2.69	74.25
	5.00	3.14	104.90
	5:30	3.15	105.60
	6.00	3.57	139.50
	6:30	4.08	187.50
	7.00	4.31	211.70
	7:30	4.21	201.00
	8.00	3.95	174.60
	8:30	3.79	159.30
	9.00	3.57	139.50
	9:30	3.33	119.50
	10.00	3.11	102.60

	10.00	1.84	31.58
Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
17/7/85	10:30	1.80	30.04
	11.00	1.77	28.73
	11:30	1.73	27.45
	12.00	1.71	26.74
18/7/85	0:30AM	1.70	26.20
	1.00	1.69	25.85
	1:30	1.68	25.68
	2.00	1.67	25.16
	3.00	1.65	24.75
	4.00	1.65	24.75
	4:30	1.66	24.99
	6.00	1.66	24.99
	7.00	1.63	23.97
	8.00	1.59	22.64
	9.00	1.56	21.52
	10.00	1.53	20.58
	11.00	1.50	19.82
	12.00	1.48	19.07
	1PM	1.47	18.77
	2.00	1.45	18.34
	3.00	1.43	17.76
	4.00	1.42	17.47
	5.00	1.40	16.91
	6.00	1.39	16.64
	7.00	1.38	16.23
	8.00	1.37	15.96
	9.00	1.36	15.82
	10.00	1.35	15.55
	10:30	1.41	17.19

	10:30	2.95	91.25
Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
5/7/1989	11.00	2.85	84.49
	11:30	2.73	76.74
	12.00	2.66	72.41
6/7/1989	0:30AM	2.61	69.40
	1.00	2.57	67.04
	1:30	2.52	64.16
	2.00	2.50	63.02
	2:30	2.46	60.78
	3.00	2.43	59.13
	3:30	2.42	58.59
	4.00	2.41	57.78
	4:30	2.39	56.97
	5.00	2.37	55.90
	5:30	2.34	54.33
	6.00	2.32	53.03
	6:30	2.28	51.25
	7.00	2.25	49.74
	7:30	2.22	48.02
	8.00	2.18	46.33
	8:30	2.14	44.43
	9.00	2.10	42.58
	9:30	2.06	40.55
	10.00	2.03	39.44
	10:30	2.00	38.14
	11.00	1.97	36.86
	11:30	1.95	36.02
	12.00	1.92	34.77
	0:30PM	1.90	33.96
	1.00	1.88	33.16
	2.00	1.83	31.19
	2:30	1.83	31.19
	3.00	1.89	33.56

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
15/11/88	6PM	0.69	3.17
	7.00	0.68	3.06
	7:55	0.68	3.06
	8.00	1.88	33.16
	8:30	1.95	36.02
	9.00	2.00	38.14
	9:30	2.04	39.88
	10.00	2.12	43.50
	10:30	2.17	45.85
	11.00	2.18	46.09
	11:30	2.17	45.85
	12.00	2.06	40.77
16/11/88	0:30AM	1.93	35.19
	1.00	1.84	31.58
	1:30	1.76	28.54
	2.00	1.71	26.74
	2:30	1.67	25.16

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
8/10/1985	3PM	1.08	9.16
	4.00	1.07	9.06
	4:30	1.10	9.67
	5.00	1.45	18.34
	5:30	1.65	24.64
	6.00	1.80	30.04
	6:30	1.87	32.76
	7.00	1.87	32.76
	7:30	1.82	30.80
	8.00	1.79	29.66
	8:30	1.77	28.92
	10:20	1.77	28.92
	10:30	1.76	28.54
	11.00	1.74	27.81
	11:30	1.72	27.09
	12.00	1.70	26.38
9/10/1985	0:30AM	1.70	26.20

	3.00	1.63	23.97
	3:30	1.60	22.97
Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
	4.00	1.58	22.16
	4:30	1.56	21.52
	5.00	1.54	20.89
	6.00	1.49	19.52
	7.00	1.44	18.05
	8.00	1.39	16.64
	9.00	1.34	15.29
	10.00	1.31	14.38
	11.00	1.27	13.51
	12.00	1.23	12.54
	1PM	1.19	11.68
	2.00	1.14	10.51
	3.00	1.10	9.67
	4.00	1.06	8.86
	5.00	1.03	8.19
	6.00	1.00	7.73
	7.00	0.97	7.24
	8.00	0.95	6.76
	9.00	0.92	6.34
	10.00	0.90	5.97
	11.00	0.88	5.70
	12.00	0.86	5.40
17/11/88	1AM	0.85	5.30
	2.00	0.85	5.18
	3.00	0.84	5.03
	4.00	0.83	4.89
	5.00	0.82	4.75
	6.00	0.81	4.68
	7.00	0.80	4.54
	8.00	0.79	4.40
	9.00	0.78	4.27
	10.00	0.77	4.14
	11.00	0.77	4.10
	12.00	0.76	4.05
	1PM	0.76	4.01
	2.00	0.76	4.01
	3.00	0.76	3.95
	4.00	0.75	3.91
	5.00	0.75	3.84
	6.00	0.74	3.76
	8.00	0.74	3.76
	9.00	0.73	3.67
	10.00	0.73	3.64
	11.00	0.73	3.59
	12.00	0.72	3.54
18/11/88	1AM	0.72	3.52
	2.00	0.72	3.49
	3.00	0.71	3.43
	4.00	0.71	3.40
	5.00	0.71	3.34
	6.00	0.70	3.30
	7.00	0.70	3.28
	11.00	0.70	3.28

