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RUNOFF ESTIMATION BY GIUH BASED CLARK AND NASH MODELS

FOR SHAYA RIVER

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DEPARTMENT OF CIVIL ENGINEERING

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ABSTRACT

For catchments with stream gauging, the traditional techniques for runoff estimation use historical rainfall-runoff data to derive the unit hydrograph. However, for ungauged catchments some indirect approaches have been used. Due to scarcity of data, more emphasis is being given to the conceptual models which are based on the geomorphological characteristics of the catchment than the physically based models.

Geomorphological Instantaneous Unit Hydrograph (GIUH) approach is among the various techniques available for the runoff estimation, especially for ungauged catchments.

In this study, the mathematical model developed by other investigators for the estimation of Clark model parameters using the geomorphological characteristics for ungauged basins and GIUH based Nash model have been used to simulate the direct surface runoff (DSRO) hydrographs of Shaya river basin up to Robe gauging site.

GIS package Integrated Land and Water Information System (ILWIS) has been used to evaluate the geomorphological parameters of Shaya basin. Arc GIS is used to draw the isochrones maps, of the selected catchment, required to draw the time area diagram for the GIUH based Clark model. Velocity vs. excess rainfall intensity relationship has also been developed based on the stream gauging station data obtained from the Ministry of Water and Energy.

The direct surface runoff (DSRO) hydrographs estimated by the two GIUH based approaches have been compared with the direct surface runoff hydrographs estimated by HEC-HMS package Clark and Nash IUH models and also with the observed direct surface runoff hydrographs.

Some of the error functions viz. (i) efficiency (EFF), (ii) root mean square error (RMSE), (iii) percentage error in peak (PEP) and (iv) error function (Err), have also been used to evaluate the performance of the GIUH based models.

From the models evaluation it is observed that GIUH based Clark approach provided the best estimation in peak discharge, whereas Nash IUH model performed best in estimating the time to peak discharge. It is also observed that the direct surface runoff hydrographs estimated using GIUH based Clark and GIUH based Nash model approaches, are quite accurate.

Finally, this paper concludes that GIUH based conceptual models can be used to simulate rainfall-runoff relations with better estimation, especially for ungauged catchments. However, further works are required in implementing the GIUH based approaches in different parts of

Ethiopian ungauged catchments, with varying geomorphology. The researcher recommends further investigations on GIUH based models using satellite based data, which can substitute the conventional historical gauging records.

KEY WORDS: Geomorphologic Instantaneous Unit Hydrograph (GIUH), HEC-HMS, Nash IUH Model, ILWIS (Integrated Land and Water Information System), GIS, Arc GIS, DSRO, rainfall-runoff, Ungauged Catchment.

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LIST OF SYMBOLS

Symbol	Meaning
DEM:	Digital Elevation Model
DRH:	Direct Runoff Hydrograph
DSRO:	Direct Surface Run Off
DUH:	Dimensionless Unit Hydrograph
EFF:	Efficiency
ERH:	Excess Rainfall Hydrograph
GIS:	Geographical Information System
GIUH:	Geomorphological Instantaneous Unit Hydrograph
HEC-HMS:	Hydrologic Engineering Center – Hydrologic Modeling System
ILWIS:	Integrated Land and Water Information System
IUH:	Instantaneous Unit Hydrograph
K:	Storage coefficient (Nash parameter)
L Ω :	Length of highest order stream
n:	Number of linear reservoirs (Nash parameter)
PEP:	Percentage Error in Peak
R _A :	Stream Area Ratio
R _B :	Bifurcation Ratio
R _L :	Stream Length Ratio
RMSE:	Root Mean Square Error
SCS:	Soil Conservation Service
TAC:	Time Area Concentration
TUH:	Triangular Unit Hydrograph
UH:	Unit Hydrograph
V:	Stream flow Velocity
Ω :	Stream network order

1. INTRODUCTION

1.1. General

In water resources planning, development and operation of various schemes, rainfall- runoff relationship study and accurate estimation of hydrologic response of the basin is the main concern of hydrologists. In surface hydrology rain fall- runoff process is a non linear process that involves various climatic, topographic, soils, and land use information. Despite of the fact that, it is the most complex real world phenomena, the transformation of rain fall into runoff has been a very active area of research.

Different theories and models have been developed to determine the runoff response of a basin for a given rainfall intensity. The simplest theory proposes the multiplication of the rain fall with some runoff coefficient to get the runoff. The hydrological models developed for this process 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 (Nash, 1957) is based on the concept of ‘a cascade of linear reservoirs’ in the watershed. The physically based distributed models on the other hand, need rigorous 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 a, b).

When catchments are gauged and their historical data are found to be adequate, these data can be used to estimate the amount of runoff using the appropriate model. Availability and accuracy of runoff data for small and medium catchments is always under question, especially in developing countries.

One of the most popular approaches for the simulation of runoff for the ungauged catchments is the application of regionalization. However it has limitations since the hydrological behavior of many nearby catchments may not establish the same regional formula for the unit hydrograph parameters. The other approach for ungauged catchments utilizes geomorphologic characteristics. This approach has many advantages over the regionalization approach as it avoids the requirement of data and computations in the neighboring gauged catchments.

The geomorphic characteristics are the channel network and surrounding landscape, which translate the rainfall input into an output hydrograph at the outlet of the watershed. One of the simplest approaches to rainfall runoff modeling is the application of unit hydrographs. A significant advance in the unit hydrograph method for an ungauged area was the development of the Geomorphological Instantaneous Unit Hydrograph (GIUH) (Lee K.T., 1998). The concept of the Geomorphologic Instantaneous Unit Hydrograph (GIUH) was first introduced by Rodriguez-Iturbe and Valdes (1979) and further simplified by Gupta et al. (1980), coupling the hydrologic characteristics of a catchment with the geomorphologic parameters.

In GIUH based approach, a unifying synthesis of the hydrological response of a catchment to a surface runoff is attempted by linking the instantaneous unit hydrograph (IUH) with the geomorphological parameters of the basin. Equations of general character is derived which expresses the IUH as a function of Horton's numbers i.e. area ratio (R_a), bifurcation ratio (R_b) and length ratio (R_l) (Strahler, 1957); an internal scale parameter ($L\Omega$) denoting the length of highest order stream and the peak velocity of the stream flow (V) expected during the storm (Kumar et al., 2000).

In this study, GIUH based Clark; GIUH based Nash, Clark-IUH using HEC-HMS and Nash IUH runoff modeling approaches have been used for runoff estimation for shaya river catchment up to Robe in Genale Dawa basin of Ethiopia. The catchment is assumed as ungauged for GIUH based models and the observed run off data at different events are used for the computation of the other models parameters.

1.2. Problem Description

Ethiopia is considered as the water tower of Africa, due to the availability of abundant water resources. Most of the basins are suitable both for Irrigation and hydropower developments. According to IDCOF, 2002 about 111 billion cubic meter of water is expected annually from surface water flow of the twelve river basins existing in the country.

As a developing country it is planning and starting to use the best out of this resource. The millennium dam project which is expected to develop 6000 MW of hydropower and other water resource development projects which are under progress and in planning stage are good indications of the country's attention towards the development of this golden resource the so called "the white coal".

For proper planning, development and utilization of water resource projects a good understanding and accurate estimation of the catchment runoff output is vital. This can be achieved by properly studying the runoff response of a basin for a given quantity of rainfall falling on the upstream area. A good estimation of the transformation of rain fall in to runoff needs an adequate and properly recorded historical data of stream flow and rainfall gauging stations. The provision of such recording instruments and stations at every river is capital intensive and difficult not only for developing country like Ethiopia, but also for the developed once.

The best recommendable approach of runoff estimation is to utilize and examine historical data of the catchment. The required data can be hydro meteorological data series and/or physical parameters describing the catchment such as catchment's area, slope, land use, infiltration capacity, and permeability.

However like most of the developing countries the quantity and quality of hydrological and meteorological stations available in the country are not sufficient. According to the information from the experts, some of the available stations are also not functioning properly due to technical and other problems. As a result the necessary rainfall-runoff data may not be reliable, with missing records, or unavailable at all.

In the absence of reliable observed data, the other approach developed, that does not require the availability of a long time data series, is the application of regionalization to develop synthetic unit hydrographs.

In the process of regionalization, the parameters of unit hydrograph models are related with physiographic and climatologic characteristics for gauged catchments. 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 does not only require 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).

Due to global changes in meteorological conditions from time to time, the rainfall pattern and climatic conditions are undergoing a dramatic change. Consequently, parameters of the physical based models also need revision from time to time. Especially in countries like Ethiopia, with mountainous and varying topography, it is quite difficult to lean on these regionalization techniques.

The interest of searching for another approach which does not require long historical data and that can be applied on the specified catchment without correlating with other catchments has conceived the idea of this research work.

Geomorphology based approach could be the most suitable technique for modeling the rainfall-runoff process for ungauged catchment. The geomorphological parameters are mostly time invariant and only geomorphological characteristics of the specific catchment are required to estimate the runoff response.

In this study the geomorphology based approach is applied to estimate the runoff response of Shaya River near Robe town. This geomorphology based hydrological analysis is hoped to fill the gap of rain fall runoff transformation process problems for the mostly ungauged catchments of Ethiopia and it is also expected to add some new insight to the area of hydrologic study of the country.

1.3. Objectives of the Study

This research work has the following general and specific objectives.

1.3.1. General Objective

Solving the problem of runoff estimation specifically in ungauged river basins, where the stream gauging records are not available and the topographic and climatic characteristics of nearby catchments are varying, is the basic concern of this study. Therefore, the general objective of this research work is to apply and evaluate the geomorphological instantaneous unit hydrograph (GIUH) models to estimate the direct surface runoff response of Shaya river basin, assuming it as ungauged catchment.

1.3.2. Specific Objective

The specific objectives of this study are:

- i. To extract the geomorphological characteristics of the catchment and Horton ratio's using GIS based ILWIS (Integrated Land and Water Information System).
- ii. To determine the geomorphological parameters of the selected catchments.
- iii. To draw the time area diagram of the Catchment using Arc GIS software
- iv. To derive the geomorphological instantaneous unit hydrographs (GIUH) using the GUIH based Clark and GIUH based Nash models for different events.
- v. To estimate the direct surface runoff hydrographs using the derived unit hydrographs.
- vi. To compare the estimated direct surface runoff hydrographs based on GIUH based approaches with the Nash IUH model and HEC-HMS based Clark model.

1.4. Thesis Outline

The thesis documents the concepts, data requirements, data processing procedures, development and application of the GIUH based and other models for the Shaya river basin. The first chapter is an introduction, which gives background information on hydrologic processes and modeling, statement of the problem, and objectives of the research are also addressed. Chapter two contains detailed description of literature review of rainfall-runoff relationships, hydrologic models and their classifications, the concept of unit hydrograph and its derivation and Chapter three deals with the brief description of the study area and hydro meteorological data collection. Chapter four is devoted to the methodology and detailed data analysis procedures of all the models. Chapter five contains the results and discussions, based on the error computations and the comparison of the observed and computed direct surface runoff hydrographs. The recommendation and conclusions followed by references are described in chapter six. The appendices contain some graphs, tables and computer out puts. It also contains data used in the research work.

2. LITRATURE REVIEW

2.1. Rainfall-Runoff Relationship and Models

2.1.1. General

Hydrology is concerned with study of the motion of the earth's waters through the hydrologic cycle, and the transport of constituents such as sediment and pollutants in the water as it flows. A hydrological model is a simplified simulation of the complex hydrological system.

Modeling is one field of scientific activity which has developed the capability of delivering customized solutions through identifying a variety of arrangements or changes within a system to comply with both external and internal highly developed mathematical capabilities and versatile software tools (R.H.Khatabi et.al.)

Runoff means the draining or flowing off of precipitation from a catchment area through a surface channel. It represents the output from the catchment in a given unit of time.

The rainfall runoff process can be considered as the main component of the hydrologic system; therefore, either 'detailed models' that can serve a wide range of modeling requirement, or "parsimonious models" meeting a specific requirement are required to be developed.

2.1.2. Catchment Runoff Generation

When a catchment receives precipitation; the evapo-transpiration, initial loss, infiltration and detention storage requirements must be satisfied first. The excess precipitation after these abstractions moves over the land surfaces to reach smaller channels. This portion of the runoff is called overland flow. Flows from several small channels join bigger and bigger channels till the flow reaches the catchment outlet. The flow in this mode, where it travels all the time over the surface as overland flow and through the channels as open channel flow and reaches the catchment out let is called surface runoff.

A part of the precipitation that infiltrates moves laterally through upper crusts of the soil and returns to the surface at some location away from the point of entry in to the soil. This component of runoff is known as interflow or subsurface flow.

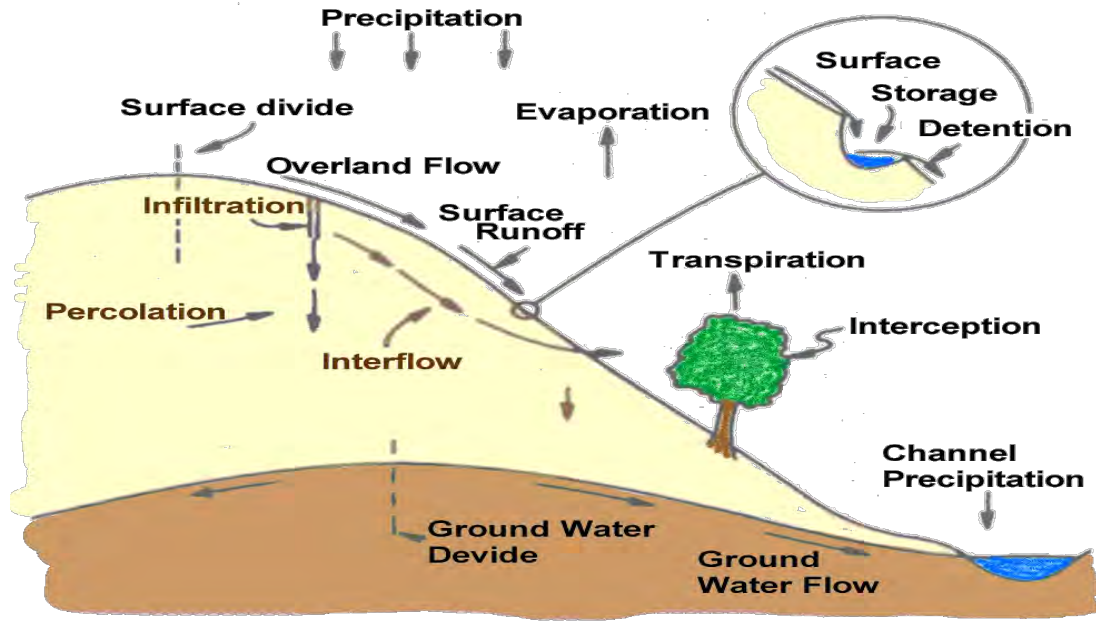


Figure 2-1 Different routes of runoff

Another route for the infiltrated water is to undergo deep percolation and reach the ground water storage in the soil. The groundwater follows a complicated and long path of travel and ultimately reaches the surface. This part of runoff is called groundwater runoff.

Based on the time delay between the precipitation and the runoff, the runoff is categorized in to two categories; as (1) Direct runoff, and (2) Base flow.

Direct runoff: it is that part of runoff which enters the stream immediately after the rain fall. It includes surface runoff, prompt interflow and rainfall on the surface of the stream. In case of snow melt the resulting flow entering the stream is also a direct runoff.

Base flow: the delayed flow that reaches a stream essentially as groundwater flow. Many times delayed interflow is also included under this category. In the annual hydrograph of a perennial stream the base flow easily recognized as the slowly decreasing flow of the stream in rainless periods (K Subramanya, 2008).

2.1.3. Rainfall-Runoff Models

A hydrologic system model is an approximation of the actual system; its inputs and outputs are measurable hydrologic variables and its structure is a set of equations linking the inputs and outputs. Central to the model structure is the concept of a system transformation (Chow, 1988).

This may be represented by:

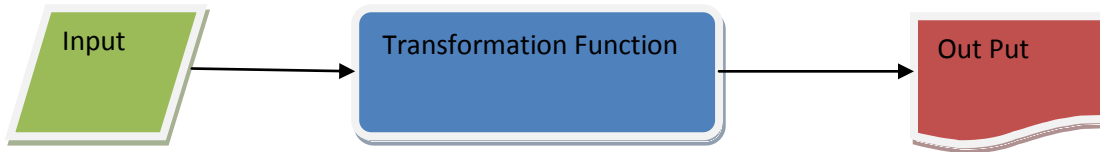


Figure 2-2 Conceptual representation of the system process

Models are normally characterized or classified to help describe and discuss their capabilities, strengths, and limitations. There is no universal method to characterize rainfall-runoff models, and models have been classified in several ways depending on the criteria of interest. Hydrologists have tried to classify rain fall runoff models according to their specific approach as well as their characteristics.

The 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 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. In contrast, a distributed model considers the hydrological process taking place at various points in space, in which parameters, inputs and outputs vary spatially. 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.

However, the most common applied classification of rainfall-runoff models divides models into three classes:

1. Empirical (black box),
2. Conceptual (grey box), and
3. Physically based (white box) model structures.

1. Empirical “black-box” models: use observed discharge data and regression procedures to establish model structure and corresponding model parameters by fitting a function of hydrological characteristics with observed discharge. Empirical models avoid the underlying physical processes; hence they exclusively depend upon the information obtained from the observed data. Artificial neural network models and multiple regression models are among the presently used empirical models.
2. 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. Conceptual models include cascade model, geomorphologic instantaneous unit hydrograph, HBV and IHACRES (Identification of unit Hydrograph And Component flows from Rainfall, Evaporation and Stream flow data).
3. 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).

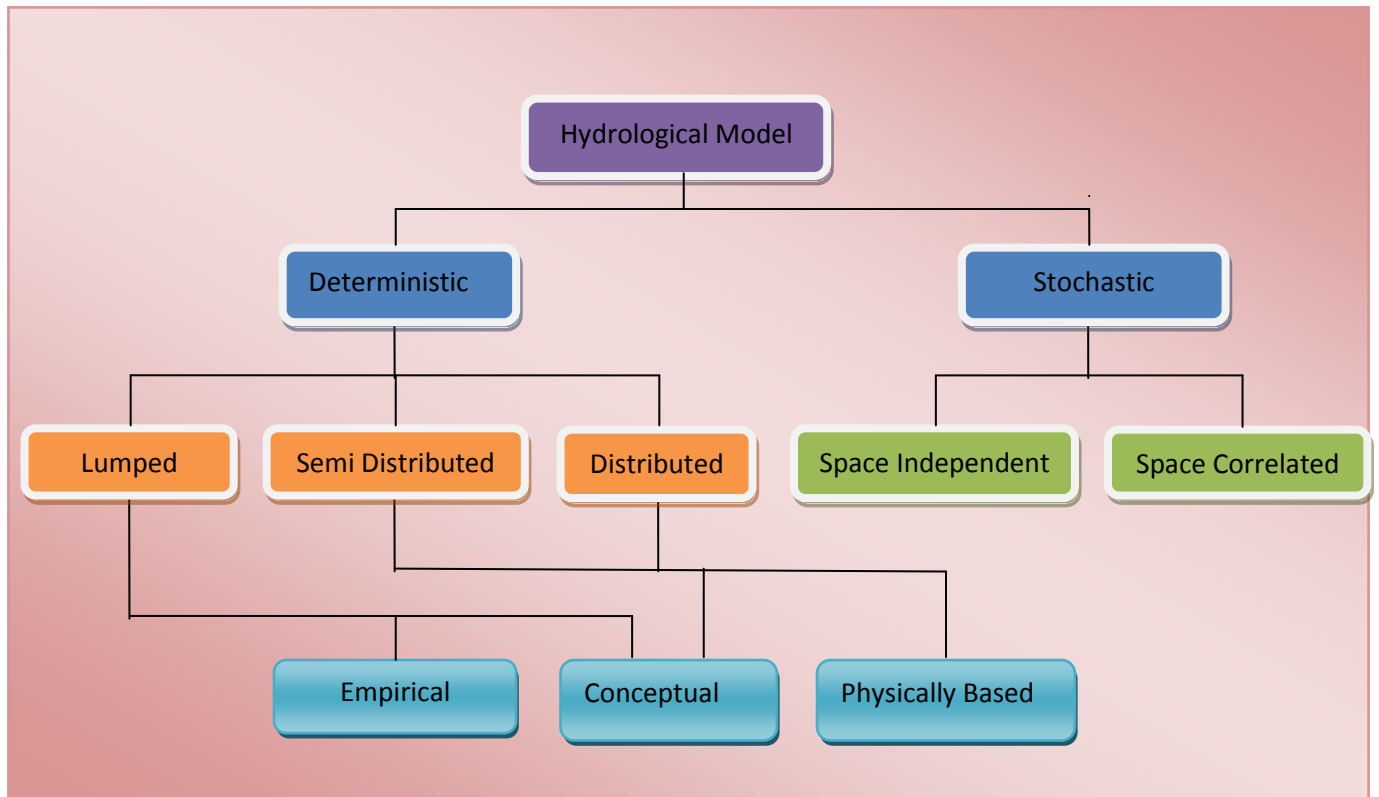


Figure 2-3 Classification of Hydrological models

2.1.4. Event based Models

Rainfall-runoff models are either event based models or continuous simulation (CS) models. Event based models typically estimate the runoff from an individual storm event, i.e., describing a relatively short period within the hydrologic record. Event based models ordinarily evaluate a partial set of the hydrologic processes that affect the watershed: infiltration, overland and channel flow, and possibly interception and detention storage. Most event based models use a constant time interval, whose value may typically range from minutes to several hours.

For an event based model, the initial conditions (antecedent soil moisture, stream and reservoir levels, etc.) must either be subjectively assigned by the user, calibrated with some type of error-reduction procedure, or approximated by an external procedure. Of these three approaches, the first two are common. Event models are typically applied to fewer storms, but increased confidence can be gained by calibrating the model to as many storms as possible.

2.1.4.1. Event based Conceptual Models

Clark (1945) suggested that the unit hydrograph due to an instantaneous rainfall over a watershed can be determined by routing its Time-Area-Concentration (TAC) curve through a single linear reservoir.

O'Kelly (1955) produced instantaneous unit hydrograph for watershed by defining the TAC curve by isosceles triangle and routing it through a linear reservoir.

Nash (1957) developed one of the most popular models based on a cascade of equal linear reservoir for derivation of instantaneous unit hydrograph.

Dooge(1959) developed a general unit hydrograph theory by including the three elements: TAC curves, linear channel and linear reservoir. According to Dooge's model a watershed can be represented by some combination of linear channel and reservoirs. A network of channels composed of a complex network of linear channels and linear reservoirs placed in series drains the watershed.

