SURFACE HYDROLOGIC APPRAISAL OF A NATURAL CATCHMENT

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

HYDROLOGY

By

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JUNE, 2008

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this dissertation entitled **"SURFACE HYDROLOGIC APPRAISAL OF A NATURAL CATCHMENT"** in partial fulfillment of the requirement for the award of the **Degree of Master of Technology in Hydrology,** submitted in the **Department of Hydrology, Indian Institute of Technology, Roorkee,** is an authentic record of my own work carried out during the period from July, 2007 to June, 2008 under the supervision of **Dr. B.S. Mathur** and **Dr. M.K. Jain.**

The matter embodied in this dissertation has not been submitted by me for the award of any other degree of this Institute or any other University.

Date: 30/06/08

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

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ACKNOWLEDGEMENTS

I wish to express my deep sense of gratitude to Dr. B.S. Mathur, Professor, and Dr. M.K. Jain, Assistant Professor, Department of Hydrology, Indian Institute of Technology, Roorkee for his valuable guidance, constant encouragement and whole hearted co-operation in carrying out this study, without which it would not have been possible to complete this dissertation at all.

I am also grateful to Dr. N.K Goel, Professor and Head, Department of Hydrology, Indian Institute of Technology, Roorkee for his encouragement and moral support during the study. I also take this opportunity to express my sincere thanks to all other faculty members of the Department of Hydrology for their excellent teachings of the concepts and fundamentals which have been used in the studies and their inspiration throughout the course.

I am also thankful to all the staff members of Department of Hydrology who have extended all sort of co-operation whenever required in connection with this work. Useful co-operation and help received from my fellow trainee officers and friends is hereby acknowledged with thanks.

June, 2008 (SUNIL BAGDE)

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ABSTRACT

Simple hydrological model such as unit hydrograph is being used for making surface hydrological appraisal from natural catchments. Several variations are available in literature for the same. The unit hydrograph method proposed by Clark (1945) is selected for use in present study for simulating direct runoff hydrograph for different discritized sub-watershed of Chambal river basin upstream of Gandhi Sagar dam. A geographic database for evaluation of catchment characteristics and discritization of bigger watershed into smaller sub-watershed is prepared. HEC-geoHMS interface is used to make geo-processing of the data. Clark model parameter for all eight discritized sub-catchment have been calculated and related to measurable catchment characteristics. Result of the study indicate that the method of Clark is suitable for simulating peak, time to peak discharge as well as overall shape of the flood hydrograph. Developed relation for estimation of Clark model parameters exhibit R^2 value between 0.97 and 0.99 indicating suitability of developed relations for estimation of Clark parameter for Chambal basin.

LIST OF NOTATIONS / ABBREVIATION

A	=	Total catchment area
DEM	=	Digitial elevation model
E	=	Error
ft/ml	=	Feet per mile
HEC-HMS	=	Hydrologic Engineering Centre – Hydrologic Modelling System
Hr	=	Hour
Km	=	Kilometre
L	=	Length of channel
m^3/sec	=	Meter cube per second
NSE	=	Nash Sutcliff Efficiency
Q(t)	=	Ordinate of direct runoff hydrograph at time t
R	=	Storage coefficient
S	=	Slope
Sq.km	=	Square kilometre
Tc	=	Time of concentration
%	=	Percent

CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER -1

INTRODUCTION

1.1 GENERAL CONCEPTS

Water is the most essential element to grow, develop and sustain the living things. Human creature is not an exception. All known civilizations have been found in the vicinity of the perennial river flows. Therefore, human beings have been concerned with the river discharge since the ancient times. Sometimes high floods troubled them as they washed away their shelters and sometimes low flows concerned them when they were too low to meet their requirements. Such events compelled them to predict the river discharge in advance and prepare accordingly.

In the long run, the concept of the hydrological cycle developed and their minds started to correlate the river flows with the clouds in the sky. Work of the Frenchman, Perrault, P. (1674) provided convincing evidence in the form of the hydrologic cycle which is currently accepted. Since then efforts to study the processes involved in the movement of water round the hydrologic cycle continued. The main concern of the hydrologists today is with the quantities and time distribution of water passing through the land phase of the hydrological cycle which constitutes the translation of precipitation into runoff and evapotranspiration.

Today, when running water is short, development and management of water resources systems has drawn special attention all around the world. Runoff estimation is a key element of it. In the last 150 years, techniques of the runoff estimation have successively grown from very crude guesswork to the development of present complicated mathematical models. Initially, empirical relations came up in different parts of the world. They were having local relevance and could be used only in the situation in which they were developed. These relationships mostly related the peak flood discharges with the catchment areas. Some of them also included the slope and the rainfall intensity. Although these relationships are still popularly used for the design and maintenance of flood works, there are many evidences when such estimations failed and caused unexpected devastations and loss of lives.

As records of events increased and the knowledge of statistics improved, hydrologists drew some realistic correlations among hydrological variables. This approach is still used to some problems like data generation, filling the missing records and forecasting of the runoff and flood discharges.

Evolution of the concept of unit hydrograph is supposed to be an important mile stone in the development of rainfall-runoff relationship after the Rational Formula. Although it does not consider the physical processes involved, it is more rational than any other of the earlier age because it inherits the catchment characteristics crucial in the runoff generation. This approach is also popular and provides the basis of some advanced models. Later on, it resulted in the evolution of the instantaneous unit hydrograph which had provided a fertile field for many research workers in hydrology.

After the advent of the digital computers with large electronic memories and development of computational techniques, it became possible to make a large number of iterative arithmetical and logical computations in accordance with the real hydrological processes to get the catchment outputs. This opened an era of mathematical modelling and many (hydrological) models came up in the last four decades. Water balance is the basis of most of them. They however differ in concepts, simplifications and the nature of mathematical functions used to represent the hydrological processes and accordingly they can be classified.

Large drainage basin requires greater attention regarding hydrologic investigations in view of growing development of water resources. The transformation of rainfall excess into direct runoff is a complex process, which is the basic problem of hydrologic investigations, occupies a central place in applied hydrology. The transformation needs for proper design of hydro structures and also for reservoir regulation. Hence correct estimation of this requires not only the knowledge of the peak flood but also the time distribution of discharges throughout the period of flows. The flood peak and time distribution of runoff from a drainage basin during a storm depend upon the meteorological conditions and also on the physiographical characteristics of the basin. The available concept, regarding the transformation of rainfall excess in the direct runoff, the synthetic approach and empirical formulae give only the knowledge of flood peak, but the time distribution of runoff throughout the period of flows can conventionally be predicted only by Unit Hydrograph approach proposed by Sherman (1932). This approach is based on the availability of gauging data and unable to predict the direct runoff considering the distributed input.

1.2 OBJECTIVES OF THE STUDY

Among the Indian states, Madhya Pradesh and Rajasthan form the biggest area of Indian mainland which is mainly catered by the Chambal river which is the tributary of the Yamuna river. There are many tributaries which form a part of Chambal catchment. In order to cater to the needs of the large population living in this part of the Yamuna basin, it is necessary to have an estimate of the water potential of different tributaries. With these objectives in view a literature survey was carried out. The area is broadly covered by the network of the Indian Meteorological Department (IMD) for measuring the rainfall. However, the runoff data of different sub-basin of the Chambal river are not available. Under the circumstances in order to estimate the availability of water during the monsoon months, it is necessary to develop synthetic unit hydrograph (SUH) for Chambal river basin. This becomes all the more essential because this area receives rainfall only, and the component of snowmelt is not involved. Keeping the above in view, the following objectives for this study have been outlined:

I. Delineation of the Chambal river basin within the states of Rajasthan and Madhya Pradesh with its outlet at Gandhi Sagar dam site.

II. Delineation of sub-catchments of all the tributaries of the basin.

III. Detail physiographic investigation of all the sub basin and Chambal river as a whole.

IV. Development of synthetic unit hydrograph using the Clark's model (1945) for all the sub-basin of the river system.

Keeping the above objectives in view, the content of this dissertation has been arranged accordingly.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION

A hydrograph is a recording of stage, discharge or velocity values at a given point in a watershed as a function of time. Historically, the results have been presented as a two dimensional plot, but the series of ordered pairs (time, value) used to generate the plot is also a hydrograph. Many methods have been developed to predict the runoff hydrograph for a given rainfall event. Rainfall is represented using a hyetograph i.e. a time-series of rainfall. The resulting hydrographs are used to design engineering projects, predict required storage volumes, and as input for pollutograph predictions.

2.2 HYDROLOGIC MODELS

Hydrologic models in general can be classified as lumped models or distributed models. In lumped models, spatial variability in hydrologic parameters or meteorological related data are not accounted for, meaning are averaged or assumed uniform over the system, whereas, in distributed models spatial variability is explicitly accounted by assuming uniformity over smaller modeling units by sub- dividing the bigger system based on physical properties. In most of the distributed hydrologic models, these units are delineated by combining topography, soil properties, land use properties and other pertinent properties. Distributed models are especially useful, for example, when impacts of land use change are to be studied or for analyzing spatially varying flood responses. As the topic of distributed modeling is of importance to this thesis, a discussion of related back ground is provided

2.2.1 HEC-HMS (Hydrologic Modeling System)

The HMS is a comprehensive hydrologic model developed by Hydrologic Engineering Center (HEC) of United States Army Corps of Engineers (USACE). It is an event – based overall lumped model (HEC, 2000). HMS offers several options to model various physical processes occurring in a watershed system. One such process is the direct runoff computations. Most of runoff models available with HMS are lumped in nature except for two which are distributed. Most of the lumped runoff models derive their roots from the Unit Hydrograph (UH) concept. This model provides a lumped model option called Clark's UH. To overcome its lumped character, a modified version called ModClark method was developed for HMS (Daniel and Arlen 1998).

ModClark's method requires that watershed be further divided into sub-areas by intersecting it with a grid. Each of these sub-areas is assigned individual lag time, instead of one value for the whole watershed, as in the case of Clark's UH. The precipitation excess at each sub-area is transported to the watershed outlet using the corresponding lag time. Thus the inflow contributions due to all the subareas to linear reservoir are computed. These flows are then routed through a linear reservoir (only a single value for storage coefficient being defined for all the sub areas) to obtain the hydrograph at the outlet, which will later be routed through the channels.

2.2.2 DEM-BASED METHOD

Since the time Jenson and Domingue (1988) have developed the D-8 algorithm, it has been incorporated in a number of watershed parameterization and hydrologic models. The concept of this method is that each cell in a DEM is assumed to flow to one of the eight neighboring cells according to the direction of steepest slope. Though a number of other flow direction determination methods that are not based on D-8 algorithm have been developed, because of its simplicity, the D-8 algorithm has been employed in a number of DEM-based models. Among the noted ones is the Watershed Delineator developed by ESRI which can be used for delineating streams and watersheds (Djokic et.al., 1997).

Later, based on the same D-8 algorithm, CRWR-PrePro was developed at Center for Research in Water Resources (CRWR), to create input files for HEC-HMS (Hellweger and Maidment 1999, Olivera 2001). The capabilities of this model like terrain analysis, topologic analysis, watershed delineation helps create basin model for HEC-HMS models (Olivera 2001).

HEC-GeoHMS is a preprocessor similar to CRWR-PrePro (HEC 2000). AVSWAT, an ArcView interface developed with the aim of creation of input files for SWAT, whose watershed delineation function is based on D-8 algorithm. This model also incorporates in itself a parameter calculation function (Neitsch, et.al 2000).

To overcome the limitation of D-8 algorithm like flow direction being restricted to one of the eight possible directions, Tarboton (1997) explained a method to determine the flow direction based on a single angle among the infinite possible directions, which was called D ∞ method. Several other researchers came up with alternatives for determining flow directions (Costa-Cabral.M and S. Burges, 1994).

2.3 RUNOFF PROCESS

Runoff is the surface water flow collected at a location in a watershed. Conceptually, the watershed integrates all the physiographic and hydro-meteorological processes that produce runoff. From this definition it should be clear that runoff varies both with location and time. The runoff in stream channels is classified as direct runoff and base flow. The total rainfall over a watershed is considered to consist of rainfall excess and rainfall abstractions or losses. Rainfall excess is the fraction of the total rainfall that contributes directly to the surface runoff. The part of the rainfall that contributes entirely to the direct runoff is called the effective rainfall. The effective rainfall consists of rainfall excess and that part of the rainfall that becomes prompt subsurface runoff.

2.3.1 STRUCTURE OF A HYDROGRAPH

A typical runoff hydrograph produced by a concentrated storm rainfall is generally a single-peaked skew distribution curve. Different parts of a simple hydrograph are:

a) Rising limb or concentration curve: period of time elapses before the flow begin to rise due to interception, infiltration, soil-moisture deficits. After losses, rainfall excess contributes to stream flow.

b) Crest segment with peak discharge.

c) Recession curve or falling limb: after rainfall ceases, still there is some contribution to stream flow until the inflection point. After this time water comes from soil storage (interflow).

The following are the properties of a typical hydrograph:

a) Lag time (L): time interval from the center of mass of rainfall excess to the peak of the resulting hydrograph.

b) Time to peak (tp): time interval from the start of rainfall excess to the peak of the resulting hydrograph.

c) Time of concentration (tc): the time interval from the end of rainfall excess to the inflection point (change of slope) on the recession curve. Also, the longest time for water to flow to a discharge point from any point in the watershed.

d) Recession time (tR): time from the peak to the end of surface runoff.

e) Time base (tb): time from the beginning to the end of surface runoff.

Figure 2.1 below is a typical hydrograph indicating the above-mentioned parts and properties.

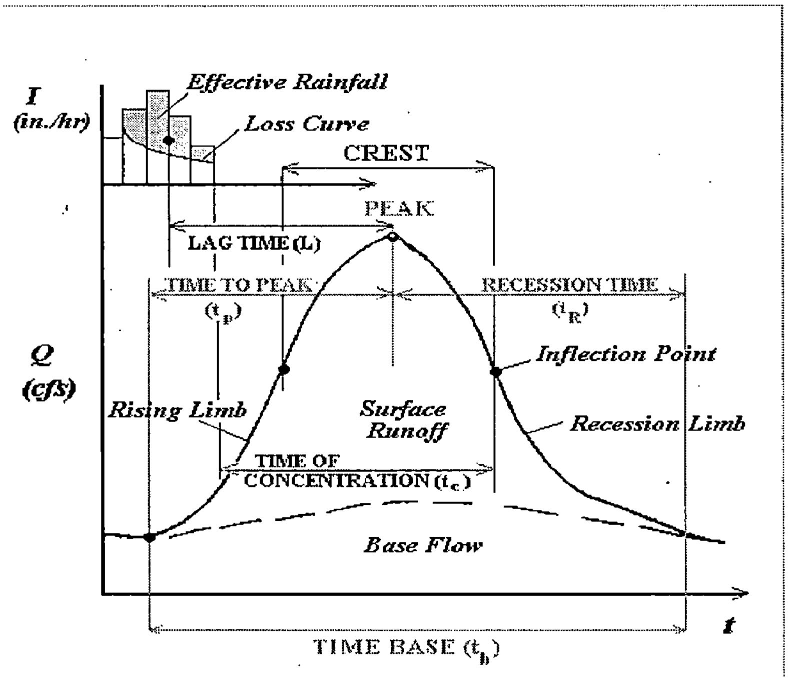

Figure: 2.1 Components of hydrograph

2.3.2 METHODS OF ANALYSIS

Numerous methods have been proposed for the estimation of runoff from the given effective rainfall (total rainfall void losses). A hydrograph can be developed using the unit hydrograph method developed by Sherman (Sherman, 1932), Snyder's synthetic unit hydrograph method (Snyder, 1938), and Commons dimensionless hydrograph (Commons, 1942).

In general a model is constructed so that the model parameters can be related to the physical parameters of the corresponding watershed. Studies showed that the model parameters estimated this way exhibited considerable regional stability (Snyder, 1938).

Mostly these models were proven to be more accurate in the respective regions for which they have been developed.

2.3.3 UNIT HYDROGRAPH

According to Chow (Chow, 1964) recognized the relationship between rainfall and runoff was as early as in 1929. Three years later a similar concept involving the successive ordinates of a 24-hr unit hydrograph was published by Sherman (Sherman, 1932).

Sherman's work is considered the seminal paper on unit hydrographs. A unit hydrograph for a drainage basin has been defined as the direct runoff hydrograph resulting from 1 inch of effective rainfall generated uniformly over the basin area at a uniform rate during a specified period of time or duration. The word "unit" as used by Sherman in his study was the 'unit of time' of the effective rainfall A runoff hydrograph for a drainage basin can be determined from the unit hydrograph given the amount of effective rainfall. The unit hydrograph defined above can be used to derive the hydrograph of runoff due to any amount of effective rainfall. The two basic principles to be satisfied to use the unit hydrograph theory are the linearity and time invariance. The ordinates of the direct-runoff hydrographs are mutually proportional and thus can be added or superimposed numerically in proportion to the total amount of direct runoff featuring the principle of linearity. The direct runoff hydrograph from a watershed due to a given pattern of effective rainfall at whatever time it may occur is invariable. This is known as the principle of time invariance.

In reality, all these assumptions are violated. In practice however, the unit hydrograph method has proved to be a very useful method to obtain engineering estimates for design purposes. Though initially developed for large drainage basins, studies have shown that UHs can be applied for smaller watersheds varying in area from 4 acres to 10 sq mi (Brater, 1940).

Snyder (1956) obtained unit hydrographs by least squares analysis of rainfall and runoff data. Nash (1959) studied the relation between the number of parameters (moments about origin) and the stream catchment's characteristics for the instantaneous unit hydrograph.

Eagleson et. al. (1966) applied the Weiner-Hopf theory to determine unit hydrographs from the observed rainfall and runoff data.

High frequency oscillations observed in the unit hydrographs were related to colinearity, the linear relation between the elements in a linear system. Multiple events were used in the deconvolution process in deriving the unit hydrographs (Bree, 1978).

A similar approach of deconvolution was used in overcoming the high frequency oscillations in the unit hydrographs derivation (Mawdsley et.al., 1981).

Extended research in this direction involved the development of a linear programming approach for the optimal determination of unit hydrographs (Mays et al., 1980). Non-linear programming models for the development of unit hydrographs were also developed (Unver et. al., 1984).

Commons (1942) suggested that a dimensionless hydrograph, the so-called basic hydrograph, would give an acceptable approximation of the flood hydrograph on any Texas basin. This hydrograph was developed from flood hydrographs in Texas. It is divided so that the base time is expressed as 100 units, the peak discharge as 60 units and the area as a constant of 1,196.5 units.

A form of NRCS dimensionless unit hydrograph was developed using the data from 40 Midwestern and Eastern watersheds (Holton et al., 1963). Synthetic hydrographs can be developed using a two-parameter Gamma distribution (Croley II, 1980). A model when developed using the correlation methods will constitute of some parameters, which can be related to the physical characteristics of the watersheds for the case of ungaged watersheds.

2.4 INSTANTANEOUS UNIT HYDROGRAPH (IUH)

The unit hydrograph theory is the application of linear systems theory to the rainfallrunoff process (Dooge, 1973; Chow, et al, 1988). Chow and others have applied various theories to hydrologic modeling since the late 1960's. One of the simpler approaches to rainfall-runoff modeling has been through the applications of linear systems theories (Dooge, 1973).

