SIMULATION OF FLOOD FLOWS USING GIS

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree
of
MASTER OF TECHNOLOGY
in
HYDROLOGY

By

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in this dissertation titled SIMULATION OF FLOOD FLOWS USING GIS in the partial fulfillment for the award of the Degree of Master of Technology in Hydrology, submitted in the Department of Hydrology of the Indian Institute of Technology, Roorkee, is an authentic record of my work done during the period from July, 2007 to June, 2008 under the guidance of Dr. M. K. Jain, Asstt. Professor, Department of Hydrology, Indian Institute of Technology, Roorkee.

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

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

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LIST OF NOTATIONS AND ABBREVIATIONS

cumec	Meter cube per second
hrs	Hours
km	Kilometer
m^3	Cubic meter
m^3/s	Cubic meter per second
min	Minutes
mm	Millimeter
Sq. km	Square kilometer
T_b	Time base
t_c	Time of concentration
t_p	Time to peak
Δt	Time step
Q_p	Peak discharge
R	Storage coefficient (Clark model)
$\overline{A}$ (i)	Cumulative area
$\overline{T}$	Fraction of time of concentration t_c
k	Storage coefficient (Nash model)
n	No. of reservoirs
L	Longest channel length
S	Basin slope
M_1	Ist moment about centroid
M_2	IInd moment about centroid

ABSTRACT

Since its inception, the unit hydrograph has been used as a key concept in estimation of storm runoff from a catchment. Significant contribution to the unit hydrograph theory was given by Clark (1945), Nash (1957) and SCS (1972) in the form of development of synthetic unit hydrographs. Present study was taken up to determine the most suitable model for simulation of storm runoff for Jolarpet watershed having outlet at railway bridge No. 719. A digital elevation model has been developed for the simulation of rainfall-runoff process. The HEC-GeoHMS interface has been used for terrain processing to prepare input files for use in HEC-HMS model. The storm transformation options for SCS unit hydrograph and Clark unit graph of HEC-HMS have been used. Computations for Nash model have been done using Excel worksheets. In all six storm events have been used for judging performance of different models. In addition a comparison between two procedures, method of moments and multi-storm processing, for calculating basin average parameters for Nash model has also been done. Performance of different models used is evaluated using visual comparison and comparison based on statistical criteria such as Nash-Sutcliffe efficiency (NSE), peak weighted root mean square error (PWRMSE) and standard error (SE). Visually all models are showing approximate similar results but statistically the difference is clear. Among three models, based on average value of statistical indices used, Clark model with average NSE 83.32 is adjudged best followed by the Nash model with average NSE 79.45. The SCS Dimensionless unit hydrograph model performed poorest.. For derivation of representative values of Nash model parameters, method of moments give slightly better results compared to multi-storm processing.

CHAPTER I

INTRODUCTION

Among the available methods for rainfall runoff simulation and flood forcasting, the unit hydrograph(UH),having as its limiting case the instantaneous unit hydrograph(IUH),is the simplest of those most widely and successfully used. Since its conception by Sherman (1932), the unit hydrograph (UH) has been widely used in hydrological rainfall-runoff modeling. Unit hydrograph of a watershed is defined as direct runoff hydrograph resulting from a unit depth of excess rainfall generated uniformly over the drainage area at a constant rate for an effective duration (Chow et al. 1988).

The unit hydrograph is a lumped linear model of a watershed where it is assumed that a catchment acts on an input of effective precipitation in a linear and time-invariant manner to produce an output of direct storm runoff (Dooge, 1959).The above definition and some assumptions constitute the unit hydrograph theory. The two basic principles to be satisfied to use the unit hydrograph theory are 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 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. Methods for determining a unit hydrograph from storm events with observed direct runoff hydrograph and effective rainfall hyetograph are umpteen. They have been systematically categorized by Singh (1988).However few developments are discussed below.

1.1 DEVELOPMENTS IN UNIT HYDROGRAPH METHOD

According to Chow (Chow, 1964), Folse recognized the relationship between rainfall and runoff 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. 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 numbers 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 co linearity, the linear relation between the elements in a linear system. Multiple events were used in the disconsolation process in deriving the unit hydrographs (Bree, 1978).