2.2. Runoff Hydrograph

When a concentrated storm producing a uniform rainfall over a catchment for duration of D hours, the excess rainfall reaches the stream through overland and channel flows after the initial and infiltration losses are met. In the process of translation a certain amount of storage is built up in the first phase of flows and gradually depletes after the rainfall has ceased. The runoff measured at the stream gauging station will give a typical hydrograph, due to an isolated storm, which is known as the runoff hydrograph or storm hydrograph.

2.2.1. Factors affecting Runoff Hydrographs

The shape of runoff hydrographs can be affected by different factors. These factors are broadly grouped in to climatic factor and physiographic factors. The most important factors affecting the shape are described below:

2.2.1.1. Shape of the Catchment

A catchment that is shaped in the form of a pear, with the narrow end towards the upstream and the broader end nearer the catchment outlet shall have a hydrograph that is fast rising and has a rather concentrated high peak. A catchment with the same area but shaped with its narrow end towards the outlet has a hydrograph that is slow rising and with a somewhat lower peak for the same amount of rainfall. Though the volume of water that passes through the outlets of both the catchments is same, the peak in case of the latter is attenuated.

2.2.1.2. Size of the Catchment

Naturally, the volume of runoff expected for a given rainfall input would be proportional to the size of the catchment. The response characteristics of large catchment is found to be significantly different from a small catchment due to the relative importance of the different phases of runoff (overland flow, inter flow, base flow, etc.) for these two catchments.

2.2.1.3. Slope

Slope of the main stream cutting across the catchment and that of the valley sides or general land slope affects the shape of the hydrograph. Larger slopes generate more velocity than smaller slopes and hence can dispose off runoff faster. Hence, for smaller slopes, the excess rainfall obtained from the difference between rainfall input and the runoff gets stored temporally over the area and is able to drain out gradually over time. Hence, for the same rainfall input for two catchments of the same area but with different slopes, the one with a steeper slope would generate a hydrograph with steeper rising and falling limit.

2.2.2. Base Flow Separation

In many hydrograph analyses a relationship between the surface flow hydrograph and the effective rainfall is sought to be established. The surface flow hydrograph is obtained from the total storm hydrograph by separating the quick- response flow from the slow response runoff (base flow).

There are four arbitrary methods of base flow separation as shown in fig 2-4.

Method 1: By drawing a line AC tangential to both the limbs at their lower portion.

Method 2: here the base flow curve existing prior to the commencement of the surface runoff is extended till it intersects the ordinate drawn at the peak, and then draw a straight line DE.

Method 3: By drawing a straight line AE, from the point of rise to the point E. The ordinate of the point on the recession limb can be obtained from the relation:

$$N = 0.83A^{0.2} \quad 2.01$$

Where A = drainage area in km² and N in days.

Method 4: By constructing a line AFG by projecting backwards the ground water recession curve after the storm,

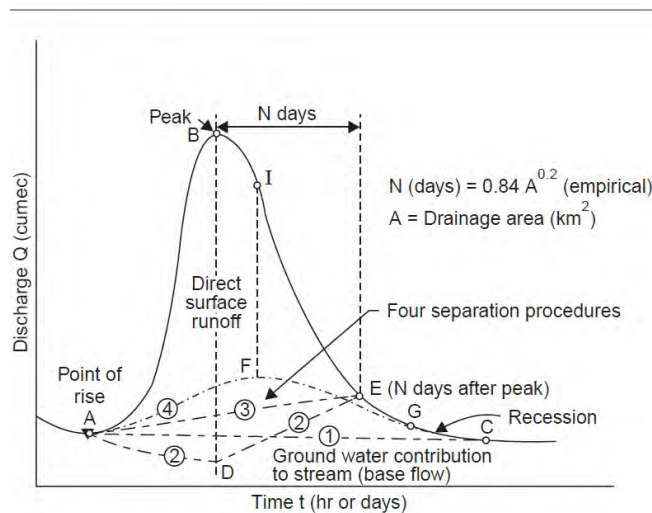


Figure 2-4 Base flow separation techniques

2.2.3. Effective Rainfall

Effective rainfall is the total rainfall in a given duration from which the rainfall abstractions are deducted. It is part of the total rain fall which becomes direct runoff at the outlet of the catchment. When the initial loss and infiltration losses are subtracted from the total rainfall hyetograph the resulting hyetograph is the effective rainfall or excess rainfall hyetograph (ERH). The total quantity of excess rainfall hyetograph and direct runoff hydrograph are the same but with different units.

2.2.4. The Unit Hydrograph

The Unit Hydrograph of a drainage basin is defined as a hydrograph of direct runoff resulting from one unit of rainfall excess which is uniformly distributed over the basin at a uniform rate during the specified period of time known as unit time or unit duration. The unit quantity of effective rainfall is generally taken as 1mm or 1cm and the outflow hydrograph is expressed by the discharge ordinates.

The Unit Hydrograph (UH) theory is also classified in the conceptual model category and has been widely and successfully used over the past few decades. First introduced by Sherman in 1932 as a basic tool that represents the hydrologic response of a catchment through which effective rainfall is transformed to direct runoff (Chow et al., 1988).

2.2.4.1. Unit Hydrograph Assumptions

The following assumptions are made while using the unit hydrograph principle:

- Effective rainfall should be uniformly distributed over the basin.
- Effective rainfall is constant over the catchment during the unit time.
- The direct runoff hydrograph for a given effective rainfall for a catchment is always the same irrespective of when it occurs. Hence, any previous rainfall event is not considered.
- The ordinates of the unit hydrograph are directly proportional to the effective rainfall hydrograph ordinate.
- The hydrograph resulting from excess rainfall reflects the unique characteristics of the watershed.

2.2.4.2. Derivation of Unit Hydrograph

The following steps are adopted to derive a unit hydrograph from an observed flood hydrograph;

- Select from the records isolated intense storms, which occurring uniformly over the catchment have produced flood hydrographs with appreciable runoff.
- Select a flood hydrograph, which has resulted from a unit storm chosen.
- Separate the base flow from the total runoff.
- From the ordinates of the total runoff hydrograph deduct the corresponding ordinates of base flow, to obtain the ordinates of direct runoff.

- Divide the volume of direct runoff by the area of the drainage basin to obtain the net precipitation depth over the basin.
- Divide each of the ordinates of direct runoff by the net precipitation depth to obtain the ordinates of the unit hydrograph.
- Plot the ordinates of the unit hydrograph against time since the beginning of direct runoff. This will give the unit hydrograph for the basin, for the duration of the unit storm selected.

2.2.4.3. Synthetic Unit Hydrographs

In order to develop unit hydrograph to a catchment, detailed information about the rainfall and the resulting runoff hydrograph are required. However, such information would be available only at a few locations. In remote areas where data is difficult to find, synthetic unit hydrographs can be derived from empirical equations of regional validity which relate the salient hydrograph characteristics to the basin characteristics. 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;

- Those based on a dimensionless unit hydrograph (SCS unit hydrograph).
- Those based on models of watershed storage (Clark unit hydrograph), and
- Those relating hydrograph characteristics (peak flow rate, base time, etc.) to watershed characteristics (Snyder unit hydrograph).

2.2.4.4. Instantaneous Unit Hydrograph

Different shapes of unit hydrographs can be observed on a single catchment based on its duration time, D . The less the value of D is, the more skewed the hydrograph will be. As the duration D approaches zero a finite unit hydrograph will be derived. This limiting case of a unit hydrograph of zero duration is the instantaneous unit hydrograph (IUH). Thus IUH is a fictitious conceptual unit hydrograph which represents the surface runoff from the catchment due to an instantaneous precipitation of the rainfall excess volume of one unit.

The main advantage of IUH is that it is independent of the duration of excess rainfall hyetograph and thus has one parameter less than a D -h unit hydrograph. This fact makes it eminently

suitable for theoretical analysis of rainfall excess - runoff relationship of catchment (K Subramanya, 2008).

IUH designated by $u(t)$ is a single-peaked hydrograph with a finite base width and the following properties:

1. $0 \leq u(t) \leq$ a positive value for $t > 0$;
2. $u(t) = 0$ for $t \leq 0$;
3. $u(t) \rightarrow 0$ as $t \rightarrow \infty$;
4. $\int_0^{\infty} u(t)dt =$ unit depth over the catchment; and
5. Time to peak is the time to the centroid of the curve.

If an effective rainfall $i(\tau)$ of duration t_0 is applied to a catchment as shown in fig. 2-5, the discharge at time t of a DRH produced when each infinitesimal element of the ERH operates on the IUH is given by the convolution integral:

$$Q(t) = \int_0^{t'} u(t - \tau)I(\tau)d\tau \quad 2.02$$

Where: $t' = t$ when $t < t_0$ and
 $t' = t_0$ when $t \geq t_0$

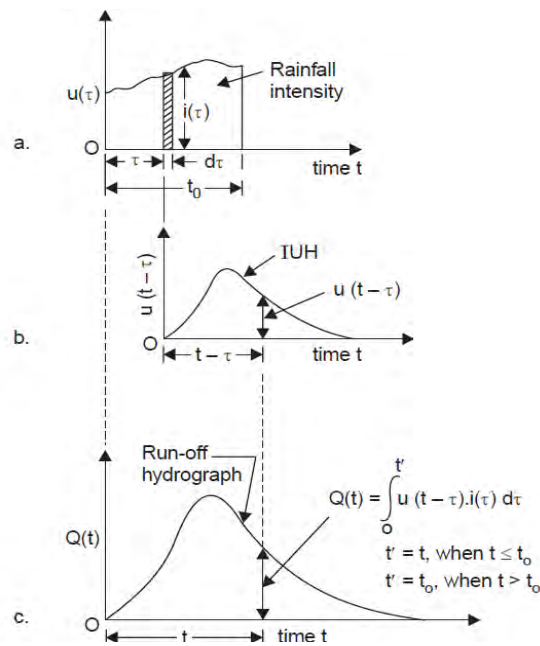


Figure 2-5 Convolution of $I(\tau)$ and IUH

i. Derivation of IUH

If S-curves, S_1 and S_2 , are derived from a D-h unit hydrograph where their excess rainfall intensities are $i = 1/D$ unit/h and i unit/hr respectively. And S_2 is separated from S_1 by a time interval dt . When the ordinates are subtracted, a DRH due to a rainfall excess of duration dt and magnitude $i dt = dt/D$ h is obtained. A unit hydrograph of dt hours is obtained from this by dividing the obtained DRH by $i dt$.

Thus the ordinates of dt -h unit hydrograph will have the value which is equal to $\left(\frac{S_2 - S_1}{i dt}\right)$. As dt is made smaller and smaller approaching zero an IUH results.

Hence for an IUH the ordinate at any time t can be approximately obtained from:

$$u(t) = \lim_{dt \rightarrow 0} \left(\frac{S_2 - S_1}{i dt} \right) = \frac{1}{i} \frac{ds}{dt} \quad 2.03$$

$$\text{If } i = 1, \text{ then } u(t) = dS'/dt, \quad 2.04$$

Where:

S' represents a S-curve of intensity 1 unit/h.

Thus the ordinate of an IUH at any time t is the slope of the S-curve of intensity 1 unit/h at the corresponding time. IUHs can also be derived in other ways like conceptual models, harmonic analysis and Laplace transform (K Subramanya, 2008).

ii. Derivation of D-hour Hydrograph from IUH

D-hour unit hydrograph for simple geometric forms can be obtained from eq. (2.02). For complex shaped IUHs the numerical computation techniques can be used.

From eq (2.04) $dS' = u(t) dt$

Integrating between two points 1 and 2

$$s'_2 - s'_1 = \int_{t_1}^{t_2} u(t) dt \quad 2.06$$

If $u(t)$ is essentially linear within the range 1-2, then for small values of $\Delta t = (t_2 - t_1)$, by taking

$$u(t) = \bar{u}(t) = \frac{1}{2} [u(t_1) + u(t_2)] \quad 2.07$$

$$S'_2 - S'_1 = \frac{1}{2}[u(t_1) + u(t_2)](t_2 - t_1) \quad 2.08$$

But $(S'_2 - S'_1)/(t_2 - t_1) =$ ordinate of a unit hydrograph of duration $D_1=(t_2 - t_1)$. Thus in general terms, for small values of D_1 , the ordinates of a D_1 -hour unit hydrograph are obtained by the equation:

$$(D_1 - \text{hour } UH)_t = \frac{1}{2}[(IUH)_t + (IUH)_{t-D_1}] \quad 2.09$$

After obtaining the ordinates of a D -hour unit hydrograph from eq (2.09), the ordinates of any D -hour UH can be obtained by the superposition method or S-curves method (K Subramanya, 2008).

2.2.5. Runoff Models for Ungauged Catchments

2.2.5.1. 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 q_p and whose time base, t_b , is equal to $2.67 t_p$.

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 (B'ardossy, 2000). Furthermore, SCS dimensionless unit hydrograph provides only empirical approximation of flood runoff characteristics; its reliability is limited to the type and the size of the catchments being considered.

2.2.5.2. 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). Clark used a linear reservoir to reflect the storage effects of watersheds.

The limitations of Clark method are: In his original work Clark did not taken in to consideration the ungauged catchments, his method of estimating R depends 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.

2.2.5.3. 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 ($q_p R$), the basin lag ($t_l R$), 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.

Limitations of Snyder method are: Identification of a nearby and physiographically similar gauged basin from which parameters can be transferred is necessary to develop Snyder parameters for an ungauged basin. The greatest limitation of the method is the difficulty in verifying that the two, gauged and ungauged basins of interest, are sufficiently alike to transfer C_t and C_p from one to the other. Typical hydrologic features that should be evaluated include soils and geology, topography, drainage pattern, drainage area, and land cover. However, in the absence of gauged data on both basins, it can never be known with certainty.

2.3. Review on the Selected Model Approaches

The four model approaches selected in this study are:

- Geomorphologic based Clark Instantaneous Unit Hydrograph Model.
- Geomorphologic based Nash Instantaneous Unit Hydrograph Model.
- Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) based Clark Model.
- Nash Instantaneous Unit Hydrograph Model.

2.3.1. Geomorphologic based Clark Instantaneous Unit Hydrograph Model

The rainfall pattern, in general is undergoing a change due to global changes in atmospheric conditions. Further, because of different activities in the catchment, its land-use is also undergoing a gradual change and this has an impact on the run-off characteristics of the catchment. The coupling of the quantitative hydrologic characteristics of the catchment with the geomorphologic parameters is the main idea of this approach. Rodriguez-Iturbe & Valdes (1979) first introduced the GIUH, which led to the renewal of research in Hydro geomorphology. The concept was re-stated by Gupta, et al. (1980). The Horton's morphometric parameters including area ratio (R_A), bifurcation ratio (R_B) and length ratio (R_L) are mainly used to develop the geomorphological instantaneous unit hydrograph (GIUH).

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:

$$q_p = 1.31R_l^{0.43} \left(\frac{V}{L\Omega}\right) (hr^{-1}) \quad 2.10$$

$$t_p = 0.44R_l^{-.38} \left(\frac{R_B}{R_A}\right)^{0.55} \left(\frac{L\Omega}{V}\right) (hr^{-1}) \quad 2.11$$

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

Where,

$L\omega$ is the length in kilometers of the highest order and

V is the expected peak velocity in m/s.

Transformation of rainfall excess to runoff using the Clark unit hydrograph is based on the method of convolution. This method convolutes rainfall excess increments with the unit hydrograph ordinates to determine the catchment hydrograph,

$$Q_n = \sum P_m * U_{n-m+1} \quad 2.13$$

Where

Q = direct runoff

P = rainfall excess depth

U = unit hydrograph ordinates

n = the number of runoff steps

m = the number of rainfall excess steps as m from 1 to n .

The Clark IUH model is based on the concept that IUH can be derived by routing the unit excess rainfall in the form of a time–area diagram through a single linear reservoir. For the derivation of the IUH, the Clark model uses two parameters viz. time of concentration (T_c) in hour and storage coefficient (R) in hour of a single linear reservoir, in addition to the time–area diagram. The governing equation of the Clark IUH model is expressed as:

$$U_i = C I_i + (1 - C) U_{i-1} \quad 2.14$$

Where,

U_i is i^{th} ordinate of the IUH,

C and $(1 - C)$ are the routing coefficients, and C is $\Delta t / (R + 0.5\Delta t)$,

Δt is computational interval in hour,

I_i is i^{th} ordinate of the time–area diagram.

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

$$U_i = \frac{1}{N} (0.5U_{i-N} + U_{i-N+1} + \dots + U_{i-1} + 0.5U_i) \quad 2.15$$

Where,

U_i is i^{th} ordinate of the UH of D -hour duration and computational interval Δt hour, and

N is number of computational intervals in D -hour and is equal to $D/\Delta t$.

Equation 2.10 is used to determine the optimum storage coefficient (R) in GIUH based Clark model by trial and error procedure.

2.3.2. 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 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. These parameters can be determined from the relation:

$$\frac{(n-1)}{\Gamma(n)} * e^{-(n-1)} * (1-n)^{(1-n)} = 0.5764 \left(\frac{R_B}{R_A}\right)^{0.55} * Rl^{0.05} \quad 2.16$$

$$n = 3.29 \left(\frac{R_B}{R_A}\right)^{0.78} * Rl^{0.07} \quad 2.17$$

$$K = \frac{0.7}{V} * \left(\frac{R_A}{R_B R_l}\right)^{0.48} L_\Omega \quad 2.18$$

Where, L_Ω is length of the basin of order Ω ; R_l , R_A and R_B are Horton's ratios; and v is the uniform stream flow velocity which is taken as constant.

2.3.3. Hydrologic Engineering Center-Hydrologic Modeling System (HEC-HMS) based Clark Model

The HEC-HMS software is a well known hydrological model specially designed for the simulation of flood events. In HEC-HMS model the transformation of excess rainfall in to runoff is accomplished by the unit hydrograph procedures. Three unit hydrograph transform methods, namely the Clark unit hydrograph transform, the SCS unit hydrograph transform and the Snyder unit hydrograph transform, are available in the package. All the precipitation losses are incorporated in the program to determine the excess rainfall using the convenient method among the available loss methods.

After creating the appropriate basin model and feeding the control specification and time series data, the Clark's parameters and loss rate parameters can be determined using the optimization technique provided, based on the observed precipitation and stream flow data.

2.3.4. Nash Instantaneous Unit Hydrograph Model

In the Nash model the catchment is assumed to be made up of a series of n identical linear reservoirs each having the same storage constant K . The first reservoir receives a unit volume equal to 1 unit of effective rain from the catchment instantaneously. This inflow is routed through the first reservoir to get the outflow hydrograph. The out flow from the first reservoir is considered as the input to the second; the outflow from the second reservoir is the input to the third and so on for all the n reservoirs. The outflow hydrograph from the n^{th} reservoir is taken as the IUH of the catchment.

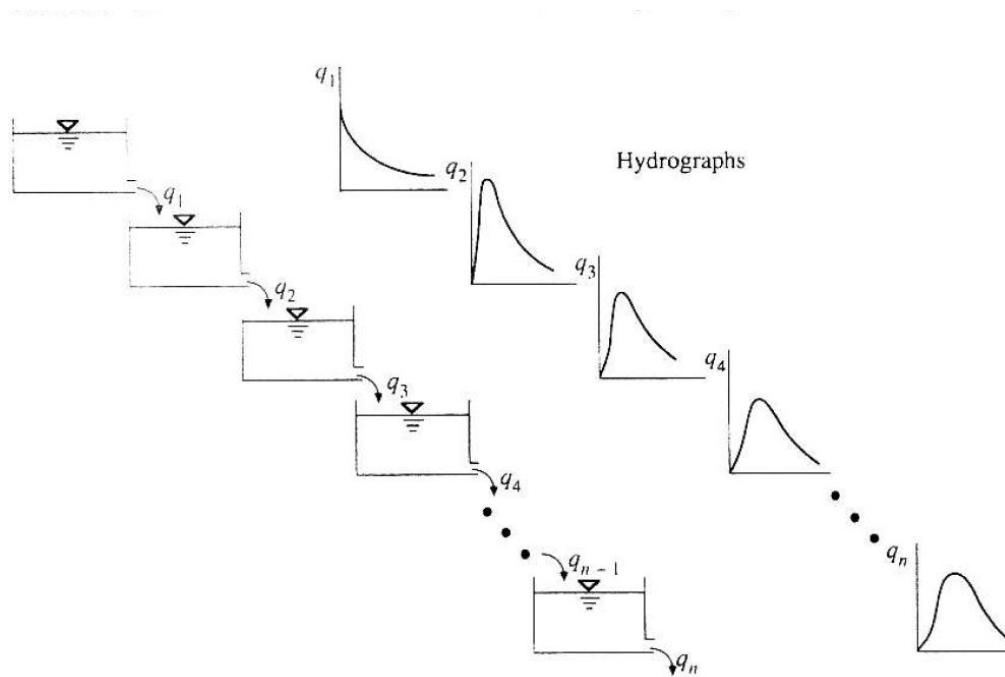


Figure 2-6 linear reservoirs in series

The parameters n and k can be determined from the relations:

$$M_1 = nk \quad 2.19$$

And

$$M_2 = n(n + 1)k^2 \quad 2.20$$

Where M_1 and M_2 are the first and second moments of the IUH respectively, about the origin, $t = 0$, if the ERH and a corresponding DRH are available.

2.4. Previous Study

Rodriguez-Iturbe and Valdes (1979) introduced an approach to derive the Geomorphological unit hydrograph. Gupta, Waymire and C.T.Wang(1980) have examined the approach and reformulated, simplified and made it more general.

Rodriguez-et.al (1982) rationalized that velocity, which was the difficulty in the derivation of IUH, must be a function of the effective rainfall intensity and duration, and proceeded to eliminate velocity from the result.

Bhaskar et al. (1997) have presented the study on flood estimation for ungauged catchments using the GIUH. In this study, the GIUH is derived from the watershed geomorphological characteristics and is then related to the parameters of the Clark and Nash instantaneous (IUH) models for deriving their complete shape.

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).

Tsehay Zeray(2009) studied the application GIUH concept for runoff prediction and Dereje Getachew (2011) studied the derivation of synthetic unit hydrograph using GIUH. Both of them used GIUH and GIUH based Nash models for ungauged catchments found in Abbay basin of Ethiopia and found good results. But they used only two models for comparison, which are also both GIUH based. The conventional models, Nash IUH and HEC-HMS package Clark IUH, and the other GIUH based Clark model used in this study are not included in their work.