The unit hydrograph from an effective precipitation of infinitesimally small duration is called an Instantaneous Unit Hydrograph (IUH). Generally the IUH is represented by $u(t)$ in the literature. For an IUH the effective precipitation is applied to the drainage basin over a very short duration of time (impulse). The major advantage of the IUH over the unit hydrograph is that the IUH is independent of the duration of the effective rainfall reducing the number of variables in the hydrograph analysis. Of course the zero time duration is a fictitious situation that will be violated in the hydrograph analysis, but it can be approximated from the slope of a finite-duration precipitation depth plot. In a linear unit hydrograph theory by the principle of superposition, when an effective rainfall of function $I(\tau)$ of duration $t0$ is applied , each infinitesimal element of the effective rainfall hydrograph (ERH) will produce a direct runoff hydrograph (DRH). This DRH will be equal to the product of $I(\tau)$ and the IUH expected by $u(t\text{-}\tau)$. Therefore the ordinate of the DRH at time t is given as

$$Q(t) = \int_{0}^{t' \le t_0} u(t-\tau) I(\tau) d\tau$$

The ordinate of the DRH at time t shown above is called the convolution integral, also known as Duhamel integral, in which $u(t-\tau)$ is a kernel function, $I(\tau)$ is the input function and $t'=t0$ when $t > t0$ and $t'=t$ when $t' \leq t0$. The shape of an IUH resembles a single peaked hydrograph. If the rainfall and the runoff in the convolution integral are measured in the same units, the ordinates of the IUH must have a dimension of [T-1]. The following are the properties of IUH:

$0 \leq u(t) \leq$ a positive peak value, for $t>0$

$u(t) = 0$, for $t \leq 0$

$u(t) \rightarrow 0$, for $t \rightarrow \infty$

$$\int_0^\infty u(t)dt = 1$$

$$\int_0^\infty u(t)t\, dt = t_L$$

where t_L is the lag time of the IUH (Chow, 1964).

Since convolution is a linear process it can be shown that tL is also equal to the time interval between the centroid of the effective rainfall and that of the direct runoff. The idea of applying an IUH to derive a unit hydrograph was originally attributed to Clark in 1945 (Clark, 1945).

Nash in 1957, instead of characterizing the runoff as translation followed by storage in a single reservoir as Clark did, viewed the watershed as a series of n identical linear storage reservoirs. Once the IUH is obtained for a watershed it can be used to synthesize any other hydrograph from a rainfall time series on the watershed by convolution.

Unit hydrograph procedures should be limited to watershed drainage areas that are less than about 2000 square miles. If the storm patterns are thought to impact runoff hydrographs, then the watershed can be subdivided into smaller sub-watersheds and each of those subjected to a hydrograph analysis. To calculate a flood hydrograph, the unit hydrograph is applied to the hyetograph of rainfall excess to estimate the hydrograph of surface runoff, and base flow is added to produce the flood hydrograph.

To develop a unit hydrograph, one should acquire as many rainfall records as possible within the study area to ensure the amount and distribution of rainfall over the watershed is accurately known. The final unit hydrograph will represent the variations in the rainfall-runoff processes over a larger period of time allowing a generalized UH.

For a specific watershed, unit hydrographs can be developed using two approaches. Given the rainfall-runoff data, different techniques can be applied to estimate the unit hydrographs from the measurements. For watersheds with no rainfall-runoff record, methods of synthetic hydrology must be applied. This thesis uses the analysis of unit hydrographs from measured rainfall-runoff events.

2.5 WATERSHED RESPONSE FUNCTION

The unit hydrograph (UH) or the IUH can be treated as a function that converts the rainfall to the observed runoff on a given watershed. This transfer function is often termed as response function in hydrology. On a gaged watershed, determination of model parameters from the observed rainfall-runoff data is one of the objectives of UH analysis. Similar analysis can be done on the ungaged watersheds except that the model parameters are obtained from the physical characteristics of the watershed (area, perimeter, etc.,). For gaged watersheds, the unit hydrograph is obtained by the following procedure:

1. Determine the direct runoff hydrograph by subtracting the base flow from the hydrograph using an appropriate base flow separation technique.
2. Compute the volume of the runoff under the hydrograph.
3. Divide the ordinates of the direct runoff hydrograph by the direct runoff volume. The resulting hydrograph is the unit hydrograph for the watershed (this step essentially forces the integral of the UH to equal one).
4. Duration of the unit hydrograph can be obtained from the effective rainfall hyetograph (ERH), which can be obtained by assuming a suitable method for the abstractions. The time-duration of the pulses of effective precipitation is the duration of the unit hydrograph.

Using the above method, for more than one event, a set of unit hydrographs with different durations will be obtained for a given watershed. Morgan and Hullinghors were the first to suggest the S-hydrograph technique (Chow, 1964) that can be used to obtain a common duration for all the derived unit hydrographs. This set of common time-duration unit hydrographs can be averaged to arrive at a single UH that can be applied to the watershed. One way that an average unit hydrograph may be constructed is by taking the arithmetic means of the peak flows (*Up*) and the times to peak (*Tp*), plotting the average peak at the appropriate mean value of *Tp*, and drawing the hydrograph to match the general shapes of the individual unit hydrographs.

Deconvolution is the process of extracting the unit response function (UH) from a direct runoff hydrograph and the generating precipitation sequence. Deconvolution can be used to obtain the UHs for complex storms with no non-linearity or errors in the data.

The error between the observed and the estimated direct runoff hydrograph (DRH) can be minimized using the least squares fitting or an optimization technique (Singh, 1976).

Collins (1939) proposed a method of successive approximation putting forward a unique convolution. The drawbacks in estimating the unit hydrograph as a solution for a set of linear equations for different rainfall pulses is overcome by the Collins procedure.

Historically the response functions have been treated as statistical distributions although researchers have linked simplified physics to the distributions (Nash, 1958; Leinhard, 1971). Linking a series of reservoirs in a feed forward (cascade) fashion, Nash (1958) developed his IUH. The Nash model, gamma-hydrograph, and Pearson Type III hydrograph are identical distributions (under certain circumstances).

Lienhard and Meyer (1967) showed that the gamma family of distributions can be explained using statistical-mechanical principles, establishing a rigorous physical basis for IUHs.

The unit hydrograph procedure should be limited to watershed drainage areas that are less than about 2,000 square miles. If storm patterns are thought to impact runoff hydrographs, then the watershed can be subdivided into smaller sub watersheds and each of those subjected to a hydrograph analysis. The development of the procedure has been documented many times.

2.6 SYNTHETIC UNIT HYDROGRAPH

As mentioned earlier, actual or observed unit hydrographs can not be determined for all the basins since there are not available rainfall and runoff data everywhere. Therefore, for such basins unit hydrographs are determined synthetically, to be used in the design of hydraulic structures. Synthetic unit hydrographs are developed using two main concepts;

1) Each watershed has a unique unit hydrograph

2) All unit hydrographs can be represented by a single family of curves or a single equation.

Several methods have been developed for estimating synthetic unit hydrographs for locations where observations of input and response are lacking.

Chow et al (1988) group synthetic unit hydrographs into three types:

(1) Those relating hydrograph characteristics (peak flow, time to peak, base time, etc.) to watershed characteristics (Snyder, 1938; Gray, 1961);

(2) Those based on conceptual models of watershed storage (Clark, 1943; Nash 1957)

(3) Those based on a dimensionless unit hydrograph DUH (Soil Conservation Service 1972).

Types (1) and (2) involve empirical coefficients whose validity is limited to a particular watershed or region. Type (3) is based on the expectation that, by selecting proper dimensionless ratios, all individual unit hydrographs can be transformed into one more-or-less universally applicable DUH.

A number of parameters are important in determining the shape of the unit hydrograph for a watershed. The discharge parameter which is mostly used is the peak discharge (Qp). Lag time (tL), time to peak (tp), time of concentration (tc) and base time (Tb) are often used as the time parameters. Watershed parameters of most concern, influencing the shape of the outflow hydrograph, include area (A in sq. mi.) and its shape, main stream length (L in ft), length to watershed centroid from the outlet (Lc in ft) and average slope of basin (y in %).

In the next chapter, the area of study has been described for which the physiographic analysis have been conducted to derive the SUH.

CHAPTER -3

THE STUDY AREA AND ITS ANALYSIS

3.1 LOCATION OF CHAMBAL CATCHMENT

River Chambal is one of the principal tributaries of the river Yamuna. The length of river is about 965 kilometre. Mostly flows of the river through the states of Madhya Pradesh and Rajasthan in India. The catchment of the Chambal river falls within latitudes 22° N to 25° N and longitudes 74° 40'E to 76° 20'E . This river rises in the northern slopes of Vindhya Mountain about 32 kilometre south-west of Mhow in Madhya Pradesh. at an elevation of about 853 meter above the mean sea level. It flows first in northernly direction for a length of about 362 kilometre and after passing through the historic part of Chaurasigarh, it flows in north-easterly direction for a length of about 603 kilometre through Rajasthan, Madhya Pradesh and Utter Pradesh before joining the river Yamuna (i.e. south west of Etawah at an elevation of 122 meter above mean sea level). Only 4% of the catchment is covered by forest while agricultural land covers 72%. The upper catchment has numerous minor tanks meant for irrigation of Rabi crops.

The present study pertain to the Chambal river basin upto Gandhi Sagar dam which is constructed on river Chambal in Mandsour district of Madhya Pradesh at a distance of about 350 kilometre from the source.

3.2 DISTRIBUTED WATERSHED OF CHAMBAL CATCHMENT AND ITS CHARACTERISTICS

3.2.1 SIZE OF BASIN

The total drainage area at Gandhi Sagar was reported to be 23025 sq. km. In present study while working on the toposheet 1" = 4 miles, the area has been computed as 22920 sq. km.

3.2.2 DRAINAGE PATTERN

The four important tributaries of the Chambal river are Shipra, Chhoti-Kalisindh, Shivana and Retum .The first two tributaries are running almost parallel to the main Chambal river. The catchments of those tributaries while taken together form nearly about two thirds of the total catchment of the Chambal river. The Shipra and Chotti – Kalisindh river are closely spaced. The other two tributaries Shivana and Retum drain about one-half of the remaining area and join the Gandhi Sagar reservoir in its middle reaches from West.

As the basin is traversed by monsoon depressions and cyclonic storms in the same general direction as the main river i.e. in the West-North-Westerly direction, the flow concentrations from different areas synchronize at the outlet.

3.2.3 SLOPE:

Originating at an elevation of about 853 metre above mean sea level, the Chambal river drops to a level of 533 metre above mean sea level after traversing a length of about 16 km. Thereafter, it has a fairly uniform gradient of about 3.30 feet per mile (m/m) except near the confluences of river Chambal with shipra and Chhoti-Kalisindh where it has relatively steeper gradient which varies from 1.82 to 3.04 metre(above mean sea level) and the length of the river is about 350 km.

CHAPTER – 4

PHISIOGRAPHIC PARAMETERS ESTIMATION USING HEC-Geo-HMS MODEL

4.1 INTRODUCTION

Water resources related problems such as mapping of flood plains require realistic flow predictions. Accurate prediction of flows requires accurate representation of the hydrologic processes occurring in the system. An effective way to improve this accuracy is by employing spatially-distributed models. The advancement of computer technology and the relative ease in the availability of spatial data has made it possible to efficiently process spatial data for deriving physical parameters needed by hydrologic models. The data visualization and analyzing capabilities of Geographic Information Systems (GIS) present a convenient platform for hydrologic modeling. Currently, a number of hydrologic models are interfaced with GIS. The principal tasks in such models consist of discretization of the watershed system into units of uniform properties, extraction of hydrologic parameter information, and interfacing with hydrologic models. There are different formats in which the spatial data are available. The raster based representation has the structure of a grid in which each cell stores the value of the property it represents. For example, in a Digital Elevation Model (DEM), cells represent elevation values. Another type of surface data representations is Triangulated Irregular Network (TIN) in which the surface is represented as a set of connected points forming triangles. Apart from these surface based representations, the third type is the vector data. Vector data is used to represent geographic objects that have shape and size. A hydrologic system consists of streams and their corresponding drainage areas. As these objects have shape and size, they are better represented in vector data format. This kind of data is derived from traditional paper maps, photographs or by surveys (Garbrecht et al. 2001). So, development of distributed models by employing vector based hydrologic information seems to be a good option because they represent the real world objects. Additionally, inherent topology (i.e. connectivity and adjacency information) that vectors possess and the existence of spatial relationships among them, make the vector - based environment very conducive for hydrologic modeling. Significant amount of resources have been devoted to developing models in the raster-based environment. Raster based techniques, though attractive and simple to develop, have some limitations that have to do with the accuracy in determination of drainage patterns. More often than not, such surface models require vector data to be imposed on them to accurately delineate streams and their corresponding drainage areas (Neitsch et al. 2000). In the absence of such required vector data, high resolution raster files are needed. The resolution enhancement in raster for better capturing the landscape, however, leads to computational challenges, resulting in diminished efficiency. On the other hand, vector data file size increase is not as dramatic upon betterment of resolution, resulting in almost the same or reasonable computational speeds.

4.2 HEC-GeoHMS

HEC-GeoHMS [Hydrologic Engineering Center (HEC) 2000] is a geospatial hydrologic modeling extension software package that uses a graphical user interface and is linked to the ArcView and Spatial Analyst GIS. HEC-Geo-HMS uses DEM data to determine drainage paths and watershed boundaries and transforms them into hydrologic data structures representing the watershed response to rainfall events. The current version of HEC-Geo-HMS creates a background map file, lumped basin model, grid-cell parameter file for use in running the HEC-HMS hydrologic model.

4.3 HYDROLOGIC NETWORK PARAMETER DERIVATION

This section of the chapter focuses on the techniques developed for calculating hydrologic network parameters. It emphasises on algorithms developed for network tracing. The tools developed for calculating network parameters, estimate drainage areas, calculate flow lengths and determine drainage divides. The tools developed for network tracing; provide capabilities for tracing streams, upstream and downstream in the network. These methods assume that the flow system is represented by two datasets, one containing set of lines forming a dendritic network to represent streams and the other containing a set of polygons that represent the drainage areas as discussed and shown in forthcoming sections.. The lines and polygons possess relationship based on location (i.e., line inside polygon), whereas lines with other lines possess neighborhood (i.e. upstream and downstream) relation. To perform intersection and build topology for streams; as a result, streams will be:

- Attributed with unique identification numbers
- Attributed with their geometric lengths
- Attributed with the sub-basin polygon identification number
- Optionally, attributed with the length weight

Also, the polygons will be:

- Attributed with unique identification numbers.
- Attributed with their geometric areas.
- Optionally, attributed with the area weight.

Though the model provides methods to extract all these information from datasets, user has an option to use these methods directly without applying the data treatment methods such as dendrification, intersection, building topology and treatment of inconsistencies. This is valid only if stream and watershed data cater to the model requirements and the attribute information requirements. Apart from the attribute information mentioned above, lines and polygons are also attributed with optional information that will be used to capture features of data that are not captured in the spatial datasets. Each segment in the network can be attributed with a weight factor that is used. Also, each polygon can be assigned a weight factor that is used to weight the upstream areas differently based on spatially distributed pattern. The procedure starts by making the streams know who their neighbours are, then goes onto calculating the parameters pertaining only to streams, next and finally calculates the parameters that are common to both streams and watersheds. In turn, to make the calculations of rank efficient, segments in the network are attributed with their downstream segment numbers. Once the downstream segment numbers have been assigned to all the streams, their upstream segment information is computed. As each segment can have only one downstream segment and as they can have more than one upstream segment, segments have one value stored in the network table for their downstream segment number and have a collection of values for upstream segment numbers. Once all the segments are attributed with their downstream segment information, the upstream segment information is computed and then stored in collections, the logic for computing which is as follows: For each segment in the network, reading its downstream segment number, a query is performed on the segment identification number column, until the segment which bears this identification number is found. Once such a segment is found, its upstream segment list is updated with the identification number of the segment under question. This process is performed for all the segments. These lists are stored in a collection, in which a list corresponds to an array of numbers. An advantage of using collections is that information access becomes efficient. With segments being attributed with their downstream segment numbers, they are then used in computation of ranks for the segments. Rank of a segment is defined by:

$$R= 1+\max\{Ru1, Ru2, Ru3 \ldots\ldots\ldots\ldots\ldots\ldots, Rui, \ldots\ldots\ldots\ldots Run\} \quad (4.1)$$

Where R is the rank of the segment, and Ru1, Ru2, Ru3..........., Rui,.......... Run are the ranks of 'n' segments upstream of it. The algorithm for computing rank starts by finding all the segments that represent headwaters, which constitutes the first iteration. These are found by querying the downstream segment column of the network table. The segments whose identification numbers are missing in the downstream segment number column represent headwaters. All such segments are assigned a value of one for their rank attribute. In the next iteration, the algorithm loops through all the headwater segments. During a single execution, a head water segment is traced downstream and all the segments that fall on the trace are attributed a value equal to one plus rank of its immediate upstream segment found on the trace. The trace is continued until the outlet segment is reached, or a segment is visited whose rank is greater than or equal to one plus the rank of its immediate upstream line located along the trace. The network table is then sorted with respect to the rank attribute. The advantage of sorting the network table by the rank is, for a given segment all its upstream segments are found above it in the table and all the downstream segments are found in the table below it. By doing so, parameters that vary in the upstream to downstream direction can be calculated in the table from top to bottom without leaving any calculations pending and parameters varying in the opposite direction are calculated in the bottom to top direction. Taking the advantage of the sorting the table with respect to rank, parameter calculation methods would now be presented. One of them is the computation of upstream flow lengths of the segments in the stream network. Upstream flow length of a segment is the length of the longest upstream flow path out of all upstream flow paths flow paths starting at its downstream node. The upstream flow length of a segment is defined as:

$$U = \max\{Uu1, Uu2, Uu3 \ldots\ldots\ldots\ldots\ldots, Uui, \ldots\ldots\ldots Uun\} + L \quad (4.2)$$

Where, U is the upstream flow length of a segment, Uu1, Uu2, Uu3…………………., Uui,……….,Uun are the upstream flow lengths of n segments located immediately upstream of the segment and L is the length of the segment. Notice that the upstream flow length of a rank one segment is the length of the segment itself. Additionally, notice that segments located upstream of a given segment are always above in the table, and are assigned their upstream flow lengths prior to the calculation of the upstream flow length of the segment. Similarly, weighted upstream flow lengths of the segments can be computed, the only difference is that instead of adding the lengths of the segments of the longest upstream flow path, their weighted lengths are added. The weighted upstream flow length is calculated as:

$$WU = \max\{WUu1, WUu2, WUu3 \ldots\ldots\ldots, WUui, \ldots\ldots\ldots WUun\} + L \times WL \quad (4.3)$$

Where, WU is the weighted upstream flow length of a segment, WUu1, WUu2, WUu3…………………., WUui,………. WUun are the weighted upstream flow lengths of n segments located immediately upstream of the segment, L is the length of the segment and WL is the length weight of the segment itself. Downstream flow length of a segment is the length of the downstream flow path from segment to the network outlet. Downstream length is measured from the downstream node of the segment, so the length of the segment itself is not included in the calculation. Since there is only one downstream segment for a segment in the network, there exists only downstream flow path. The downstream flow length of a segment is defined as:

$$D = Dd + Ld \quad (4.4)$$

Where, D is the downstream flow length of a segment, Dd is the downstream flow length of the segment located immediately downstream of the segment and Ld is the length of the segment located immediately downstream. Note that the downstream flow length of the segment with highest rank, which is the network outlet, is zero, which is the last record in the network table. Starting at the last record in the network table, the calculations move all the way up until the downstream length of all the segments is completed. Similarly, weighted downstream flow lengths of the segments can be computed, the only difference is that instead of adding the lengths of the segments of the downstream flow path, their weighted lengths are added. The weighted downstream flow length is measured from the downstream node of the segment. The weighted downstream flow length is calculated as follows:

$$WD = WDd + Ld * WLd \qquad (4.5)$$

Where, WD is the weighted upstream flow length of a segment, WDd is the weighted downstream flow length of the segment located immediately downstream of the segment, Ld is the length of the segment located immediately downstream and WLd is the length weight of the segment located immediately downstream itself. This concludes the part of the methodology that deals with techniques for computing parameters, that are only stream based. To this point, all parameters calculated are based on the network topology and geometry. However, by relating the segments of the network with the polygons of the watershed dataset, additional information concerning drainage areas can be obtained. To establish a one-to-one relation between stream segments and watershed polygons, the outlet segment of each polygon is identified by flagging the segment of highest order out of all segments with the same polygon identification number. The logic behind this is as follows: While looping through the segments of the network, a segment is chosen, its polygon number is compared with its downstream segment's polygon number. If they are equal, it is assigned a value of 0 for its outlet segment attribute; otherwise it is assigned a value of 1 for its outlet segment attribute. Figure 4.8 shows the stream network in which the outlet segments are selected green in colour. Also, the outlet segment is attributed with the downstream polygon number, which is the polygon number of its downstream segment. This attribute is transferred to all other segments in Where, WD is the weighted upstream flow length of a segment, WDd is the weighted downstream flow length of the segment located immediately downstream of the segment, Ld is the length of the segment located immediately downstream and WLd is the length weight of the segment located immediately downstream itself. This concludes the part of the methodology that deals with techniques for computing parameters, that are only stream based. To this point, all parameters calculated are based on the network topology and geometry. However, by relating the segments of the network with the polygons of the watershed dataset, additional information concerning drainage areas can be obtained.