A similar approach of disconsolation 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).

Another dimension in the theory of unit hydrograph was introduced by Clark (1945) by utilizing the watershed time-area histogram for derivation of unit hydrograph. The Clark IUH is based on the premise that the unit hydrograph can be constructed from one inch of excess precipitation, which is then translated and routed through a reservoir to account for the storage effects of the basin. Synthetic hydrographs can be developed using a two-parameter Gamma distribution (Nash, 1957, Croley II, 1980).

Based on the concept of IUH, Nash (1959) conceptualized a drainage basin as series of 'n' identical linear reservoirs in series, whose response function can be represented by two parameter gamma distribution. Later Dooge (1959) improved the Nash (1957) model by introducing translation time into the Cascades that was ignored earlier. However, the model was not amenable to practical applications (Chow, 1964). To overcome this difficulty Singh (1964) derived the IUH using a nonlinear model considering the overland and channel flow components separately. Methods of UH derivation are used widely in developing countries like India (CWC, 1982), Turkey and others.

1.2 OBJECTIVES:

The objectives of the thesis are:

- To generate geo-database for a small catchment for which short interval rainfall runoff data is available.

- To use HEC-GEOHMS model to develop HEC-HMS's basin model files.

- To use HEC-HMS models (SCS UH and Clark's UH) of direct runoff, to simulate the flood flows.

- To derive average unit hydrograph for the study catchment using Nash model by using: a) Multistorm processing b) method of moments.

- To present comparative analysis of results obtained with different methods used.

CHAPTER II

LITERATURE REVIEW

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.

Croley, 1980, presented a two-parameter Gamma distribution as a basis for synthetic hydrographs with a review of existing applications and non-feasible applications are identified. Several approaches for fitting this function to practical boundary condition parameters are identified and presented in a unified treatment. They are especially designed for use on small programmable calculators since the synthetic hydrograph is extremely sensitive to the Gamma distribution parameters. Nomographs would give large errors in the fit for small errors in the boundary condition parameters. Although non-dimensionalization of the synthetic hydrograph is possible with the Gamma distribution, it is shown to be unnecessary.

Aron and White, 1982, presented a simple method to fit a smooth gamma distribution over a single point specified by the unit hydrograph peak and the time Speak with a guaranteed unit depth of runoff. Several methods for synthetic unit hydrographs, available in the literature, involve the hand fitting of s-curve over a set of a few hydrograph points, which can sometimes be a objective task. Besides, the user often finds it difficult or simply neglects to adjust the generated unit graph to a runoff volume of one unit (inch, cm, or mm).this paper solves the problem upto some extant.

Bhunya, 2003, introduced a simplified two-parameter gamma distribution for derivation of synthetic unit hydrograph to derive a synthetic hydrograph more conveniently and accurately than the popular gray, soil conservation service, and Snyder methods. The revised version incorporates the approximate, but accurate, empirical relations developed for the estimation of factors governing the shape of the dimensionless unit hydrograph from the Nash parameter n (=number of reservoirs). The applicability of the simplified version is tested on both text and field data.

Bhunya et al., 2005, introduced a hybrid model for derivation of a synthetic unit hydrograph by Splitting the Nash single linear reservoir into two serially connected reservoirs of unequal storage coefficients (one hybrid unit) for a physically realistic response. Empirical relations are given for estimation of the two storage coefficients from known peak flow and time to peak. The hybrid model with two serially connected units is found to work significantly better than the most widely used methods such as those of Snyder, the soil conservation service and the two-parameter gamma distribution when tested on synthetically generated data and the data from four catchments from India and one from turkey. The workability of the proposed approach was also tested for partial and no data availability situations.

Singh 2007, proposed a simplified procedure for runoff modeling using a Gamma IUH or Nash model's IUH The use of the gamma distribution as instantaneous unit hydrograph (IUH) for modeling runoff is simple and reasonably accurate. A simplified procedure was proposed for modeling runoff using the gamma IUH, in which the incomplete gamma function was computed indirectly, but the gamma function was computed. This proposed procedure does not require computation of either the gamma function or incomplete gamma function.