	1.00	1.69	25.85
	2.00	1.66	24.81
Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
	3.00	1.61	23.30
	4.00	1.58	22.16
	5.00	1.54	20.89
	6.00	1.49	19.52
	7.00	1.45	18.19
	8.00	1.41	17.19
	9.00	1.38	16.36
	10.00	1.35	15.55
	11.00	1.32	14.77
	12.00	1.30	14.20
	1PM	1.27	13.51
	2.00	1.25	12.90
	3.00	1.22	12.30
	4.00	1.20	11.84
	5.00	1.18	11.39
	6.00	1.17	11.05
	7.00	1.15	10.62
	8.00	1.13	10.29
	9.00	1.12	10.08
	10.00	1.11	9.87
	11.00	1.10	9.67
	12.00	1.09	9.46
10/10/1985	1AM	1.08	9.26
	2.00	1.07	9.06
	3.00	1.06	8.92
	4.00	1.06	8.80
	5.00	1.05	8.67
	6.00	1.05	8.57
	7.00	1.04	8.47
	8.00	1.04	8.47
	9.00	1.04	8.42
	10.00	1.03	8.19
	12.00	1.03	8.19
	1PM	1.02	8.10
	5.00	1.02	8.10
	6.00	1.01	7.91
	7.00	1.01	7.82
	8.00	1.00	7.73
	9.00	0.99	7.60
	10.00	0.99	7.55
	11.00	0.99	7.47
	12.00	0.98	7.37
11/10/1985	1AM	0.98	7.37
	2.00	0.97	7.24
	3.00	0.97	7.19
	4.00	0.97	7.14
	5.00	0.96	7.07
	6.00	0.96	7.02
	8.00	0.96	7.02
	9.00	0.96	6.93
	10.00	0.95	6.78
	11.00	0.94	6.72
	12.00	0.94	6.72

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
	12.00	0.70	3.24
	1PM	0.69	3.20
	2.00	0.69	3.20

Continued

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
	3.00	0.69	3.17
	5.00	0.69	3.17
	6.00	0.69	3.12
	7.00	0.68	3.09
	8.00	0.68	3.06
	11.00	0.68	3.06
	12.00	0.68	3.01
19/11/88	1AM	0.67	2.95
	3.00	0.67	2.95
	4.00	0.67	2.89
	6.00	0.67	2.89
	7.00	0.66	2.84
	12.00	0.66	2.84

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
	1PM	0.94	6.67
	6.00	0.94	6.67
	7.00	0.94	6.59
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
	8.00	0.93	6.51

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
11/9/1986	1PM	1.37	16.09
	2.00	1.37	15.96
	2:30	1.37	16.09
	3.00	1.49	19.52
	3:30	1.82	30.80
	4.00	2.09	42.12
	4:30	2.21	47.77
	5.00	2.26	50.24
	5:30	2.31	52.78
	6.00	2.36	55.11
	6:30	2.35	54.85
	7.00	2.21	47.77
	7:30	2.06	40.77
	8.00	1.93	35.19
	8:30	1.86	32.36
	9.00	1.83	31.00
	9:30	1.80	30.04
	10.00	1.79	29.66
	10:30	1.77	28.92
	11.00	1.75	28.18
	11:30	1.73	27.27
	12.00	1.71	26.56
	1AM	1.67	25.33
	2.00	1.64	24.41
	3.00	1.61	23.14
	4.00	1.57	21.84
	5.00	1.53	20.83
	6.00	1.49	19.52
	7.00	1.47	18.77
	8.00	1.44	18.13
	9.00	1.42	17.56
	10.00	1.41	17.05

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
24/7/87	1PM	1.400	16.91
	2.00	1.390	16.64
	2:30	1.460	18.63
	3.00	1.580	22.32
	3:30	1.790	29.66
	4.00	2.120	43.50
	4:30	2.250	49.74
	5.00	2.290	51.75
	5:30	2.320	53.29
	6.00	2.500	63.02
	6:30	2.760	78.64
	7.00	2.880	86.48
	7:30	2.790	80.57
	8.00	2.680	73.64
	8:30	2.560	66.46
	9.00	2.500	63.02
	9:30	2.350	54.85
	10.00	2.230	48.75
	10:30	2.110	43.04
	11.00	2.030	39.44
	11:30	1.970	36.86
	12.00	1.920	34.77
25/7/87	0:30AM	1.885	33.36
	1.00	1.860	32.36
	2.00	1.820	30.80
	3.00	1.800	30.04
	4.00	1.770	28.92
	5.00	1.745	27.99
	6.00	1.727	27.34
	7.00	1.720	27.09
	8.00	1.715	26.91
	9.00	1.713	26.84

Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
11/9/1986	11.00	1.39	16.64
	12.00	1.38	16.23
12/9/1986	1PM	1.36	15.90
	1:30	1.37	16.09
	2.00	1.37	16.09
	2:30	1.41	17.19

Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
25/7/87	10.00	1.710	26.74
	11.00	1.700	26.38
	12.00	1.693	26.13
	1PM	1.680	25.68
	1:30	1.700	26.38

Hoha Catchment Stream flow Data

Site: Nr. Asossa

Date	Time (hr)	Gauge Height(m)	Discharge(m3/s)
28/6/85	3	1.080	0.71
	3:30	1.160	1.71
	3:45	1.215	2.75
	4	1.190	2.24
	4:30	1.160	1.71
	5	1.190	2.24
	6AM	1.215	2.75
	7	1.226	3
	8	1.268	4.06
	9	1.390	8.35
	10	1.470	12.26
	11	1.510	14.58
	12AM	1.535	16.16
	1PM	1.550	17.15
	2	1.556	17.56
	3	1.553	17.36
	4	1.550	17.15
	5	1.545	16.82
	6PM	1.540	16.49
	7	1.537	16.29
	8	1.533	16.03
	9	1.530	15.84
	10	1.525	15.52
	11	1.515	14.89
	12PM	1.490	13.39
29/6/85	1AM	1.470	12.26
	2	1.450	11.2
	3	1.430	10.19
	4	1.410	9.24
	5	1.390	8.35
	6AM	1.370	7.51
	7	1.360	7.11
	8	1.345	6.54
	9	1.335	6.18
	10	1.320	5.65
	11	1.310	5.32
	12AM	1.305	5.16
	1	1.295	4.84
	2	1.285	4.54
	3	1.280	4.4

Date	Time(hr)	Gauge Height(m)	Discharge(m^3/s)
1/7/1985	4AM	1.120	1.14
	5	1.120	1.14
	6AM	1.120	1.14
	6:30	1.120	1.14
	7	1.160	1.71
	8	1.190	2.24
	9	1.195	2.34
	10	1.200	2.44
	11	1.250	3.58
	12AM	1.345	6.54
	1PM	1.430	10.19
	2	1.480	12.82
	3	1.505	14.28
	4	1.530	15.84
	5	1.535	16.22
	6PM	1.530	15.84
	7	1.520	15.2
	8	1.505	14.28
	9	1.490	13.39
	10	1.470	12.26
	11	1.455	11.46
	12PM	1.430	10.19
2/7/1985	1AM	1.410	9.24
	2	1.385	8.13
	3	1.365	7.31
	4	1.350	6.73
	5	1.340	6.36
	6AM	1.325	5.82
	7	1.315	5.52
	8	1.305	5.16
	9	1.296	4.87
	10	1.285	4.54
	11	1.280	4.4
	12	1.275	4.25
	1PM	1.265	3.97
	2	1.260	3.84
	3	1.256	3.73
	4	1.250	3.58
	5	1.243	3.4
	6PM	1.240	3.33