Calculation of total drainage areas can now be done based on the area of the watershed polygons and the network topology. The total drainage area is the sum of the areas of all polygons located upstream of a segment; however, the area of the watershed polygon in which the segment itself is located is added only at the outlet segments. The total drainage area is calculated as:

$$TA = \sum TAu\text{-}i + A \qquad (4.6)$$

Where TA is the total drainage area of the segment,TAu-1, TAu-2, ...TAu-i ...and TAu-n are the total drainage areas of the n segments located immediately upstream of it, and A is the area of the watershed polygon in which the segment is located. The weighted total drainage area is similar to the total drainage area, but adding weighted areas (i.e., the product of the area by the area weight) instead of areas. The weighted total drainage area is the sum of the weighted areas of all polygons located upstream of a segment; however, as in the previous case, the weighted area of the watershed polygon in which the segment itself is located is added only at the outlet segments. The weighted total drainage area is calculated as:

$$WTA = \sum WTAu\text{-}i + A^{*}W\,A \qquad (4.7)$$

Where WTA is the weighted total drainage area of the segment, WTAu-1, WTAu-2, ... WTAu-i ... and WTAu-n are the weighted total drainage areas of the n segments located immediately upstream of it, and A and WA are the area and the area weight of the watershed polygon in which the segment is located. Moreover, as oppose to the calculation of flow lengths, network tracing consists of identifying the actual segments located upstream or downstream of a given segment. Because of the large number of segments involved in the tracing of each segment, tracing has to be performed "on the fly" for a limited number of user-defined segments. Downstream tracing consists of populating a collection with the number of the downstream segments of the selected segment. The process starts by adding the number of the immediate downstream segment, and then adding the number of the downstream segment of the just added segment. Likewise, upstream tracing consists of populating a collection with the numbers of all the upstream segments of the selected segment. If a number of segments are selected, and at least one of them is upstream of one of the others, the upstream trace collection of the downstream segment will not include the segments already included in the collection of the upstream one.

The algorithm is based on a technique called 'Breadth First Search' Technique (Weiss 1994). The algorithm starts by adding the identification numbers of the selected segments to a collection. It starts visiting elements in the order they are added to the list. During a visit to an element (i.e. segment identification number) in the list, its immediate upstream segment number list is appended to the collection. This process is performed until the last segment in the collection, which does not have any upstream segment numbers, is visited.

4.4 HYDROLOGICALLY CORRECTED & DEPRESSIONLESS TERRAIN MODEL

The preparation of "hydrologically corrected" terrain data often requires much iteration through drainage path computations. To represent the movement of water through the watershed, the "hydrologically corrected" DEM must have the proper accuracy and resolution to capture details of the stream alignments and watershed divides. The problems often arise when the watershed has low relief and the resolution is not fine enough to delineate the needed details.

Construction of a "hydrologically corrected" terrain model involves more complexity than combining tiled USGS's DEMs into a unified DEM grid. The DEM assembled from the USGS represented by elevation averages at regular intervals may not accurately represent stream locations and watershed boundaries. For example, stream and watershed delineation sometimes does not coincide with published data sources like the EPA's RFI and the USGS's watershed in the Hydrologic Unit Code (HUC). A "hydrologically corrected" terrain model must represent accurate stream patterns across the landscape, stream confluences, internal drainage areas, and drainage facilities. Many factors, such as cell resolution, accuracy, topographic relief, and drainage facilities deserve careful consideration because they often affect the quality of the terrain model. In theory, combining GIS data sets of different resolutions is generally not recommended because of the difficulty in assessing the accuracy and the precision of the resulting data set. In practice, however, combining data sets of various resolutions is necessary due to lack of uniform data and data coverage.

In contrast to the effort required for the "hydrologically corrected" DEM, the "depressioless" DEM is simply constructed using automated algorithms to fill in the sinks or depressions in the assembled DEM. In a "depressionless" DEM, all area is contributing to the most downstream outlet and therefore does not address closed basins or substantial non-contributing areas. Because of the complexity and effort required for constructing a "hydrologically corrected terrain model, a "depressionless" terrain model often serves as a simpler substitute in the analysis. For study regions with moderate to high topographic relief, the "depressionless" terrain model may be adequate for the analysis. For low-relief regions, however, the "depressionless" terrain model often needs additional work to adequately represent the terrain. For example, a watershed with flat terrain often requires editing to force proper drainage location.

Until better data quality and editing techniques are available, users may struggle with terrain data assembly. It is important to identify the issues with the data so that the user can understand and fixes the problems. As an encouraging note, many governmental institutions, including the USGS and the EPA, are working to develop seamless terrain information and streams and watersheds information, which will ease the data assembly efforts.

4.4.1 TERRAIN PREPROCESSING

This chapter will discuss terrain pre-processing features and functionality HMS model setup, and related utilities.

A terrain model is used as an input to drive eight additional data sets that collectively describe the drainage patterns of the watershed and allows for stream and sub-basin delineation. The first five data sets in grid representation are the flow direction, flow accumulation, stream definition, stream segmentation, and watershed delineation. The next two data sets are the vectorized representation of the watersheds and streams, and they are the watershed polygons and the stream segments. The last data set, the aggregated watersheds, is used primarily to improve the performance in watershed delineation. The objectives of terrain pre-processing are:

- Terrain is processed and analyzed using the 8-pour point approach to determine flow paths. Terrain analysis is computer intensive and some steps may require several hours, depending on the amount of data and computer resources.
- After terrain pre-processing is completed, the resulting data sets serve as a spatial database for the study. With the information centralized in the spatial database, pertinent data sets can be extracted for subsequent work on building the hydrologic models.
- Preliminary watershed and stream delineation provides results that can be verified with published information to detect possible errors in the terrain model. If errors are detected in the terrain model, the DEM should be edited outside of GeoHMS. When the DEM has been revised to better represent field conditions, it should be processed again to update the spatial database.

(i) **Depressionless DEM:** The depressionless DEM is created by filling the depressions or pits by increasing the elevation of the pit cells to the level of the surrounding terrain in order to determine flow directions. The pits are often considered as errors in the DEM due to re-sampling and interpolating the grid.

For example, in a group of three-by-three cells, if the center cell has the lowest elevation compared to its eight neighboring cells, then the center cell's elevation will be increased equaling the next lowest cell. Filling the depressions allows water to flow across the landscape. The assumption is generally valid when a large event storm fills up the small depressions and any incremental amount of water that flows into the depression will displace the same amount of water from the depression (see Fig. 4.1).

(ii) **Flow Direction:** This step defines the direction of the steepest descent for each terrain cell. Similar to a compass, the eight-point pour algorithm specifies the following eight possible directions :

1 = east, 2 = southeast,

4 = south, 8 = southwest,

16 = west, 32 = northwest,

64 = north, 128 = northeast

The results of the Flow Direction operation is the "FdirGrid" is shown in Figure 4.2

(iii) **Flow Accumulation:** This step determines the number of upstream cells draining to a given cell. Upstream drainage area at a given cell can be calculated by multiply the flow accumulation value by the cell area. The result of the Flow Accumulation operation is the "faceGrid", as shown in Figure 4.3

(iv) **Stream Definition:** This step classifies all cells with flow accumulation greater than the user-defined threshold as cells belonging to the stream network. Typically, cells with high flow accumulation, greater than a user-defined threshold value, are considered part of a stream network. The user-specified threshold may be specified as an area in distance units squared, e.g. square miles, or as a number of cells. The flow accumulation for a particular cell must exceed the user-defined threshold for a stream to be initiated. The default is one percent (1%) of the largest drainage area in the entire basin. The smaller the threshold chosen, the greater the number of sub-basins delineated by Geo-HMS. The result of the Stream Definition operation is the "strgrid" as shown in Figure 4.4

(v) **Stream Segmentation:** This step divides the stream into segments. Stream segments or links are the sections of a stream that connect two successive junctions, a junction and an outlet, or a junction and the drainage divide.

The stream segmentation operation results in 14 stream segments as shown in the "strlnkgrid" theme in Figure 4.5

(vi) **Watershed Delineation**: This step delineates a sub-basin or watershed for every stream segment. The watershed delineation operation results in 14 sub-basins as shown in the "wshedgrid" theme in Figure 4.6

(vii) **Stream Segment processing.** This step converts streams in the grid representation into a vector representation. The stream processing operation vectorized the grid-based streams into line vectors as shown in the "River-shp" theme in Figure 4.7

(viii) **Watershed Aggregation.**: This step aggregates the upstream sub-basins at every stream confluence. This is a required step and is performed to improve computational performance for interactively delineating sub-basins and to enhance data extraction. The watershed aggregation operation results are shown in the "wshedgrid23" theme in Figure 4.7

(ix) **Stream and Watershed Characteristics:** HEC-GeoHMS computes several topographic characteristics of streams and watersheds. These characteristics are useful for comparing of basins and for estimating hydrologic parameters. The user should compare and verify the physical characteristics with published information prior to estimating the hydrologic parameters. The stream and watershed physical characteristics are stored in attribute tables, which can be exported for use with a spreadsheet and other programs. When more experience is gained from working with GIS data, initial estimates of hydrologic parameters will be provided in addition to the physical characteristics.

The next operations will discuss the tools for extracting topographic characteristics of the watershed and river.

(x) **River Length:** This step computes the river length for all sub-bains and routing reaches in the "River.shp" file as shown in Figure 4.9.

(xi) **River Slope:** This step extracts the upstream and downstream elevation of a river reach and computes the slope.

(xii) **Basin Centroid:** The basin centroid location can be estimated in four ways. The engineering approach to locating the centroid with momentum calculations around the X-and Y-axis is not implemented here because the centroid may be outside of U-shaped and other odd-shaped sub-basins. The bounding box method has been used for estimating the basin centroid. The bounding box method encompasses a sub-basin with a rectangular box and approximates the centroid as the box center. This method works really fast. The result of the operation is a point shapefile, "WshCentroid.Shp", showing the basin centroids as shown in Figure 4.11

(xiii) **Longest Flow Path:** The Longest Flow Path operation computes a number of basin physical characteristics: the longest flow length, upstream elevation, downstream elevation, slope between the endpoints, and slope between 10% and 85% of the longest flow length.. These characteristics are stored in the "WaterShd.shp" theme. The result of the longest flow path operation is shown in Figure 4.10

(xiv) **Centroidal Flow Path:** This operation computes the centroidal flow path length by projecting the centroid onto the longest flow path. The centroidal flow path is measured from the projected point on the longest flow path to the sub-basin outlet as shown in Figure 4.12.

CHAPTER – 5

HYDROLOGIC ANALYSIS

5.1 SELECTION OF A HYDROLOGIC MODEL

There are numerous criteria which can be used for choosing a hydrologic model. These criteria are always project-dependent, since every project has its own specific requirements and needs. Furthermore, some criteria are also user-depended, and therefore subjective (such as GUI or OS preference). Among the various project-depended selection criteria, there are four fundamental ones that must always be answered: 1) required model outputs important to the project and therefore to be estimated by the model; 2) hydrologic processes that need to be modeled to estimate the desired outputs adequately; 3) availability of input data; and 4) price.

The US Army Corps of Engineers (USACE) Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS) was selected for the project. The current version of the HEC-HMS model is a highly flexible package (7 infiltration methods, 6 stream flow routing, 3 base flow and 3 reservoir routing methods). The model uses HEC-DSS data format, which is the format used in the study area by the Upper Thames River Conservation Authority (UTRCA). Its modular structure allows taking advantage of other HEC products, such as HEC-ResSim for regulated reservoir simulation (USACE, 2000a). The model has been applied successfully in numerous studies (see e.g. Yu et al., 1999, Yu et al., 2002, Moges et al., 2003, Fleming and Neary, 2004).

5.1.1. HEC-HMS MODEL

The HEC-HMS hydrologic model was used for modeling both individual (single) rainfall events as well as long, continuous sequences of precipitation data. The Hydrologic Modeling System, HEC-HMS was developed to simulate the rainfall runoff processes in watershed systems that have multiple branches. The HEC-HMS software can be applied in solving a large range of problems such as large river basin water supply, flood hydrology, urban or natural watershed runoff, water availability, urban drainage, flow forecasting, future urbanization impact, reservoir spillway design, flood damage reduction, floodplain regulation, and systems operation. The HEC-HMS uses algorithms used in HEC-1 (HEC, 1998), HEC-1F (HEC, 1989), PRECIP (HEC, 1989), and HEC-IFH (HEC, 1992) in conjunction with new algorithms to form a comprehensive library of simulation routines.

When a mathematical model is used to optimize rainfall-runoff loss-rate parameters from observed rainfall-runoff data, it is important that the observed hydrograph and the hydrograph generated by using the optimization trial are as identical as possible. An objective function is a mathematical tool to measure the goodness of fit between the observed and generated hydrographs. The objective functions available in the HEC-HMS software are peak weighted root mean square, percentage of error in peak flow, percentage of error in volume, sum of absolute residuals, sum of squared residuals and time weighted errors. The HEC-HMS software contains two search algorithms, namely the univariate method and the Nelder and Mead (1965) method to find the lowest objective function value and optimum parameter values. The univariate gradient method computes and adjusts one parameter at a time while locking the other parameters. Alternatively, the Nelder and Mead method evaluates all parameters simultaneously and determines which parameter to adjust. The search algorithms are also known as optimization methods.

The search algorithms used to obtain the minimum value for an objective function can sometimes delude the modeler by providing a set of solution parameter values, but the objective function value may not be the least possible value. A solution set with a lesser objective function value could be available in the solution space. A global minimal solution may be defined as the solution with the lowest objective function value in the solution space while a local minimal solution may be defined as a solution with objective function values lower than those in the surrounding space. A local minimal solution can possibly occur if the seed values are in the close vicinity of the local minimum solution, or if the slope toward the local minimum is larger than that pointing toward the global minimum.

The Clark unit hydrograph (Clark, 1945) was used for modeling direct runoff. In the Clark method, overland flow translation is based on a synthetic time–area histogram and the time of concentration. The movement of water in aquifer was modeled by the base flow component. The event model used simpler exponential recession model and the continuous model a linear reservoir base flow model. Both overland flow and base flow enter river channels. The translation and attenuation of stream flow in river channels was simulated by the modified Puls method. This method can simulate backwater effects, can take into account floodplain storage, and can be applied to a broad range of channel slopes. Finally, the effect of hydraulic facilities (reservoirs, detention basins) and natural depressions (lakes, ponds, wetlands) was reproduced by the reservoir component of the model.

5.2 SYNTHETIC UNIT HYDROGRAPH METHOD

Synthetic unit-hydrograph methods are utilized to describe the entire unit hydrograph for a gauged watershed with only a few hydrograph parameters. Needed hydrograph parameters vary among the different synthetic unit-hydrograph methods. These hydrograph parameters can be related to the characteristics of the watersheds and storms from which the parameters were determined. This method can be applied to ungauged watersheds with geomorphology, soils, land cover/land use, and climate similar to the gauged watersheds. Many synthetic unit hydrograph methods have been proposed in the hydrologic literature. In this thesis, only the Clark (1945) unit hydrograph method is considered because this method commonly is applied for hydrologic design and analysis.

5.2.1 CLARK UNIT HYDROGRAPH METHOD

The processes of translation and attenuation dominate the movement of flow through a watershed. Translation is the movement of flow down gradient through the watershed in response to gravity. Attenuation results from the frictional forces and channel storage effects that resist the flow. Clark (1945) noted that the translation of flow throughout the watershed could be described by a time-area curve, which expresses the curve of the fraction of watershed area contributing runoff to the watershed outlet as a function of time since the start of effective precipitation. Effective precipitation is that precipitation that is neither retained on the land surface nor infiltrated into the soil (Chow and others 1988, p. 135). The time-area curve is bounded in time by the watershed Tc. Thus, Tc is a hydrograph parameter of the Clark unit-hydrograph method. Attenuation of flow can be represented with a simple, linear reservoir for which storage is related to outflow as

$$S = RO \tag{5.1}$$

Where S is the watershed storage, R is the watershed-storage coefficient, and O is the outflow from the watershed. The value of R can be estimated by considering the point of inflection of a surface runoff hydrograph. At this point the inflow into the channel has ceased and beyond this point the flow is entirely due to withdrawal from the channel storage. The continuity equation (1) can be rewritten as follows.

$$I-Q=\frac{ds}{dt} \qquad (5.2)$$

$$-Q=\frac{ds}{dt}=K\frac{dQ}{dt} \qquad (5.3)$$

Hence, $K=-Q_i/(dQ/dt)_i$ (5.4)

Where, i refers to the point of the inflection, and R can be estimated from a known surface runoff hydrograph of the catchment. The constant R can be estimated from the data on the recession limb of a hydrograph. Knowing R of the linear reservoir, the inflows at various times are routed by the Muskingum method (Chow et al., 1988), which is stated as follows.