Bruen and Dooge, 1984, describes least square fitting method to compute discrete unit hydrograph (DUH) to reduce both the data storage and arithmetic computation requirements of computer programs for estimating DUHs, for small computers when memory space is limited or when large amounts of data are involved. The algorithm can be used to provide smoothed least-squares estimates of the DUH using a single extra arithmetic addition. Estimates of DUHs for multiple-event data can easily be calculated.

Zhao et al., 1994, discusses the derivation of a unit hydrograph by multiple storm analysis using least squares methods. Variations of leant squares method was generalized using the framework of weighted ridge analysis. The paper also shows two theorems to support using multi-storm analysis to derive UH. In addition, an issue was addressed on the scaling effect in the conventional multi-storm analysis which could create potential bias toward large storm in deriving a multi-storm UH. For that, a simple scaling procedure was proposed to reduce such potential bias problem. Numerical investigations were conducted to examine the performance of the scaling procedure by comparing with the conventional multistorm analysis (without scaling) and the HEC-1 weighing procedures. Based on various performance criteria using a total of 39 storms from three watersheds, the proposed scaling procedure was found to produce a quite desirable UH.

Unver and Mays, 1984, presented an optimization model that can be used in the determination of loss rate functions and unit hydrographs using observed rainfall and runoff data. Composite unit hydrographs and loss rate parameters can be determined by using several multi period storms simultaneously or using individual multiperiod storms. The model is a nonlinear programming problem so that a generalized reduced gradient method is used as the solution technique. Kostiakov's, Philip's, Horton's, and the ϕ-index methods are used to illustrate the model for comparative purposes. An infiltration equation that includes rainfall intensity is also introduced and is compared with the others by using the optimization model. The model is applied to storms for a hypothetical example.

Wang, 1985, determined the structural relationship between the model with N-linear reservoirs in series and its discrete form, and the parameters of the discrete form directly from rainfall and runoff records by linear programming. Generally to solve a rainfall-runoff problem through the use of conceptual hydrological models usually involves determining the instantaneous unit hydrograph (IUH) from rainfall and runoff records by using moment matching. In use, it is necessary to convert the IUH into a unit hydrograph of finite duration *D,* DUH. For this the S-curve is used.

Chen and Singh, 1986, presented a new variable instantaneous unit hydrograph (NVIUH) derived by employing n second order representation of the convolution integral and n nonlinear storage-discharge relationship. This derivation removed n conceptual inconsistency in Ding's variable-instantaneous hydrograph (DVIUH), and led to an analytical expression for second-order kernal. An alternative procedure for derivation of DVIUH was also formulated. The NVIUH was verified on two watersheds in China.

Turner et al., 1989, used De Laine's method of deriving the unit hydrograph from the common roots of polynomial corresponding to different storms as a basis for proposing a new procedure in which the hydrograph roots can be selected from among the polynomial roots for the runoff of a single storm. The selection is made on the basis that the complex unit hydrograph roots form a character "skew circle" pattern when plotted on an Argand diagram. The application of the procedure to data is illustrated for both a single-peaked and double-peaked event. The method of unit hydrograph derivation by root selection was developed, during a study of the flood, response of the River catchment prior to hydroelectric development (Turner, 1982). Since there no record of rainfall but an ample supply of runoff records, a method of unit hydrograph derivation that did not use rainfall data was required. De Laine (1970) had proposed such a method using the runoff data for several storms and the present approach was developed from De Laine's work to overcome some weaknesses of his method and to identify the ' hydrograph from the runoff data of a single storm. This method of unit hydrograph derivation is proposed on a hypothetical basis.

Allam, 1990, presented a watershed discharge hydrograph simulation model, based on the hydraulically based geomorphologic instantaneous unit hydrograph (IUH), developed by kirshen and bras (1983). This IUH was derived as a function of watershed geomorphology and the response of streams to lateral inflows, determined by solving one-dimensional linearized equations of motion. The effective rainfall is calculated here to be equal to the gross rainfall minus the infiltration losses, presented with Philip's expression coupled with an empirical equation for soil moisture computation. Consideration is given to the role of mountainous terrain in runoff generation. The simulation model is verified through applications for three gauged watersheds in Saudi Arabia. A comparison is made between the observed and simulated hydrographs.