3. STUDY AREA AND DATA AQUISATION

3.1. Study Area

Shaya watershed is found in south-eastern part of Ethiopia in Oromia Regional State, Bale Zone. The watershed is situated in Genale-Dawa basin at the upper most parts of the Weyb basin, located between 6° 52' - 7° 15' N latitudes and 39° 46' - 40° 02' E longitudes as shown in Fig.3-1. The Shaya River originates from the northern flanks of the Bale Mountains and first flows generally north-eastwards before joining the Weyb river which flows to east and south eastwards for the remainder of its course. Finally, it joins with Genale and Dawa River near Ethiopia-Somalia border to strengthen its journey to Somali lowlands. It originates from an elevation of 4,343 meter above sea level (m.a.s.l), in the Bale Mountains extreme point locally called Sanetti Mountains to an elevation of 2,357 meter (m.a.s.l) at the outlet of the watershed. The average annual rainfall distribution is 1071 mm and the annual maximum and minimum temperature of the watershed area is about 19.7 °C and 6.1 °C, respectively. The Shaya river watershed is a region of rich environmental diversity, but with increasing levels of environmental stress in recent years from a rapidly expanding human population.

3.2. Geomorphologic Characteristics

The runoff hydrologic response of a drainage basin mostly depends on its watershed characteristics. Therefore it is important to understand the different characteristics of the river basin in order to work with different models of estimations. Geomorphological characteristics of a stream basin is among the most important parameters as far as run off estimation is concerned. The geomorphological characteristics of a watershed deals with the physical characteristics; which includes watershed shape, drainage area, ground slope and centroid of the watershed, and the channel characteristics; including the channel length, channel order, channel slope and drainage density.

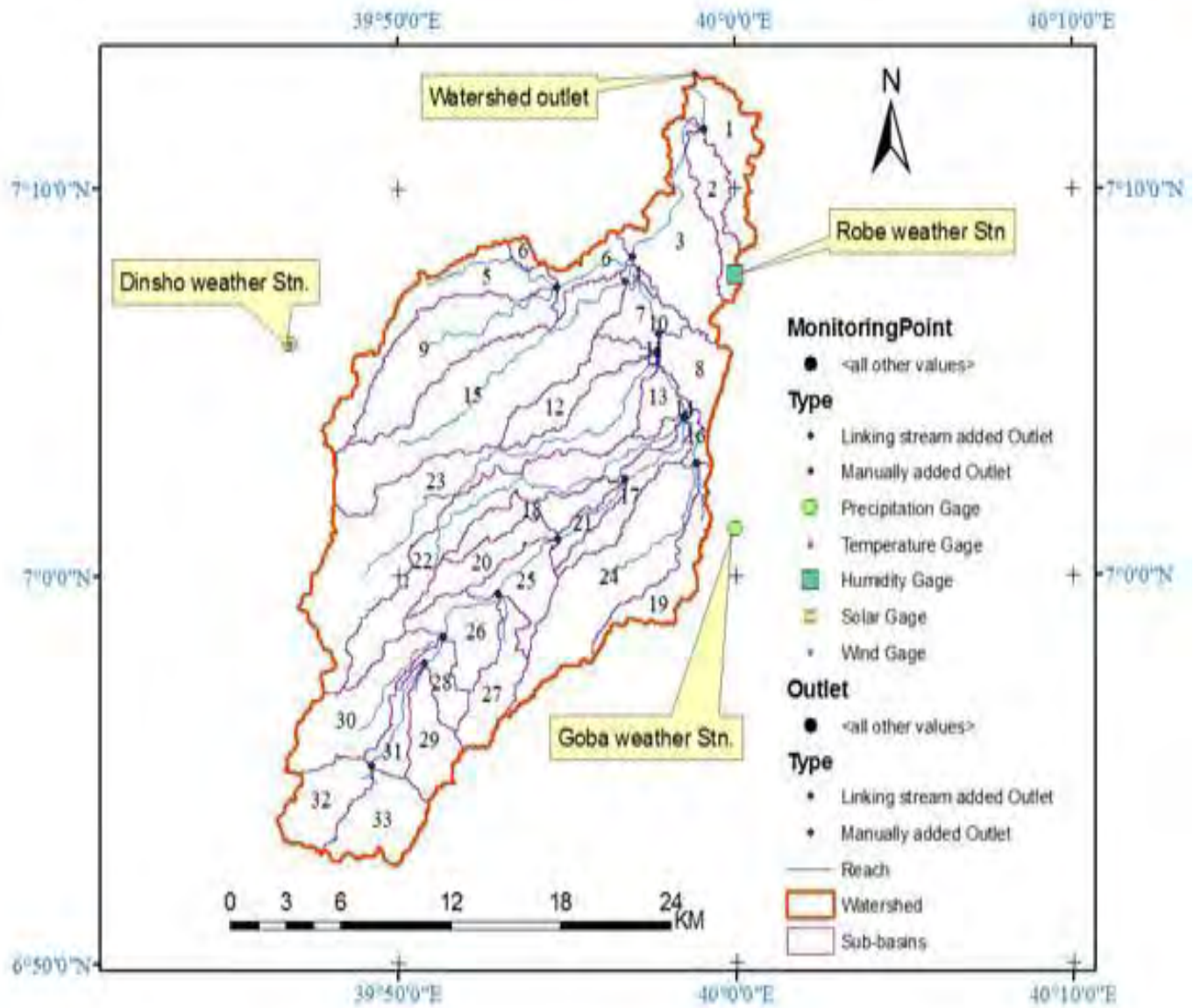


Figure 3-1 Location map and sub-basins of Shaya watershed

3.2.1. Watershed Shape

As described in section 2.2.1 the shape of the runoff hydrograph can be influenced by the shape of the watershed. Numerous symmetrical and irregular forms of drainage areas are encountered in practice. When a catchment is shaped in the form of a pear, with the narrow end towards the upstream and the broader end nearer the catchment outlet its hydrograph rise faster with a rather concentrated high peak. A catchment with the same area but opposite shape has a hydrograph that is slow rising and with a somewhat lower peak for the same amount of rainfall. From fig. 3-1 it can be observed that the Shaya river basin is shaped nearly in the form of a pear resulting with a fast rising and concentrated high peak hydrograph as clearly shown in the observed data.

3.2.2. Watershed Area

The vertical projection of the drainage divide on a horizontal plane gives the watershed area of the drainage basin. All the areas contained in the drainage basin may not contribute to its runoff hydrograph. Sink areas like lake, depression, swamps etc. termed as closed drainage are among those which are not contributing to the runoff. The actual delineated watershed area of Shaya river basin up to Robe stream gauging station is found to be 441.58 km².

3.2.3. Watershed Slope

Slope of the main stream and that of the land slope affects the shape of the hydrograph. 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, 1992)

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

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.

Larger slopes generate more velocity than smaller slopes and hence can dispose off runoff faster. As it can be observed from fig.3-2 the slope of the main stream of Shaya river basin varies from sharp steeply slope at the up stream areas to a smooth flatter slope in some areas around the out

let. In general the river basin can be considered as a watershed with a steep slope. This steepness of the slope has great impact on the shape of the runoff hydrograph, especially with its shorter time to peak.

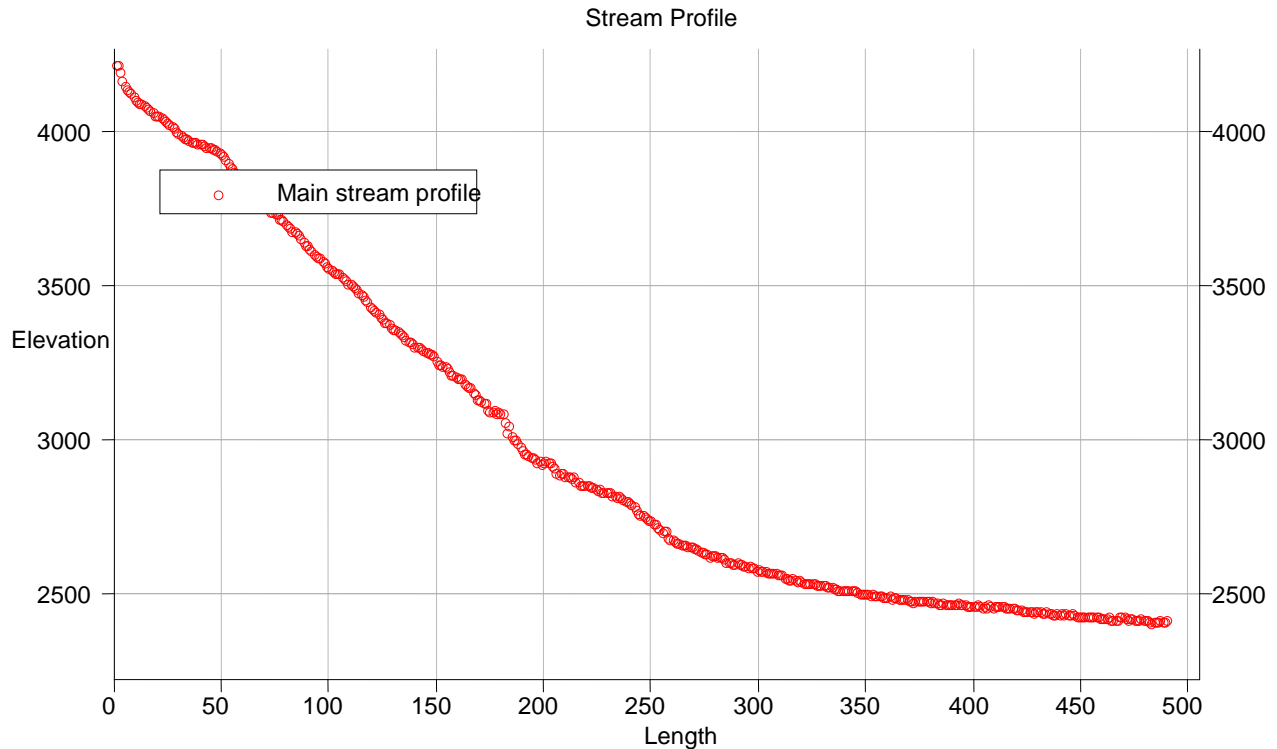


Figure 3-2 Cross sectional profile of the main stream of Shaya river basin

3.2.4. Watershed Order

Drainage areas may be characterized in terms of the hierarchy of stream ordering. The order of a watershed is the order of its highest order channel. The first order streams are defined as those channels that have no tributaries. The junction of two first order channels forms a second order channel. A third order channel is formed by the junction of two second order channels. The junction of two third order channels forms a fourth order channel and so on. This scheme of stream ordering is referred to as the Horton-Strahler ordering scheme. According to this stream ordering scheme the selected Shaya river watershed is found to be fourth order stream network.

3.3. Data Collection

3.3.1. General

Most of the rainfall- runoff modeling needs a properly measured and reliable stream flow and precipitation data. In addition data of soil type, land use, temperature, humidity and others are also required in some models. Obtaining reliable data over time and space is an essential step before modeling the rainfall-runoff. The required data for the models used, in estimating the runoff response of this study area, are the stream flow and precipitation data. Stages vs. mean velocities data are also used to relate rainfall intensity with stream velocity. These data are found from Ministry of Water and Energy and National Meteorological Service Agency of Ethiopia respectively.

3.3.2. Selection of Gauged Catchment

For the purpose of validation of the models used and calibration of parameters, collection of rainfall and stream flow data of a gauged catchment is required. In order to apply event based GIUH models, the first step in collection of the rainfall and stream flow data is the selection of a gauged catchment with automatic recorder. The next step is selecting a stream gauging stations where an automatic or hourly rainfall recording stations can be found in or near the boundary of the stream gauged catchment. The third step is checking the availability of a reliable hourly stream flow data and the corresponding hourly rainfall intensity data for the selected events.

Based on these and following the recommendation of Shaw, catchments whose size do not exceed 1000km², Shaya river basin near Robe town was selected.

3.3.3. Stream Flow Data

The source of stream flow and other hydrological data is the Ethiopian Ministry of Water and Energy Hydrology Department. The Shaya river stream gauging station near Robe town is found at 7° 10' N latitudes and 39° 58' E longitudes. The hydrological data required for this study was the stream flow record taken from automatic and continuous water level recorder. Among the different continuous water level recordings of the stream flow, found in a chart form, eight peak discharge hydrographs were selected. The corresponding rating equations are also obtained from

the same department. The water level readings from each chart were discretized for each flood event selected and the discharge hydrograph was calculated using the rating equations provided. Table 3-1 and figure 3-3 shows samples of stream flow data and runoff hydrographs.

Table 3-1 Sample of stream flow data

Event - 04				
1		2	3	4
Date	Time	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)
14/8/98	0	6:00 PM	1.400	11.61
	1	7:00 PM	1.400	11.61
	2	8:00 PM	1.980	43.23
	3	9:00 PM	2.960	148.28
	4	10:00 PM	2.740	118.79
	5	11:00 PM	2.500	90.58
	6	12:00 AM	2.230	63.68
15/8/98	7	1:00 AM	2.100	52.52
	8	2:00 AM	1.900	37.58
	9	3:00 AM	1.795	30.79
	10	4:00 AM	1.770	29.28
	11	5:00 AM	1.730	26.96
	12	6:00 AM	1.690	24.73
	13	7:00 AM	1.650	22.60
	14	8:00 AM	1.620	21.08
	15	9:00 AM	1.590	19.61
	16	10:00 AM	1.570	18.66
	17	11:00 AM	1.540	17.29
	18	12:00 PM	1.520	16.40
	19	1:00 PM	1.500	15.54
	20	2:00 PM	1.480	14.71
	21	3:00 PM	1.465	14.10
	22	4:00 PM	1.455	13.70
	23	5:00 PM	1.440	13.11
	24	6:00 PM	1.435	12.92
	25	7:00 PM	1.420	12.35

3.3.4. Rainfall Data

The source of meteorological data in Ethiopia is the National Meteorological Agency (NMA). The only available rain gauging station with automatic recorder, within and around the boundary of the study area, is the Bale Robe weather station. The Bale Robe rain gauging station established since 1968 is a class one station located at 7° 13' N latitudes and 40° 00' E longitudes. The corresponding discretized hourly rainfall intensity values of this station, for the peak stream flow events, were collected from NMA. These hourly rainfall intensity data were used as a precipitation input for this study.

Table 3-2 gives the details of selected periods of various storm events. These events are selected among different peak events based on the availability of coinciding rainfall runoff data only. And Table 3-3 shows sample of rainfall intensity data.

Table 3-1 Periods of various rainfall runoff events

Event No	Period of the event
1	22.07.1998 at 7:00 PM. to 23.07.1998 at 7:00PM.
2	24.07.1998 at 7:00 PM. to 25.07.1998 at 3:00PM.
3	31.07.1998 at 3:00 PM. to 01.08.1998 at 11:00PM.
4	14.08.1998 at 6:00 PM. to 15.08.1998 at 7:00PM.
5	15.08.1998 at 9:00 PM. to 16.08.1998 at 7:00PM.
6	11.09.1998 at 3:00 PM. to 12.09.1998 at 12:00PM.
7	03.10.1998 at 5:00 PM. to 04.10.1998 at 8:00PM.
8	20.10.1998 at 4:00 PM. to 21.10.1998 at 7:00AM.

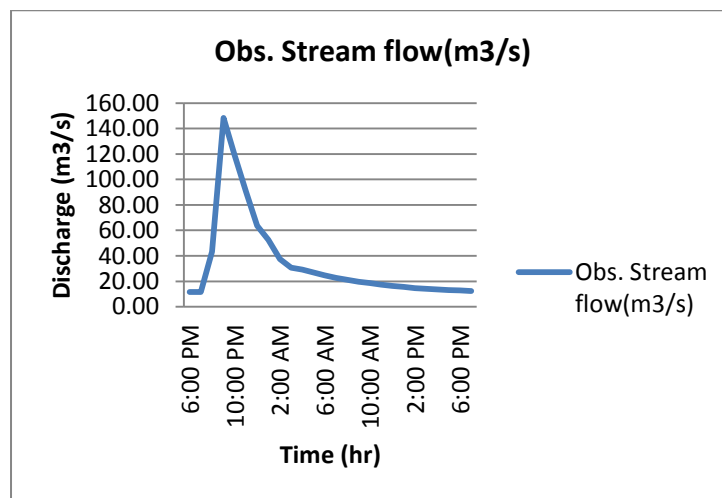


Figure 3-3 Sample of stream flow hydrograph

Table 3-2 Sample of rainfall intensity data

Station: Robe

Region : Balie

Element: Rain fall Intensity

Month: Oct. Year : 1998

Date/ Time	3	4	5	6	7	8	9	10	12	18		
0 - 1												
1 - 2									1.00			
2 - 3												
3 - 4												
4 - 5		7.60							2.20			
5 - 6				2.60	3.20							
6 - 7					2.80							
7 - 8												
8 - 9			1.20									
9 - 10							4.00					
10 - 11												
11 - 12												
12 - 13												
13 - 14						1.60						
14 - 15							6.00		2.60			
15 - 16	12.20						1.00	4.00		0.20		
16 - 17				2.20				4.30		2.40		
17 - 18	0.80		0.50		6.80					0.80		
18 - 19										1.20		
19 - 20										0.50		
20 - 21												
21 - 22												
22 - 23												
23 - 24									1.20			
Total												
H in 10m												
H in 20m												
H in 30m												
H in 1 hr	12.20	7.60	1.20	2.60	6.80	1.60	6.00	4.30	2.20	2.40		

Time is local standard
time

H = Highest rain fall
m =
Minute
hr = Hour

4. METHODOLOGY

4.1. Excess Rainfall and Direct Runoff Determination

4.1.1. Excess Rainfall Determination

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. Among the other techniques the Soil Conservation System (SCS) curve number method is very often used for the estimation of excess rainfall particularly when the catchment is ungauged. For this study a Φ -index method was selected for infiltration rate computation, to determine the excess rainfall hyetograph.

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).

The following are 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 .

$$\text{Volume of direct runoff } (V_d) = \sum_{n=1}^n Q_n * \Delta t \quad 4.01$$

$$\text{Depth of direct runoff } r_d = \frac{\text{volume of direct runoff } V_d}{\text{watershed area}(A)} \quad 4.02$$

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

$$r_d = \sum_{m=1}^M (R_m - \Phi \Delta t) \quad 4.03$$

Where:

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

Δt : time interval length (hr),

R_m : observed rainfall (mm) in the 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.

The ordinate of the ERH are found by subtracting $\Phi\Delta t$ from the ordinate of the observed rainfall hyetograph, neglecting all time intervals in which the observed rainfall depth is less than $\Phi\Delta t$.

4.1.2. Base Flow Separation and Direct Surface Runoff Determination

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 among the four methods available. 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.

The observed direct surface runoff hydrographs have been computed by deducting the base flows from each stream flow hydrograph. These hydrographs are used only for the estimation of excess rainfall hyetographs and for comparison purpose with the direct runoff hydrographs derived by the conceptual models. However, this is not always possible especially for the ungauged catchments, since stream flow data can not be found. Alternatively other methods such as the SCS method may be applied for estimation of the excess rainfall provided that all the required information like land use, soil type, and hydrological and antecedent soil moisture conditions are well known for the estimation of the curve number. In the absence of this information the value of the Φ -index can be estimated from the rainfall runoff records of the neighboring catchments having the same hydro meteorological characteristics.

4.2. Geomorphological Characteristics Extraction from Digital Elevation Model (DEM)

Extraction of geomorphological characteristics was being done by manually processing the toposheets of the catchment of the required basin. This difficult, time consuming and tedious task had been discouraging the users from adopting the various approaches available for runoff estimation. However, the provision of Geographical Information System (GIS) package soft wares like ILWIS, GRASS, ERDAS; etc have helped the users to overcome the difficulty, and to process the information required easily without consuming more time and effort.

The GIS software Integrated Land and Water Information System (ILWIS) used in this study has been developed in 1988 at ITC, the Netherlands. The software imports different types of data like raster maps, vector file remote sensing and tabular data etc, using its conversion program. It integrates tabular and spatial data bases and performs different map and tabular calculations. ILWIS is designed to execute activities like image processing and creating DEM maps from contours. Spatial filter and functions are available for producing slope, aspect map, data processing and others in a simplified and user friendly approach.

Newly implemented routine, called DEM hydro processing module allows to extract, through several steps (Figure 4-1), 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 RB, RL, and RA.

4.2.1. 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 and Energy GIS Department, and the river basin extraction was operated in Arc GIS using the Arc Hydro extension.

DEM preprocessing was executed through the following consecutive Arc Hydro process: Fill sink, flow accumulation, flow direction, stream definition, Stream Segmentation, Catchment Grid Delineation, Catchment Polygon processing, Drainage line processing, Adjoint Catchment processing, Longest flow path for Adjoint Catchment and Point Delineation.

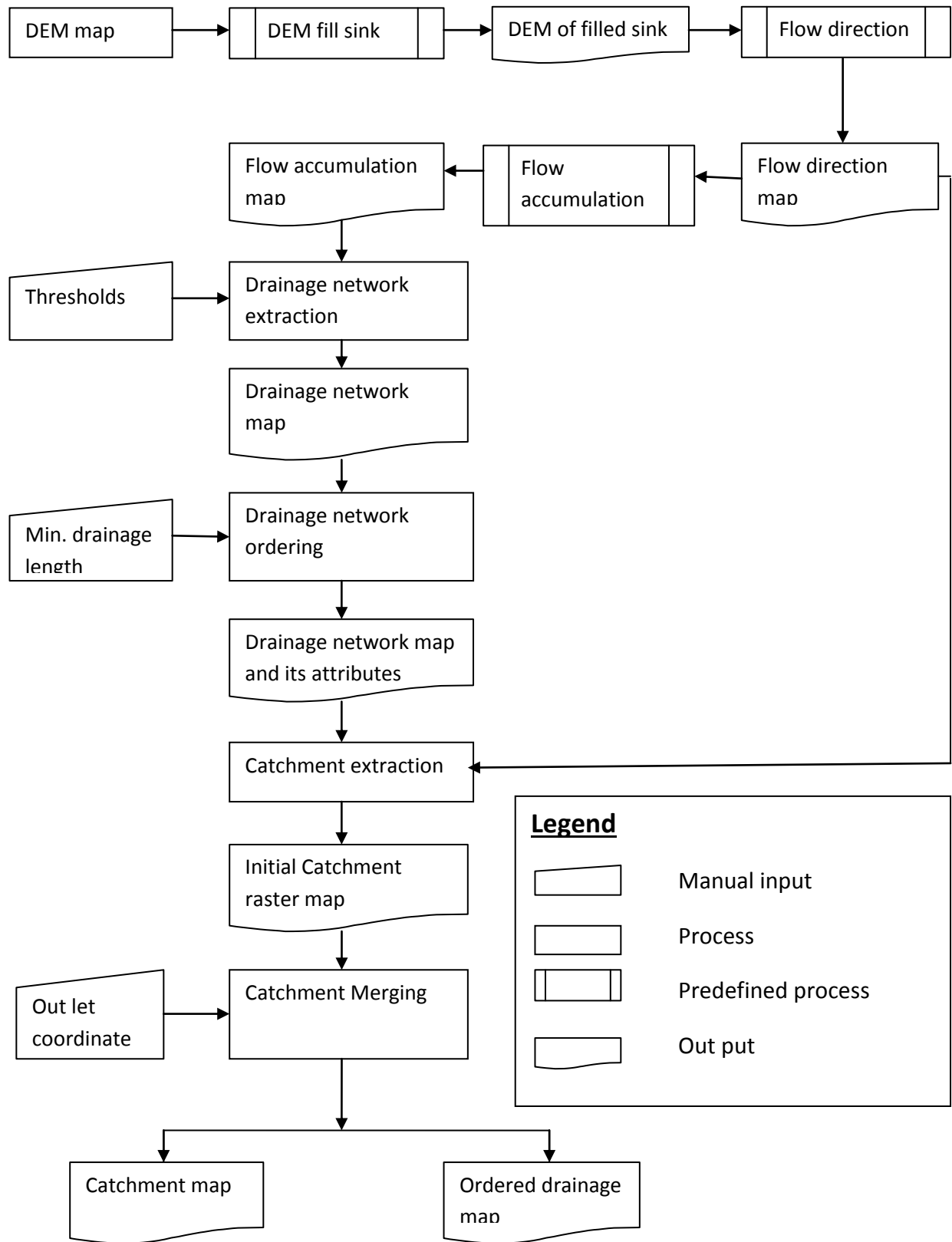


Figure 4-1 DEM processing algorithm of ILWIS software

The DEM of the catchment of Shaya river basin extracted following the above procedure was imported by ILWIS to determine the Horton's ratio and other geomorphological parameters. The same DEM was used to determine the time area diagram component of the GIUH based Clark's model by using Arc GIS.