The inflow rate between an inter- isochrone area A_r km^2 with a time interval Δt_c (hr) can be sated as follows.

$$I=\frac{A_r\times 10^4}{3600\,\Delta t_c}\quad (\mathrm{m^3/s}) \qquad (5.5)$$

The following equations are used for routing the translation hydrograph through the linear reservoir with storage coefficient value (R) hours to calculate the IUH ordinates.

$$Q_n=C_0I_n+C_1I_{n-1}+C_2Q_{n-1} \qquad (5.6)$$

$$C_0=\frac{-Kx+0.5\Delta t}{K-Kx+0.5\Delta t} \qquad (5.7)$$

$$C_1 = \frac{Kx + 0.5\Delta t}{K - Kx + 0.5\Delta t} \tag{5.8}$$

$$C_2 = \frac{K - Kx - 0.5\Delta t}{K - Kx + 0.5\Delta t} \tag{5.9}$$

Where, $C_0 + C_1 + C_2 = 1.0$ and x is a weighting factor having the range $0 \le x \le 0.5$. The value of x depends on the shape of the modeled wedge storage. The value of x ranges from zero for reservoir- type storage to 0.5 for a full wedge. When the value is $x = 0$, there is no wedge hence no backwater ,i.e. level-pool reservoir. In natural stream the values of x varies from 0 to 0.3.

$$Q_n = C_0 I_n + C_1 I_{n-1} + C_2 Q_{n-1} \tag{5.10}$$

$$C_0 = \frac{0.5\Delta t}{K + 0.5\Delta t} \tag{5.11}$$

$$C_1 = \frac{0.5\Delta t}{K + 0.5\Delta t} \tag{5.12}$$

$$C_2 = \frac{K - 0.5\Delta t}{K + 0.5\Delta t} \tag{5.13}$$

Where, $C_0 + C_1 + C_2 = 1.0$.

Therefore, Clark (1945) proposed that a synthetic unit hydrograph could be obtained by routing 1 in. of direct runoff to the channel in proportion to the time-area curve and routing the runoff entering the channel through a linear reservoir. Numerous researchers have found that determining the time-area curve for the watershed was not needed to obtain a reasonable unit hydrograph. For example, Turner and Burdoin (1941) and O'Kelly (1955) found that reasonable unit hydrographs were obtained when simple geometric shapes were substituted for the actual time-area curve. Experience with the Clark unit-hydrograph method at the U.S. Army Corps of

Engineers, Hydrologic Engineering Center, indicates that a detailed time-area curve usually is not necessary for accurate synthetic unit-hydrograph estimation (Ford and others, 1980). In most instances, the dimensionless time-area curve included in HEC-1 (U.S. Army Corps of Engineers, 1990) is satisfactory for obtaining a reliable synthetic unit hydrograph. In Chambal river basin, HEC-HMS model typically is utilized to compute the Clark unit hydrograph. *Tc* and *K* are the hydrograph parameters required for HEC-HMS computation of the Clark unit hydrograph. The *Tc*for the Clark unit hydrograph is slightly different than the typical definition applied in storm water management, such as that in the Rational method (Kuichling, 1889). In the typical definition, the time of concentration (*Tc*) is the travel time for the first drop of effective precipitation at the hydraulically most distant point in the watershed to reach the watershed outlet. In the Clark unit-hydrograph method, *Tc* is the time from the end of effective precipitation to the inflection point of the recession limb of the runoff hydrograph. The inflection point on the runoff hydrograph corresponds to the time when overland flow to the channel network ceases and beyond that time the measured runoff results from drainage of channel storage. Therefore, Clark's *Tc* is the travel time required for the last drop of effective precipitation at the hydraulically most distant point in the watershed to reach the channel network. From a linear-system theory and the conceptual model of pure translatory flow, the two definitions of time of concentration are equivalent. The subtle differences, however, between the definition of time of concentration in the Rational method and in the Clark unit-hydrograph method imply the time of concentration estimation equations commonly applied in the Rational method may not be appropriate for application to the Clark unit-hydrograph method. In most applications of HEC-HMS, *Tc* is determined from values calibrated with measured rainfall and runoff data either directly, by scaling from hydrologically similar watersheds, or from equations, such as those developed in this study.

5.2.2 IMPLEMENTATION OF CLARK MODEL ON CHAMBAL BASIN USING HEC-HMS

The present study is aimed at simulating the direct runoff hydrograph proposed by Clark (1945) for the Chambal basin using a lumped parameter model for gauged sub basin. The proposed model is to be developed keeping in view the availability of the following data;

(i) Short term data is available at different points in the catchment.

(ii) Corresponding runoff data available at the couple of sections on major tributing network.

(iii) Topographic and physiographic details of the catchment.

Further the distributed nature of the proposed model has been attempted by splitting the catchment into different sub-basin. Also distributed input are taken care of by dividing these sub-basin into sub-areas. The sub-area's are arrived at keeping in view the meteorological homoginity.These sub-basins have been marked on the catchment topographic sheet given in Fig. No. 3.1 and are detailed as below:

(i) Pat sub-basin

(ii) Mahidpur sub-basin

(iii) Nagda sub-basin

(iv) Mandsour sub-basin

(v) Chaldu sub-basin

Above sub-basin have been demarketed keeping in view the drainage characteristic of the catchment. Thus these are representing te surface drainage of all the major tributaries in te catchment. The rest of the catchment i.e. in between the gauged of the sub-basin and the Gandhi Sagar reservoir has further been sub-divided, and these divisions are termed as Intermediate Sub-basins (ISB). The following is the list of ISB adopted in the study and indicated in Fig. No. 3.1

(i) Choumahla ISB

(ii) Kalakhedi ISB

(iii) Nahargarh ISB

5.2.2.1 DETERMINATION OF CLARK UNIT HYDROGRAPH PARAMETER

For computation of flood event using Clark model estimates for the parameter *Tc* and *R* are required. *Tc* is a parameter which denotes time of concentration and *R* denotes the catchment storage characteristics. Several studies are reported in past wherein value of *Tc* and *R* where estimated using hydrograph characteristics. For the present study initial estimates for *Tc* and *R* where made using the method Graf *et.al*,(1982b). Mathematically the equation given by Graf is given as

$$Tc = 1.54\, L^{0.875}\, S^{-0.181} \quad (5.14)$$

$$R = 16.4\, L^{0.342}\, S^{-0.790} \quad (5.15)$$

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

Observed value for single storm for all eight sub-basin was available and using this observed storm value of *Tc* and *R* is refined using optimization in HEC-HMS model. The computed value of *Tc* and *R* which produced minimum value of sum of the square difference between the observed and computed hydrograph ordinate are given in table 5.1

Sl.No.	Sub-basin	Tc(hours)	R(hours)
1.	Choumahla	26	1.9
2.	Pat	40	2.44
3.	Mahidpur	38	2.6
4.	Kalakhedi	15	1.8
5.	Nagda	29	1.9
6.	Mandsour	19	1.8
7.	Chaldu	29.51	2.1
8.	Nahargarh	20	1.6

Table 5.1 Optimized value of *Tc* and *R*

5.2.3 EVALUATION OF MODEL RESULTS

The result obtained for different sub-watersheds are evaluated using visual comparision and statistical measures of Nash-Sutcliff (Nash & Sutcliff 1970). The Nash-Sutcliff efficiency is given by

$$NSE = 1 - \{\sum (\hat{y}_i - y_i)^2 / \sum(y_i - \bar{y})^2\} \qquad (5.16)$$

Where, $\hat{y}_i$ is the computed runoff, y_i is the observed runoff and $\bar{y}$ is the mean of the observed runoff. Nash (NSE) ranges from $-\infty$ to 1 with higher values indicating better agreement. As per NSE criteria simulation results are considered to be very good for the value of NSE > 0.75, where as for values of NSE between 0.75 and 0.36, simulation results are considering as satisfactory (Motovilov et al, 1999). If the value of NSE is negative the model prediction is worst than the mean observation. In other words a negative value for NSE indicates that the average measured value give a better estimate than the simulation value. Simulated results are further evaluated based on percent error in peak and time to peak discharge.

For visual comparision the plot between observed and computed runoff are presented in Figure (5.1-5.8). As can be seen from these figures the model could simulate peak and time to peak discharge as well as overall shape of the flood hydrograph regionably well. Computed value for statistical evaluation criteria such as NSE, percent error in peak and time to peak discharges between observed and computed values are given in Table (5.2). As can be seen from Table (5.2), percent error between observed and computed peak discharge is within ± 14%. Also it can be seen from Table 5.2 that the NSE for 50% of events are analysed is more than 0.81 indicating very good match and for rest of the events it is between 0.37 and 0.81 indicating satisfactory match(Motovilov *et.al,* 1999).

5.2.4 DEVELOPMENT OF RELATIONSHIP BETWEEN CLARK MODEL PARAMETER AND CATCHMENT CHARACTERISTICS

To establish relation between dimensionless ratio of Tc/R and catchment characteristics, values for various sub-watershed parameters such as length of main channel, drainage area of watershed and slope of main channel are plotted. Easily measurable catchment characteristic from topographic map such as length of main channel and catchment area for all eight sub-watersheds are calculated using HEC-geoHMS. Calculated values for these parameters are given in Table 5.3 for all eight sub-basin of study area. Further efforts are made to establish relationship between Tc/R and different ratios for measured catchment characteristics such as $L/\sqrt{S}$, $L\sqrt{A}/\sqrt{S}$, L^2/A and also relationship between R/(Tc+R) and L^2/A is attempted. Plot between these four combinations are given in Fig. No. (5.9 - 5.12). Developed relations for Chambal basin for estimation of Clark parameters are as under:

$Tc/R = 3.909L^2/A + 6.107$ $\quad (R^2 = 0.98) \quad$ (5.17)

$Tc/R = 0.0085L\sqrt{A}/\sqrt{S} + 3.196$ $\quad (R^2 = 0.98) \quad$ (5.18)

$Tc/R = 0.370L/\sqrt{S} + 3.684$ $\quad (R^2 = 0.99) \quad$ (5.19)

$R/(Tc+R) = 0.023L^2/A - 0.029$ $\quad (R^2 = 0.97) \quad$ (5.20)

As can be seen from developed relation the R^2 values ranges from 0.97 to 0.99 indicating a very good fit and the developed relation can be used to estimate Clark model parameter for Chambal sub-basin.

CHAPTER – 6

SUMMARY OF CONCLUSION

The present study is aimed at simulating the direct runoff hydrograph for the Chambal river basin at Gandhi Sagar site, using Clark model and to retated model parameters with easily measurable topographical characteristics of the sub-watershed. In a hydrologic system the catchment response can be simulated better when the input is considered as distributed. In the proposed model the same has been achieved by splitting the vast catchment, (having the drainage area of about 22920 sq. km), into its different sub-basins and intermediate sub-basins keeping in view the drainage character of the basin. Further, the non-uniformity of rainfall is taken care of by dividing them into sub-areas. Using HEC-geo HMS for all discritize sub-watersheds input network file for used in HEC-HMS model are generated in HEC-geoHMS interface. Pertinent topographic attributes are calculated in HEC-geoHMS. For transforming excess rainfall into direct surface runoff for all eight discritize sub-watersheds, Clark transformation method is selected. Initial estimates for parameter Tc and R is made using method of Graf et al, (1982b). These initial estimates for Tc and R further refined by optimization using of single observed storm event available for these sub- watershed. Relation between dimensionless ratio of Tc/R and four different ratios for measured catchment characteristics are also established.

Based on the study following conclusion are made:

(i) The GIS based HEC-geoHMS interface is found very effective for discritization of large basin into smaller sub-watershed and generation of input information for use in HEC-HMS model.

(ii) Transformation method of Clark (1945) is found to simulate peak and time to peak hydrograph reasonably well.

(iii) Developed relation between dimensionless ratio of Tc/R and ratios for measurable catchment characteristics can be used with confidence for computation of Clark model parameter for Chambal sub-basin as developed relations exhibit R^2 values ranging from 0.97 to 0.99

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Attributes of Watershd.shp

Shape	Id	Gridcode	Area	Wshid	Perimeter	Elevation	Slp_endpt	Slp_1085	Longestfl	Centroidal
Polygon	1	7	64124993.47	7	54538.84905	395.0000	0.003	0.003	16550.151	6779.188
Polygon	2	4	1442622011.	4	284933.9869	390.0000	0.002	0.001	67308.170	25806.066
Polygon	3	8	2354189936.	8	349732.6195	433.0000	0.001	0.001	99024.969	34473.662
Polygon	5	10	675066007.5	10	186656.0609	431.0000	0.002	0.002	49641.407	21963.926
Polygon	7	11	144335908.0	11	85318.19950	412.0000	0.003	0.003	21463.865	10023.803
Polygon	8	12	578882567.1	12	153896.7522	477.0000	0.002	0.002	47762.044	13275.590
Polygon	9	9	2363860928.	9	399951.5597	473.0000	0.001	0.001	129709.665	65636.582
Polygon	10	13	1298836880.	13	254694.6251	456.0000	0.001	0.001	89858.409	42196.506
Polygon	11	15	1036262162.	15	256854.5795	468.0000	0.001	0.001	92833.190	43902.884
Polygon	12	18	2754782829.	18	431090.9025	506.0000	0.001	0.001	143940.151	77045.147
Polygon	14	14	1930027039.	14	322913.1854	478.0000	0.001	0.001	88563.352	40226.690
Polygon	15	16	3132615681.	16	517849.0717	479.0000	0.001	0.001	165384.828	74217.927
Polygon	17	20	1458902324.	20	338032.8664	519.0000	0.001	0.001	119412.603	64821.301
Polygon	18	21	1992451104.	21	406971.4115	508.0000	0.002	0.001	146358.859	60711.173
Polygon	19	17	1581142364.	17	415791.2254	506.0000	0.002	0.001	145943.498	74403.222

TABLE No. – 4.1 PHYSIOGRAPHIC PARAMETERS OF CHAMBAL CATCHMENT

TABLE NO. - 5.2

MODEL RESULTS

Name of Subbasin	Peak Discharge (cumec)			Time to Peak (hrs)			Nash Sutcliffe Efficiency
	Observed	Computed	% Error	Observed	Computed	% Error	
Pat	2648	2368	10.57	24	24	0	93.78%
Mandsour	1813	1937	-6.83	14	18	-28.57	92.45%
Nagda	4207	4237	-0.72	20	20	0	91.28%
Choumahla	4030	3486	13.49	44	44	0	81.59%
Mahidpur	5878	5807	1.2	52	44	15.38	60.42%
Chaldu	623	622	0.12	20	24	-20	42.24%
Kalakhedi	1649	1559	5.44	40	40	0	36.87%
Nahargarh	5230	5103	2.41	28	24	14.28	40.52%

TABLE NO. - 5.3

CLARK MODEL PARAMETERS AND CHACHMENT CHARACTERISTICS

Station	A Sq.km	L Km	S	Tc Hr	R Hr	Tc/R	L2 Sq.km	L2/A	R/(Tc+R)	L*√ A Sq.km	√S	L√A/√ S Sq.km	L/ √S km
Pat	1432	74	2.89	40	2.44	16.39	5476	3.82	0.05	2800.29	1.70	1647.23	43.52
Choumahla	1349	51	3.84	26	1.9	13.68	2601	1.93	0.06	1873.16	1.95	955.89	26.02
Mahidpur	4421	122	12.08	38	2.6	14.62	14884	3.37	0.06	8111.85	3.47	2333.92	35.10
Nagda	3825	115	11.27	29	1.9	15.26	13225	3.46	0.06	7112.35	3.35	2118.61	34.25
Mandsour	1072	72	1.87	19	1.8	10.56	5184	4.84	0.08	2357.38	1.36	1723.88	52.65
Chaldu	500	38	2.62	29.51	2.1	14.05	1444	2.89	0.06	849.70	1.61	524.95	23.47
Kalakedi	1010	57	3.07	15	1.8	8.33	3249	3.22	0.10	1811.48	1.75	1033.87	32.53
Nahargarh	2700	38	2.88	20	1.6	12.5	1444	0.53	0.07	1974.53	1.69	1163.50	22.39

TABLE No.5.4

Time Series result for Kalakhedi sub-basin

(All figures in SI units)

Date	Time	Precip	Loss	Excess	Direct Flow	Base flow	Total Flow	Obs Flow
18-Aug-74	12:00				0	0	0	0
18-Aug-74	16:00	0	0	0	0	0	0	0
18-Aug-74	20:00	13.21	0	13.21	90.2	0	90.2	0
19-Aug-74	00:00	7.11	0	7.11	302.9	0	302.9	0
19-Aug-74	04:00	17.27	0	17.27	569.3	0	569.3	28
19-Aug-74	08:00	5.08	0	5.08	745.5	0	745.5	185
19-Aug-74	12:00	6.1	0	6.1	721.6	0	721.6	438
19-Aug-74	16:00	22.35	0	22.35	695.6	0	695.6	831
19-Aug-74	20:00	34.54	0	34.54	968.4	0	968.4	1303
20-Aug-74	00:00	9.14	0	9.14	1378.7	0	1378.7	1530
20-Aug-74	04:00	26.42	0	26.42	1559.2	0	1559.2	1649
20-Aug-74	08:00	15.24	0	15.24	1475.8	0	1475.8	1536
20-Aug-74	12:00				1220.1	0	1220.1	800
20-Aug-74	16:00				821	0	821	650
20-Aug-74	20:00				358.1	0	358.1	567
21-Aug-74	00:00				67.6	0	67.6	547
21-Aug-74	04:00				0	0	0	544
21-Aug-74	08:00				0	0	0	542
21-Aug-74	12:00				0	0	0	536
21-Aug-74	16:00				0	0	0	524
21-Aug-74	20:00				0	0	0	467
22-Aug-74	00:00				0	0	0	432
22-Aug-74	04:00				0	0	0	401
22-Aug-74	08:00				0	0	0	284
22-Aug-74	12:00				0	0	0	141
22-Aug-74	16:00				0	0	0	0

TABLE No. -5.5
Time Series result for Choumahla sub-basin
(All Figures in SI units)

Date	Time	Precip	Loss	Excess	Direct Flow	Base flow	Total Flow	Obs Flow
18-Aug-74	16:00				0	0	0	0
18-Aug-74	20:00	27.43	0	27.43	109.7	0	109.7	7
19-Aug-74	00:00	19.3	0	19.3	387.3	0	387.3	14
19-Aug-74	04:00	36.58	0	36.58	824.6	0	824.6	113
19-Aug-74	08:00	6.1	0	6.1	1303	0	1303	269
19-Aug-74	12:00	24.38	0	24.38	1674.5	0	1674.5	666
19-Aug-74	16:00	21.34	0	21.34	1941.2	0	1941.2	1161
19-Aug-74	20:00	32.51	0	32.51	2064.6	0	2064.6	1671
20-Aug-74	00:00	78.23	0	78.23	2340.3	0	2340.3	2053
20-Aug-74	04:00	20.32	0	20.32	2772.3	0	2772.3	2464
20-Aug-74	08:00	37.59	0	37.59	3180.6	0	3180.6	3200
20-Aug-74	12:00	18.29	0	18.29	3486.2	0	3486.2	4030
20-Aug-74	16:00				3362.2	0	3362.2	3682
20-Aug-74	20:00				2823.6	0	2823.6	2407
21-Aug-74	00:00				1977.6	0	1977.6	1685
21-Aug-74	04:00				1143.1	0	1143.1	694
21-Aug-74	08:00				567.9	0	567.9	120
21-Aug-74	12:00				187.4	0	187.4	70
21-Aug-74	16:00				25.8	0	25.8	50
21-Aug-74	20:00				0	0	0	0