Wilson and Brown, 1992, developed a generalized unit hydrograph method and evaluate it for ungaged watersheds by A key component in this method is the value of a dimensionless storage coefficient. Procedures to estimate this coefficient are given using calibrated values from 142 rainfall-runoff events gaged in watershed located mainly in the East US. Only limited success was obtained in predicting this storage sufficient. Thirty-seven, independent rainfall-runoff events were used to test the proposed technique. The generalized unit hydrograph predicted the runoff hydrographs fairly well with considerable improvement in accuracy over the SCS dimensionless unit hydrograph. Approximately one-half of test storms had percent error in predicted peak flow rates that were less than 34 percent compared to percent error of 88 percent with the SCS method.

Bruen and Dooge, 1992, expanded the method of regularization for estimating unit hydrographs, to allow the inclusion of prior information about the unit hydrograph shape. This may give smooth estimates without any loss in volume. The method is illustrated with prior information from a regression on catchment characteristics and with catchment lag determined from the data.

Zhao and Tung, 1994, proposed two alternative LP formulations for obtaining optimal UH. The objective functions in commonly used linear programming (LP) formulations for obtaining an optimal UH are (1) minimizing the sum of absolute deviations (MSAD) and (2) minimizing the largest absolute deviation (MLAD). In this paper two alternative LP formulations for obtaining an optimal UH, namely, (1) minimizing the weighted sum of absolute deviations (M WS AD) and (2) minimizing the range of deviations (MRNG).The predicted DRHs as well as the regenerated DRHs by using the UHs obtained from different LP formulations were compared using a statistical cross-validation technique. The golden section search method was used in determine the optimal Weights for the model of MWSAD. The numerical results show that the UH by MRNG is better than that by MLAD in regenerating and predicting DRHs. It is also found that the model MWSAD with a properly selected weighing function would produce a UH mat is better in predicting the DRHs than the commonly used MSAD.

Zhao et.al., 1995, presented five potentially useful statistical validation methods. For illustration, they were applied lo examine the predictability of unit hydrographs derived by various methods in the framework of the least squares and its variations. It was found that storm-stacking (conventional multistorm analysis) together with storm-scaling yields the most desirable UH. The general framework of these validation methods can also be applied to a validation study of other hydrologic and hydraulic models.

Krishna p. Jonnalagadda, 2003, determined an instantaneous unit hydrographs for small watersheds of central Texas. A significant number of individual storm events were contained within these studies. No data pertinent to unit hydrograph research from these studies are digitally available and the USGS reports represent the sole data source. The data obtained from the studies was digitized and a database containing all the recorded events of rainfall and runoff was constructed for the small watersheds of central Texas. The database was divided into five different modules each with certain number of watersheds. The database was used to derive the instantaneous unit hydrographs for all the stations operated by USGS. A form of convolution model was used to duplicate the observed hydrographs. The model parameters were analyzed for their dependencies on the location of the station.

Xin he, 2004, compared three candidate models derived from a linear-system analysis with nrcs model, along with an early empirical model. The models are gamma model, Rayleigh model, Weibull model and the empirical model by commons. In this research comparison of gamma, Rayleigh, Weibull and nrcs models with observed runoff data for central Texas small watersheds the watersheds being studied by are from central Texas. Results show that all the models have produced acceptable prediction of runoff discharge, when supplied historical precipitation events. The Weibull model produced the best "fit" as was expected because it has the most adjustable parameters.

Biaskar, 1992, used a geomorphologic instantaneous unit hydrograph (GIUH) method to estimated hydrologic parameters using geographic information system for select watersheds within the big sandy river basin in northeastern Kentucky using a GIS software program called arc/info. Although runoff simulation results using the (WAHS) model did not compare well with the observed data, this study clearly demonstrates the advantages and disadvantages of using a GIS in runoff modeling that require geomorphic and other spatial data bases.