4.2.2. Geomorphological Parameters and Horton's Ratio Computation

The main characteristics of drainage basins that reflect hydrologic behavior of the catchment are related to the physical characteristics of the drainage basin as well as of the drainage network. Physical characteristics of the drainage basin include drainage area, basin shape, ground slope, and centeroid. Where as, channel characteristics include channel order, channel length, channel slope, channel profile, and drainage density.

The geomorphological parameters of a catchment and the three 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.

The bifurcation ratio, for instance, is to be determined from the stream ordering characteristics of the basin. The order of the basin is the order of its highest-order channel. It can be stated that the uppermost channel receives water from overland surface and direct towards the outlet through the drainage network. This upper most channel joins another channel and form the higher order channel, and so on. The first order channel is defined as those channels that receive water entirely from overland surface and does not have tributaries. The junction of two first order streams forms the second order stream. It means the higher order drain carries more water than the lower order drains. A second order channel receives flow from the two first order channels and from overland flow from surface and it might receive the flow from another first order stream. When the two second order streams join together form the third order stream, and so on. This scheme of stream ordering is referred to as the Horton-Strahler ordering scheme (Horton, 1945; Strahler, 1957).

4.2.2.1. Bifurcation Ratio (RB)

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.

$$R_B = \frac{N_i}{N_{i+1}} \quad 4.04$$

Where:

R_B is the bifurcation ratio, and

N_i and N_{i+1} are the numbers of streams in order i and $i+1$ respectively, $i = 1, 2 \dots \Omega$ where Ω is the highest stream order of the watershed. The value of bifurcation ratio; R_B for watersheds varies between 3 and 5.

4.2.2.2. Stream Length Ratio (RL)

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. The R_L is mathematically expressed as follows.

$$R_L = \frac{\bar{L}_{i+1}}{\bar{L}_i} \quad 4.05$$

Where,

\bar{L}_i is the average length of channels of order i and is

$$\bar{L}_i = \frac{1}{N_i} \sum_{j=1}^{N_i} L_{j,i} \quad 4.06$$

Generally R_L varies between 1.5 and 3.5.

4.2.2.3. Stream Area Ratio (RA)

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. The area ratio among different orders can be quantified as:

$$R_A = \frac{\bar{A}_{i+1}}{\bar{A}_i} \quad 4.07$$

Where

\bar{A}_i is the average area of order i and is computed as:

$$\bar{A}_i = \frac{1}{N_i} \sum_{j=1}^{N_i} A_{i,j} \quad 4.08$$

R_A is the stream area ratio,

The value of R_A for watersheds varies between 3 and 6.

4.2.2.4. Stream Length

The summation of the length of each stream of order i give the total stream length of the order (L_i). The total stream length divided by the number of stream segments (N_i) of that stream order gives the mean stream length of the order. The stream length of a basin (L_Ω) is the total length of the stream segments of the maximum stream order or the basin order.

4.3. Excess Rainfall and Velocity Relationship Development

The most sensitive model parameters identified in different literatures are the dynamic velocities of the hill slope and stream flow. Rodriguez and Valdes (1979) assumed that the flow velocity at any given moment during the storm event can be taken as constant through the basin. They used the peak discharge velocity as the constant stream flow velocity in the derivation of GIUH for a given rainfall runoff moment. This peak discharge velocity can easily be determined where stream gauging facility is available. However in the case of ungauged catchment this approach can no longer be applied, since the peak discharge is unknown. For such conditions developing a relationship between excess rainfall intensity and velocity is proposed as a solution for the problem (Kumar et.al.2002). According to this proposal, two approaches are commonly used based on the availability of information at the outlet of the catchment. The first one is where

cross section properties of the outlet station are known and the other approach is where stage and velocity relationship is already determined or can easily be determined. This study is done based on the second approach.

4.3.1. Based on the Stage Velocity Relationship

This approach is used where the geometric properties and the Manning's roughness coefficient (n) of the stream gauging station are not known, but the velocities corresponding to discharges passing through the gauging station at different depths of flow can be obtained from the observations.

The steps involved include:

1. For different depths of flow the discharge and the corresponding velocities are known by observation.
2. Let these velocities and discharges be the equilibrium velocities V_e and the corresponding equilibrium discharge Q_e .
3. For these Q_e values, find the corresponding intensities I of excess-rainfall from the expression:

$$i = Q_e / (0.2778 \times A_c) \quad 4.09$$

4. From the pairs of such V_e and I develop the relationship between the equilibrium velocity and the excess rainfall intensity in the form of; $v = ai^b$ using method of least squares. Here a, and b are the regression coefficients.

It is to be noted that, for ungauged catchments, the stage and velocity relationship information can be easily obtained by gauging the stream intermittently for all ranges of depths of flow at the assumed outlet cross section.

4.4. Time Area Diagram Preparation

The time-area methods were developed in recognition of the importance of the time distribution of rainfall on runoff (Singh, 1992). The basic idea of these methods is the time-area histogram, which indicates the distribution of partial watershed areas contributing to runoff at the watershed outlet as a function of travel time. These areas are bounded by isochrones. An isochrone is a

contour joining those points in the watershed that are separated from the outlet by the same travel time. The time-area diagram is a graph of cumulative watershed area whose time of travel is less than or equal to a given value, say $t = i\Delta t$, where $i = 1, 2 \dots n$, plotted against the value of t . In other words, it is the graph of the watershed area enclosed by an isochrone against time.

Empirical formulas are proposed to determine the time of concentration and travel time of the catchment, required to draw the isochrones. Drawing of the isochrones was the difficult and time demanding task for the preparation of time area diagram. However, the application of GIS based software makes it simple and less time consuming. Arc GIS is used in this study to draw the isochrones map and time area histograms for the provided time intervals.

4.4.1. Arc-GIS Application

The time-area histogram represents the area of the basin contributing to the flow at the basin outlet at any given time after the application of a unit excess rainfall. Reflecting the shape and drainage properties of the basin, it is the most important parameter for derivation of the translation hydrograph. The filled sink Digital Elevation Model (DEM) of the basin, which is 10x10 meter cell sized grids having elevation values, is used for this purpose. Firstly the flow direction grid is drawn using the filled sink DEM and finally the flow length grid of the basin, which calculates the distance or weighted distance along the flow path from the outlet point is determined by the special analyst tool hydrology extension of the software. These distances are turned to travel time values. Finally by converting the number of cells to area, the time-area histogram is derived. These steps are explained in the following sections

4.4.2. Time of Concentration

Time of concentration is the time required for excess rainfall to travel from the most remote point of the basin to the outlet. At the end of this time, entire basin will be contributing to the flow at the outlet. In literature, several equations are available for calculation of time of concentration.

In this study for a given storm event the estimation of peak velocity V using the highest rainfall excess of the event is made by using the relationship between velocity and intensity of excess rainfall developed in section 4.3.

Hence the time of concentration is calculated from the relation;

$$T_c = 0.2778L/V \quad 4.10$$

Where:

L is length of the main channel in Km, and

V is the peak velocity in m/sec.

4.4.3. Flow Length Grid

The Flow length Grid represents for each cell, the total travel length of the water droplet from that cell to the basin outlet along the direction of flow. Required inputs for the calculation are flow direction grid and weight grid, where the use of weight grid is optional.

In the weighted grid condition the weighted grid is prepared and used with flowdirection grid for the calculation of flowlength grid. And if the weighted grid condition is not considered only the flowdirection grid is used in calculation, where the weight of each cell is accepted as the same and equal to 1. No weighted grid condition is selected for this study.

After properly inserting the required raster inputs and running the function the flow length grid is obtained for the entire river basin.

4.4.4. Travel Time Grid

After computing the flow length grid, the next step is calculating the travel time values. The maximum value of the flow length grid belongs to the remotest cell of the basin to the outlet. Travel time of flow from that cell to outlet gives the time of concentration value of the basin.

The travel time grid of the basin is then determined from equation:

$$\text{Travel Time Grid} = \frac{T_c}{\text{Max.of cell travel length}} \times \text{flow length grid} \quad 4.11$$

by using the raster calculator option of the spatial analyst tool.

The time-area histogram of the basin is determined from travel time grid of the basin. First, histograms of the travel time grid are derived for different time intervals. It is observed that as the interval gets smaller shape of the histogram resembles complex hydrographs, and as it gets bigger the shape roughly looks like a single peaked hydrograph.

4.5. GIUH Derivation using Geomorphological Parameters

The GIUH theory was introduced by Rodriguez-Iturbe and Valdes (1979) by relating the peak discharge and time to peak discharge with the geomorphologic characteristics of the catchment and a dynamic velocity parameter. This work which integrates the geomorphologic details and the climatologic characteristics of the basin, in a framework of travel time distribution, is a benefit for stream flow synthesis in the basin having no information of flow data. This formulation of GIUH is based on the probability density function (*pdf*) of the time history of a randomly chosen drop of effective rainfall arrived to the trapping state of a hypothetical basin, treated as a continuous Markovian process, where the state is the order of the stream in which the drop is located at any time. The value at the mode of this *pdf* produces the main characteristics of GIUH. Rodriguez-Iturbe and Valdes (1979) derived the peak and time to peak characteristics of the IUH as function of Horton's order ratios (Horton, 1945), and are expressed as follows (Kumar et al., 2007).

$$q_p = 1.31R_L^{0.43} V/L_\Omega \quad 4.12$$

$$t_p = 0.44(L_\Omega/V)(R_B/R_A)^{0.55}(R_L)^{-0.38} \quad 4.13$$

Where;

L_Ω = the length in Kilometer of the stream of order Ω .

V = the expected peak velocity, in m/sec.

q_p = the peak flow, in units of inverse hours.

t_p = the time to peak, in hours

R_B , R_L and R_A = the bifurcation, length and area ratios given by the Horton's laws of the stream numbers, lengths and areas respectively.

Empirical results indicate that for natural basins the value for R_B normally ranges from 3 to 5, for R_L from 1.5 to 3.5 and for R_A from 3 to 6 (Smart, 1972).

Multiplying equations (4.12) and (4.13) a non-dimensional term $q_p * t_p$ is derived as;

$$q_p * t_p = 0.5764(R_B/R_A)^{0.55}(R_L)^{0.05} \quad 4.14$$

The term $q_p * t_p$ is not dependent on the velocity and thereby on the storm characteristics and hence, it is a function of only the geomorphologic characteristics of the basin. The dynamic velocity parameter in the formulation of GIUH incorporates the effect of climatic variation. Rodriguez-Iturbe et al. (1979) showed the dynamic velocity parameter of the GIUH can be taken as the velocity at the peak discharge time for a given rainfall-runoff event in the catchment.

4.6. GIUH based Clark Model Development

The Clark IUH model is based on the concept that IUH can be derived by routing the unit excess rainfall in the form of a time–area diagram through a single linear reservoir. The mathematical model developed for Clark parameters estimation is used in this study. For the derivation of the IUH, the Clark model uses two parameters viz. time of concentration (T_c) in hour and storage coefficient (R) in hour of a single linear reservoir, in addition to the time–area diagram.

4.6.1. Derivation of Clark Model IUH and D-hour Unit Hydrograph

After preparing the time area diagram and determining the time of concentration (T_c) and storage coefficient (R), the following governing equation can be applied to determine the Clark model IUH:

$$U_i = CI_i + (1 - C)U_{i-1} \quad 4.15$$

Where;

$U_i = i^{\text{th}}$ ordinate of the IUH

C & $(1-C)$ = the routing coefficients.

And $C = \Delta t / (R + 0.5\Delta t)$

Δt = computational interval in hours

$I_i = i^{\text{th}}$ ordinate of the time-area diagram

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

$$U_i = \frac{1}{N} (0.5U_{i-N} + U_{i-N+1} + \dots + U_{i-1} + 0.5U_i) \quad 4.16$$

Where,

U_i = the i^{th} ordinate of the UH of D-hour duration and computational interval Δt hour, &
 N = the number of computational intervals in D-hour and is equal to $D/\Delta t$.
 u_i = the i^{th} ordinate of the IUH.

4.6.2. Derivation of Unit Hydrograph

According to previous studies, the steps involved in derivation of a UH of a specific duration using the GIUH based Clark model approach are as follows:

1. Evaluate the geomorphological parameters like RA, RB and RL for consecutive orders and the overall values of these parameters for the catchment as a whole.
2. Compute the excess-rainfall hyetograph either by uniform loss rate procedure or by soil conservation service (SCS) curve number method or by any other suitable method.
3. For a given storm, estimate the peak velocity V for the highest rainfall excess by using the relationship between velocity and intensity of rainfall excess (Kumar *et al.*, 2002).
4. Compute the time of concentration (T_c) in hour using the equation:

$$T_c = 0.2778 L/V \tag{4.17}$$

Where,

L is length of the main channel in km, and

V is peak velocity in m/sec.

Considering this T_c as the largest time of travel, compute the ordinates of cumulative isochronal areas corresponding to integral multiples of computational time interval with the help of non-dimensional relation between cumulative isochronal area and the percent time of travel. This describes the ordinates of the time–area diagram at each computational time interval.

5. Compute the peak discharge (q_p) of IUH given by Equation (4.12).
6. Assume two trial values of the storage coefficient of GIUH based Clark model as R_1 and R_2 . Compute the ordinates of two IUHs by Clark model using time of concentration T_c as obtained in step (4) and two storage coefficients R_1 and R_2 , respectively using Equation (4.15). Compute the IUH ordinates at a very small time interval say 0.1 or 0.05 h so that a better estimate of peak value may be obtained.
7. Find out the peak discharges Q_{pc1} and Q_{pc2} of the IUHs obtained for Clark model for the storage coefficients R_1 and R_2 , respectively at step (6).

8. Compute the value of objective function, using the relation:

$$FCN1 = (q_p - Q_{pc1})^2 \quad 4.18$$

$$FCN2 = (q_p - Q_{pc2})^2 \quad 4.19$$

9. Compute the first numerical derivative FPN of the objective function FCN with respect to parameter R as:

$$FPN = \frac{FCN1-FCN2}{R_1-R_2} \quad 4.20$$

10. Compute the next trial value of R using the following governing equations of Newton–Raphson’s method:

$$\Delta R = \frac{FCN1}{FPN} \quad 4.21$$

And

$$R_{NEW} = R_1 + \Delta R \quad 4.22$$

11. For the next trial, consider $R_1 = R_2$ and $R_2 = R_{NEW}$ and repeat steps (6) to (10) till one of the following criteria of convergence is achieved.

- (a) $FCN2 = 0.000001$
- (b) No. of trials exceeds 200
- (c) $ABS(\Delta R)/R_1 = 0.001$

12. The final value of storage coefficient (R_2) obtained as above is the required value of the parameter R corresponding to the value of time of concentration (T_c) for the Clark model.

13. Compute the IUH using the GIUH based Clark model with the help of final values of storage coefficient (R) as computed in step (12), time of concentration (T_c) and the time–area diagram.

14. Compute the D-hour UH using the relationship between IUH and UH of D-hour as given by Equation (4.16).

4.7. GIUH based Nash Model Development

The Nash model (Nash, 1957) 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 viz. number of linear reservoirs (n), which is dimensionless and storage coefficient (k) in hour.

4.7.1. Derivation of Nash Model IUH and D-hour Unit Hydrograph

Once the model parameters are determined, the following governing equation can be used to derive the Nash IUH;

$$U(t) = \frac{1}{k\Gamma(n)} (t/k)^{n-1} e^{-t/k} \quad 4.23$$

Where,

- U (t) = denotes IUH ordinates in hour₋₁,
- t = sampling time interval in hour,
- n and k are parameters of Nash IUH model and
- Γ is γ function.

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

$$U(D, t) = \frac{1}{D} (I(n, t/k) - I(n, (t - D/k))) \quad 4.24$$

Where,

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

4.7.2. Derivation of Unit Hydrograph

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 (4.23) with respect to time t to zero, t becomes the time to peak discharge, t_p

$$\frac{\partial}{\partial t} \ln[U(t)] = \left[-\frac{1}{k} + \frac{(n-1)}{t} \right] = 0 \quad 4.25$$

$$t = t_p = K(n - 1) \quad 4.26$$

Substituting the value of t_p of equation (4.26) in equation (4.23) and simplifying gives;

$$q_p = \left[\frac{1}{K\Gamma(n)} \right] e^{-(n-1)} (n - 1)^{(n-1)} \quad 4.27$$

From equations (4.26) and (4.27),

$$q_p * t_p = \left[\frac{(n-1)}{\Gamma(n)} \right] e^{-(n-1)} (n - 1)^{(n-1)} \quad 4.28$$

From GIUH formula,

$$q_p * t_p = 0.5764(R_B/R_A)^{0.55} (R_L)^{0.05} \quad 4.29$$

The relation of Nash model parameters and that of the GIUH parameters can be expressed by equating the two equations, (4.28) and (4.29) as written below,

$$\left[\frac{(n-1)}{\Gamma(n)} \right] e^{-(n-1)} (n - 1)^{(n-1)} = 0.5764(R_B/R_A)^{0.55} (R_L)^{0.05} \quad 4.30$$

All the terms in the right hand side of equation (4.30) 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 expressed as,

$$n = 3.29 \left(\frac{R_B}{R_A} \right)^{0.78} R_L^{0.07} \quad 4.31$$

The Nash model parameter k for the given velocity V is obtained using the above relations and the known value of the parameter n as follows,

$$k = \frac{0.44L\Omega}{V} \cdot \left[\frac{R_B}{R_A} \right]^{0.55} \cdot R_L^{-0.38} \cdot \frac{1}{(n-1)} \quad 4.32$$

The derived values of n and k are used to determine the complete shape of GIUH based Nash model using Equation (4.23). Subsequently, the D-hour UH is obtained using the relationship between IUH and UH of D-hour as given by Equation (4.24).

4.8. Computation of Direct Surface Runoff (DSRO)

For a storm event whose excess rainfall values are known at D-hour interval the direct surface runoffs (DSRO) are computed using convolution based on the D-hour unit hydrograph. The convoluted hydrograph ordinates can be computed as shown below;

$$Q(t) = \Delta t \sum_i^n (U(D, t - (i - 1)\Delta t) * I_i) \quad 4.33$$

Where;

U (D, t) = ordinate of D hour unit hydrograph at time t,

I_i = excess-rainfall intensity at ith interval (i.e., at time = Δt * i),

n = number of excess rainfall blocks, and

Δt = computational time interval.

4.9. HEC-HMS based Clark Model Development

The Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) is designed to simulate the precipitation runoff process of watershed systems. It is designed to be applicable in a wide range of geographic areas for solving the widest possible range of problems.

In the HEC-HMS model the transformation of rainfall excess to stream flow is accomplished by the unit hydrograph procedure. Three unit hydrograph methods are available in the model, namely Clark's, Snyder's and SCS-methods. Among the three, Clark's unit hydrograph method of the model is selected for this study for better estimation of the runoff. All the precipitation losses like; interception, depression storage and infiltration are incorporated in the program to determine the excess rainfall. Initial and constant loss rate functions are used for this purpose.

Clark (1945) suggested that IUH can be derived by routing the unit inflow in the form of time area concentration curve, which is prepared from isochronal map, through a single linear reservoir. There are two parameters of the Clark model: the time of concentration (T_c) and the storage coefficient (R). The parameter (T_c) represents the travel time of a water particle from the farthest point in a basin to its outlet; while the parameter R is an attenuation constant which has the dimension of time. The parameter R is used to account for the effect of storage in the river channel on the hydrograph. Apart from the two parameters the Clark model uses time area concentration curve also (Kumar et al., 2007).

The loss rate parameters along with Clark's parameters are optimized using the parameter optimization technique of the HEC-HMS model.

4.10 Nash IUH Model Development

Nash considered that the IUH can be obtained by routing the instantaneous inflow through a cascade of linear channels (n numbers) with equal storage coefficient. The out flow from the first reservoir is considered as inflow into the second reservoir, and so on. The outflow from the n th reservoir yields the IUH. The two parameters n and k may be computed by making an analysis of the observed rainfall-runoff data on the catchment as follows:

The first moment of the IUH about the origin ($t = 0$): $M1 = nk$

The second moment of the IUH about the origin ($t = 0$):

$$M2 = n(n + 1) k^2 \quad 4.34$$

By routing through the cascade of n -reservoirs, it can be shown that

$$MQ1 - MI1 = nk \quad 4.35$$

$$MQ2 - MI2 = n(n + 1)k^2 + 2nk MI1 \quad 4.36$$

Where;

$MQ1, MQ2$ are the first and second moments of direct runoff about the origin, and

$MI1, MI2$ are the first and second moments of the effective rainfall about the origin.

The parameters n and k may be evaluated by solving these two equations. The value of the parameter n , which is a shape parameter, is a measure of the catchment channel storage, which defines the shape of the IUH. The parameter K (delay time, hr), which is a scale parameter, represents the dynamics of rainfall-runoff process in the catchment.

4.11 Model Evaluation

The following objective functions were employed for evaluation of the DSRO hydrographs computed by GIUH based Clark and GIUH based Nash model approaches in comparison with the observed DSRO hydrographs as well as with the DSRO hydrographs estimated by the HEC-HMS package Clark and Nash IUH models.

4.11.1. Efficiency

Efficiency (EFF) is computed as follows:

$$EFF = \frac{\sum_{i=1}^n (Q_{oi} - \bar{Q})^2 - \sum_{i=1}^n (Q_{oi} - Q_{ci})^2}{\sum_{i=1}^n (Q_{oi} - \bar{Q})^2} * 100 \quad 4.38$$

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.