Continued			
Date	Time (hr)	Gauge Height(m)	Discharge(m3/s)
29/6/85	4	1.273	4.2
	5	1.265	3.97
	6PM	1.260	3.84
	7	1.250	3.58
	8	1.240	3.33
	9	1.230	3.09
	10	1.220	2.86
	11	1.213	2.71
	12PM	1.205	2.54
30/6/85	1AM	1.200	2.44
	2	1.195	2.34
	3	1.185	2.15
	4	1.180	2.06
	5	1.175	1.97
	6AM	1.170	1.88
	7	1.163	1.76
	8	1.160	1.71
	9	1.157	1.67
	10	1.153	1.6
	11	1.150	1.56
	12AM	1.145	1.48
	1PM	1.142	1.44
	2	1.140	1.41
	3	1.140	1.41
	4	1.140	1.41
	5	1.140	1.41
	6PM	1.140	1.41

Continued			
Date	Time (hr)	Gauge Height(m)	Discharge(m3/s)
2/7/1985	7	1.233	3.16
	8	1.225	2.97
	9	1.220	2.86
	10	1.215	2.75
	11	1.210	2.65
	12PM	1.205	2.54
3/7/1985	1	1.200	2.44
	2	1.197	2.38
	3	1.193	2.3
	4	1.190	2.24
	5	1.186	2.17
	6AM	1.183	2.11
	7	1.180	2.06
	8	1.178	2.02
	9	1.173	1.93
	10	1.168	1.85
	11	1.163	1.76
	12AM	1.160	1.71
	1	1.160	1.71
	1:30	1.160	1.71
	2	1.196	2.36
	2:30	1.180	2.06
	3	1.183	2.11
	4	1.180	2.06
	5	1.176	1.99
	6PM	1.167	1.83
	7	1.163	1.76
	8	1.160	1.71
	9	1.160	1.71
	10	1.160	1.71
	11	1.163	1.76
	12PM	1.165	1.8
4/7/1985	1AM	1.173	1.93
	2	1.180	2.06

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
11/8/1985	1	1.240	3.33
	2	1.240	3.33
	2:30	1.240	3.33
	3	1.280	4.4
	3:30	1.300	5
	4	1.270	4.11
	5	1.290	4.69
	6AM	1.320	5.65
	7	1.360	7.11
	8	1.700	29.24
	8:30	1.810	40.77
	9	1.760	35.24
	9:30	1.750	34.19
	10	1.780	37.39
	10:30	1.790	38.5

Date	Hour	Gauge Height(m)	Discharge(m3/s)
14/8/85	7PM	1.230	8.7
	8	1.225	8.62
	9	1.230	8.7
	10	1.300	9.87
	11	1.500	13.61
	12PM	1.600	15.7
	12:30	1.620	16.14
15/8/85	1AM	1.590	15.48
	2	1.550	14.64
	3	1.500	13.61
	4	1.460	12.81
	5	1.420	12.04
	6AM	1.380	11.3
	7	1.360	10.93
	8	1.330	10.39

Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
11/8/1985	11	1.785	37.94
	12AM	1.760	35.24
	1PM	1.720	31.16
	2	1.660	25.63
	3	1.640	23.93
	4	1.585	19.62
	5	1.550	17.15
	6PM	1.530	15.84
	7	1.500	13.98
	8	1.480	12.82
	9	1.460	11.72
	10	1.435	10.43
	11	1.415	9.47
	12AM	1.390	8.35
	1	1.360	7.11
	2	1.350	6.73
	3	1.360	7.11
	4	1.345	6.54

Continued			
Date	Hour	Gauge Height(m)	Discharge(m3/s)
15/8/85	9	1.315	10.13
	10	1.300	9.87
	11	1.290	9.7
	12AM	1.275	9.45
	1	1.260	9.2
	2	1.250	9.03
	3	1.240	8.87
	4	1.230	8.7
	5	1.225	8.62
	6PM	1.215	8.46
	7	1.205	8.31
	8	1.200	8.23
	9	1.195	8.15
	10	1.190	8.07
	11	1.190	8.07
	12AM	1.190	8.07

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
12/8/1985	5AM	1.350	6.73
	6	1.370	7.51
	6:30	1.400	8.79
	7	1.400	8.79
	8	1.460	11.72
	9	1.640	23.93
	10	1.680	27.4
	11	1.720	31.16
	12AM	1.800	39.62
	12:30	1.830	43.13
	1	1.805	40.19
	2	1.780	37.39
	3	1.740	33.16
	4	1.720	31.16
	5	1.725	31.66
	6PM	1.740	33.16
	7	1.735	32.65
	8	1.720	31.16
	9	1.700	29.24
	10	1.660	25.63
	11	1.630	23.11
	12AM	1.600	20.74
13/8/85	1	1.580	15.27
	2	1.560	14.85

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
25/8/85	12AM	1.120	7.02
	1	1.120	7.02
	2	1.120	7.02
	2:30	1.430	12.23
	3	1.210	8.38
	3:40	1.360	10.93
	4	1.340	10.57
	5	1.580	15.27
	6PM	2.050	26.94
	7	2.000	25.55
	8	2.120	28.96
	9	2.080	27.8
	10	1.980	25
	11	1.900	22.86
	12PM	1.790	20.08
26/8/85	1AM	1.690	17.71
	2	1.630	16.36
	3	1.590	15.48
	4	1.550	14.64
	5	1.520	14.01
	6AM	1.485	13.31
	7	1.450	12.62
	8	1.420	12.04
	9	1.390	11.48

Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
13/8/85	3	1.540	14.43
	4	1.520	14.01
	5	1.500	13.61
	6AM	1.485	13.31
	7	1.480	13.21
	8	1.470	13.01
	9	1.450	12.62
	10	1.435	12.33
	11	1.420	12.04
	12AM	1.405	11.76
	1	1.390	11.48
	2	1.380	11.3
	3	1.370	11.11
	4	1.360	10.93
	5	1.350	10.75
	6PM	1.340	10.57
	7	1.330	10.39
	8	1.320	10.22
	9	1.315	10.13
	10	1.310	10.04
	11	1.300	9.87
	12PM	1.295	9.79
14/8/85	1AM	1.290	9.53
	2	1.280	9.53
	3	1.270	9.36
	4	1.265	9.28
	5	1.263	9.25
	6AM	1.260	9.2
	7	1.260	9.2
	8	1.258	9.16
	9	1.254	9.1
	10	1.250	9.03
	11	1.245	8.95
	12AM	1.242	8.9
	1	1.240	8.87
	2	1.240	8.87
	3	1.245	8.95
	4	1.240	8.87
	5	1.230	8.7
	6PM	1.230	8.7

Continued			
Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
26/8/85	10	1.370	11.11
	11	1.340	10.57
	12AM	1.320	10.22
	1	1.310	10.04
	2	1.300	9.87
	3	1.290	9.7
	4	1.280	9.53
	5	1.270	9.36
	6PM	1.260	9.2
	7	1.250	9.03
	8	1.240	8.87
	9	1.230	8.7
	10	1.215	8.46
	11	1.200	8.23
	12PM	1.190	8.07
27/8/85	1	1.180	7.92
	2	1.170	7.76
	3	1.160	7.61
	4	1.150	7.46
	5	1.145	7.38
	6AM	1.140	7.31
	7	1.145	7.38
	8	1.140	7.31
	9	1.140	7.31
	10	1.130	7.16
	11	1.130	7.16
	12AM	1.125	7.09
	1PM	1.125	7.09
	2	1.120	7.02
	3	1.120	7.02
	4	1.110	6.87
	5	1.110	6.87
27/8/85	6PM	1.110	6.87
	7	1.110	6.87
	8	1.110	6.87
	9	1.110	6.87

Neshi Catchment Streamflow Data

Site: Nr. Shambo

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
9/6/1987	3AM	1.520	7.42
	4	1.520	7.42
	5	1.540	7.71
	7	1.540	7.71
	8	1.610	8.79
	9	1.810	12.31
	10	1.870	13.49
	12	1.870	13.49
	1PM	1.850	13.09
	2	1.830	12.7
	3	1.810	12.31
	4	1.790	11.93
	5	1.790	11.93
	6	1.770	11.55
	7	1.750	11.18
	8	1.750	11.18
	9	1.730	10.82
	10	1.745	11.09
	11	1.710	10.47
10/6/1987	1AM	1.710	10.47
	2	1.700	10.29
	3	1.690	10.12
10/6/1987	6	1.690	10.12
	7	1.685	10.03
	8	1.680	9.95
	9	1.680	9.95
	10	1.675	9.86
	11	1.670	9.78
	12	1.670	9.78

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
3/7/1987	10PM	1.430	6.17
4/7/1987	1AM	1.430	6.17
	2	1.610	8.79
	3	1.670	9.78
	4	1.700	10.29
	5	1.710	10.47
	6	1.710	10.47
	7	1.700	10.29
	8	1.690	10.12
	9	1.650	9.44
	10	1.620	8.95
	11	1.610	8.79
	12	1.600	8.63
	1PM	1.590	8.47
	2	1.575	8.24
	3	1.570	8.16
	4	1.550	7.86
	5	1.550	7.86
	6	1.535	7.64
	7	1.530	7.56
	8	1.525	7.49
	9	1.510	7.27
4/7/1987	10	1.510	7.27
	11	1.500	7.13
	12	1.490	6.99

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
4/7/1987	12PM	1.490	6.990
5/7/1987	1AM	1.620	8.950
	2	1.700	10.290
	3	1.740	11.000
	4	1.760	11.370
	5	1.780	11.740
	6	1.820	12.500
	7	1.900	14.100
	8	1.930	14.730
	9	1.930	14.730
	10	1.910	14.310
	11	1.880	13.690
	12	1.840	12.890
	1PM	1.800	12.120
	2	1.780	11.740
	3	1.760	11.370
	4	1.740	11.000
	5	1.720	10.650
	6	1.700	10.290
	7	1.680	9.950
	8	1.660	9.610
	9	1.640	9.280
	10	1.620	8.950
	11	1.610	8.790
	12	1.600	8.630
6/7/1987	1AM	1.590	8.470
	2	1.580	8.320
	3	1.570	8.160
	5	1.570	8.160
	6	1.560	8.010
	8	1.560	8.010
	9	1.550	7.860
	10	1.540	7.710
	2PM	1.540	7.710
	3	1.530	7.560
	4	1.530	7.560
	5	1.520	7.420
	7	1.520	7.420
	8	1.510	7.270
	9	1.500	7.130
	12	1.500	7.130

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
4/8/1987	9PM	1.900	14.1
	10	1.900	14.1
	11	1.895	14
	12	1.890	13.9
5/8/1987	1AM	2.040	17.16
	2	2.120	19.06
	3	2.150	19.8
	4	2.160	20.05
	5	2.140	19.55
	6	2.120	19.06
	7	2.110	18.82
	8	2.100	18.57
	9	2.095	18.45
	10	2.090	18.33
	11	2.080	18.1
	12	2.070	17.86
	1PM	2.060	17.63
	2	2.040	17.16
	3	2.020	16.7
	4	2.000	16.26
	5	1.980	15.81
	6	1.960	15.37
	7	1.955	15.27
	8	1.940	14.94
	9	1.940	14.94
	10	1.930	14.73
	11	1.920	14.52
	12	1.915	14.42
6/8/1987	2AM	1.915	14.42
	3	1.910	14.31
	4	1.910	14.31
	5	1.900	14.1
	9	1.900	14.1
	10	1.915	14.42
	11	1.910	14.31
	2PM	1.910	14.31
	3	1.900	14.1
	4	1.900	14.1
	5	1.890	13.9
	6	1.880	13.69
	10	1.880	13.69
	11	1.870	13.49
7/8/1987	5AM	1.870	13.49