Sorman, 1995, estimated the peak discharge using GIUH model in Saudi Arabia. Many of the basins, called wadis, in Saudi Arabia lack long records of hydrologic data. Most of these wadis located in the southwestern part of the kingdom are under flooding danger for every 5-10 yr return period. Flash floods threaten downstream villages, towns, and agricultural areas because they are uncontrollable and difficult to predict. Therefore, the subject requires special attention by researchers in arid climates to estimate the magnitude, volume, and time to peak of hood hydrographs. As a result, the optimal planning and managing of water resources can be achieved using the limited water resources available (surface, subsurface) at the potential sites in arid climates. Geomorphologic instantaneous unit hydrograph (GUIH) is one of the possible approaches to predicting the hydrograph characteristics. Because of the study the designer and project engineer will be able to estimate the hydrograph properties (peak discharge and time to peak) in the design of various projects such as culverts, levees, and dams in arid climates with limited hydrologic information.

Ben Chie Yen, 1997, tested the GIUH model on two hilly watersheds in the eastern United States and two relatively hat-slope watersheds in Illinois. The recently proposed geomorphologic instantaneous unit hydrograph (GUIH) method is perhaps the most promising development to relate the hydrologic response of watersheds as runoff production from rainfall to watershed topographic structures in this direction; if successful, it would allow the derivation of the unit hydrograph (UH) for ungauged or inadequately gauged watersheds without the need of observed runoff and rainfall data. In this method unit hydrograph derivation for ungauged watersheds by stream-order laws, thegeomorphic ratios of the Horton-Strahler stream-ordering laws are incorporated in the GIUH model for UH generation.. Comparison between the simulated and observed hydrographs for a number of rainstorms indicates the potential of this model as a useful tool in watershed rainfall-runoff analysis.

Sahoo, 2006, developed a geomorphologic instantaneous unit hydrograph (GIUH) based on models by Clark in 1945 and Nash in 1957 and applied to the Ajay river basin at Jamtara in northern India. The geomorphologic parameters of the basin were estimated using Erdas imagine 8.5 image processing and geographic information system (GIS) software. The direct surface runoff (dsro) hydrographs derived by the GIUH-based models without using historical runoff data, the conventional Clark IUH model option of the hec-1 package, and the Nash IUH model, were hydrographs employing four performance criteria. The DSRO hydrographs are computed with reasonable accuracy by the GUIH-based Clark and Nash models, when compared to the Clark IUH model option of the hec-1 package and the Nash IUH model. It is observed that the GIS supported GIUH-based models hold great potential of estimating floods from ungauged basins, the dsro hydrographs computed using the GIUH-based Clark and Nash models, were estimated with reasonable accuracy

Singh, 2006, proposed an optimization method for estimating an optimal smooth instantaneous unit hydrograph (IUH) from multistorm data taken simultaneously. A gamma IUH is used for representing the analytical IUH. The parameters of the analytical IUH are optimized. The method avoids oscillations in the tail end of IUH. It automatically takes care of the volume and nonnegativity constraints. The application of the method is illustrated through examples. The new method reliably estimates the optimal IUH.

Charng, 2002, introduced a method to convert a known unit hydrograph to unit hydrographs of different durations, as an alternative to the s-hydrograph method The method called the complementary hydrograph method involves a process of decomposing a known unit hydrograph of duration i_0 into a pair of "complementary" hydrographs associated with two sequential rainfalls with the sum of their durations equal to tc. In hydrograph conversion, the complementary hydrograph method produces an identical result to the well-established S-hydrograph method and involves a comparable number of computational steps. In certain special cases, such as converting a unit hydrograph into one with half its duration, the new method requires fewer computational steps. While the two methods employ different approaches in their solutions of hydrograph conversion problems, the agreement in their results stems from the fact that both methods are founded on the same principles of superposition and linearity of the unit hydrograph method. The s-hydrograph method can also be viewed as a special case of the complementary hydrograph method in which the known hydrograph is associated with a storm of infinite duration and uniform intensity.

Bunya et al., 2004, developed the approaches to evaluate the unknown parameters using an analogy between the three-parameter beta-distribution shape and the

SUH shape. Based on nondimensional analysis and optimization, a simple accurate relation is introduced to estimate the three parameters of the beta distribution that is useful for unit hydrograph derivation. The relation yields results closer to those obtained by an available trial and error procedure. The unit hydrographs from the proposed method fit observed hydrographs better than those from the widely used two-parameter gamma distribution.