4.11.2. Root Mean Square Error (RMSE)

Root mean square error (RMSE) is computed as follows:

$$RMSE = \sqrt{\frac{\sum_{i=1}^n (Q_{oi} - Q_{ci})^2}{n}} \quad 4.39$$

Where;

. n = number of ordinates

4.11.3. Percentage Error in Peak (PEP)

Percentage error in peak (PEP) is computed as follows:

$$PEP = \frac{(Q_{op} - Q_{cp})}{Q_{cp}} \times 100 \quad 4.40$$

Where, Q_{op} = *observed peak discharge*

Q_{cp} = *computed peak discharge*

4.11.4. Cumulative Error Function (ERR)

Cumulative error function in time to peak and peak discharge (ERR) according to Lee.et.al (1972) is computed as follows:

$$ERR = \left[\left(\frac{(Q_{op} - Q_{cp})}{Q_{cp}} \right)^2 + \left(\frac{(T_{Op} - T_{Cp})}{T_{Cp}} \right)^2 \right] \quad 4.41$$

Where, T_{Op} = *Time to peak of observed peak discharge*

T_{Cp} = *Time to peak of computed peak discharge*

5. RESULTS AND DISCUSSION

5.1. Computation of Excess-Rainfall and DSRO Hydrographs

The single peaked stream flow hydrographs at different events were selected from stream gauging flow charts drawn by automatic recorder at the out let of the study area. These hydrograph chart readings were interpreted and the values at an hourly interval were recorded in a tabular form. The corresponding rainfall storm data were also collected from the automatic rain gauge recording chart and the hourly rainfall intensity data were computed. The direct surface runoff was computed by subtracting the base flow from the observed discharge of the corresponding rainfall runoff event. Among the different methods available for the estimation of rainfall excess the Φ -index approach was used in this study.

The observed direct surface runoff data have been used for the comparison purpose of the runoff hydrographs computed by using GIUH based Clark and Nash models and HEC-HMS based Clark model. These data are also used in determining the Nash IUH model parameters (n and K), which have also been used as additional comparison model. Using the same procedure mentioned in section 4.1 all the rainfall-runoff data of the eight events were analyzed. The rainfall hyetographs are shown in fig. 5-11 to 5-18 along with the direct surface runoff hydrographs computed by the proposed models.

5.2. Geomorphological Characteristics of the Catchment

The 90 X 90 m DEM map of Ethiopia was used to delineate the catchment of Shaya river basin using Arc GIS application. The DEM map of Shaya river catchment shown in fig. 5-1 was imported by the GIS based ILWIS software in order to determine the necessary geomorphological parameters of the catchment, following the procedures and steps specified in section 4.2.1 and 4.2.2.

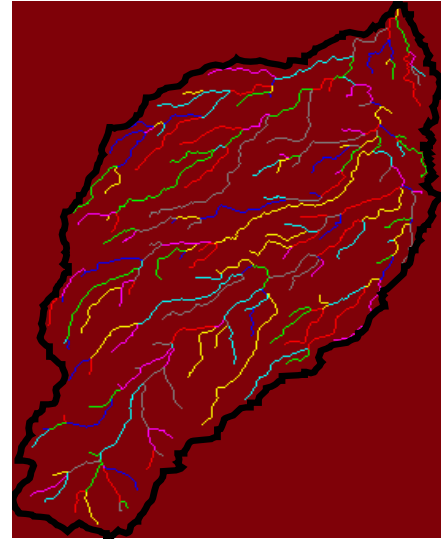


Figure 5-1 Shaya river catchment DEM map Figure 5-2 Shaya river basin stream ordering map

The stream ordering map of the catchment is shown in fig. 5-2 and the river is found to be 4th order according to the stream ordering definition.

The Horton Statistics operation calculated; the stream order, the number of streams, the average stream length (km), and the average area of catchments (km²). The output, which was used to construct so-called Horton plots, is then stored in a table. The Horton plot, 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 (2006). 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 plots and the Horton statistics outputs of the catchment are shown in figure 5-3 and table 5-1 respectively.

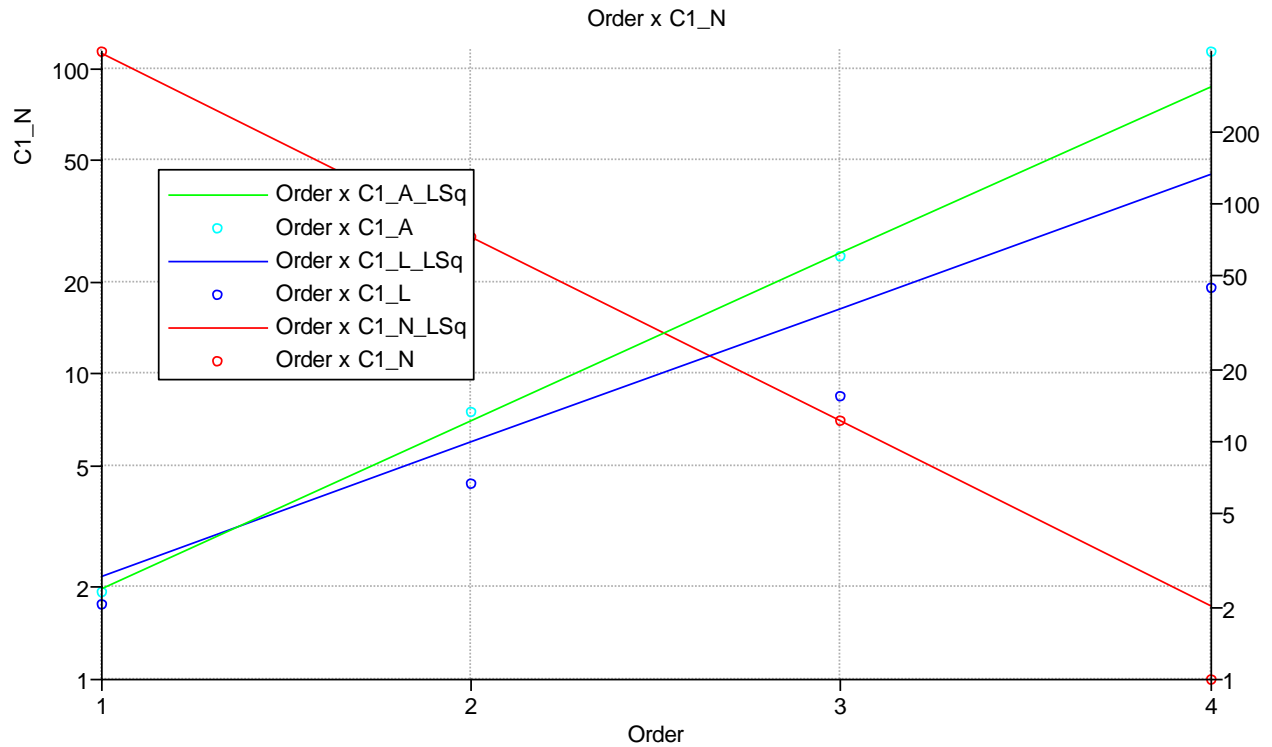


Figure 5-3 Horton plot of Shaya river catchment

In the Horton plot the meaning of the legends are:

- C1_N is the number of streams,
- C1_L is the average stream length (km),
- C1_A is the average area of catchments (km²),
- 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 5-1 Horton statistical outputs of the catchment

Order	C1_N	C1_L	C1_A	C1_N_LSq	C1_L_LSq	C1_A_LSq	R _B	R _L	R _A	L _Ω
1	113	2.05	2.32	112.841	2.165	2.411	4.02	2.75	5.07	44.64
2	28	6.63	13.21	28.084	5.947	12.233				
3	7	15.47	59.73	6.99	16.335	62.07				
4	1	44.64	441.58	1.74	44.871	314.944				

It is shown in the above table that the three Horton's ratios namely bifurcation ratio ($R_B=4.02$), stream-length ratio ($R_L=2.75$) and stream-area ratio ($R_A=5.07$) are found to be within their range of limitation; 3 to 5, 1.5 to 3.5 and 3 to 6 respectively. The length of the main stream of Shaya river basin up to Robe gauging site is calculated to be 52.47 kilometers. And also the length of the highest order stream is the length in kilometers of the stream of the highest order. In this study the length of the highest stream order L_Ω , i.e. the fourth order stream is found to be 44.64 kilometers.

5.3. Excess Rainfall and Velocity Relationship

The peak stream flow velocity is the dynamic parameter of the geomorphological models that needs to be estimated as accurate as possible. As discussed in section 4.3 two approaches are recommended to estimate the peak velocity at the stream gauging section for a given rainfall intensity. As the geometric properties of the gauging section and the Manning's roughness for the river under study are not known the first approach could not be applied. Instead the second approach was used to determine the velocity and rainfall intensity relationship. The stage vs. mean velocity and stage vs. discharge relationship data were collected from the Ministry of Water and Energy of Ethiopia. These tabular data were plotted using excel sheet charts, and the least square fit was made to forecast and include all unmeasured values. These graphs are shown in figure 5-4 and 5-5.

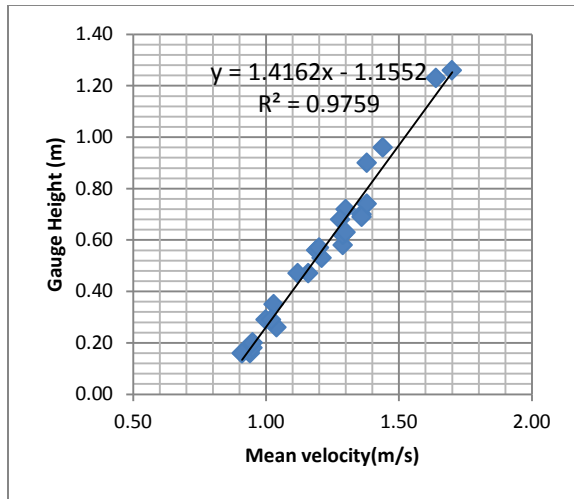


Figure 5-4 Stage vs. velocity relation

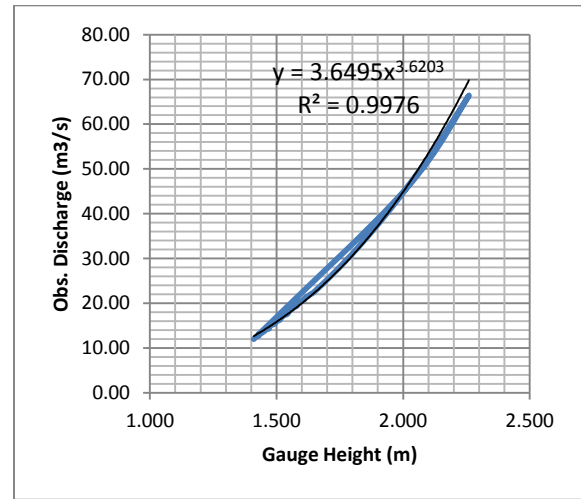


Figure 5-5 Stage vs. discharge relation

Based on these two observation data a relationship is developed between the equilibrium discharges (Q_e) and equilibrium velocities (V_e) as shown in figure 5-6. Using equation (4.09), as mentioned in section 4.3, for each equilibrium discharge the corresponding intensity I of the excess rainfall was found. Finally a relationship is developed between the equilibrium velocities and the computed excess rainfall intensities as shown in figure 5-7. Accordingly the peak velocities of each of the eight storm events were estimated based on this velocity and intensity relationship, for the excess rainfall intensities determined. These peak velocities are used in the derivation of geomorphological instantaneous unit hydrographs of the GIUH based Clark model.

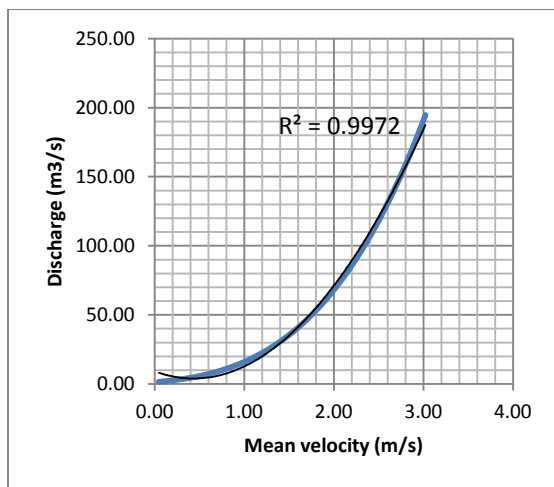


Figure 5-6 Discharge – velocity relationship

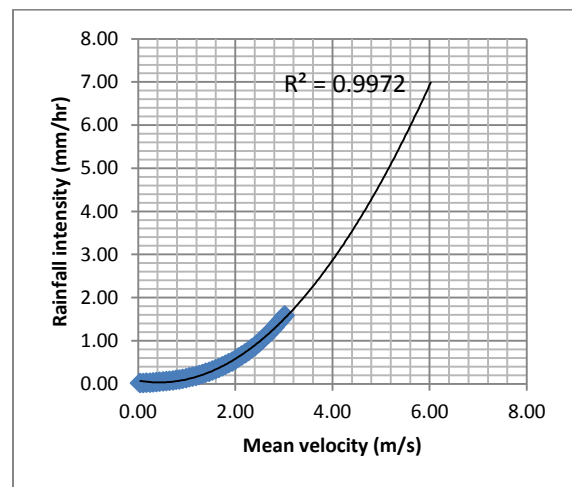


Figure 5-7 Excess rainfall-velocity relationship

5.4. Preparation of Time Area Diagram (TAD)

As described in section 4.4.3 the flow length grid was determined based on the filled sink and flow direction grids of the Shaya river catchment, using Arc GIS. By substituting values of L and the estimated peak velocities V in equation (4.10) the time of concentrations Tc of all the events were determined. These Tc values and the flow length grid were inserted in equation (4.11) to derive the travel time grid of the catchment.

The travel time grid starting from the gauging site of the catchment to various locations over the catchment is calculated using the raster calculation tool of Arc GIS. This grid was further classified in one hour time interval and re drawn as shown in figure 5-8 below.

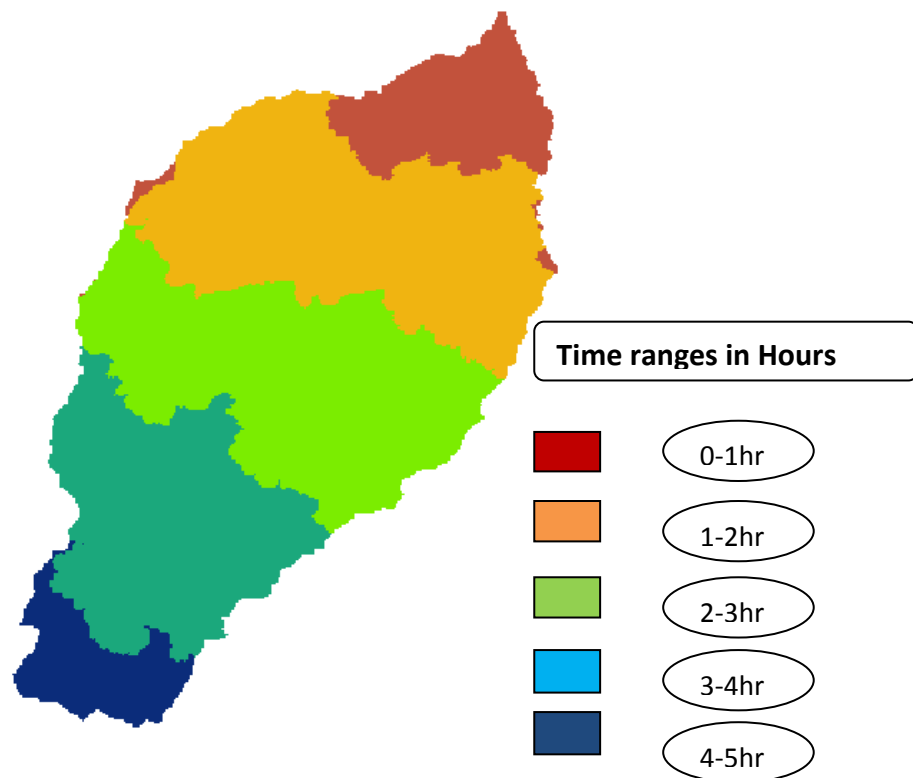


Figure 5-8 Time area diagram of Shaya river basin

The third parameter of the Clark's method, the time-area histogram of the basin is determined from travel time grid of the basin. First, histograms of the travel time grid are derived for different time intervals (figure 5-9). Histogram of travel time grid has time values on the abscissa and number of cells on the ordinate. The time-area histogram of the basin is calculated from histogram of travel time grid by converting number of cells to area. The time vs. cumulative area of Shaya River shown in fig.5-10 is determined from the time area histogram.

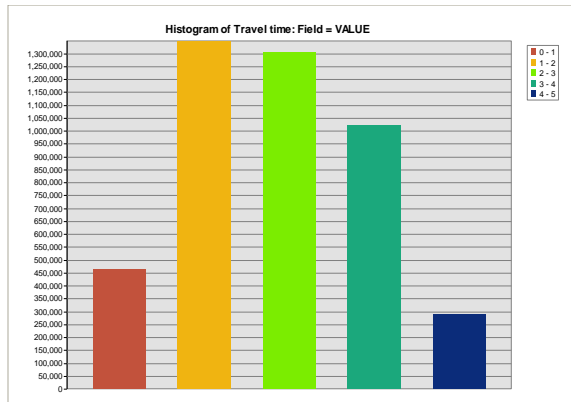


Figure 5-9 Time area histogram

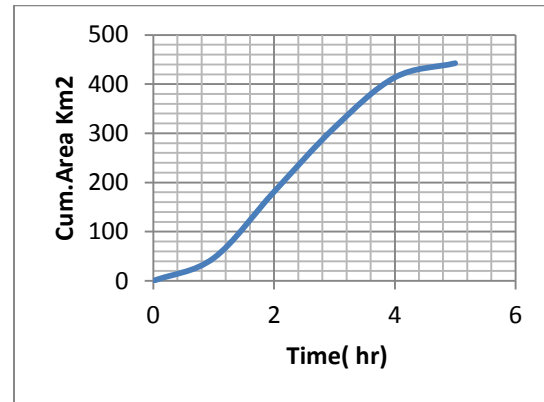


Figure 5-10 Cum. area vs. time diagram

5.5. Model Application

The methodology described in section 4 was applied and GIUH based Clark and Nash models are used for computing the DSRO hydrographs of the eight events. For applying the GIUH models, excess-rainfall hyetographs are separated from the observed rainfall by Φ - index method. The same excess-rainfall hyetographs have been used for the four models.

The values of the parameters of the GIUH based Clark and Nash models are obtained using the geomorphological characteristic of the catchment. Whereas, the parameters of the Nash IUH model and the Clark IUH model are estimated using historical data of the eight hourly rainfall-runoff events. The DSRO hydrographs and excess-rainfall hyetographs are analyzed to derive the parameters of Clark and Nash IUH models.

5.6. Comparison of Observed and Computed DSRO Hydrographs

The direct surface runoff hydrographs DSRO are estimated by convoluting the unit hydrographs derived based on the following approaches

- i. GIUH based Clark model considering the basin as ungauged.
- ii. GIUH based Nash models considering the basin as ungauged
- iii. Clark IUH model option of the HEC- HMS Package and
- iv. Nash IUH models

For evaluation of the performances of the GIUH based Clark and Nash models, the unit hydrographs derived using the GIUH based approaches have been compared with the unit hydrographs derived using Clark and Nash IUH models. And also the DSRO hydrographs

derived using the GIUH based approaches have been compared with the DSRO hydrographs computed using the Clark and Nash IUH models as well as with the observed DSRO hydrographs.

All the parameters of the GIUH based Clark and Nash models and HEC-HMS package Clark and Nash IUH models are particularly estimated for the eight rainfall runoff events and the same parameters have been used to compute the DSRO hydrographs of the corresponding eight rainfall-runoff events.

5.6.1. Comparison of Derived Unit Hydrographs

The parameters of GIUH based Clark and Nash models and Clark and Nash IUH models for the selected rainfall runoff events are given in table 5-2. As it can be observed from the table all the parameters of the eight rainfall runoff events are different in both the GIUH based and IUH approaches, except the parameter ‘n’ of the GIUH based Nash model approach which is equal to 2.76 for all the events. This is due to the fact that the dynamic parameter, peak velocity of the stream based on which all the parameters are estimated, is dependent on the excess rainfall intensity for every particular event. And only the parameter ‘n’ of the GIUH Nash model is purely dependent on the geomorphological characteristics of the basin as described in equation (4.31). The ratio between the storage coefficient (R) and the sum of time of concentration (Tc) and storage coefficient i.e. ($R / (Tc + R)$) has been computed as 0.495 and 0.490 for GIUH based Clark model and Clark IUH model HEC-HMS package respectively. The ratio has a unique value for a particular catchment.

Table 5-2 Parameters of GIUH based and IUH models

Event No.	GIUH Clark		HEC-HMS Clark IUH		GIUH Nash		Nash IUH	
	Tc	R	Tc	R	n	k	n	k
1	4.40	4.32	3.7	5.33	2.76	2.02	1.33	4.03
2	4.31	4.22	4.21	3.63	2.76	1.98	1.26	3.77
3	2.90	2.82	2.96	3.06	2.76	1.33	1.08	4.82
4	2.88	2.80	2.94	3.06	2.76	1.32	1.28	3.69
5	3.94	3.86	4.02	3.59	2.76	1.81	1.80	3.18
6	5.03	4.95	7.36	4.59	2.76	2.31	2.00	2.77
7	3.61	3.53	3.19	3.23	2.76	1.66	0.91	5.73
8	3.66	3.58	3.73	3.58	2.76	1.68	1.98	2.53

Table 5-3 and 5-4 shows the 1-hour unit hydrographs derived by the GIUH based Clark and Nash models, the HEC-HMS package Clark and the Nash IUH models. The values of peak discharge (QP) time to peak (TP) and their product (Qp*Tp) for 1-hour unit hydrographs derived using the GIUH based and the IUH models for the eight rainfall runoff events are shown in table 5-5.