TABLE No. -5.6
Time Series Result for Mahidpur sub-basin
(All Figures in SI units)

Date	Time	Precip	Loss	Excess	Direct Flow	Base flow	Total Flow	Obs Flow
18-Aug-74	12:00				0	0	0	0
18-Aug-74	16:00	0	0	0	0	0	0	56.7
18-Aug-74	20:00	12.6	0	12.6	81.4	0	81.4	85
19-Aug-74	00:00	11.58	0	11.58	315.7	0	315.7	170.
19-Aug-74	04:00	23.77	0	23.77	748	0	748	297.5
19-Aug-74	08:00	24.79	0	24.79	1427.1	0	1427.1	566.6
19-Aug-74	12:00	26.82	0	26.82	2329.3	0	2329.3	1195.5
19-Aug-74	16:00	23.77	0	23.77	3361.3	0	3361.3	1699.8
19-Aug-74	20:00	12.6	0	12.6	4341.5	0	4341.5	2436.4
20-Aug-74	00:00	11.58	0	11.58	5114.6	0	5114.6	3054
20-Aug-74	04:00	11.58	0	11.58	5613.4	0	5613.4	3790.6
20-Aug-74	08:00	16.66	0	16.66	5807.4	0	5807.4	4980.4
20-Aug-74	12:00				5636.7	0	5636.7	5722.7
20-Aug-74	16:00				5074.7	0	5074.7	5878.5
20-Aug-74	20:00				4270.1	0	4270.1	5708.5
21-Aug-74	00:00				3376	0	3376	5297.7
21-Aug-74	04:00				2496.6	0	2496.6	4688.6
21-Aug-74	08:00				1738.3	0	1738.3	3867
21-Aug-74	12:00				1156.1	0	1156.1	2733.8
21-Aug-74	16:00				695.1	0	695.1	1204
21-Aug-74	20:00				308.4	0	308.4	291.8
22-Aug-74	00:00				71.8	0	71.8	150.1
22-Aug-74	04:00				0	0	0	0

TABLE No.- 5.7

Time Series result for Mandsour sub-basin

(All Figures in SI units)

Date	Time	Precip	Loss	Excess	Direct Flow	Base flow	Total Flow	Obs Flow
19-Aug-74	16:00				0	0	0	0
19-Aug-74	20:00	31.5	0	31.5	160.2	0	160.2	368.3
20-Aug-74	00:00	26.42	0	26.42	587.3	0	587.3	977.4
20-Aug-74	04:00	38.61	0	38.61	1217.7	0	1217.7	1473.2
20-Aug-74	08:00	17.27	0	17.27	1796.4	0	1796.4	1813.1
20-Aug-74	12:00	3.05	0	3.05	1937	0	1937	1756.5
20-Aug-74	16:00	5.08	0	5.08	1590.7	0	1590.7	1416.5
20-Aug-74	20:00				1014.3	0	1014.3	878.2
21-Aug-74	00:00				493.8	0	493.8	439.1
21-Aug-74	04:00				192.2	0	192.2	113.3
21-Aug-74	08:00				69.9	0	69.9	56.7
21-Aug-74	12:00				0	0	0	0

TABLE No.-5.8
Time Series Result for Nagda sub-basin
(All Figures in SI unit)

Date	Time	Precip	Loss	Excess	Direct Flow	Base flow	Total Flow	Obs Fl
19-Aug-74	04:00	0	0	0	0	0	0	0
19-Aug-74	08:00	12.7	0	12.7	122.8	0	122.8	226.
19-Aug-74	12:00	12.7	0	12.7	469.9	0	469.9	594.
19-Aug-74	16:00	16.76	0	16.76	1024.3	0	1024.3	1005.
19-Aug-74	20:00	15.75	0	15.75	1753.6	0	1753.6	1614.
20-Aug-74	00:00	12.7	0	12.7	2503.5	0	2503.5	2124.
20-Aug-74	04:00	23.88	0	23.88	3235.6	0	3235.6	2549.
20-Aug-74	08:00	16.76	0	16.76	3902.9	0	3902.9	325
20-Aug-74	12:00	9.65	0	9.65	4237.7	0	4237.7	420
20-Aug-74	16:00				4143.9	0	4143.9	3852.
20-Aug-74	20:00				3696.7	0	3696.7	3215.
21-Aug-74	00:00				2966.3	0	2966.3	2932.
21-Aug-74	04:00				2132.6	0	2132.6	2614
21-Aug-74	08:00				1265.3	0	1265.3	2266
21-Aug-74	12:00				529.6	0	529.6	906.
21-Aug-74	16:00				130.4	0	130.4	396.
21-Aug-74	20:00				0	0	0	0

TABLE No.- 5.9

Time Series Result for Nahargarh sub-basin

(All Figures in SI units)

Date	Time	Precip	Loss	Excess	Direct Flow	Base flow	Total Flow	Obs Flow
19-Aug-74	12:00				0	0	0	0
19-Aug-74	16:00	28.45	0	28.45	337.4	0	337.4	56.6
19-Aug-74	20:00	31.5	0	31.5	1327.7	0	1327.7	269
20-Aug-74	00:00	26.42	0	26.42	2745.1	0	2745.1	821.3
20-Aug-74	04:00	35.56	0	35.56	4206	0	4206	2293.9
20-Aug-74	08:00	13.21	0	13.21	5103.6	0	5103.6	5230
20-Aug-74	12:00	1.02	0	1.02	4845.4	0	4845.4	2690.4
20-Aug-74	16:00	11.18	0	11.18	3784.1	0	3784.1	2010.7
20-Aug-74	20:00		0		2568.6	0	2568.6	1416
21-Aug-74	00:00		0		1454.2	0	1454.2	934.6
21-Aug-74	04:00		0		731.1	0	731.1	665.5
21-Aug-74	08:00		0		386.9	0	386.9	523.9
21-Aug-74	12:00		0		132.5	0	132.5	410.6
21-Aug-74	16:00		0		0	0	0	283.2
21-Aug-74	20:00		0		0	0	0	198.2
22-Aug-74	00:00		0		0	0	0	127.4
22-Aug-74	04:00		0		0	0	0	42.5
22-Aug-74	08:00		0		0	0	0	0

TABLE No.- 5.10

Time Series Result for Pat sub-basin
(All Figures in SI unit)

Date	Time	Precip	Loss	Excess	Direct Flow	Base flow	Total Flow	Obs Flow
18-Aug-74	16:00				0	0	0	0
18-Aug-74	20:00				22.5	0	22.5	170
19-Aug-74	00:00	11.2	0	11.2	84.2	0	84.2	481
19-Aug-74	04:00	9.1	0	9.1	189.2	0	189.2	708
19-Aug-74	08:00	17.3	0	17.3	396	0	396	864
19-Aug-74	12:00	42.7	0	42.7	716.2	0	716.2	977
19-Aug-74	16:00	29.5	0	29.5	1090.2	0	1090.2	1161
19-Aug-74	20:00	29.5	0	29.5	1463.8	0	1463.8	1444
20-Aug-74	00:00	18.3	0	18.3	1793.1	0	1793.1	1685
20-Aug-74	04:00	14.2	0	14.2	2065.6	0	2065.6	1926
20-Aug-74	08:00	24.4	0	24.4	2279.7	0	2279.7	2237
20-Aug-74	12:00	30.5	0	30.5	2368	0	2368	2648
20-Aug-74	16:00				2284.8	0	2284.8	1897
20-Aug-74	20:00				2068.5	0	2068.5	1728
21-Aug-74	00:00				1742	0	1742	1586
21-Aug-74	04:00				1373.2	0	1373.2	1345
21-Aug-74	08:00				1031.5	0	1031.5	1020
21-Aug-74	12:00				734.5	0	734.5	666
21-Aug-74	16:00				492.3	0	492.3	85
21-Aug-74	20:00				267.4	0	267.4	57
22-Aug-74	00:00				0	0	0	0

TABLE No. 5.11

Time series result for Chaldu sub-basin

Date	Time	Precip (MM)	Loss (MM)	Excess (MM)	Direct Flow (M3/S)	Baseflow (M3/S)	Total Flow (M3/S)	Obs Flow (M3/S)
19Aug1974	16:00				0.0	0.0	0.0	0.0
19Aug1974	20:00	38.61	0.00	38.61	46.2	0.0	46.2	70.8
20Aug1974	00:00	26.42	0.00	26.42	163.3	0.0	163.3	181.3
20Aug1974	04:00	26.42	0.00	26.42	318.6	0.0	318.6	269.1
20Aug1974	08:00	9.14	0.00	9.14	477.6	0.0	477.6	425.0
20Aug1974	12:00	6.10	0.00	6.10	589.5	0.0	589.5	623.3
20Aug1974	16:00	0.00	0.00	0.00	622.5	0.0	622.5	524.1
20Aug1974	20:00	0.00	0.00	0.00	573.0	0.0	573.0	354.1
21Aug1974	00:00	0.00	0.00	0.00	437.5	0.0	437.5	198.3
21Aug1974	04:00	0.00	0.00	0.00	269.8	0.0	269.8	113.3
21Aug1974	08:00	0.00	0.00	0.00	135.7	0.0	135.7	127.5
21Aug1974	12:00	0.00	0.00	0.00	53.1	0.0	53.1	170.0
21Aug1974	16:00	0.00	0.00	0.00	15.4	0.0	15.4	226.6
21Aug1974	20:00	0.00	0.00	0.00	2.0	0.0	2.0	184.1
22Aug1974	00:00	0.00	0.00	0.00	0.0	0.0	0.0	85.0
22Aug1974	04:00	0.00	0.00	0.00	0.0	0.0	0.0	0.0