Seann Reed, 2002, developed a computer application and national geospatial database to support the calculation of flooding flow and threshold runoff across the conterminous united states and Alaska in his study application and national geographic information system database to support two-year flood and threshold runoff estimates . Flooding how is the flow required to cause a stream to slightly overflow its bank and cause damage. Threshold runoff, defined as the depth of runoff required to cause flooding, is computed as flooding flow divided by the unit hydrograph peak flow. A key assumption in this work is that the two-year return flood is a useful surrogate for flooding flow. The application described here computes flood magnitude estimates for selected return periods. Using regression equations published by the U.S. Geological survey for each of 210 hydrologic regions. The application delineates basin boundaries and computes all basin parameters required for the flood frequency calculations. The geographic information system database that supports these calculations contains terrain data [digital elevation models and dem derivatives], reference data, and 89 additional data layers related to climate, soils, geology, and land use. Initial results indicate that there are some practical limitations associated with using q_i regression equations to estimate flooding flow.

Sabol, 1988, presented a procedure to facilitate the evaluation of *R* from recorded hydrographs using the technique of hydrograph recession analysis. The Clark unit hydrograph is a three-parameter synthetic unit hydrograph procedure that can be used in flood hydrology and other surface water runoff studies that require the development of a hydrograph or the reconstitution of a flood event. The technique is particularly valuable for unusually shaped watersheds, such as watersheds with large length-to-width ratios, or for application lo watersheds containing several different physiographic areas, such as plateaus, escarpments, and valleys. The Clark unit hydrograph can be developed completely by a mathematical routing procedure that is computationally very efficient and lends itself to convenient computer applications. Although this unit hydrograph procedure has a strong theoretical basis and is very applicable to many watersheds, it has not gained wide application. Infrequent use by practicing engineers may be because of the difficulty in evaluating the storage coefficient *R* from recorded hydrographs, and the lack of empirical procedures to estimate *R* for ungagcd watersheds.

Noorbakhsh, 2005, derived parameters for Clark's SUH using geographic information system (GIS), techniques. Clark's method requires estimation of three basin parameters for the derivation of IUH, time of concentration (Tc), storage attenuation coefficient (K) and time-area histogram of the basin. The results show good agreements between observed data hydrographs and Clark's SUH which was derived by GIS techniques. This model was applied to the Kardeh river basin, in Khorasan province located in the northeast part of Iran. The results show that the Arc View GIS software is a powerful tool for IUH estimation.

Cleveland et al., 2008, done the Synthesis of unit hydrographs from a digital elevation model Characterization of hydrologic processes of a watershed requires estimation of the specific time-response characteristics of the watershed. In the absence of observations these characteristics are estimated from watershed physical characteristics. An exploratory assessment of a particle-tracking approach for parametrizing unit hydrographs from topographic information for applicable Texas watersheds is presented. The study examined 126 watersheds in Texas, for which rainfall and runoff data were available with drainage areas. Unit hydrographs based on entirely on topographic information were generated and used to simulate direct runoff hydrographs from observed rainfall events. These simulated results are compared to observed results to assess method performance. Unit hydrographs were also generated by a conventional analysis (of the observed data) approach to provide additional performance comparison. The results demonstrate that the procedure is a reasonable approach to estimate unit hydrograph parameters from a relatively minimal description of watershed properties, in this case elevation and a binary development classification. The method produced unit hydrographs comparable to those determined by conventional analysis and thus is a useful synthetic hydrograph approach.

CHAPTER III

METHODOLOGY

In the present study simulation of flood events for Jolarpet watershed have been done using synthetic unit hydrographs of Clark (1945); Nash (1957); SCS Dimensionless unit hydrograph (USDA,1972). While simulating flood hydrograph, the watershed is treated as a single unit and also a combination of two discretized sub-watersheds.

3.1 SCS Dimensionless Unit Hydrograph

The Soil Conservation Service (SCS) dimensionless unit hydrograph procedure is one of the most well known methods for deriving synthetic unit hydrographs in use today. The dimensionless unit hydrograph used by the SCS was developed by Victor Mockus (1957) and was derived based on a large number of unit hydrographs from basins that varied in characteristics such as size and geographic location. The unit hydrographs were averaged and the final product was made dimensionless by considering the ratios of q/q_p (flow/peak flow) on the ordinate axis and t/t_p (time/time to peak) on the abscissa. This final, dimensionless unit hydrograph, which is the result of averaging a large number of individual dimensionless unit hydrographs, has time base of approximately 5 times the time to peak and approximately 3/8 of the total volume occurred before the time to peak and the UH had a curvilinear shape. The curvilinear unit hydrograph may also be represented by an equivalent triangular unit hydrograph that has similar characteristics.