Ukke Catchment Stream flow Data

Site: Nr. Nekemte

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
6/9/1988	6AM	1.510	31.05
	7	1.510	31.05
	8	1.530	32.17
	9	1.570	34.48
	10	1.610	36.88
	11	1.660	39.99
	12	1.730	44.58
	1PM	1.830	51.58
	2	1.870	54.54
	3	1.890	56.05
	4	1.870	54.54
	5	1.830	51.58
	6	1.810	50.14
	7	1.770	47.31
	8	1.750	45.93
	9	1.710	43.24
	10	1.690	41.92
	11	1.670	40.63
	12	1.650	39.36
7/9/1988	1AM	1.630	38.11
	2	1.610	36.88
	3	1.610	36.88
	4	1.590	35.67
	5	1.590	36.67
	6	1.580	35.07
	7	1.570	34.48
	8	1.570	34.48
	9	1.560	33.9
	10	1.550	33.32
	12	1.550	33.32
	1PM	1.540	32.74
	6	1.520	31.61
	7	1.525	31.89
	8	1.530	32.17
	9	1.550	33.32

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
22/9/88	7AM	1.83	51.58
	8	1.86	53.79
	9	1.94	59.93
	10	2.02	66.41
	12	2.02	66.41
	1PM	2.04	68.09
	2	2.14	76.8
	3	2.22	84.18
	4	2.28	89.95
	5	2.28	89.95
	6	2.24	86.08
	7	2.18	80.45
	8	2.12	75.02
	9	2.08	71.51
	10	2.04	68.09
	11	2.00	64.75
	12	1.98	63.12
23/9/88	1AM	1.96	61.51
	2	1.94	59.93
	3	1.92	58.36
	4	1.92	58.36
	5	1.90	56.82
	7	1.90	56.82
	8	1.89	56.05
	9	1.88	55.29
	1PM	1.88	55.29
	2	1.90	56.82

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
9/9/1988	7PM	1.49	29.95
	8	1.50	30.5
	10	1.50	30.5
	11	1.51	31.05
	12	1.53	32.17
10/9/1988	1AM	1.54	32.74
	2	1.57	34.48
	3	1.61	36.88
	4	1.69	41.92
	5	1.81	50.14
	6	1.89	56.05
	7	1.93	59.14
	8	1.94	59.93
	9	1.91	57.58
	10	1.87	54.54
	11	1.81	50.14
	12	1.77	47.31
	1PM	1.73	44.58
	2	1.71	43.24
	3	1.68	41.27
	4	1.66	39.99
	5	1.65	39.36
	6	1.63	38.11
	7	1.62	37.49
	8	1.60	36.27
	9	1.59	35.67
	10	1.58	35.07
	11	1.58	35.07
	12	1.57	34.48
11/9/1988	1AM	1.57	34.48
	2	1.56	33.9
	3	1.56	33.9
	4	1.55	33.32
	5	1.55	33.32
	6	1.54	32.74
	11	1.54	32.74
	12	1.53	32.17
	5PM	1.53	32.17
	6	1.54	32.74
	7	1.55	33.32
	8	1.56	33.9

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
17/9/88	4AM	1.61	36.88
	5	1.61	36.88
	6	1.62	37.49
	7	1.65	39.36
	8	1.69	41.92
	9	1.75	45.93
	10	1.81	50.14
	11	1.95	60.72
	12	2.01	65.58
	1PM	2.05	68.93
	2	2.13	75.91
	3	2.21	83.24
	4	2.24	86.08
	5	2.23	85.13
	6	2.19	81.37
	7	2.13	75.91
	8	2.07	70.64
	9	2.03	67.25
	10	2.19	61.37
	11	1.95	60.72
	12	1.94	59.93
18/9/88	1AM	1.93	59.14
	2	1.93	59.14
	3	1.92	58.36
	4	1.91	57.58
	5	1.89	56.05
	6	1.87	54.54
	7	1.86	53.79
	8	1.84	52.32
	9	1.85	53.05
	10	1.84	52.32
	11	1.84	52.32
	12	1.83	51.58
	1PM	1.83	51.58
	2	1.82	50.86
	3	1.81	50.14
	4	1.81	50.14
	5	1.80	49.42
	1AM	1.80	49.42
	2	1.82	50.86

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
3/10/1988	8PM	1.58	35.07
	9	1.57	34.48
	10	1.56	33.9
	11	1.57	34.48
	12	1.59	35.67
4/10/1988	1AM	1.61	36.88
	2	1.67	40.63
	3	1.75	45.93
	4	1.76	46.62
	5	1.75	45.93
	6	1.77	47.31
	7	1.89	56.05
	8	1.96	61.51
	9	1.95	60.72
	10	1.91	57.58
	11	1.89	56.05
	12	1.83	51.58
	1PM	1.80	49.42
	2	1.77	47.31
	3	1.75	45.93
	4	1.74	45.25
	5	1.74	44.91
	6	1.73	44.58
	7	1.73	44.58
	8	1.72	43.9
	9	1.71	43.24
	10	1.69	41.92
	11	1.68	41.27
	12	1.66	39.99

Date	Time(hr)	Gauge Height(m)	Discharge(m3/s)
28/9/89	11PM	1.60	36.27
	12	1.60	36.27
29/9/89	1AM	1.61	36.88
	2	1.63	38.11
	3	1.65	39.36
	4	1.69	41.92
	5	1.74	45.25
	6	1.79	48.72
	7	1.83	51.58
	8	1.85	53.05
	9	1.85	53.05
	10	1.83	51.58
	11	1.81	50.14
	12	1.79	48.72
	1PM	1.77	47.31
	2	1.75	45.93
	3	1.74	44.91
	4	1.71	43.24
	5	1.69	41.92
	6	1.67	40.63
	7	1.67	40.63
	8	1.65	39.36
	9	1.65	39.36
	10	1.63	38.11
30/9/89	1AM	1.63	38.11
	2	1.62	37.49
	5	1.62	37.19
	6	1.63	38.11
	7	1.65	39.36