Table 5-3 One- hour unit hydrographs derived by the four models (event 01 & 02)

Time(hr)	Event - 01				Event - 02			
	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00	1.14	3.32	0.05	8.41	1.24	3.47	2.45	9.79
2.00	6.29	10.17	2.26	16.66	6.93	10.57	8.78	18.79
3.00	12.77	15.37	7.85	15.60	13.71	15.84	15.92	16.66
4.00	17.04	17.15	13.66	13.67	17.90	17.49	19.85	14.00
5.00	17.45	16.40	15.86	11.61	17.48	16.58	18.32	11.49
6.00	14.10	14.32	14.35	9.69	13.86	14.33	14.13	9.29
7.00	11.18	11.77	11.88	8.00	10.93	11.66	10.71	7.45
8.00	8.87	9.26	9.84	6.55	8.63	9.08	8.11	5.93
9.00	7.04	7.05	8.17	5.33	6.81	6.85	6.12	4.70
10.00	5.59	5.24	6.77	4.31	5.37	5.04	4.64	3.71
11.00	4.43	3.82	5.59	3.48	4.24	3.63	3.52	2.92
12.00	3.52	2.73	4.62	2.80	3.34	2.58	2.65	2.30
13.00	2.79	1.93	3.82	2.24	2.64	1.80	2.04	1.80
14.00	2.21	1.35	3.17	1.80	2.08	1.25	1.53	1.41
15.00	1.76	0.93	2.63	1.44	1.64	0.86	1.17	1.10
16.00	1.39	0.64	2.20	1.14	1.30	0.58	0.87	0.86
17.00	1.11	0.44	1.83	0.91	1.02	0.39	0.66	0.67
18.00	0.88	0.30	1.51	0.73	0.81	0.26	0.51	0.52
19.00	0.70	0.20	1.24	0.58	0.64	0.17	0.41	0.41
20.00		0.13	1.02	0.46	0.50	0.12	0.31	0.32
21.00		0.09	0.86	0.36	0.40	0.08	0.20	0.25
22.00		0.06	0.70	0.29	0.31	0.05	0.15	0.19
23.00		0.04	0.59	0.23		0.03		0.15
24.00		0.03	0.48	0.18		0.02		0.11
25.00		0.02	0.38	0.14		0.01		0.09
26.00			0.32					
27.00			0.27					
28.00			0.22					
29.00			0.22					

Table 5-4 One- hour unit hydrographs derived by the four models (event 03&04)

Time(hr)	Event - 03				Event - 04			
	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00	3.47	8.13	4.83	9.52	3.49	8.26	4.87	9.78
2.00	17.43	21.12	16.30	17.70	17.53	21.37	16.44	18.85
3.00	26.66	25.49	24.23	15.04	26.79	25.66	24.28	16.80
4.00	22.50	22.28	21.94	12.57	22.56	22.31	21.88	14.14
5.00	15.79	16.61	15.77	10.43	15.79	16.54	15.72	11.59
6.00	11.07	11.27	11.32	8.62	11.05	11.16	11.31	9.36
7.00	7.77	7.18	8.14	7.10	7.73	7.07	8.13	7.49
8.00	5.45	4.38	5.85	5.84	5.41	4.29	5.83	5.95
9.00	3.82	2.59	4.21	4.79	3.78	2.52	4.20	4.70
10.00	2.68	1.49	3.03	3.93	2.65	1.44	3.01	3.70
11.00	1.88	0.84	2.18	3.22	1.85	0.81	2.16	2.90
12.00	1.32	0.47	1.56	2.63	1.30	0.45	1.55	2.27
13.00	0.93	0.25	1.13	2.16	0.91	0.24	1.12	1.77
14.00	0.65	0.14	0.81	1.76	0.63	0.13	0.80	1.38
15.00	0.46	0.07	0.58	1.44	0.44	0.07	0.59	1.07
16.00	0.32	0.04	0.43	1.18	0.31	0.04	0.42	0.84
17.00	0.22	0.02	0.30	0.96	0.22	0.02	0.30	0.65
18.00	0.11	0.01	0.21	0.78	0.11	0.01	0.21	0.50
19.00		0.01	0.00	0.64		0.01	0.00	0.39
20.00		0.00	0.00	0.52		0.00	0.00	0.30
21.00		0.00	0.00	0.43		0.00	0.00	0.23
22.00		0.00	0.00	0.35		0.00	0.00	0.18
23.00		0.00	0.00	0.28		0.00	0.00	0.14
24.00		0.00	0.00	0.23		0.00	0.00	0.11
25.00		0.00	0.00	0.19		0.00	0.00	0.08
26.00			0.00				0.00	
27.00			0.00				0.00	
28.00			0.00					
29.00			0.00					

Table 5-5 Peak discharge and time to peak of the unit hydrographs derived by the four models

Event	GIUH Clark			GIUH Nash			HEC-HMS Clark			Nash IUH		
	Q _p	T _p	Q _p T _p	Q _p	T _p	Q _p T _p	Q _p	T _p	Q _p T _p	Q _p	T _p	Q _p T _p
1	17.45	5.00	87.25	17.15	4.00	68.6	15.86	5.00	79.3	16.66	2.00	33.32
2	17.90	4.00	71.6	17.49	4.00	69.96	19.85	4.00	79.4	18.79	2.00	37.58
3	26.60	3.00	79.8	25.49	3.00	76.47	24.23	3.00	72.69	17.70	2.00	35.4
4	26.79	3.00	80.37	25.66	3.00	76.98	24.28	3.00	72.84	18.85	2.00	37.7
5	15.96	3.00	47.88	18.99	4.00	75.96	20.34	4.00	81.36	15.35	3.00	46.05
6	15.63	5.00	78.15	14.97	5.00	74.85	13.78	7.00	96.46	15.92	3.00	47.76
7	26.60	4.00	106.4	20.26	4.00	81.04	22.47	3.00	67.41	17.78	2.00	35.56
8	21.38	4.00	85.52	20.10	4.00	80.4	20.57	4.00	82.28	17.67	3.00	53.01

It can be observed from table 5-5 that the peak and time to peak of the unit hydrographs derived by using GIUH based models varies from 15.63 to 26.79 and from 3 to 5 in the case of Clark model and from 17.15 to 25.66 and from 3 to 5 in the case of Nash model respectively. And also the peak and time to peak of the unit hydrographs derived based on HEC-HMS package Clark IUH model varies from 13.78 to 24.28 and from 3 to 5 and that of the Nash IUH model varies from 15.35 to 18.85 and from 2 to 3 respectively.

5.6.2. Comparison of Computed and Observed DSRO Hydrographs

The DSRO hydrographs computed using the GIUH based Clark and Nash models are compared with the observed DSRO hydrographs and also with the DSRO hydrographs computed using the HEC-HMS package Clark and Nash IUH models for the eight events.

Table 5-6 Peak discharge and time to peak of the observed and computed DSRO hydrographs

Event	Observed		GIUH Clark		GIUH Nash		HEC-HMS Clark		Nash IUH	
	Q _p	T _p	Q _p	T _p	Q _p	T _p	Q _p	T _p	Q _p	T _p
1	30.50	2.00	32.04	5.00	31.74	5.00	29.50	5.00	30.69	3.00
2	39.34	2.00	35.08	4.00	34.29	4.00	38.90	4.00	36.82	2.00
3	123.52	2.00	125.30	3.00	119.81	3.00	113.40	3.00	83.19	2.00
4	136.28	2.00	128.59	3.00	123.17	3.00	128.20	3.00	90.46	2.00
5	40.42	2.00	38.62	3.00	45.96	4.00	47.60	4.00	37.14	3.00
6	19.23	2.00	21.57	5.00	20.65	5.00	19.70	7.00	21.97	3.00
7	73.34	2.00	63.07	4.00	59.17	4.00	65.60	3.00	51.92	2.00
8	68.12	2.00	60.29	4.00	56.68	4.00	57.60	4.00	49.83	3.00

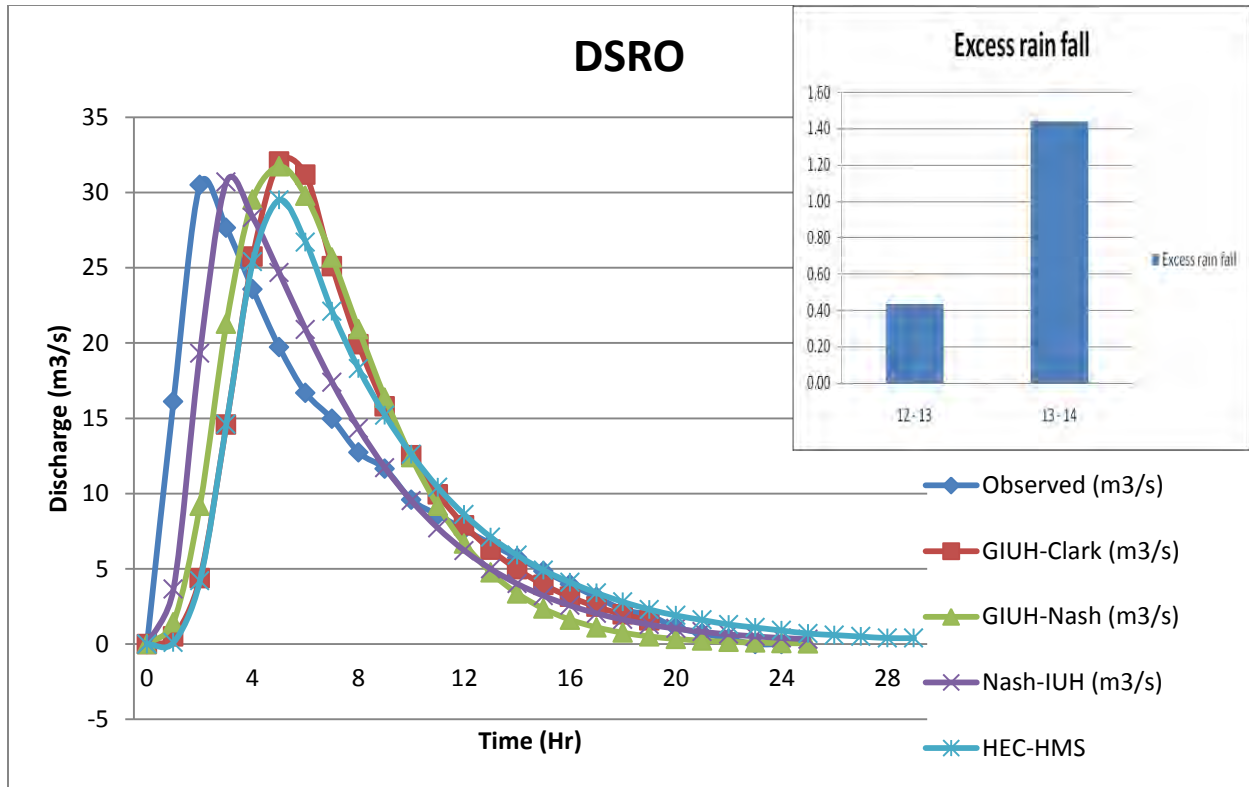


Figure 5-11 Comparisons of DSRO hydrographs of Shaya river up to Robe (Event No. 01)

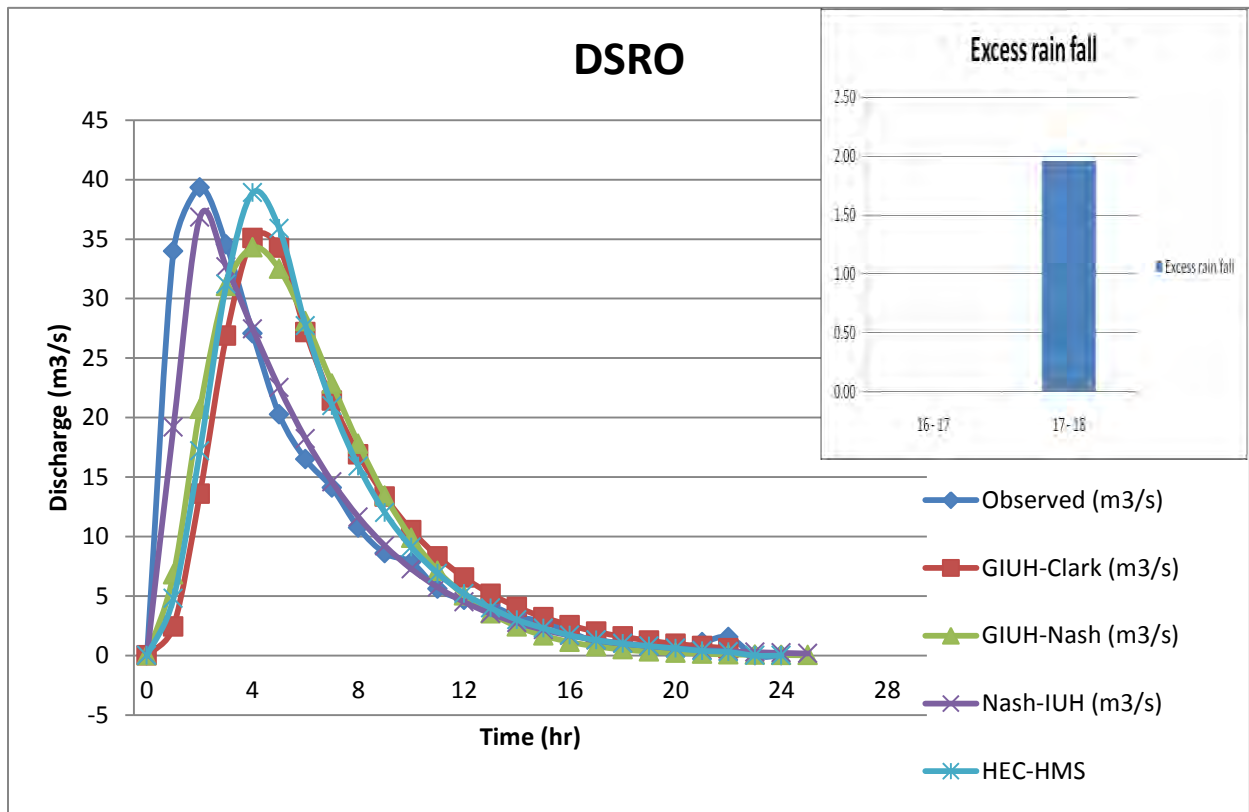


Figure 5-12 Comparisons of DSRO hydrographs of Shaya river up to Robe (Event No. 02)

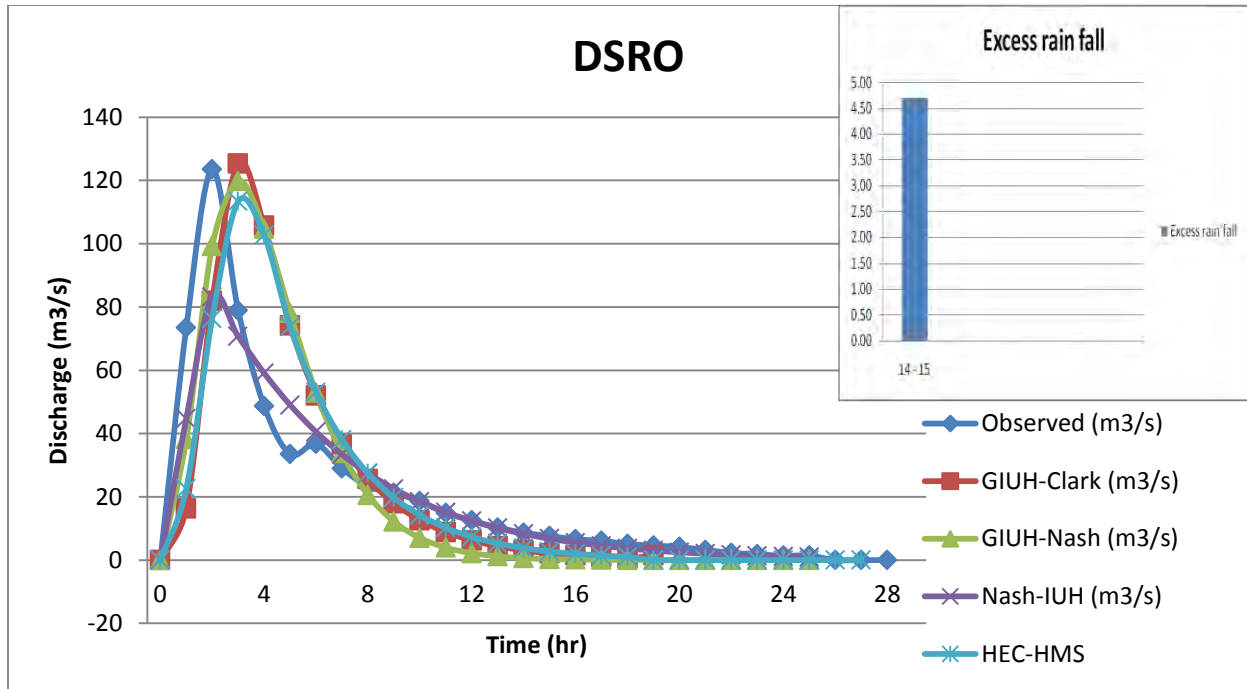


Figure 5-13 Comparisons of DSRO hydrographs of Shaya river up to Robe (Event No. 03)

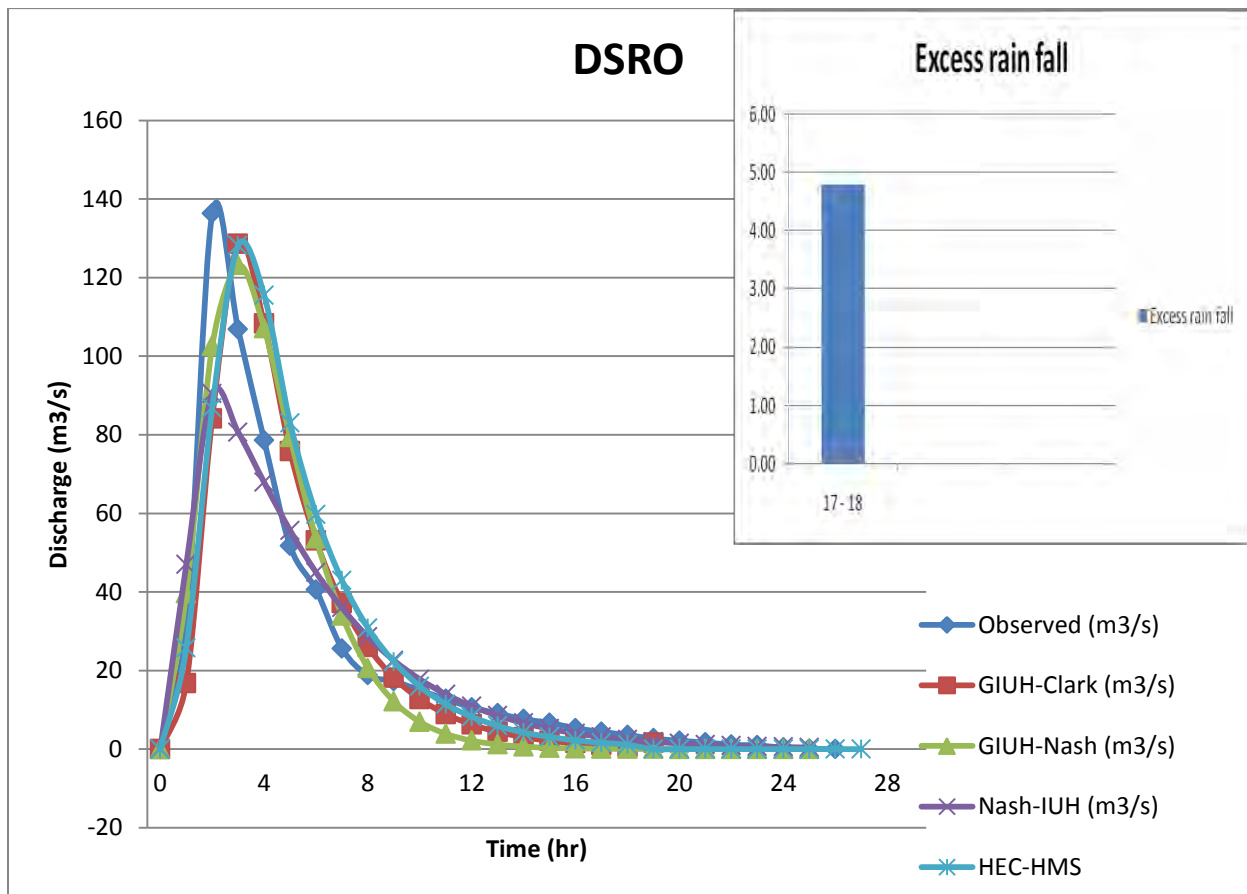


Figure 5-4 Comparisons of DSRO hydrographs of Shaya river up to Robe (Event No. 04)

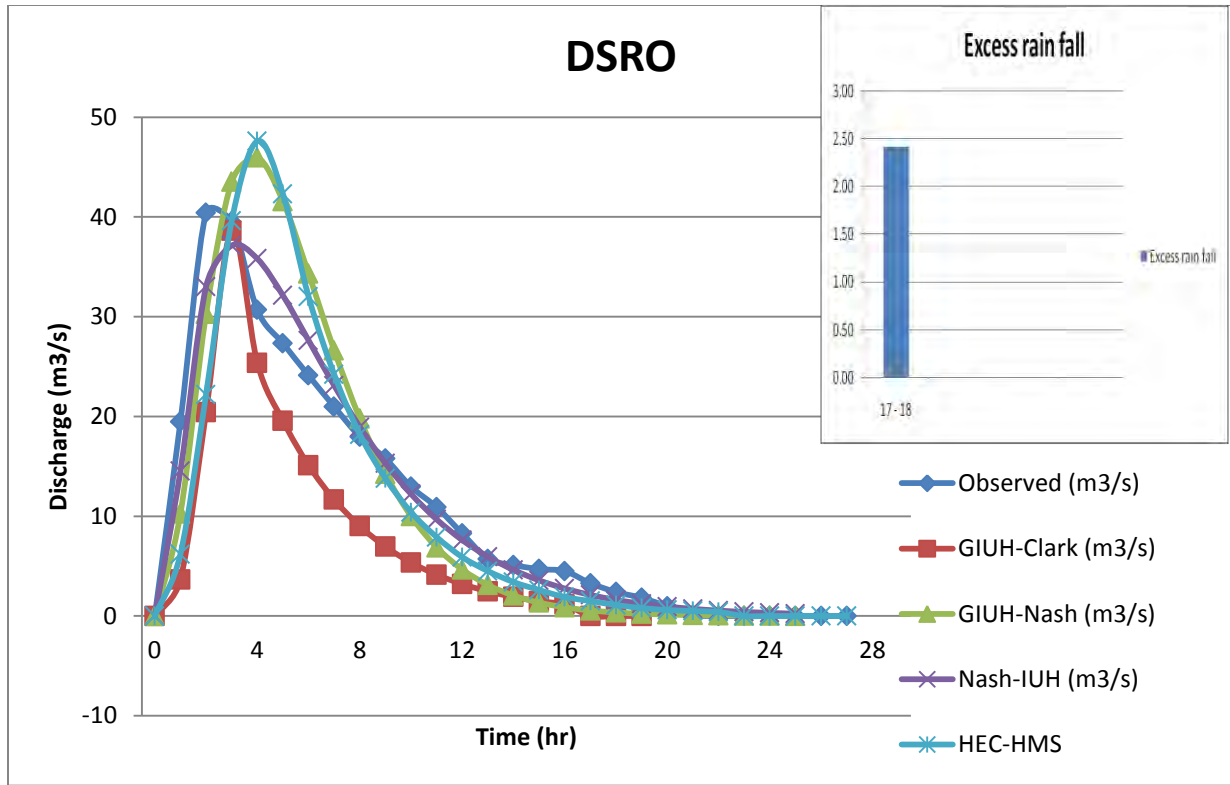


Figure 5-5 Comparisons of DSRO hydrographs of Shaya river up to Robe (Event No. 05)