Analysis of a number of hydrographs by the soil conservation services (USDA, 1972) resulted the following average relations.

$$T_r = 1.67 t_p \tag{3.1}$$

$$T_b = 2.67 t_p \tag{3.2}$$

$$t_p = t_1 + \frac{\Delta t}{2} = 0.6 t_c + \frac{\Delta t}{2} \tag{3.3}$$

Defining Q as effective rainfall

$$Q_p = \frac{2AQ}{T_b} = \frac{2AQ}{2.67 t_p} \tag{3.4}$$

For unit of effective rainfall or Q = 1 cm, eq. (3.4) reduces to

$$Q_p = \frac{2.08A}{t_p} \tag{3.5}$$

Where A is the drainage area in km^2; t_c is the time of concentration in hr; Δt is computational time interval, t_p is the time to peak in hr and t_b is time base of unit hydrograph in hr. The average dimensionless unit hydrograph is shown in fig 3.1.

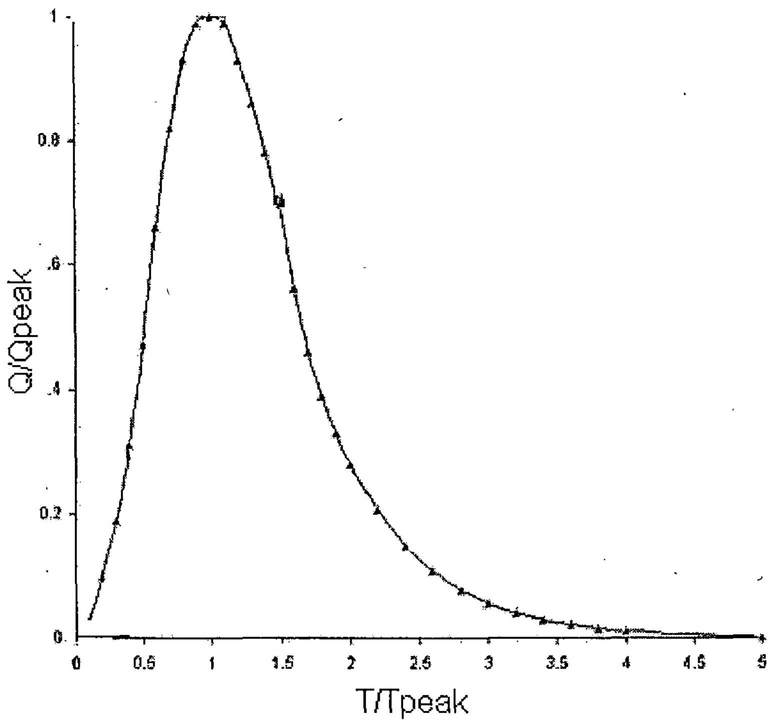

Fig. 3.1: SCS dimensionless unit hydrograph

The duration of the unit hydrograph, is based on the relationship $\Delta t = 0.2\ T_P$ (SCS, 1972).there is some latitude allowed in this relationship; however, the duration of the unit hydrograph should not exceed $\Delta t \leq 0.25 t_p$. These relations are based upon the empirical relationship, $t_l = 0.6\ t_C$ and $1.7\ T_P = \Delta t + t_C$, where t_C Is the time of concentration of the watershed.

3.2 Clark Unit Hydrograph

Clark's method also known as Time-Area histogram method aims at developing an IUH due to an instantaneous rainfall excess applied over a catchment .It is assumed that the rainfall excess first undergoes pure translation and then attenuation.

Clark's model derives a watershed UH by explicitly representing these two critical processes in the transformation of excess precipitation to runoff:

- **Translation** or movement of the excess from its origin throughout the drainage to the watershed outlet; and achieved by a travel time-area histogram.
- **Attenuation** or reduction of the magnitude of the discharge as the excess is stored throughout the watershed and achieved by routing the results of above translation through a linear reservoir at the catchment outlet.