APPENDIX-H: Rainfall Data

Station: **Finoteselam** , Region: **Gojam**,

Element: **Rainfall Intensity**, Month: **July**, year: **1985**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1															
1-2													4.9		
2-3													4.5	5.2	
3-4															
4-5															
5-6															
6-7		3.5													
7-8															
8-9															
9-10		1.2													
10-11															
11-12															
12-13													4.2		
13-14		1.0													
14-15			1.8					1.2							5.5
15-16			4					4.9	5.3		0.7				
16-17			2.1				5.5	1.9							
17-18	12.4						0.9								
18-19				0.7						1.5		2.3			
19-20										2.5					
20-21				1.0											
21-22															
22-23					9.8						2.0				
23-24	2.6					3.5									6.7
Max. 1hr.	**12.4**	**3.5**	**4.0**	**1.0**	**9.8**	**3.5**	**5.5**	**4.9**	**5.3**	**2.5**	**2.0**	**2.3**	**4.9**	**5.2**	**6.7**

Station: **Finoteselam** , Region: **Gojam**,

Element: **Rainfall Intensity**, Month: **July**, year: **1985**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1		0.4			1.8	3.6			4.9	8.5						
1-2						3.5	6.5				7.6					
2-3			1.8					7.9	5.2				5.0			10.3
3-4									7.8				7.7			

4-5									2.2							
5-6									1.5							
6-7																
7-8																
8-9																
9-10																
10-11																
11-12		0.8														
12-13		6.2														7.2
13-14		1.4														
14-15																
15-16												2.5				
16-17	1.6															
17-18									3.6			3.8				
18-19							9.0									
19-20				2.9											3.8	
20-21																
21-22								12.3	3.0			1.3				8.8
22-23						1.6	14.3		4.7			3.5				
23-24									5.3		4.9					
Max. 1hr.	**1.6**	**6.2**	**1.8**	**2.9**	**1.8**	**3.6**	**14.3**	**12.3**	**7.8**	**8.5**	**7.6**	**3.8**	**7.7**	**0.0**	**3.8**	**10.3**

Station: **Finoteselam** , Region: **Gojam**,

Element: **Rainfall Intensity**, Month: **July**, year: **1986**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1				6.0				2.0						2.0		
1-2					1.0		2.0	1.0						2.0		
2-3					0.8		2.0	4.0						0.4		
3-4								2.0						0.6		
4-5								2.5	3.0					1.5		
5-6									10.0					1.0		
6-7									3.0							
7-8									4.5							
8-9																
9-10																
10-11																
11-12											7.2					
12-13											4.8			4.5		
13-14													1.3			

14-15			16.0										6.3			
15-16			2.4		2.8	1.5	0.8						2.1			
16-17							3.7									
17-18								3.5				1.8		2.0		
18-19					0.9	2.5		4.0				4.6				
19-20							6.0	7.5				1.4		1.8		
20-21						0.8	5.0	2.0								
21-22									5.0							
22-23									6.4							
23-24				0.5				3.0		2.2						
Max. 1hr.	**0.0**	**0.0**	**16.0**	**6.0**	**2.8**	**2.5**	**6.0**	**7.5**	**10.0**	**2.2**	**7.2**	**4.6**	**6.3**	**4.5**	**0.0**	**0.0**

Station: **Finoteselam** , Region: **Gojam**,

Element: **Rainfall Intensity**, Month: **May**, year: **1987**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1			0.7									4.5			1.0	
1-2																
2-3								2.5								
3-4		4.4	0.7				2.3									
4-5																
5-6								2.0								
6-7																
7-8																
8-9																
9-10																
10-11																
11-12																
12-13								1.1								
13-14												2.3				
14-15													0.6			
15-16								2.4								
16-17																
17-18							0.6									
18-19			3.8			9.8										
19-20						1.1	0.8									
20-21				7.2			1.5									
21-22														1.3		
22-23													1.7	4.7		
23-24							1.2						0.6	2.2		
Max. 1hr.	**0.0**	**4.4**	**3.8**	**7.2**	**0.0**	**9.8**	**2.3**	**2.5**	**0.0**	**0.0**	**0.0**	**4.5**	**1.7**	**4.7**	**1.0**	**0.0**

Station: **Finoteselam** , Region: **Gojam**,

Element: **Rainfall Intensity**, Month: **July**, year: **1987**

Hours/Date	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31
0-1		1.5				3.4										
1-2	13.8		1.5													
2-3												7.2				
3-4																
4-5														7.5		
5-6																
6-7																
7-8																
8-9																
9-10																
10-11															0.7	
11-12																
12-13									2.4							
13-14			12.0						7.3							
14-15									2.5			2.9				
15-16			12.6													
16-17		1.8														
17-18			8.4													
18-19													5.7			
19-20					2.5					4.8						
20-21				4.9					8.0							5.1
21-22	3.6															
22-23														9.4		
23-24																
Max. 1hr.	**13.8**	**1.8**	**12.6**	**4.9**	**2.5**	**3.4**	**0.0**	**0.0**	**8.0**	**4.8**	**0.0**	**7.2**	**5.7**	**9.4**	**0.7**	**5.1**

Station: **Finoteselam** , Region: **Gojam**,

Element: **Rainfall Intensity**, Month: **July**, year: **1989**

Hours/Date	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0-1				1.7				2.8		0.8				1.5	
1-2			2.0						4.7				11.0		
2-3			2.0												
3-4											1.4				
4-5															

5-6															
6-7															1.0
7-8															1.5
8-9															
9-10															
10-11															
11-12								0.9							
12-13					1.4										
13-14					17.5									3.0	
14-15					2.1								2.2		
15-16				11.5											
16-17										3.0					
17-18			10.7				6.0			1.4					
18-19		1.5					1.5	2.4				2.2			
19-20								2.0	2.5		2.1				
20-21							1.0	0.8	1.3						
21-22							4.5							3.4	
22-23							3.0	6.5		6.0				2.5	
23-24					8.5		4.0			1.5				1.0	
Max. 1hr.	**0.0**	**1.5**	**10.7**	**11.5**	**17.5**	**0.0**	**6.0**	**6.5**	**4.7**	**6.0**	**2.1**	**2.2**	**11.0**	**3.4**	**1.5**

Station: **Hoha** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **June**, year: **1985**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1		8.2														
1-2													1.6		4.9	
2-3												1.5	10.0		0.9	
3-4											0.4		1.4		0.4	
4-5												0.8				
5-6					8.6						0.2	1.2		0.5		
6-7												0.6				
7-8												0.3		1.1	0.4	
8-9																
9-10																
10-11																
11-12							2.2	0.1							2.6	
12-13							0.7									
13-14																