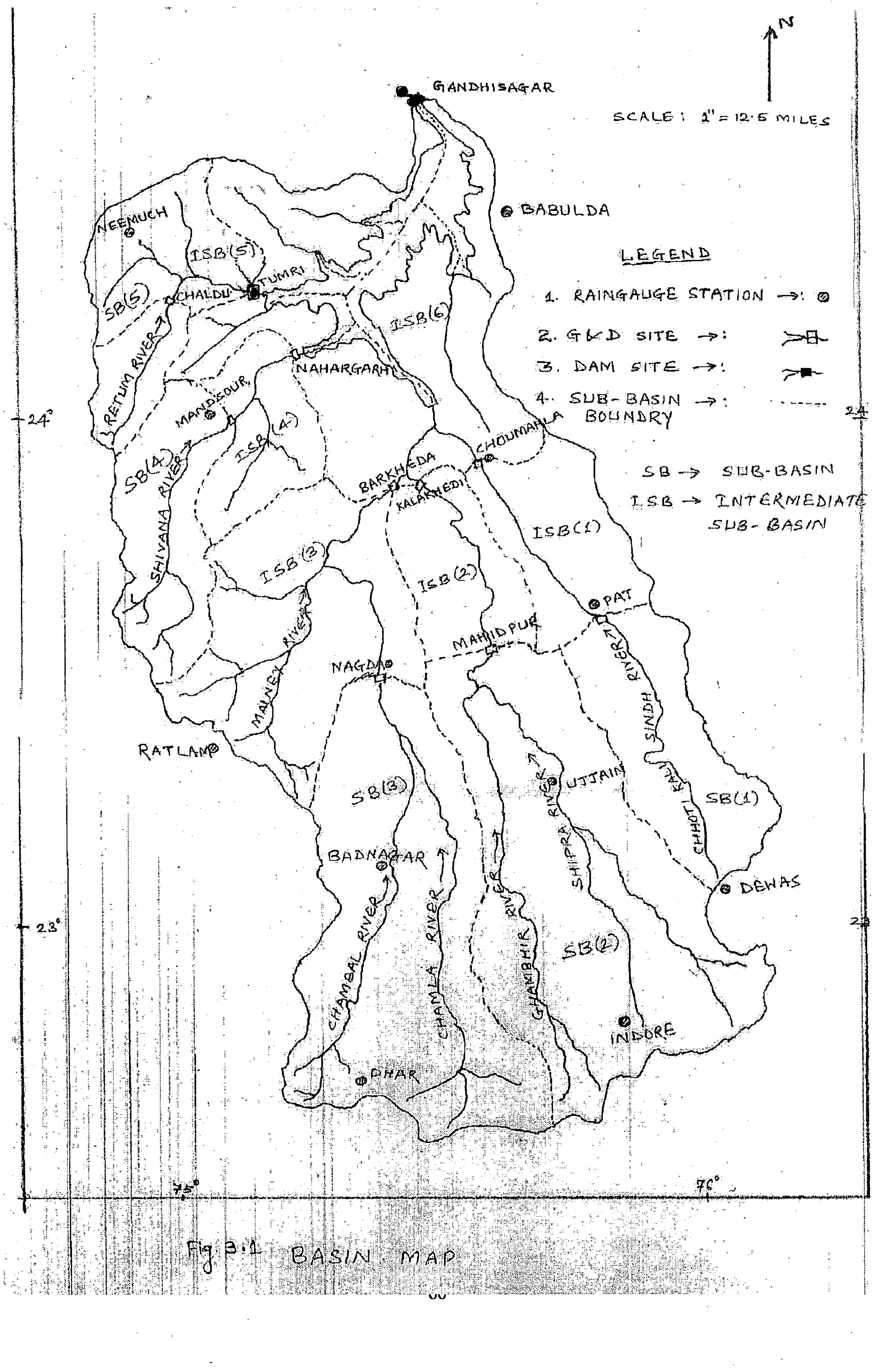

Fig 3.1 BASIN MAP

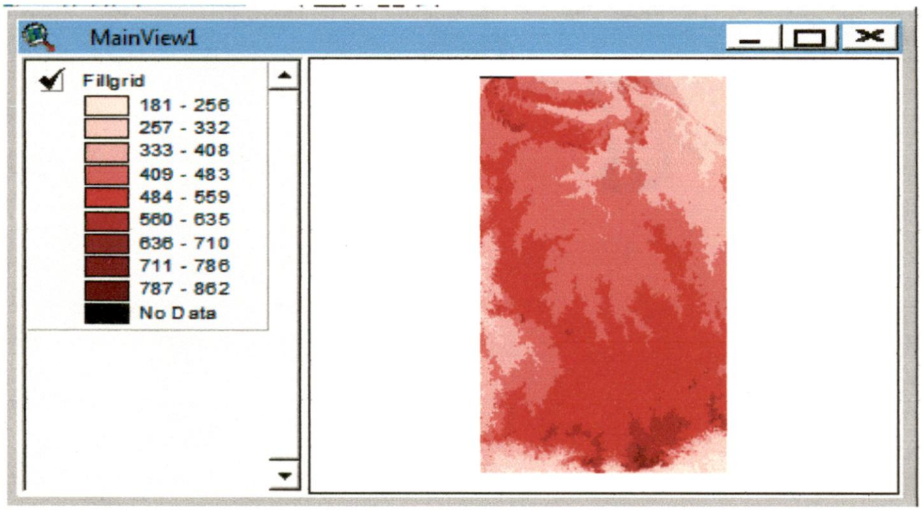

Fig. 4.1 - Raw Digital Elevation Model

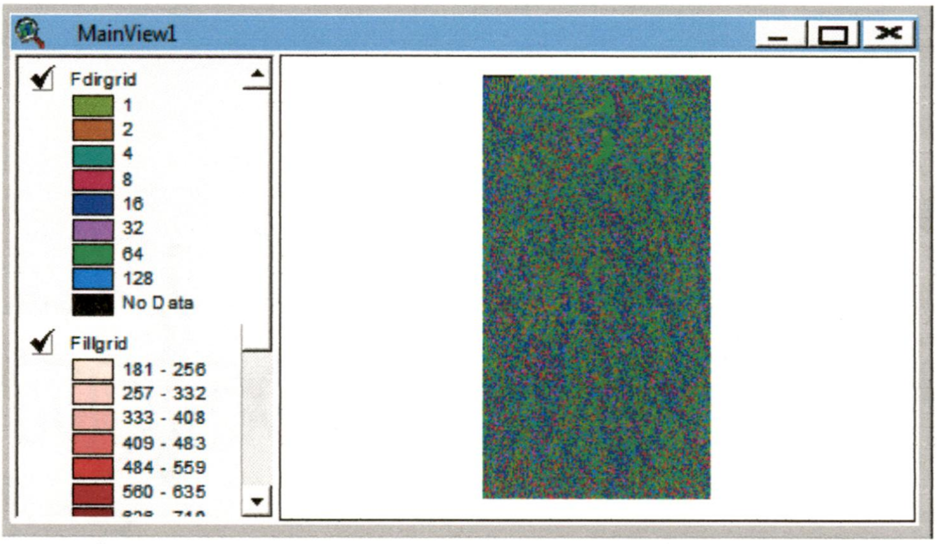

Fig. 4.2 - Fill Sink and Flow Direction Operation

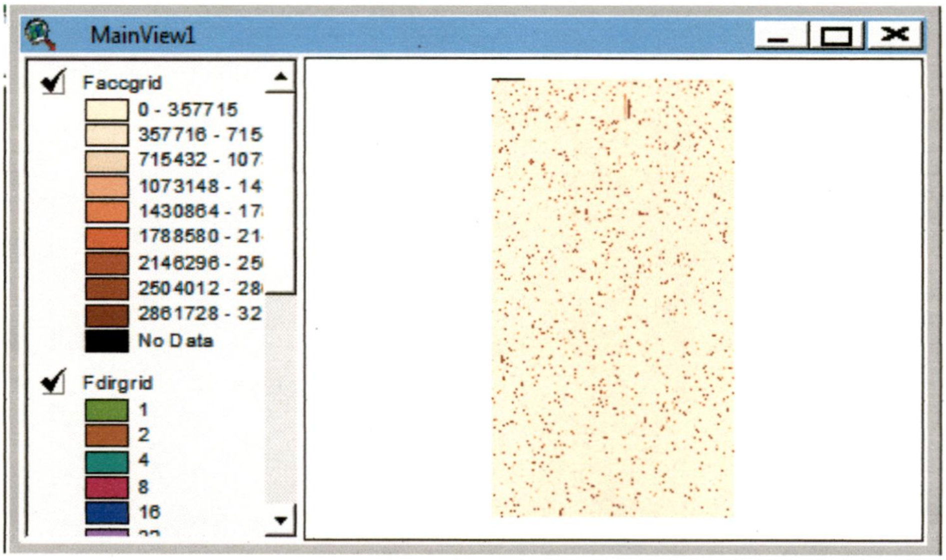

Fig. 4.3 - Flow Accumulation

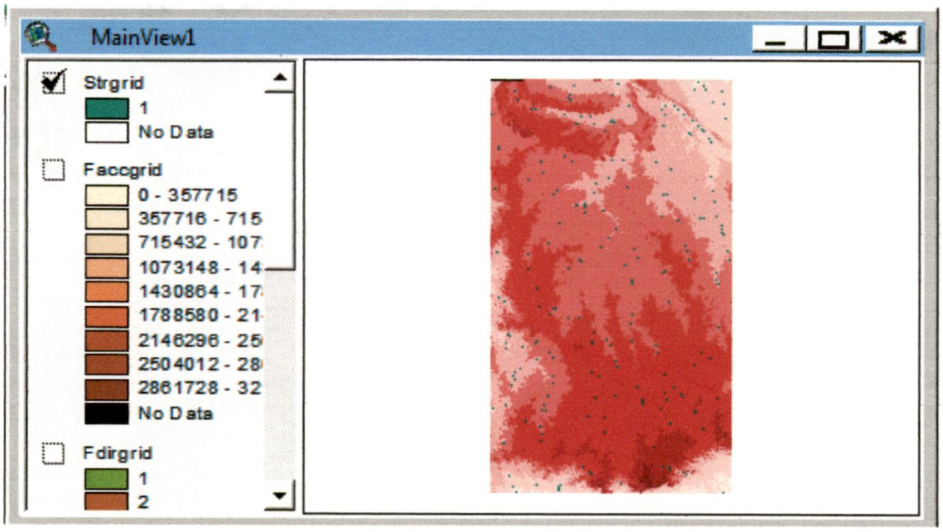

Fig.4.4- Stream Definition

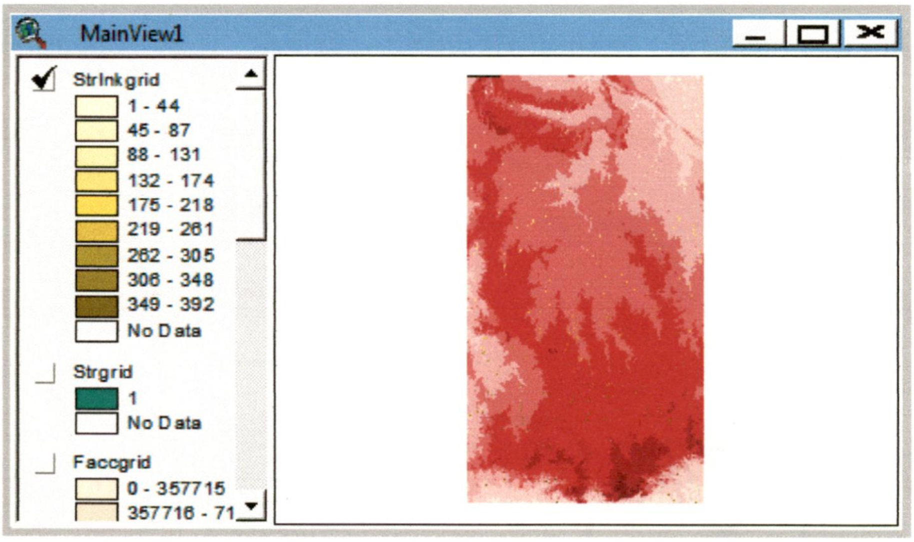

Fig. 4.5- Stream Segmentation

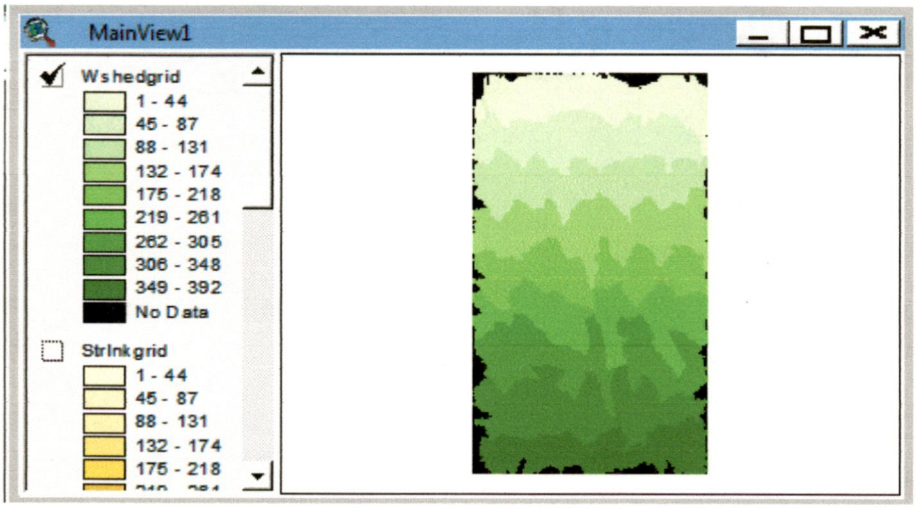

Fig.4.6 - Watershed Polygon Processing

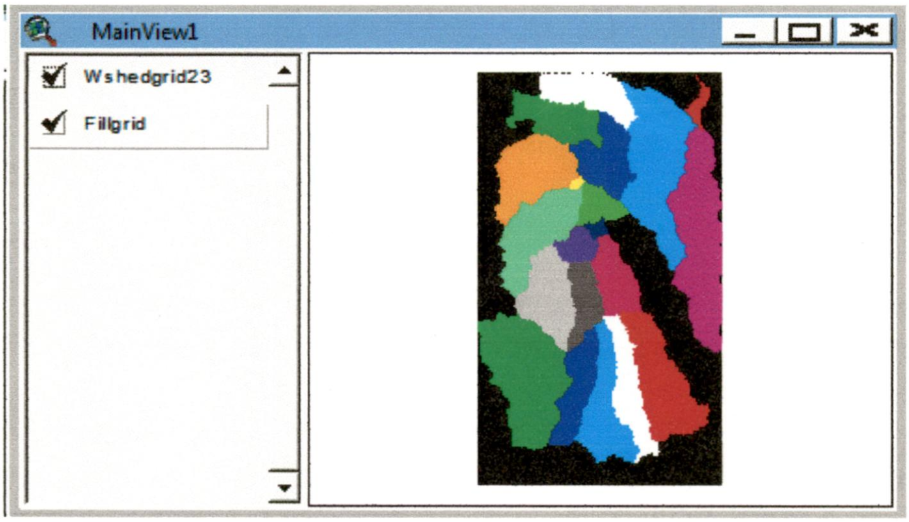

Fig 4.7 - Watershed Aggregation

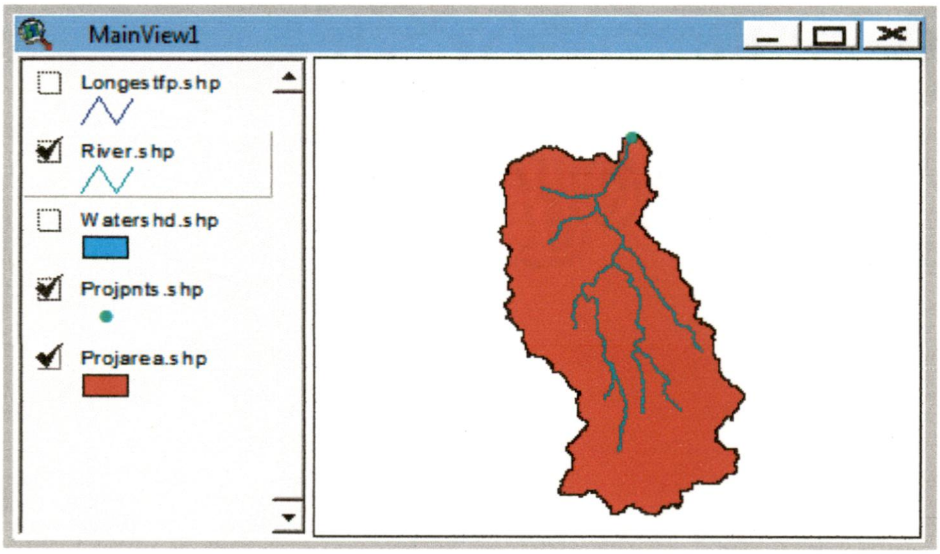

Fig. 4.8- Watershed Delineation

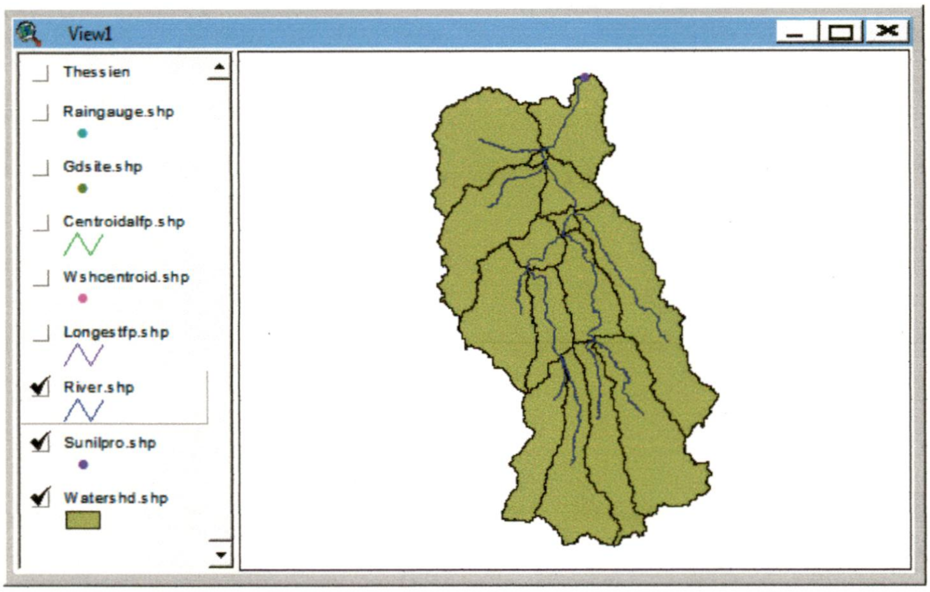

Fig. 4.9 Sub-watershed Delineation

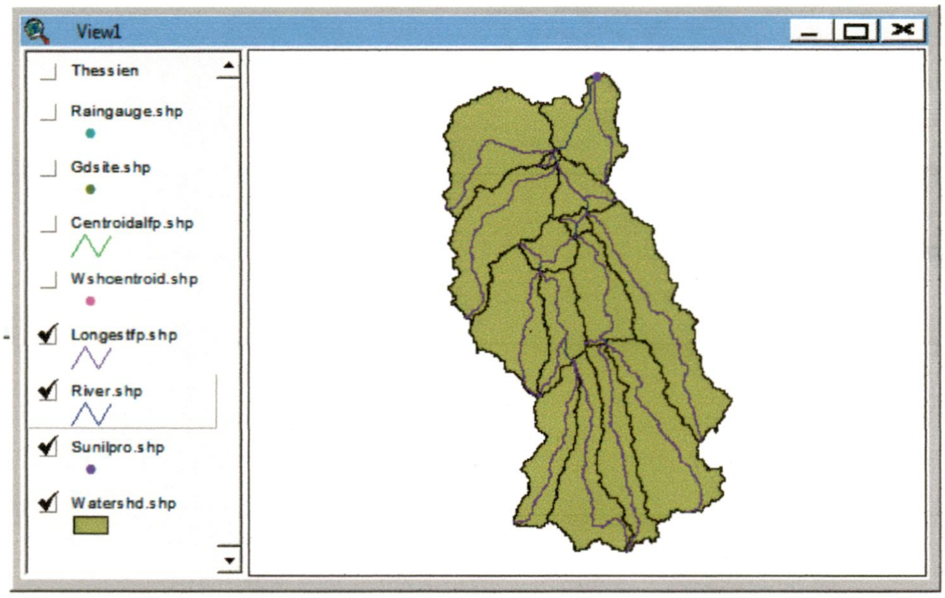

Fig. 4.10 Longest Flow Path of River

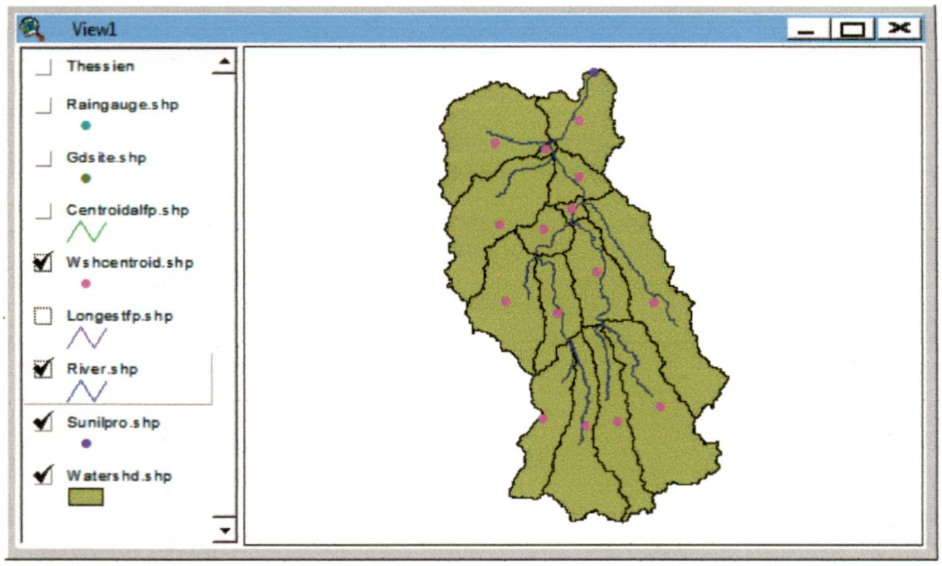