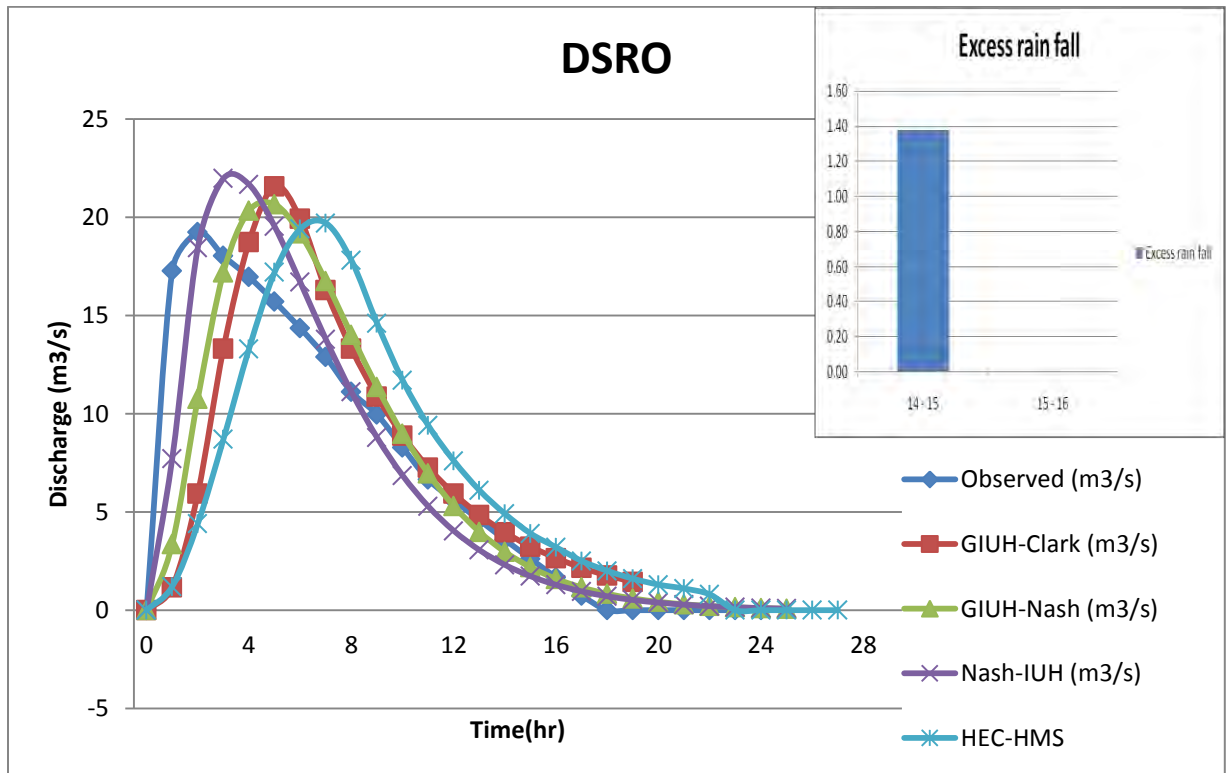


Figure 5-16 Comparisons of DSRO hydrographs of Shaya river up to Robe (Event No. 06)

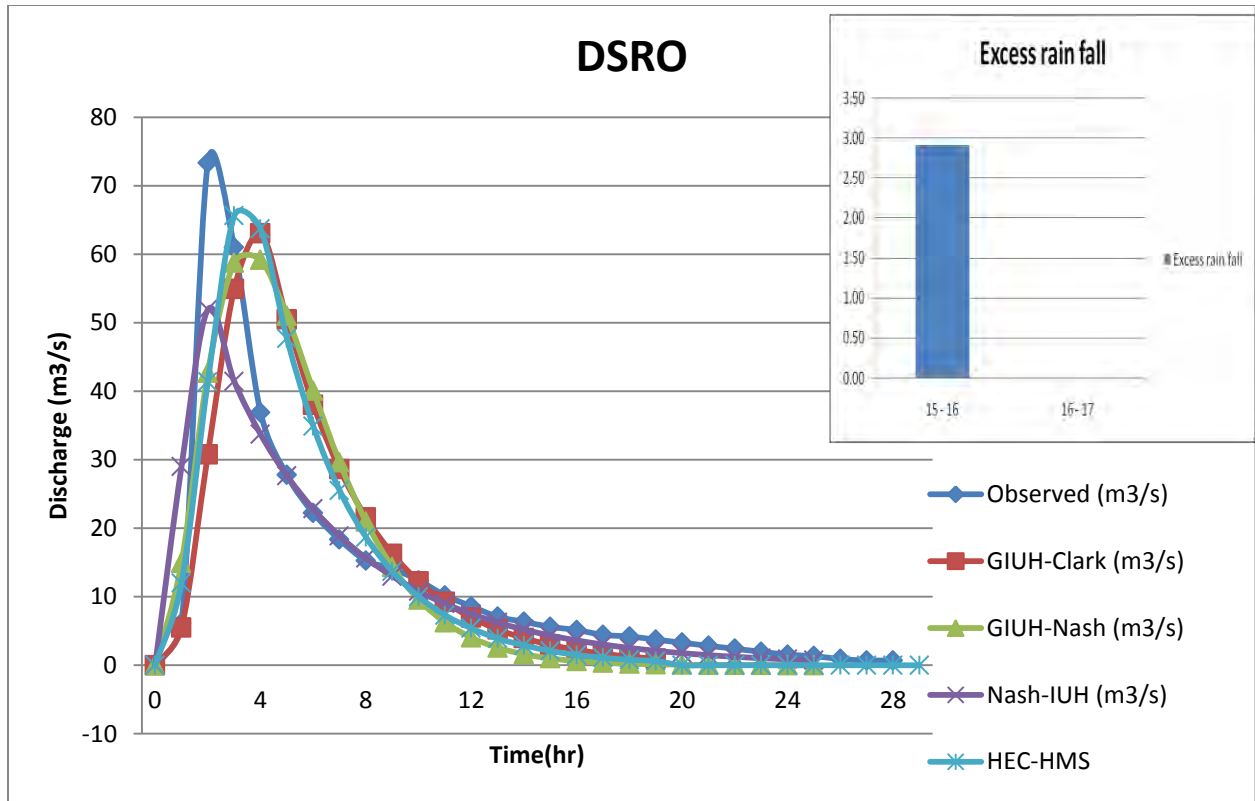


Figure 5-17 Comparisons of DSRO hydrographs of Shaya river up to Robe (Event No. 07)

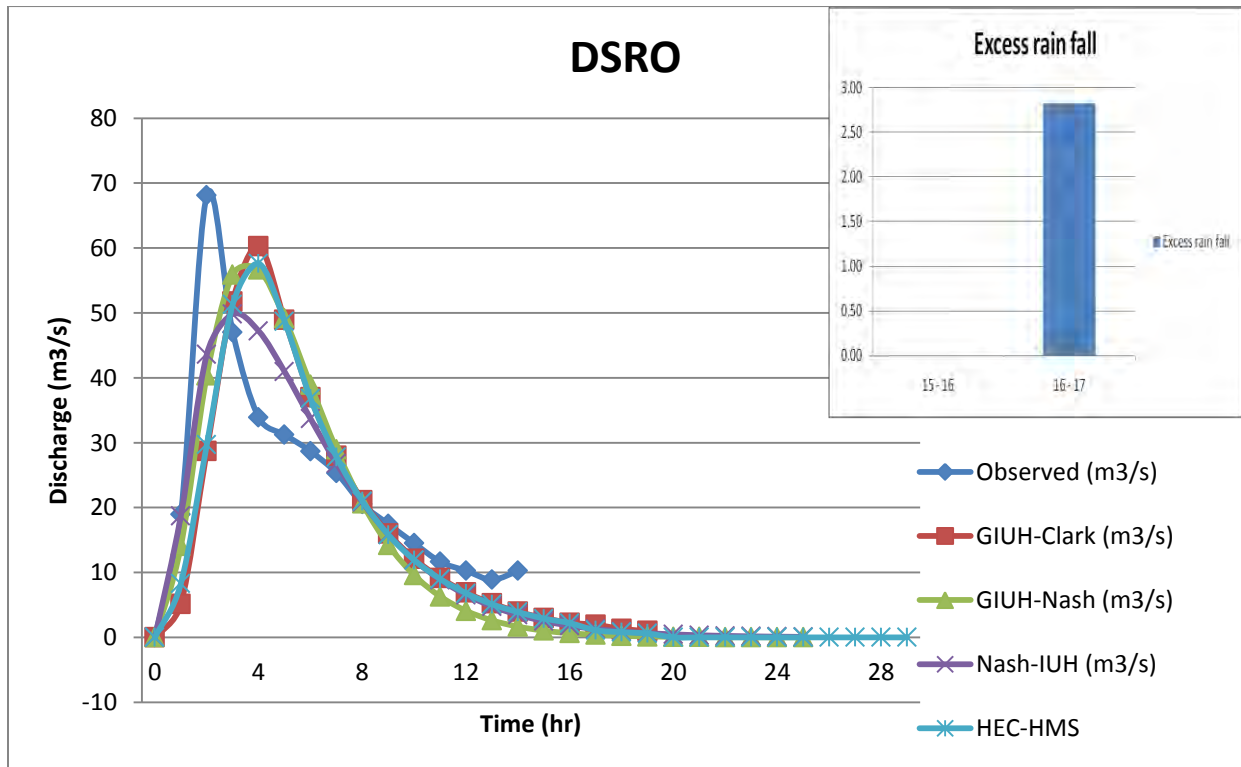


Figure 5-18 Comparisons of DSRO hydrographs of Shaya River up to Robe (Event No. 08)

Table 5-6 shows the peak discharge and time to peak of the observed and the computed DSRO hydrographs. The time to peak discharge of the observed hydrograph is two hours and that of the, GIUH based Clark, GIUH based Nash, HEC-HMS package and Nash IUH DSRO hydrographs varies from 3 to 5 , 3 to 5, 3 to 7 and 2 to 3 respectively. The observed DSRO hydrographs and the computed DSRO hydrographs using the four models are shown in fig 5-11 through fig 5-18. It can be observed from the above figures and from table 5-6 that the time to peak estimated by the models are delayed uniformly as compared to the observed hydrograph except that of the Nash IUH model. This is because of the fact that the slope of the main stream across the catchment is very steeply, as shown in figure 3-2, generating rapid rising sharp runoff hydrographs with shorter time to peak.

5.6.3. Comparison of Error Functions used for Evaluation

Table 5-7 shows the values of error functions computed to evaluate the DSRO hydrographs derived using the GIUH based Clark and Nash models, Nash IUH model and the HEC-HMS package Clark model. These error functions are: (1) Efficiency (EFF), (2) Root mean square error (RMSE), (3) Percentage error in peak (PEP) and (4) The time to peak and peak discharge cumulative error (ERR).

It can be observed from the table that the values of EFF varies from 59.65 to 92.2 for GIUH based Clark model; from 43.12 to 84.45 for GIUH based Nash model; from 77.32 to 95.13 for Nash IUH model and from 58.88 to 89.81 for HEC-HMS package Clark IUH model. And also the values of the EFF of the DSRO hydrographs are higher in four out of the eight events for the GIUH based Clark model; three out of the eight events for the Nash IUH model and in one out of the eight events for the HEC-HMS package Clark IUH model..

It is also observed that the PEP is lower in four out of the eight events for GIUH based Clark model; three out of the eight events for HEC-HMS model and one for Nash IUH model. The cumulative time to peak and peak discharges error ERR is also lower in four events for the Nash IUH model, in three events for the GIUH based Clark and in one event for HEC-HMS.

Table 5-7 Error functions computed for DSRO hydrographs.

Methods	Error functions for DSRO hydrographs			
	EFF (%)	RMSE	PEP (%)	ERR (%)
EVENT 01				
GIUH (Clark)	74.68	4.22	-5.05	36.00
GIUH (Nash)	51.27	5.86	-4.07	36.00
HEC-HMS (Clark)	78.94	3.85	3.28	36.00
Nash IUH	78.52	3.89	-0.62	11.00
EVENT 02				
GIUH (Clark)	92.20	3.23	10.83	26.00
GIUH (Nash)	84.45	4.56	12.84	27.00
HEC-HMS (Clark)	89.81	3.69	1.12	25.00
Nash IUH	92.75	3.11	6.41	0.00
EVENT 03				
GIUH (Clark)	88.04	10.23	-1.44	11.00
GIUH (Nash)	75.57	14.62	3.00	11.00
HEC-HMS (Clark)	87.70	10.38	8.19	12.00
Nash IUH	86.60	10.83	32.65	24.00
EVENT 04				
GIUH (Clark)	88.18	11.71	5.64	11.00
GIUH (Nash)	73.73	17.46	9.62	12.00
HEC-HMS (Clark)	85.55	12.95	5.93	12.00
Nash IUH	88.11	11.74	33.62	26.00
EVENT 05				
GIUH (Clark)	61.86	7.13	4.45	11.00
GIUH (Nash)	79.80	5.19	-13.71	26.00
HEC-HMS (Clark)	59.71	7.33	-17.76	27.00
Nash IUH	95.13	2.55	8.11	12.00
EVENT 06				
GIUH (Clark)	85.25	2.34	-12.17	37.00
GIUH (Nash)	81.05	2.07	-7.38	36.00
HEC-HMS (Clark)	74.78	2.70	-2.44	51.00
Nash IUH	77.32	2.56	-14.25	13.00
EVENT 07				
GIUH (Clark)	63.09	10.57	14.00	28.00
GIUH (Nash)	52.77	11.96	19.32	31.00
HEC-HMS (Clark)	75.34	8.64	10.55	13.00
Nash IUH	81.43	7.50	29.21	17.00
EVENT 08				
GIUH (Clark)	59.65	8.22	11.49	27.00
GIUH (Nash)	43.12	9.76	16.79	29.00
HEC-HMS (Clark)	58.88	8.30	15.44	28.00
Nash IUH	75.78	6.37	26.85	25.00

6. CONCLUSION AND RECOMMENDATION

6.1. Conclusions

The basic objective of this study, as mentioned in section 1.3.2, is to estimate the direct surface runoff hydrographs using the four models; GIUH based Clark model, GIUH based Nash model, Nash IUH model, and Clark IUH model option of HEC-HMS package. The observed DSRO hydrographs and the computed DSRO hydrographs using the four models for the eight rainfall runoff events have been compared. The four error functions described in section 4.11; Efficiency error (EFF), Root mean square error (RMSE), Percentage error in peak (PEP) and Cumulative Error Function (ERR) have been employed to evaluate the models. Based on the specific objectives and the analysis made in this study the following conclusions are drawn:

- The geomorphological characteristics required for the derivation of GIUH based Clark and GIUH based Nash models for Shaya catchment up to Robe stream gauging station were easily and accurately extracted from Digital Elevation Map of the basin, using GIS package ILWS. The Horton plots and the Horton statistics outputs of the catchment were also estimated from the geomorphological characteristics with reasonable accuracy.
- The geomorphological parameters required for derivation of the GIUH based Clark and Nash models, such as, length of main stream, catchment area, bifurcation ratio, length ratio and area ratio have been determined quite accurately. The extracted geomorphological characteristics of the catchment are used instead of the observed rainfall runoff data, since the basin has been considered as ungauged for the application of these models.
- Time Area diagram (TAD) of Shaya river basin required for GIUH based Clark model has been accurately drawn using Arc GIS based on the time-area histogram of the basin determined from the travel time grid. Manual drawing of the isochrones maps of the basin is a tedious and cumbersome process which might also leads to erroneous estimates. The application of modern techniques like using Arc-GIS, serves as an efficient way to overcome these discouraging methods.

- Based on the determined geomorphological parameters and the drawn time area diagram the geomorphological instantaneous unit hydrographs (GIUH) have been properly derived using both GIUH based Clark and Nash models.
- The direct surface runoff hydrographs of the eight events selected were also estimated using the derived unit hydrographs and found to be reasonable when compared with the observed runoff hydrographs.
- From model evaluation results it can be observed that, GIUH based Clark model provides higher values of EFF and lower PET as compared to the other three models. And the Nash IUH model provides lower RMSE and ERR as compared to the other three models. Therefore, from the overall output of the study, it can be conclude that GIUH based Clark model can give promising results in simulating the peak discharge of the direct surface runoff hydrographs and is a potential model in runoff estimation in an ungauged catchment.

6.2. Recommendations

- The researcher estimated direct surface runoff hydrographs after deriving GIUH based hydrographs, which in fact followed similar procedure as was used in the development of the conventional hydrographs based on the records of rainfall and stream gauging data. It is therefore, recommended that GIUH based hydrographs are sufficiently accurate laying within the limits of hydrological errors, refer tables of comparison, and can be used for runoff estimation of ungauged catchments.
- It is recommended also that future researchers undertake further similar studies in other basins existing in Ethiopia, which will help in saving money and energy avoiding permanent stream gauging stations.
- GIUH based hydrographs are derived based on the geomorphological parameters obtained using GIS based models, independent of the availability of stream gauging data. However, this research work is done based on the rainfall intensity data collected in the boundary of the catchment. Further investigations by other researchers can be carried out to see whether it is possible to avoid measurement of rainfall by conventional rain gauges and examine the possibility of using satellite recorded rain occurring in the boundary.

- Finally the researcher strongly recommends that future researchers must thoroughly study preparation and processing of satellite driven digital elevation maps (DEM) and the relevant soft ware developed such as ILWIS, Arc GIS etc. to derive maximum advantage of computing runoff from a given catchment for the purpose of developing hydropower stations on river basins in Ethiopia avoiding use of conventional techniques and wait for statistical justified number of years.

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APPENDICES

Event - 02

1		2	3	4	5	6 = 5-4
Date	Time	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)	Base flow (m³/s)	DRH (m³/s)
24/7/98	0	7:00 PM	1.445	13.25	13.25	0.00
	1	8:00 PM	2.040	47.98	13.31	34.67
	2	9:00 PM	2.110	53.34	13.37	39.97
	3	10:00 PM	2.050	48.53	13.43	35.10
	4	11:00 PM	1.950	41.06	13.50	27.56
	5	12:00 AM	1.850	34.26	13.56	20.70
25/7/98	6	1:00 AM	1.790	30.49	13.62	16.87
	7	2:00 AM	1.750	28.11	13.68	14.43
	8	3:00 AM	1.690	24.73	13.74	10.99
	9	4:00 AM	1.650	22.60	13.80	8.80
	10	5:00 AM	1.635	21.84	13.87	7.97
	11	6:00 AM	1.590	19.61	13.93	5.68
	12	7:00 AM	1.570	18.66	13.99	4.67
	13	8:00 AM	1.560	18.20	14.05	4.15
	14	9:00 AM	1.540	17.29	14.11	3.18
	15	10:00 AM	1.530	16.84	14.17	2.67
	16	11:00 AM	1.510	15.97	14.23	1.74
	17	12:00 PM	1.490	15.12	14.30	0.82
	18	1:00 PM	1.480	14.71	14.36	0.35
	19	2:00 PM	1.475	14.50	14.42	0.08
	20	3:00 PM	1.475	14.50	14.48	0.02

Event - 03

1		2	3	4	5	6 = 5-4
Date	Time	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)	Base flow (m³/s)	DRH (m³/s)
31/7/1998	0	3:00 PM	1.51	15.97	15.97	0.00
	1	4:00 PM	1.51	15.97	15.97	0.00
	2	5:00 PM	2.48	88.41	15.91	72.50
	3	6:00 PM	2.89	138.52	15.84	122.68
	4	7:00 PM	2.530	93.88	15.78	78.10
	5	8:00 PM	2.230	63.68	15.72	47.96
	6	9:00 PM	2.050	48.53	15.66	32.88
	7	10:00 PM	2.010	51.71	15.59	36.12
	8	11:00 PM	1.990	43.97	15.53	28.44
	9	12:00 AM	1.930	38.26	15.47	22.79
1/8/1998	10	1:00 AM	1.910	36.23	15.40	20.83
	11	2:00 AM	1.880	33.61	15.34	18.27
	12	3:00 AM	1.780	29.88	15.28	14.60
	13	4:00 AM	1.740	27.53	15.21	12.32
	14	5:00 AM	1.700	25.28	15.15	10.13
	15	6:00 AM	1.670	23.65	15.09	8.56
	16	7:00 AM	1.650	22.60	15.03	7.58
	17	8:00 AM	1.630	21.58	14.96	6.62
	18	9:00 AM	1.620	21.08	14.90	6.18
	19	10:00 AM	1.600	20.09	14.84	5.25
	20	11:00 AM	1.590	19.61	14.77	4.84
	21	12:00 PM	1.580	19.13	14.71	4.42
	22	1:00 PM	1.560	18.20	14.65	3.55
	23	2:00 PM	1.540	17.29	14.58	2.71
	24	3:00 PM	1.530	16.84	14.52	2.32
	25	4:00 PM	1.520	16.40	14.46	1.94
	26	5:00 PM	1.520	16.40	14.40	2.01
	27	6:00 PM	1.505	15.73	14.33	1.40
	28	7:00 PM	1.500	15.54	14.27	1.27
	29	8:00 PM	1.490	15.12	14.21	0.91
	30	9:00 PM	1.480	14.71	14.14	0.57
	31	10:00 PM	1.470	14.30	14.08	0.22
	32	11:00 PM	1.460	14.02	14.02	0.00

Event - 04

1		2	3	4	5	6 = 5-4
Date	Time	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)	Base flow (m³/s)	DRH (m³/s)
14/8/98	0	6:00 PM	1.400	11.61	11.61	0.00
	1	7:00 PM	1.400	11.61	11.61	0.00
	2	8:00 PM	1.980	43.23	11.64	31.59
	3	9:00 PM	2.960	148.28	11.67	136.61
	4	10:00 PM	2.740	118.79	11.70	107.09
	5	11:00 PM	2.500	90.58	11.73	78.85
	6	12:00 AM	2.230	63.68	11.76	51.92
15/8/98	7	1:00 AM	2.100	52.52	11.79	40.73
	8	2:00 AM	1.900	37.58	11.83	25.75
	9	3:00 AM	1.795	30.79	11.86	18.93
	10	4:00 AM	1.770	29.28	11.89	17.39
	11	5:00 AM	1.730	26.96	11.92	15.04
	12	6:00 AM	1.690	24.73	11.95	12.78
	13	7:00 AM	1.650	22.60	11.98	10.62
	14	8:00 AM	1.620	21.08	12.01	9.07
	15	9:00 AM	1.590	19.61	12.04	7.57
	16	10:00 AM	1.570	18.66	12.07	6.59
	17	11:00 AM	1.540	17.29	12.10	5.19
	18	12:00 PM	1.520	16.40	12.13	4.27
	19	1:00 PM	1.500	15.54	12.16	3.38
	20	2:00 PM	1.480	14.71	12.20	2.51
	21	3:00 PM	1.465	14.10	12.23	1.87
	22	4:00 PM	1.455	13.70	12.26	1.44
	23	5:00 PM	1.440	13.11	12.29	0.82
	24	6:00 PM	1.435	12.92	12.32	0.60
	25	7:00 PM	1.420	12.35	12.35	0.00