14-15																
15-16																
16-17									5.4							
17-18																
18-19								1.6								
19-20			0.2													
20-21																
21-22																
22-23																
23-24																
Max. 1hr.	**0.0**	**8.2**	**0.2**	**0.0**	**8.6**	**0.0**	**2.2**	**1.6**	**5.4**	**0.0**	**0.4**	**1.5**	**10.0**	**1.1**	**4.9**	**0.0**

Station: **Hoha** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **July**, year: **1985**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1										5.0			0.3		
1-2										0.8					
2-3															
3-4	2.1														
4-5	7.8														
5-6													3.0	5.8	
6-7													0.8		
7-8												6.2			
8-9															2.5
9-10					5.0										
10-11															
11-12															
12-13															
13-14															
14-15															
15-16															
16-17															0.8
17-18															2.1
18-19															1.9
19-20															
20-21															
21-22															
22-23															
23-24										5.6					
Max. 1hr.	**7.8**	**0.0**	**0.0**	**0.0**	**5.0**	**0.0**	**0.0**	**0.0**	**0.0**	**5.6**	**0.0**	**6.2**	**3.0**	**5.8**	**2.5**

Station: **Hoha** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **August**, year: **1985**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1	0.2	0.1	6.0				4.4	1.5			1.5				
1-2	0.1		13.3								8.4				
2-3	0.6		0.4						0.1						
3-4			0.4												
4-5												2.4			
5-6												12.6			
6-7					1.5	4.0						1.8		0.2	
7-8					3.0										
8-9					0.5										
9-10							5.6								0.1
10-11			5.6				13.9								
11-12			1.1				4.4								
12-13			2.5				3.7								
13-14							0.7								
14-15			2.4				0.5			7.1				0.1	
15-16										2.0				0.8	0.2
16-17				1.6											1.6
17-18				0.7											2.1
18-19				1.8										4.2	0.9
19-20				0.1					0.8						
20-21									1.2	0.5					
21-22		3.5		0.3			2.0	2.3	7.0	0.5	0.6				
22-23		2.1					13.8	3.0	0.5	1.0					
23-24	0.3	0.9	9.6	0.3			6.0	1.5							
Max. 1hr.	**0.6**	**3.5**	**13.3**	**1.8**	**3.0**	**4.0**	**13.9**	**3.0**	**7.0**	**7.1**	**8.4**	**12.6**	**0.0**	**4.2**	**2.1**

Station: **Hoha** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **August**, year: **1985**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1			0.5													
1-2		1.7	1.3			1.0										
2-3		3.5	1.0													
3-4																
4-5																

5-6																
6-7																
7-8																
8-9																
9-10																
10-11																
11-12			13.7													
12-13			8.8	1.3	1.3					2.1						
13-14			1.5							8.0						
14-15			1.2													
15-16			2.8													
16-17	1.4		2.8			1.6										
17-18			7.5													
18-19			0.5					1.4								
19-20			0.1													
20-21			0.2	0.4												
21-22												0.5				
22-23			0.4													
23-24			0.4													
Max. 1hr.	**1.4**	**3.5**	**13.7**	**1.3**	**1.3**	**1.6**	**0.0**	**1.4**	**0.0**	**8.0**	**0.0**	**0.5**	**0.0**	**0.0**	**0.0**	**0.0**

Station: **Shambu** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **June**, year: **1987**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1							1.7			0.2					
1-2		7.6	3.9	0.6			3.6			0.4					0.7
2-3		1.5	2.7				1.6		2.0	0.3					
3-4		0.5		3.2	7.8			1.4							
4-5				6.9	5.9			1.5							
5-6				3.0	2.3			0.1							
6-7				0.9											
7-8				0.1	0.1										
8-9															
9-10															
10-11															
11-12	0.5														
12-13	0.9														
13-14															
14-15										6.9					

15-16	3.4									1.1					
16-17					10.2			0.1				0.5	6.4		0.1
17-18			14.5									0.1	0.4		
18-19							3.4								
19-20	12.7						4.0								
20-21	2.8						2.5					4.7			
21-22							2.0					0.8	0.4		
22-23							1.2						1.2	0.2	
23-24							0.4								
Max. 1hr.	**12.7**	**7.6**	**14.5**	**6.9**	**10.2**	**0.0**	**4.0**	**1.5**	**2.0**	**6.9**	**0.0**	**4.7**	**6.4**	**0.2**	**0.7**

Station: **Shambu** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **July**, year: **1987**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1			3.1		2.4				1.7			2.2			0.2
1-2								0.8	0.3	0.5					
2-3						8.5		0.3				0.4			
3-4						1.4		0.3							0.1
4-5						0.5		0.3							
5-6															
6-7						0.5									
7-8															
8-9															
9-10	0.1		0.1												
10-11															
11-12															
12-13															
13-14									2.5						
14-15															
15-16								8.7							0.8
16-17							0.3	0.6	1.1						
17-18							0.6		1.5			12.5	1.2		0.2
18-19				0.7							4.0	2.2	0.8		
19-20				0.5									0.8		
20-21	3.6							0.1			22				
21-22							1.7	4.0	10.8		6.7	0.5			3.5
22-23							2.0	1.4	1.9		0.5				12.8
23-24						8.5	1.9		1.7		0.7	0.4			0.8
Max. 1hr.	**3.6**	**0.0**	**3.1**	**0.7**	**2.4**	**8.5**	**2.0**	**8.7**	**10.8**	**0.5**	**22.0**	**12.5**	**1.2**	**0.0**	**12.8**

Station: **Shambu** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **August**, year: **1987**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1	0.2									1.8					
1-2					0.2										
2-3					1.8										
3-4			11.0		0.9										
4-5			1.1												
5-6															
6-7	0.1			0.4											
7-8		0.1	0.1	0.6											0.5
8-9		1.0	3.9	0.2											0.9
9-10															
10-11						2.2									
11-12						1.0									
12-13		3.1				4.0		1.2		5.0					
13-14						1.4	0.5			3.8					
14-15							7.5	0.5							
15-16										0.3					0.1
16-17	11.8														
17-18	8.8		0.1												1.4
18-19	2.0				1.1		0.5	0.6							0.1
19-20	19.2														1.2
20-21										0.1					
21-22	2.8									0.1					0.2
22-23	9.6									0.9					
23-24	1.3	1.5		3.5						8.1					
Max. 1hr.	**19.2**	**3.1**	**11.0**	**3.5**	**1.8**	**4.0**	**7.5**	**1.2**	**0.0**	**8.1**	**0.0**	**0.0**	**0.0**	**0.0**	**1.4**