Fig. 4.11 Basin Centroid

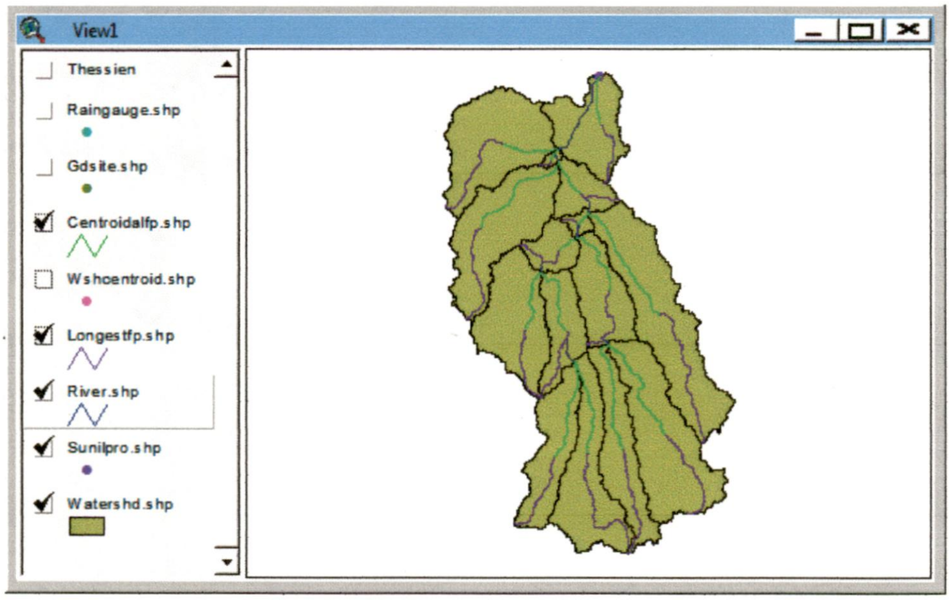

Fig. - 4.12 Basin Centroidal Flow Path

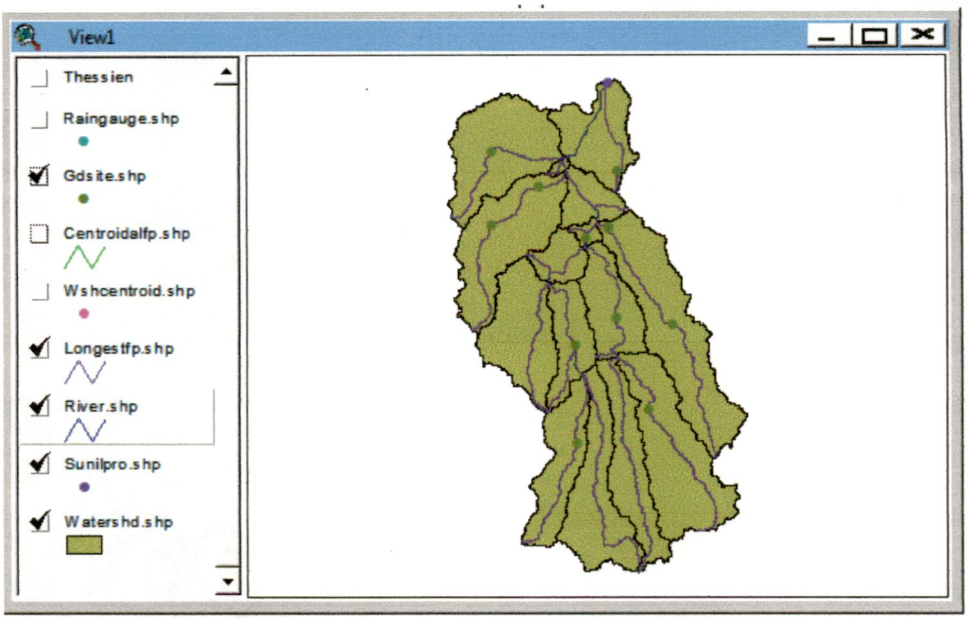

Fig. 4.13- Location of G& D Sites

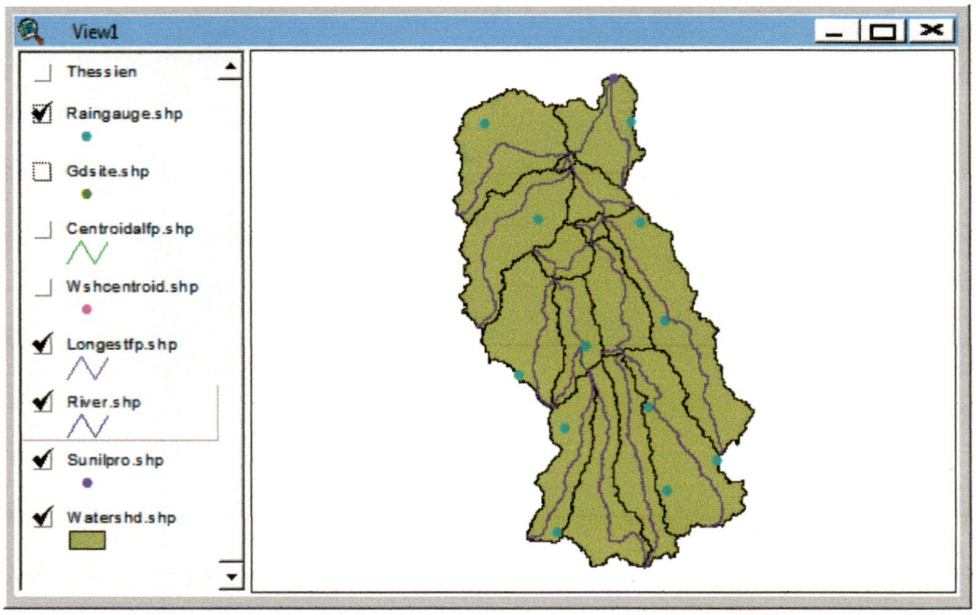

Fig. 4.14-Location of Raingauge Stations

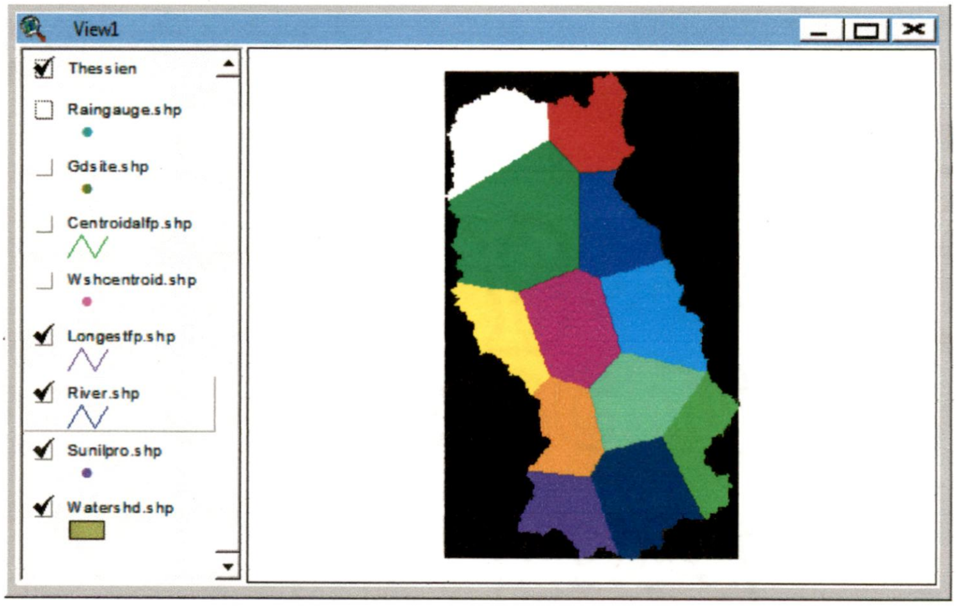

Fig. 4.15 – Thiessen polygon pattern

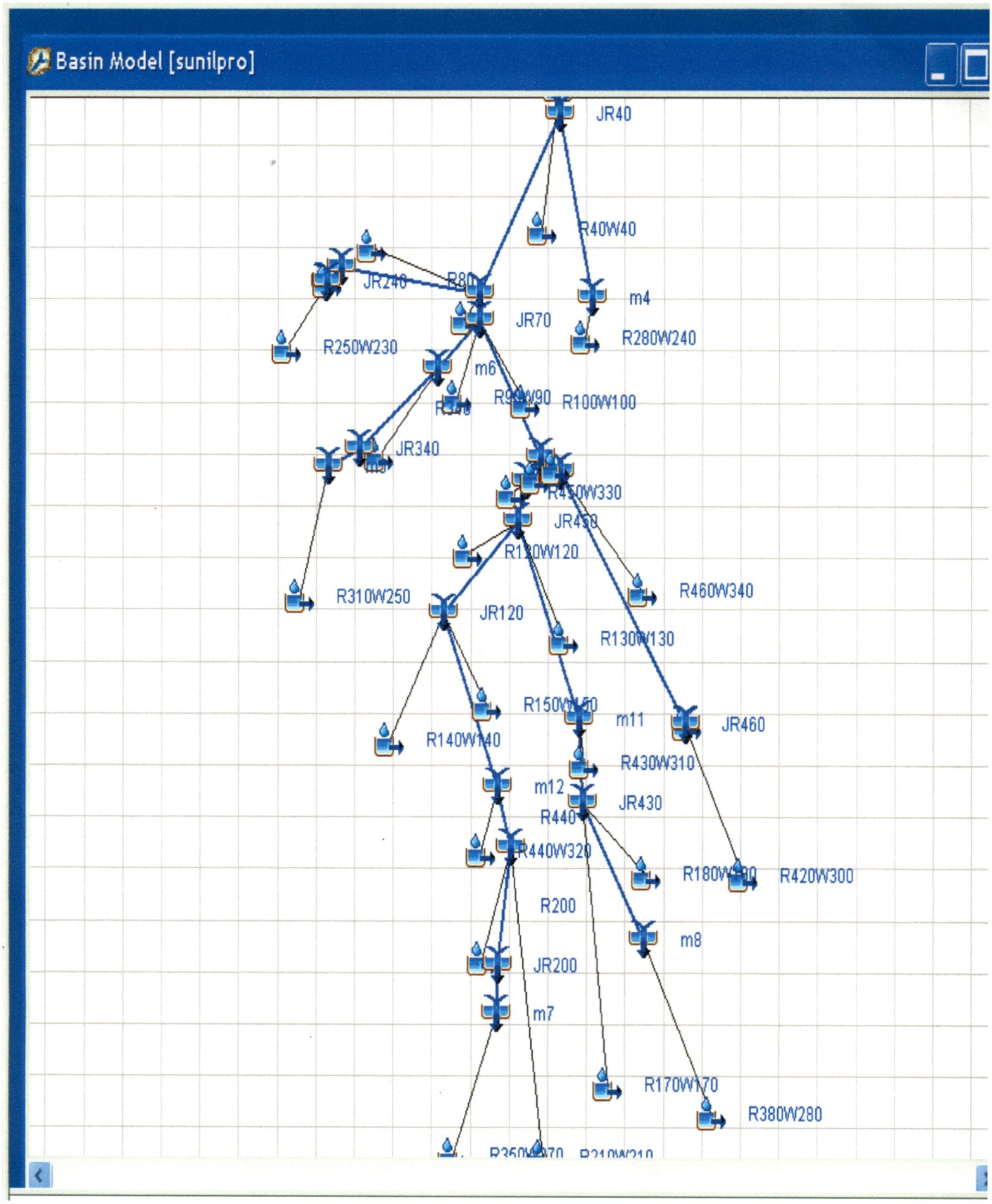

Fig. 4.16 - Lumped Basin Model of Chambal Catchment

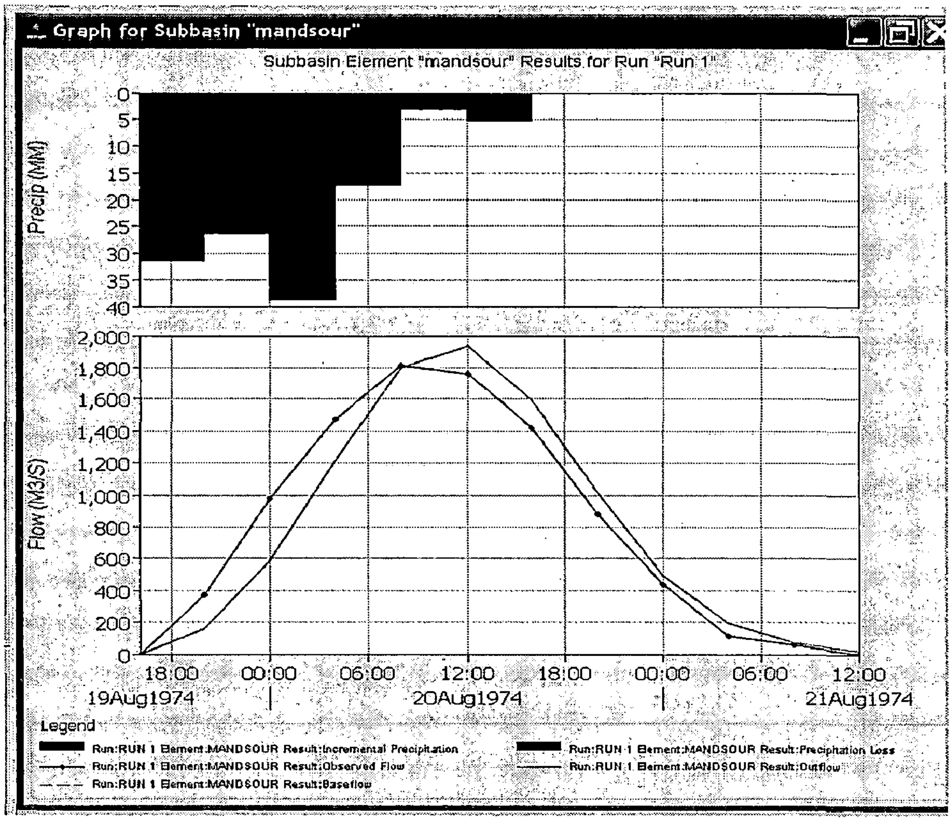

Fig. 5.1 Plot of observed and computed hydrograph for Mandsour sub-basin

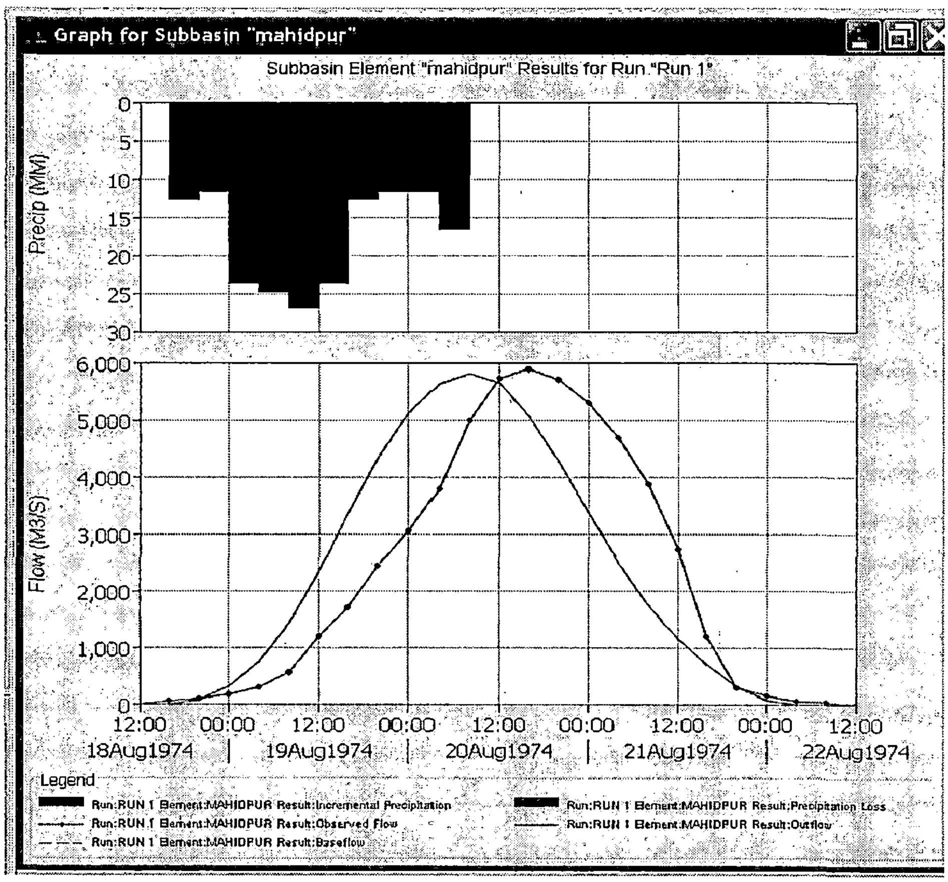

Fig. 5.2 Plot of observed and computed hydrograph for Mahidpur sub-basin

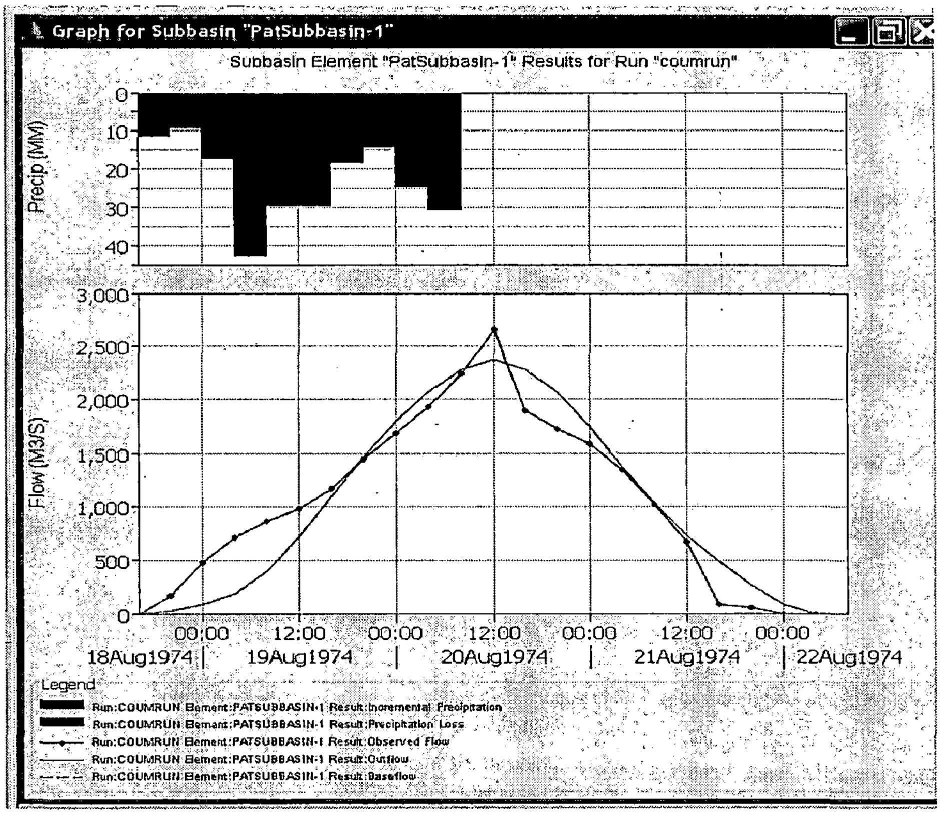

Fig. 5.3 Plot of observed and computed hydrograph for Pat sub-basin

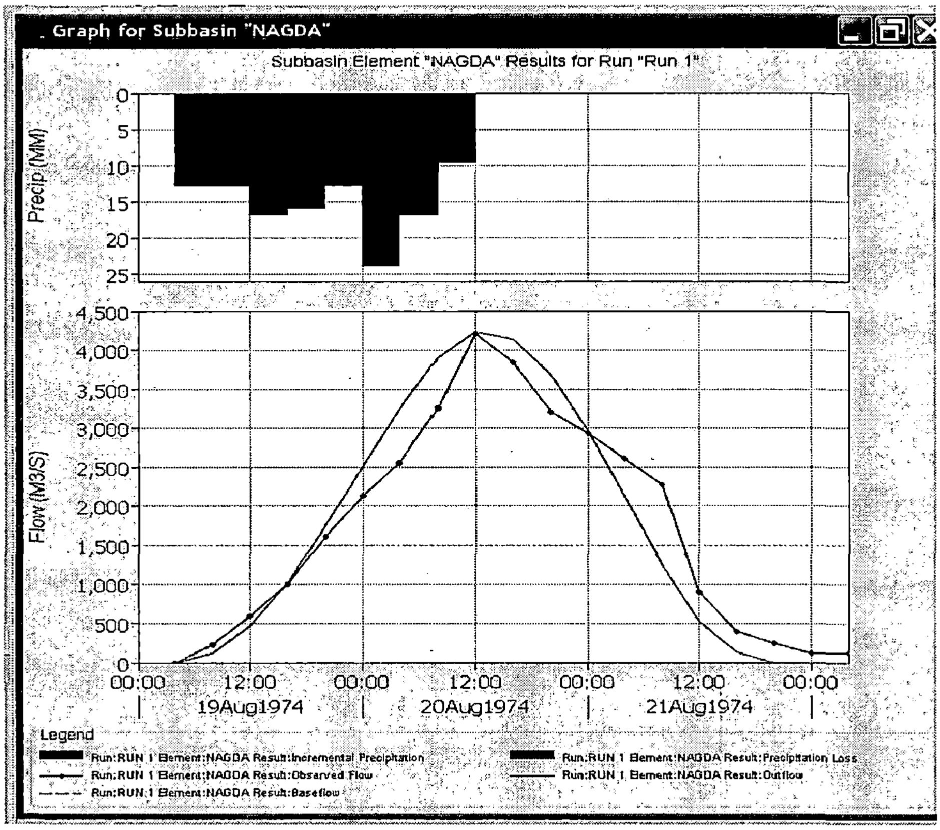