Event - 05

1		2	3	4	5	6 = 5-4
Date	Time	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)	Base flow (m3/s)	DRH (m3/s)
15/8/98	0	9:00 PM	1.410	11.98	26	0.00
	1	10:00 PM	2.010	45.46	26	19.46
	2	11:00 PM	2.260	66.42	26	40.42
	3	12:00 AM	2.250	65.50	26	39.50
16/8/98	4	1:00 AM	2.150	56.68	26	30.68
	5	2:00 AM	2.110	53.34	26	27.34
	6	3:00 AM	2.070	50.11	26	24.11
	7	4:00 AM	2.030	46.98	26	20.98
	8	5:00 AM	1.990	43.97	26	17.97
	9	6:00 AM	1.960	41.78	26	15.78
	10	7:00 AM	1.920	38.95	26	12.95
	11	8:00 AM	1.890	36.90	26	10.90
	12	9:00 AM	1.850	34.26	26	8.26
	13	10:00 AM	1.810	31.72	26	5.72
	14	11:00 AM	1.800	31.10	26	5.10
	15	12:00 PM	1.795	30.70	26	4.70
	16	1:00 PM	1.790	30.49	26	4.49
	17	2:00 PM	1.770	29.28	26	3.28
	18	3:00 PM	1.755	28.40	26	2.40
	19	4:00 PM	1.745	27.82	26	1.82
	20	5:00 PM	1.730	26.96	26	0.96
	21	6:00 PM	1.715	26.11	26	0.11
	22	7:00 PM	1.700	25.28	26	0.00

Event - 06

1		2	3	4	5	6 = 5-4
Date	Time	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)	Base flow (m3/s)	DRH (m3/s)
11/9/1998	0	3:00 PM	1.180	5.08	17	0.00
	1	4:00 PM	1.180	5.08	17	0.00
	2	5:00 PM	1.290	7.98	17	0.00
	3	6:00 PM	1.340	9.54	17	0.00
	4	7:00 PM	1.410	11.98	17	0.00
	5	8:00 PM	1.850	34.26	17	17.26
	6	9:00 PM	1.880	36.23	17	19.23
	7	10:00 PM	1.830	35.03	17	18.03
	8	11:00 PM	1.800	33.95	17	16.95
	9	12:00 AM	1.795	32.70	17	15.70
12/9/1998	10	1:00 AM	1.785	31.35	17	14.35
	11	2:00 AM	1.780	29.88	17	12.88
	12	3:00 AM	1.750	28.11	17	11.11
	13	4:00 AM	1.730	26.96	17	9.96
	14	5:00 AM	1.700	25.28	17	8.28
	15	6:00 AM	1.670	23.65	17	6.65
	16	7:00 AM	1.650	22.60	17	5.60
	17	8:00 AM	1.630	21.58	17	4.58
	18	9:00 AM	1.610	20.58	17	3.58
	19	10:00 AM	1.590	19.61	17	2.61
	20	11:00 AM	1.570	18.66	17	1.66
	21	12:00 PM	1.550	17.74	17	0.74
	22	1:00 PM	1.530	16.84	17	0.00

Event - 07

1		2	3	4	5	6 = 5-4
Date	Time(hr)	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)	Base flow (m3/s)	DRH (m3/s)
3/10/1998	0	5:00 PM	1.600	20.09	14	6.09
	1	6:00 PM	2.470	87.34	14	73.34
	2	7:00 PM	2.350	75.01	14	61.01
	3	8:00 PM	2.080	50.90	14	36.90
	4	9:00 PM	1.960	41.78	14	27.78
	5	10:00 PM	1.880	36.23	14	22.23
	6	11:00 PM	1.820	32.34	14	18.34
	7	12:00 AM	1.770	29.28	14	15.28
4/10/1998	8	1:00 AM	1.750	28.11	14	14.11
	9	2:00 AM	1.720	26.39	14	12.39
	10	3:00 AM	1.680	24.19	14	10.19
	11	4:00 AM	1.650	22.60	14	8.60
	12	5:00 AM	1.620	21.08	14	7.08
	13	6:00 AM	1.605	20.34	14	6.34
	14	7:00 AM	1.590	19.61	14	5.61
	15	8:00 AM	1.580	19.13	14	5.13
	16	9:00 AM	1.565	18.43	14	4.43
	17	10:00 AM	1.560	18.20	14	4.20
	18	11:00 AM	1.550	17.74	14	3.74
	19	12:00 PM	1.540	17.29	14	3.29
	20	1:00 PM	1.530	16.84	14	2.84
	21	2:00 PM	1.520	16.40	14	2.40
	22	3:00 PM	1.510	15.97	14	1.97
	23	4:00 PM	1.500	15.54	14	1.54
	24	5:00 PM	1.495	15.33	14	1.33
	25	6:00 PM	1.485	14.91	14	0.91
	26	7:00 PM	1.480	14.71	14	0.71
	27	8:00 PM	1.480	14.71	14	0.71

Event - 08

1		2	3	4	5	6 = 5-4
Date	Time(hr)	Time (hr)	Gauge Height	Obs. Stream flow(m³/s)	Base flow (m3/s)	DRH (m3/s)
20/10/1998	0	4:00 PM	1.760	28.69	28	0
	1	5:00 PM	1.760	28.69	28	0.00
	2	6:00 PM	2.030	46.98	28	18.98
	3	7:00 PM	2.550	96.12	28	68.12
	4	8:00 PM	2.350	75.01	28	47.01
	5	9:00 PM	2.210	61.89	28	33.89
	6	10:00 PM	2.180	59.25	28	31.25
	7	11:00 PM	2.150	56.68	28	28.68
	8	12:00 AM	2.110	53.34	28	25.34
21/10/1998	9	1:00 AM	2.050	48.53	28	20.53
	10	2:00 AM	2.010	45.46	28	17.46
	11	3:00 AM	1.970	42.50	28	14.50
	12	4:00 AM	1.930	39.65	28	11.65
	13	5:00 AM	1.910	38.26	28	10.26
	14	6:00 AM	1.890	36.90	28	8.90
	15	7:00 AM	1.910	38.26	28	10.26

Appendix – B. Measured rainfall intensity data

Station: Robe

Region : Balie

Element: Rain fall Intensity

Month : July Year :
1998

Date/ Time	2	4	18	19	22	24	26	27	28	29	30	31
0 - 1												
1 - 2												
2 - 3												
3 - 4												
4 - 5												
5 - 6						2.00						
6 - 7												
7 - 8												
8 - 9												
9 - 10												
10 - 11												
11 - 12												
12 - 13					0.80							
13 - 14					1.80							
14 - 15			1.20	8.00			1.80					6.00
15 - 16				2.50				0.80			6.50	
16 - 17		7.20		1.60		2.00			2.00		2.60	
17 - 18	3.40					3.00				7.00		
18 - 19	0.60									4.00		
19 - 20	0.30											
20 - 21												
21 - 22												
22 - 23												
23 - 24												
Total												
H in 10m												
H in 20m												
H in 30m												
H in 1 hr	3.40	7.20	1.20	8.00	1.80	3.00	1.80	0.80	2.00	7.00	6.50	6.00

H = Highest rain fall

m = Minute

hr = Hour

Time is local standard
time

Station: Robe

Region : Balie

Element: Rain fall Intensity

Month : August Year : 1998

Date/ Time	4	9	11	14	15	17	22	24	27			
0 - 1												
1 - 2												
2 - 3			7.00									
3 - 4							2.80					
4 - 5												
5 - 6												
6 - 7												
7 - 8												
8 - 9												
9 - 10		3.50										
10 - 11												
11 - 12	7.00											
12 - 13												
13 - 14												
14 - 15	2.60					1.80			9.00			
15 - 16								2.50				
16 - 17								1.40				
17 - 18				16.80	8.80		4.20	1.60				
18 - 19												
19 - 20												
20 - 21												
21 - 22												
22 - 23												
23 - 24												
Total												
H in 10m												
H in 20m												
H in 30m												
H in 1 hr	7.00	3.50	7.00	16.80	8.80	1.80	4.20	2.50	9.00			

Time is local standard
time

H = Highest rain fall

m = Minute

hr = Hour

Station: Robe

Region : Balie

Month : sept. Year :

Element: Rain fall Intensity

1998

Date/ Time	1	2	7	8	10	11	12	16	17	18		
0 - 1												
1 - 2												
2 - 3									0.80			
3 - 4			1.60						0.70			
4 - 5												
5 - 6												
6 - 7												
7 - 8												
8 - 9				0.80								
9 - 10												
10 - 11												
11 - 12												
12 - 13												
13 - 14							0.30					
14 - 15		0.30		0.30		18.00						
15 - 16	2.40					1.00				9.00		
16 - 17			0.60	2.60	2.00							
17 - 18				0.80				0.40				
18 - 19			1.80		4.30					2.50		
19 - 20												
20 - 21												
21 - 22												
22 - 23												
23 - 24												
Total												
H in 10m												
H in 20m												
H in 30m												
H in 1 hr	2.40	0.30	1.80	2.60	4.30	18.00	0.30	0.40	0.80	9.00		

Time is local standard
time

H = Highest rain fall

m = Minute

hr = Hour

Station: Robe

Region : Balie

Element: Rain fall Intensity

Month : sept. Year :
1998

Date/ Time	19	20	21	22	23	24	26	27				
0 - 1												
1 - 2												
2 - 3												
3 - 4												
4 - 5												
5 - 6												
6 - 7												
7 - 8												
8 - 9												
9 - 10												
10 - 11												
11 - 12												
12 - 13												
13 - 14				3.50								
14 - 15						8.40	2.40					
15 - 16					1.00							
16 - 17	0.30			13.00				1.70				
17 - 18	1.00	0.20		1.20								
18 - 19			0.20			0.30						
19 - 20								4.40				
20 - 21												
21 - 22								2.00				
22 - 23												
23 - 24												
Total												
H in 10m												
H in 20m												
H in 30m												
H in 1 hr	1.00	0.20	0.20	13.00	1.00	8.40	2.40	4.40				

Time is local standard
time

H = Highest rain fall

m = Minute

hr = Hour

Station: Robe

Region : Balie

Month : Oct. Year :

Element: Rain fall Intensity

1998

Date/ Time	3	4	5	6	7	8	9	10	12	18		
0 - 1												
1 - 2									1.00			
2 - 3												
3 - 4												
4 - 5		7.60							2.20			
5 - 6				2.60	3.20							
6 - 7					2.80							
7 - 8												
8 - 9			1.20									
9 - 10							4.00					
10 - 11												
11 - 12												
12 - 13												
13 - 14						1.60						
14 - 15							6.00		2.60			
15 - 16	12.20						1.00	4.00		0.20		
16 - 17				2.20				4.30		2.40		
17 - 18	0.80		0.50		6.80					0.80		
18 - 19										1.20		
19 - 20										0.50		
20 - 21												
21 - 22												
22 - 23												
23 - 24									1.20			
Total												
H in 10m												
H in 20m												
H in 30m												
H in 1 hr	22.20	7.60	1.20	2.60	6.80	1.60	6.00	4.30	2.20	2.40		

Time is local standard
time

H = Highest rain fall

m =

Minute

hr = Hour

Station: Robe

Region : Balie

Element: Rain fall Intensity

Month: Oct. Year : 1998

Date/ Time	19	20	21	23	24	28	30					
0 - 1												
1 - 2				6.50								
2 - 3			5.00	1.00								
3 - 4				1.50								
4 - 5												
5 - 6												
6 - 7				9.00		11.00						
7 - 8				10.00		1.20						
8 - 9												
9 - 10						1.20						
10 - 11												
11 - 12												
12 - 13												
13 - 14												
14 - 15	1.50											
15 - 16		5.20										
16 - 17		12.00										
17 - 18												
18 - 19					9.00		6.00					
19 - 20	1.50						1.60					
20 - 21		0.70										
21 - 22												
22 - 23												
23 - 24												
Total												
H in 10m												
H in 20m												
H in 30m												
H in 1 hr	1.50	12.00	5.00	10.00	9.00	11.00	6.00					

H = Highest rain fall

Time is local standard time

m = Minute

hr = Hour

Appendix – C. One- hour unit hydrographs

Event .NO 05 & 06

Time(hr)	Event - 05				Event - 06			
	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00	1.51	4.24	2.65	6.01	0.84	2.44	0.84	5.58
2.00	8.44	12.51	9.49	13.64	4.30	7.80	3.08	13.37
3.00	15.96	17.98	16.92	15.35	9.64	12.45	6.08	15.92
4.00	10.48	18.99	20.34	14.80	13.57	14.73	9.30	15.70
5.00	8.09	17.18	18.08	13.27	15.63	14.97	12.03	14.15
6.00	6.24	14.18	13.68	11.41	14.44	13.89	13.57	12.10
7.00	4.82	11.01	10.34	9.54	11.80	12.13	13.78	9.99
8.00	3.72	8.18	7.78	7.83	9.64	10.15	12.45	8.05
9.00	2.87	5.89	5.90	6.32	7.88	8.23	10.21	6.37
10.00	2.21	4.13	4.44	5.05	6.44	6.50	8.18	4.96
11.00	1.71	2.84	3.38	4.00	5.26	5.04	6.57	3.83
12.00	1.32	1.92	2.52	3.14	4.30	3.84	5.31	2.92
13.00	1.02	1.28	1.92	2.45	3.51	2.89	4.27	2.22
14.00	0.79	0.85	1.45	1.91	2.87	2.15	3.43	1.67
15.00	0.61	0.55	1.11	1.47	2.34	1.58	2.73	1.25
16.00	0.47	0.36	0.81	1.14	1.92	1.15	2.24	0.93
17.00	0.22	0.23	0.64	0.87	1.56	0.84	1.75	0.69
18.00	0.11	0.15	0.47	0.67	1.28	0.60	1.40	0.51
19.00		0.09	0.34	0.51	1.04	0.43	1.12	0.38
20.00		0.06	0.26	0.39		0.31	0.91	0.28
21.00		0.04	0.21	0.30		0.22	0.77	0.20
22.00		0.02	0.17	0.22		0.15	0.56	0.15
23.00		0.01	0.00	0.17		0.11	0.00	0.11
24.00		0.01	0.00	0.13		0.08	0.00	0.08
25.00		0.01	0.00	0.10		0.05	0.00	0.06
26.00			0.00				0.00	
27.00			0.00				0.00	
28.00								
29.00								

Event .NO 07 & 08

Time(hr)	Event - 07				Event - 08			
	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH	GIUH Clark	GIUH Nash	HEC-HMS Clark	Nash IUH
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.00	1.88	5.12	4.11	9.94	1.82	4.99	2.96	6.64
2.00	10.54	14.62	14.18	17.78	10.19	14.32	10.61	15.47
3.00	18.81	20.12	22.47	14.19	18.36	19.82	18.32	17.67
4.00	21.60	20.26	21.82	11.55	21.38	20.10	20.57	16.74
5.00	17.29	17.47	16.34	9.47	17.35	17.45	17.39	14.52
6.00	13.02	13.72	11.95	7.81	13.12	13.80	13.14	11.95
7.00	9.81	10.14	8.73	6.46	9.92	10.27	9.93	9.50
8.00	7.39	7.17	6.40	5.36	7.50	7.32	7.50	7.38
9.00	5.57	4.91	4.69	4.45	5.68	5.05	5.64	5.63
10.00	4.19	3.28	3.42	3.70	4.29	3.39	4.25	4.23
11.00	3.16	2.15	2.50	3.08	3.25	2.24	3.21	3.15
12.00	2.38	1.38	1.85	2.56	2.45	1.45	2.43	2.32
13.00	1.79	0.88	1.34	2.14	1.86	0.93	1.82	1.70
14.00	1.35	0.55	0.99	1.78	1.40	0.59	1.39	1.23
15.00	1.02	0.34	0.72	1.49	1.06	0.37	1.04	0.89
16.00	0.77	0.21	0.51	1.24	0.80	0.23	0.79	0.64
17.00	0.58	0.13	0.38	1.04	0.69	0.14	0.39	0.46
18.00	0.43	0.08	0.27	0.87	0.46	0.09	0.29	0.33
19.00	0.33	0.05	0.21	0.72	0.35	0.05	0.21	0.23
20.00		0.03	0.00	0.61		0.03	0.00	0.17
21.00		0.02	0.00	0.51		0.02	0.00	0.12
22.00		0.01	0.00	0.42		0.01	0.00	0.08
23.00		0.01	0.00	0.35		0.01	0.00	0.06
24.00		0.00	0.00	0.30		0.00	0.00	0.04
25.00		0.00	0.00	0.25		0.00	0.00	0.03
26.00			0.00				0.00	
27.00			0.00				0.00	
28.00			0.00				0.00	
29.00			0.00				0.00	

Appendix – D. Gauge Height versus Mean Velocity relationship

Event No 01

Item No.	Gauge Height(m)	Mean velocity (m/s)
1	1.38	0.90
2	1.03	0.35
3	1.21	0.53
4	1.70	1.26
5	0.95	0.20
6	0.94	0.16
7	1.19	0.56
8	1.44	0.96
9	1.36	0.70
10	1.29	0.58
11	1.64	1.23
12	1.29	0.62
13	1.20	0.57
14	1.30	0.72
15	1.04	0.26
16	1.16	0.47
17	1.28	0.68
18	1.38	0.74
19	1.30	0.63
20	1.36	0.69
21	1.00	0.29
22	0.91	0.16
23	0.94	0.17
24	0.95	0.18
25	0.95	0.20
26	1.02	0.29
27	0.91	0.16
28	1.12	0.47

Event No 02

Item No.	Gauge Height(m)	Mean velocity (m/s)
1	1.11	0.39
2	1.03	0.34
3	1.13	0.50
4	0.84	0.08
5	0.84	0.09
6	1.23	0.58
7	1.05	0.34
8	1.36	0.78
9	1.33	0.74
10	1.17	0.49
11	1.64	1.22
12	0.95	0.20
13	0.85	0.14
14	0.86	0.15
15	1.36	0.78
16	1.33	0.82
17	1.36	0.80
18	1.38	0.86
19	0.87	0.17
20	0.86	0.14
21	0.98	0.33
22	1.30	0.72
23	1.33	0.85
24	1.12	0.51
25	0.90	0.20
26	1.16	0.47
27	1.06	0.35
28	0.86	0.17
29	0.87	0.17

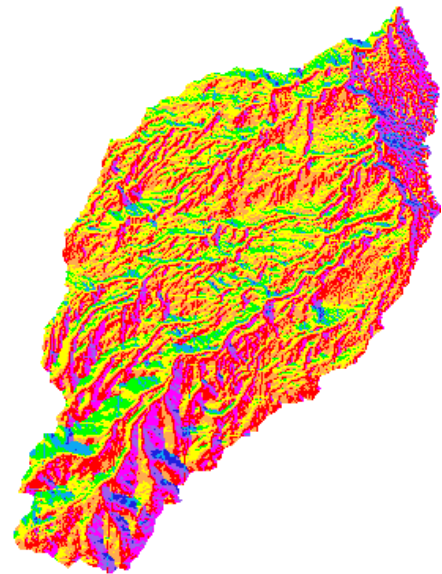
Event No 03

Item No.	Gauge Height(m)	Mean velocity (m/s)
1	1.22	0.57
2	1.12	0.43
3	1.10	0.53
4	1.37	0.80
5	1.36	0.85
6	1.50	1.00
7	1.52	1.08
8	1.35	0.80
9	1.36	0.76
10	0.82	0.14
11	0.96	0.23
12	1.00	0.29
13	1.47	1.03
14	1.22	0.65
15	1.16	0.51
16	1.31	0.73
17	1.22	0.59
18	1.19	0.52
19	1.13	0.45
20	1.08	0.40
21	1.68	1.38
22	1.40	0.89
23	1.11	0.48
24	0.98	0.23
25	1.31	0.60

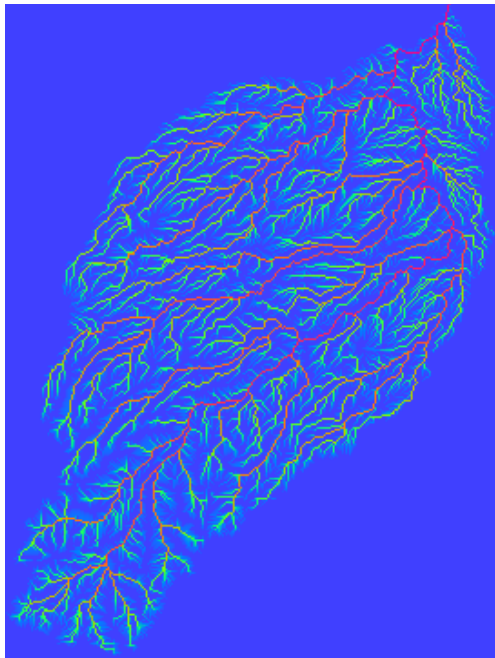
Appendix – E. ILWIS processed Raster maps of Shaya river basin



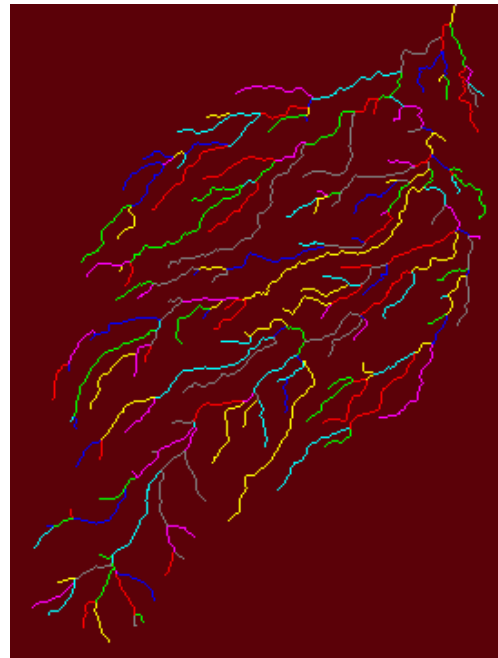
A) Filled sink map



C) flow accumulation map



Flow direction map



D) Stream ordering map