Station: **Shambu** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **September**, year: **1987**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1																
1-2					7.5											
2-3					3.3	2.2		6.3								
3-4						1.6		4.7								
4-5																
5-6																

6-7								2.8								
7-8																
8-9																
9-10							5.3									
10-11																
11-12																
12-13																
13-14																
14-15																
15-16													1.5			
16-17																
17-18		0.2				13.8										
18-19																
19-20														1.9	0.4	
20-21						3									0.1	
21-22						3.5										
22-23															0.2	
23-24	1.8															
Max. 1hr.	**1.8**	**0.2**	**0.0**	**0.0**	**7.5**	**13.8**	**5.3**	**6.3**	**0.0**	**0.0**	**0.0**	**0.0**	**1.5**	**1.9**	**0.4**	**0.0**

Station: **Nekemte** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **September**, year: **1988**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1			1.3											1.7	0.4
1-2		4.3											1.5	4.8	
2-3		5.0												1.5	2.3
3-4		1.3												2.0	2.6
4-5															0.8
5-6						6.8						0.5		0.9	2.0
6-7								0.8						1.3	
7-8			0.3					18.9			0.3	0.8	0.2		
8-9													0.6		
9-10	0.9	4.4		1.7										3.2	
10-11	0.1		0.2	5.5											
11-12															
12-13															
13-14				0.2			0.3								
14-15							0.2								
15-16															

16-17											0.4		8.0		
17-18									7.2		2.8				
18-19											20.3		6.4		0.5
19-20											8.0				
20-21			0.6								2.0				
21-22			6.1	10.4							1.3				
22-23			5.3	4.3									0.3		
23-24			0.9												
Max. 1hr.	**0.9**	**5.0**	**6.1**	**10.4**	**0.0**	**6.8**	**0.3**	**18.9**	**7.2**	**0.0**	**20.3**	**0.8**	**8.0**	**4.8**	**2.6**

Station: **Nekemte** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **September**, year: **1988**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1	1.2														0.4	
1-2	2.2												0.2	0.2		
2-3	2.2	1.4										2.4			2.2	
3-4		12.6		0.2	1.6										0.7	
4-5		2.1		0.6	2											
5-6	3.8				0.5		1.8	0.7								
6-7	1.4						9.1									
7-8												4.3				
8-9			0.2									16	0.8			
9-10									0.1		0.6		6			
10-11									0.7		12.5		4			5.3
11-12	0.5								1.8		7.5		1.9			
12-13	1.6								0.1					3		3
13-14	0.3													1.2		
14-15												9.4		4.5		
15-16	0.2								7.5			15		0.4		
16-17	1			2					1.4		10.6	1.7		0.3		
17-18	4.2			2	8.1						1.4					
18-19	4			0.2												
19-20					0.1	0.3										
20-21																
21-22	0.2			0.3								0.1				
22-23								1.2			0.1	0.9	0.5			
23-24	1			0.3	1.3											
Max. 1hr.	**4.2**	**12.6**	**0.2**	**2.0**	**8.1**	**0.3**	**9.1**	**1.2**	**7.5**	**0.0**	**12.5**	**16.0**	**6.0**	**4.5**	**2.2**	**5.3**

Station: **Nekemte** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **October**, year: **1988**

Hours/Date	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**
0-1		2.0	0.5		1.5				2.0				2.1	3.5	
1-2									0.4	0.3		4.7	0.6	12	0.2
2-3										0.5	8.2	0.8	0.9	4.9	1
3-4											0.7		6.0	6.0	1.5
4-5												0.9	1.5	1.8	1.5
5-6															0.8
6-7															0.5
7-8															1.2
8-9															
9-10															
10-11															
11-12				0.1											
12-13															
13-14															
14-15						0.4									
15-16															
16-17															
17-18													0.5		
18-19						0.1									
19-20				0.2	19.2				11					5.4	
20-21									0.2					4.5	
21-22			6.0		13								0.2		
22-23									4.5	0.2					0.6
23-24									1.4				0.7		0.6
Max. 1hr.	**0.0**	**2.0**	**6.0**	**0.2**	**19.2**	**0.4**	**0.0**	**0.0**	**11.0**	**0.5**	**8.2**	**4.7**	**6.0**	**12.0**	**1.5**

Station: **Nekemte** , Region: **Wellega**,

Element: **Rainfall Intensity**, Month: **October**, year: **1989**

Hours/Date	**16**	**17**	**18**	**19**	**20**	**21**	**22**	**23**	**24**	**25**	**26**	**27**	**28**	**29**	**30**	**31**
0-1	0.5					0.9	0.2									
1-2		0.4						6.2								
2-3							2.7	1.7								
3-4					8.2			0.7								
4-5																
5-6																

6-7																
7-8																
8-9																
9-10																
10-11																
11-12																
12-13					5.0		1.0		0.4							
13-14					5.7		1.1									
14-15																
15-16	13															
16-17	1.4											2.4		0.6		
17-18		9.2								0.4				0.4		
18-19																
19-20				0.2							0.4					
20-21											0.6					
21-22	0.4					0.8	4.2					1.5				
22-23	2.3	0.9					7.0									
23-24	0.4	1.7					2.7	0.8					5.4			
Max. 1hr.	**13.0**	**9.2**	**0.0**	**0.2**	**8.2**	**0.9**	**7.0**	**6.2**	**0.4**	**0.4**	**0.6**	**2.4**	**5.4**	**0.6**	**0.0**	**0.0**

DECLARATION

I, the undersigned, declare that this thesis is my original work, has not been presented for a degree in any other university and that all sources of materials used for the thesis have been duly acknowledged.

Name: Dereje Getachew

Signature:________________

Place: Addis Ababa Institute of Technology

Date of Submission: June 16/ 2011