Fig. 5.4 Plot of observed and computed hydrograph for Nagda sub-basin

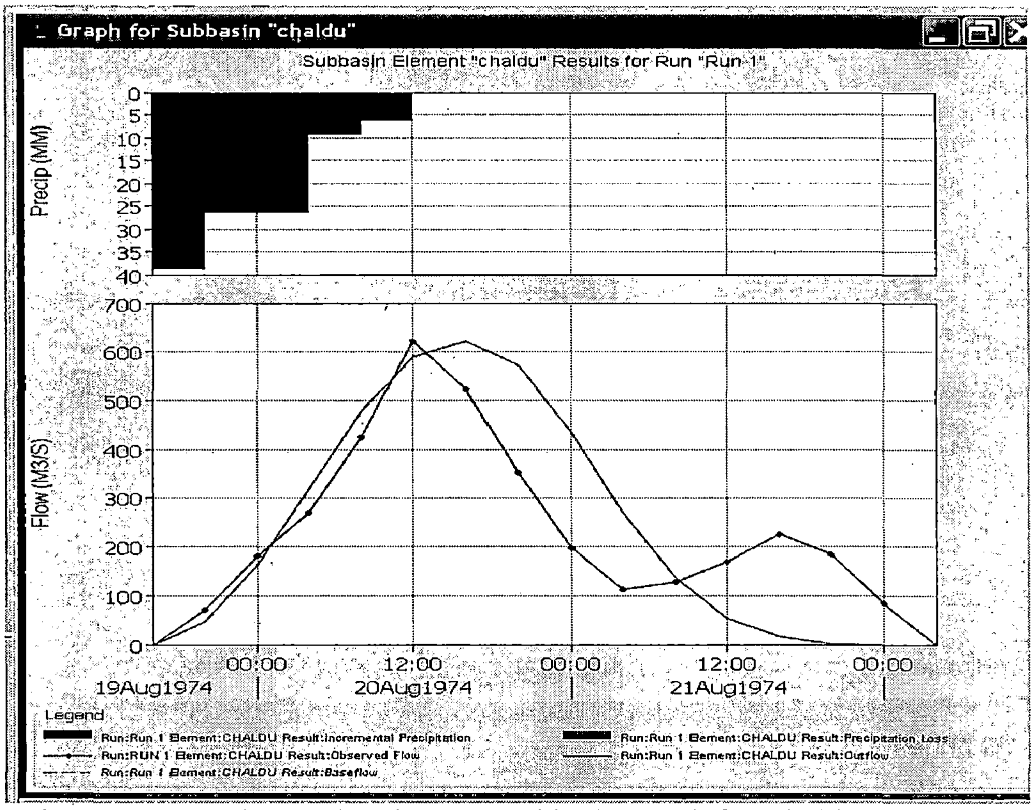

Fig 5.5 Plot of observed and computed hydrograph for Chaldu sub-basin

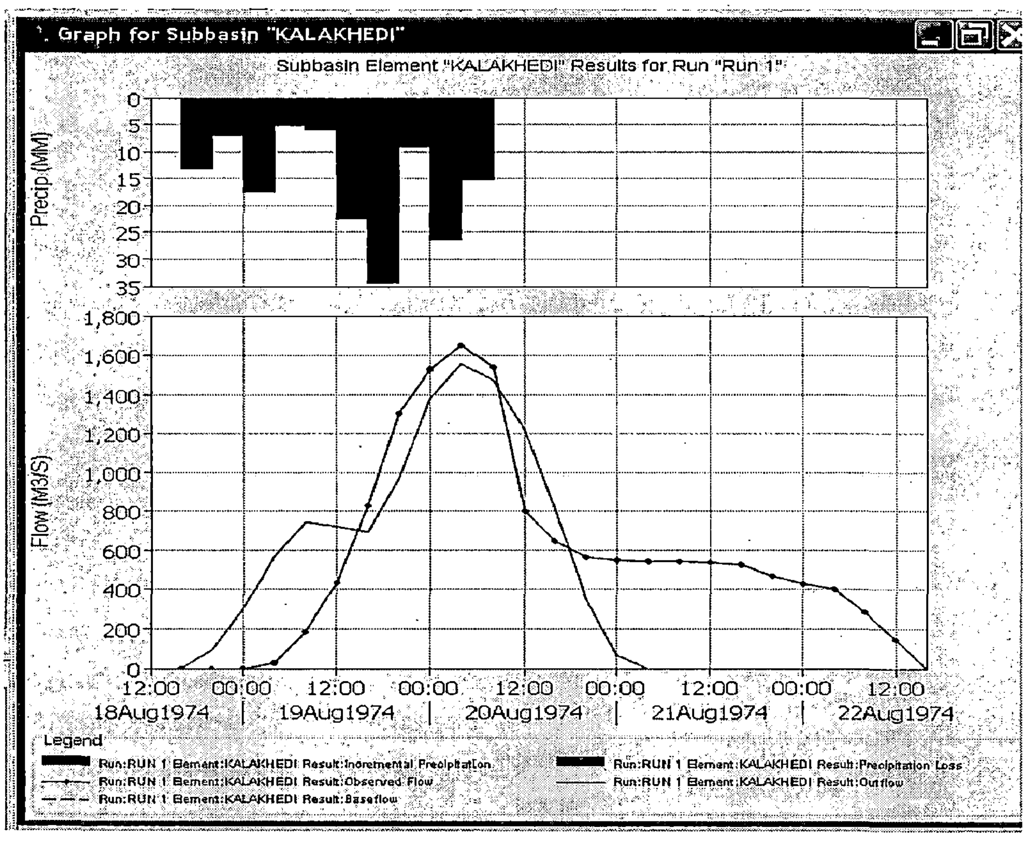

Fig 5.6 Plot of observed and computed hydrograph for Kalakhedi sub-basin

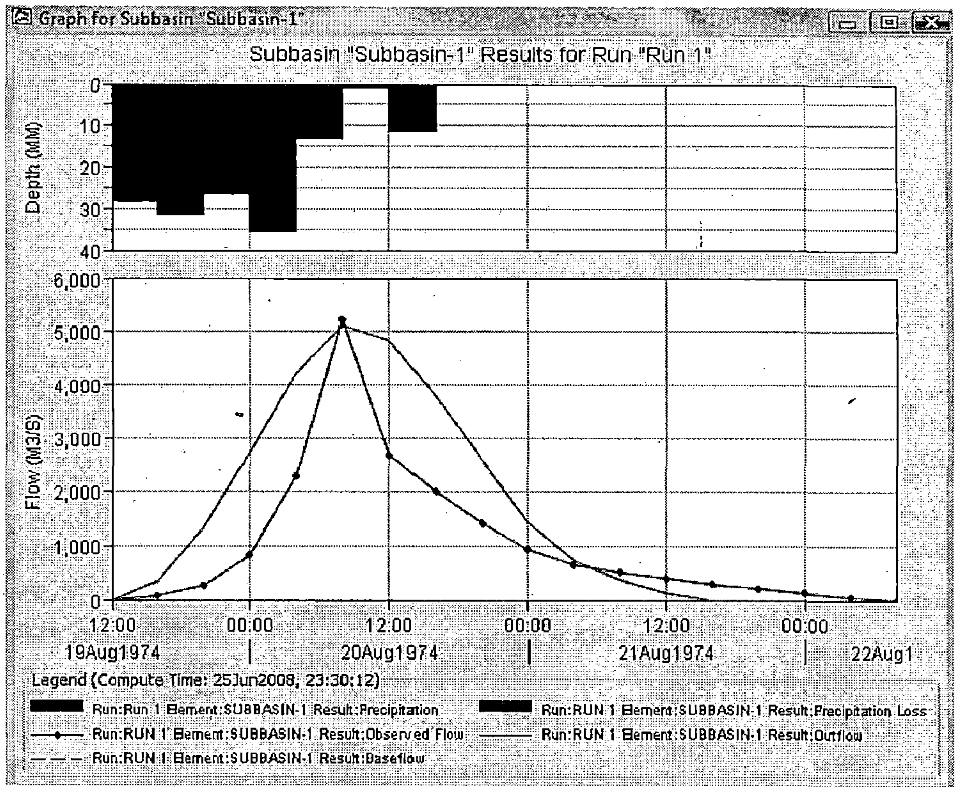

Fig. 5.7 Plot of observed and computed hydrograph for Nahargarh sub-basin

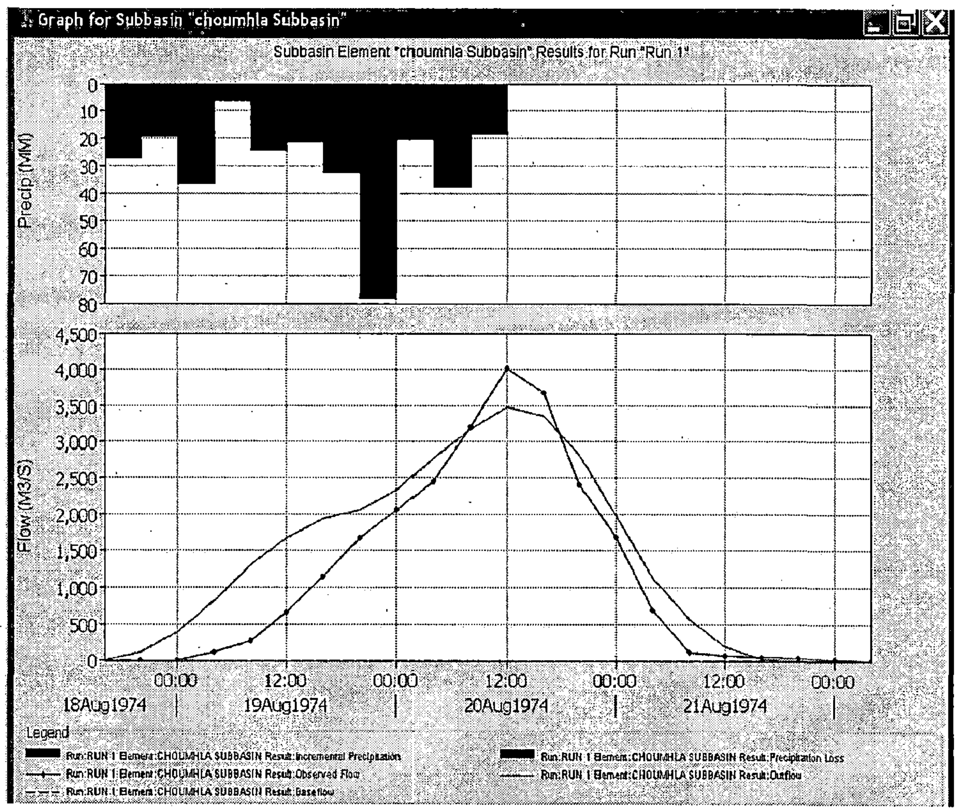

Fig. 5.8 Plot of observed and computed hydrograph for Choumahla sub-basin

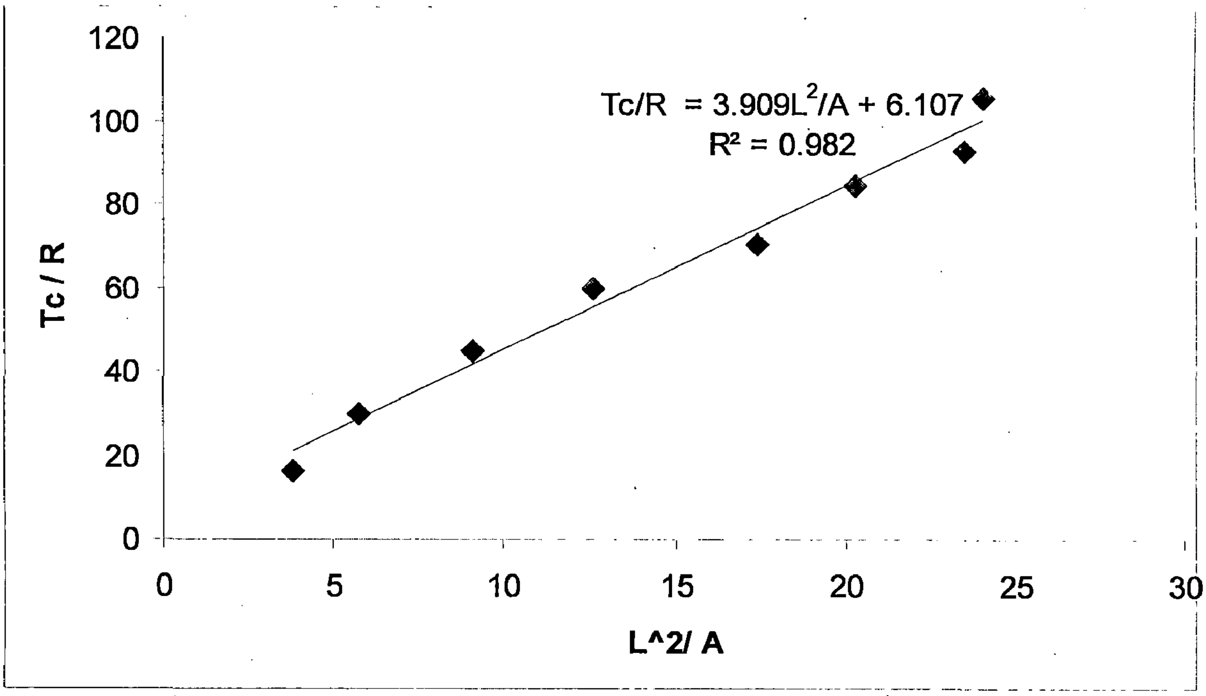

Fig. 5.9 Relation between Tc / R and L^2 / A

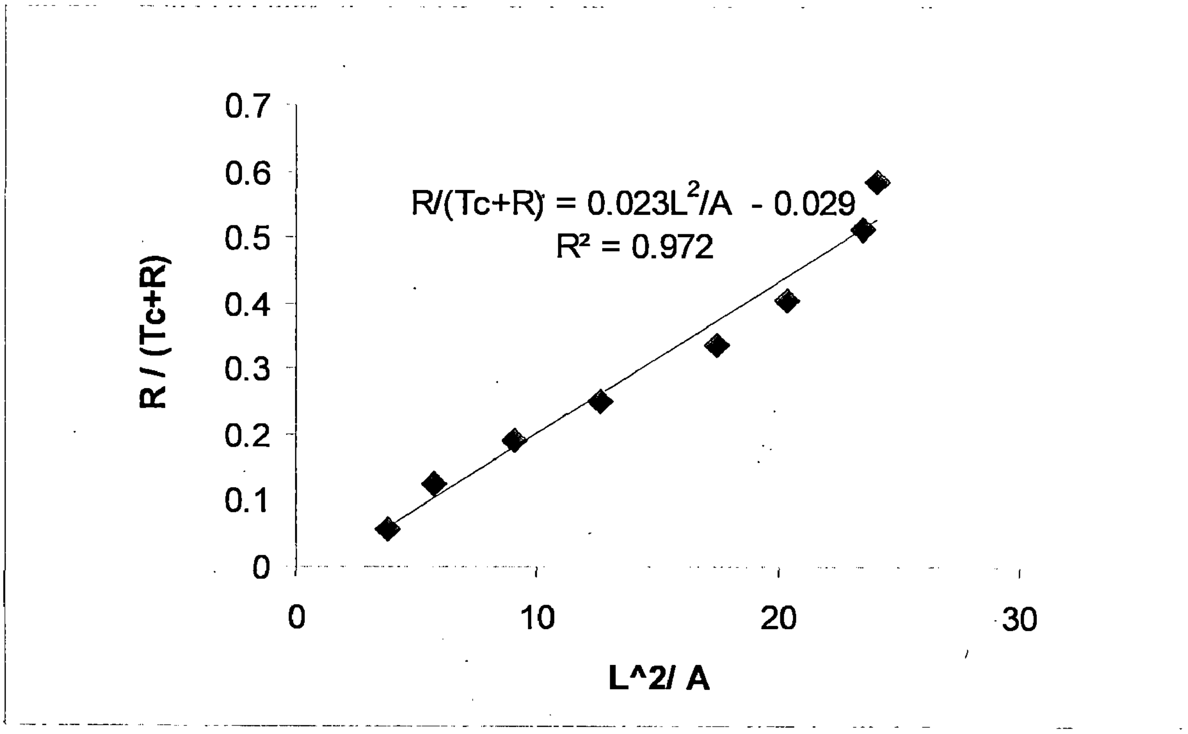

Fig. 5.10 Relation between R / (Tc+R) and L^2 / A

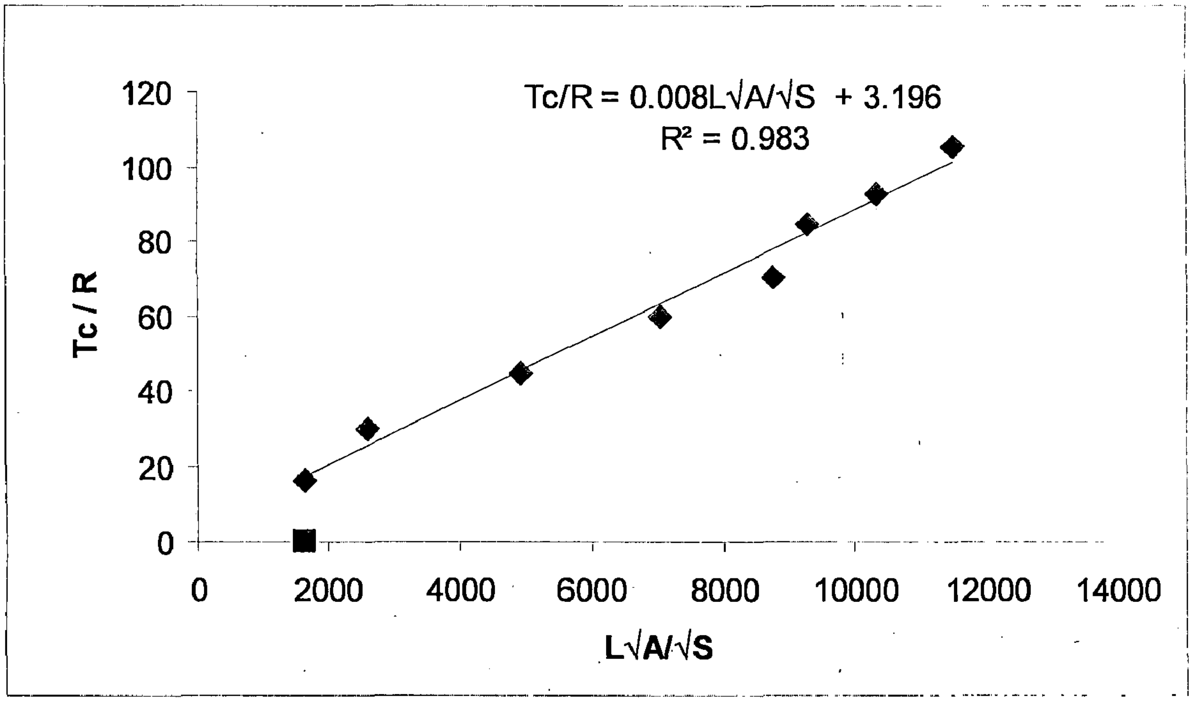

Fig. 5.11 Relation between Tc/R and L√A /√S

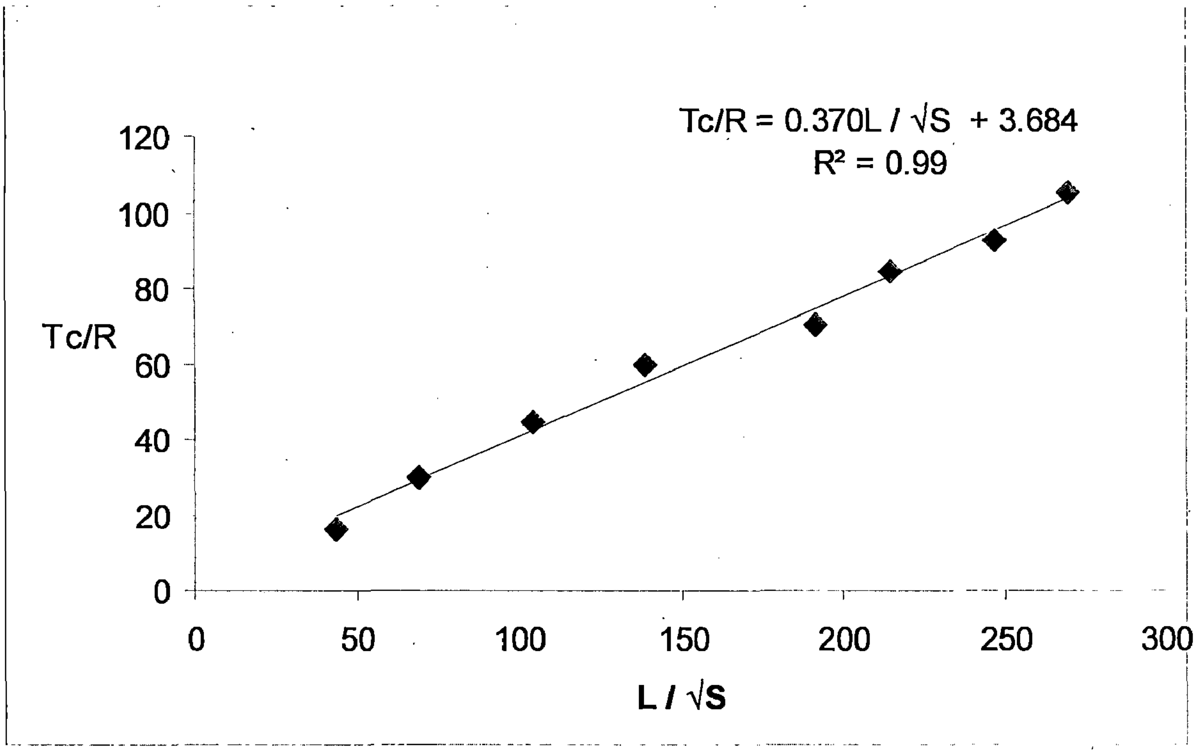

Fig.5.12 Relation between Tc/R and L /√S