The translation hydrograph is used to reflect the runoff's time of travel. To develop translational hydrograph, the watershed must first be divided into by lines of equal travel time, called ISOCHRONES. The instantaneous unit of excess precipitation is then lagged based on the isochrones to the outlet, creating a time discharge histogram. Next linear reservoir routing is used to the outlet to reflect stream channel storage attenuation effects. The routing results an instantaneous unit hydrograph (IUH).Fig.3.2 depicts a conceptual model of Clark's method.

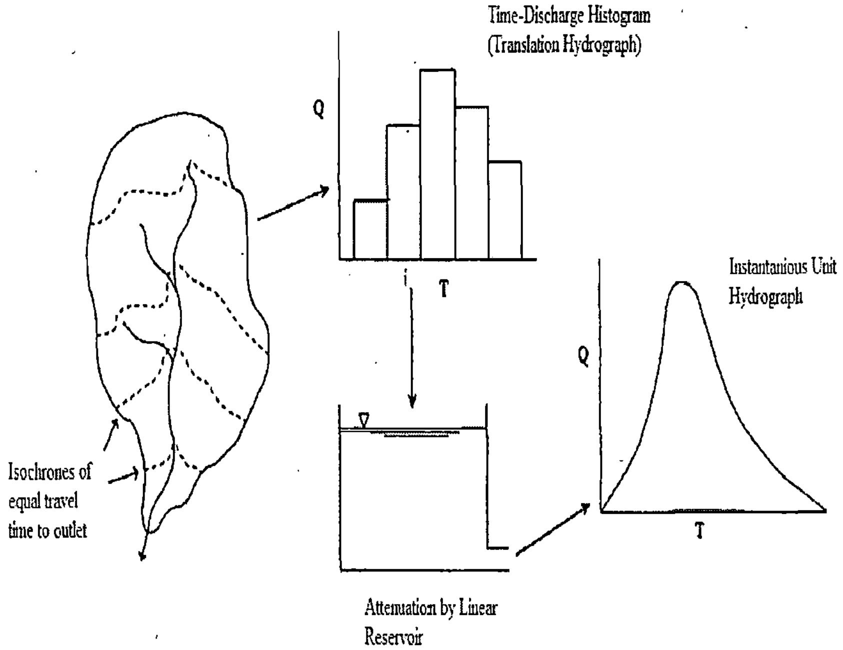

Fig.3.2: a conceptual model of Clark's method

For deriving the IUH, the Clark method uses two parameters, time of concentration (T_c) in hr, which is base length of the time area diagram, and storage coefficient (R), in hours, of a single linear reservoir.

The governing equation of Clark IUH model (Clark, 1945) is given by

$$u_i = \left(\frac{\Delta t}{R + 0.5\Delta t}\right) I_i + \left(\frac{R - 0.5\Delta t}{R + 0.5\Delta t}\right) u_{i-1} \tag{3.6}$$

Where u_i is ith ordinate of IUH; Δt is computational time interval in hr; I_i is ordinate of time area diagram.

If the time-area curve for a specific watershed is not available, then the following generalized equation may be used as an application (HEC, 1990).

$$\overline{A}(i) = 1.414\overline{T}^{1.5} \qquad 0 \le \overline{T} \le 0.5 \qquad (3.7)$$

$$\overline{A}(i) = 1 - 1.414\left(1-\overline{T}\right)^{1.5} \qquad 0.5 \le \overline{T} \le 1.0 \qquad (3.8)$$

Where $\overline{A(i)}$ is cumulative area up to isochrones for time t as fraction of total basin area and $\overline{T}$ is time t expressed as fraction of time of concentration t_c .Both $\overline{A(i)}$ and $\overline{T}$ are dimensionless.

The ordinate of time-area curve are converted to volume of runoff per unit time for unit effective rainfall, and interpolated to given time interval. The resulting translation hydrograph is routed through a linear reservoir to simulate the storage effects of the basin to synthesize the Clark IUH.

With Clark's model, the linear reservoir represents the aggregated impacts of all watershed storage. Thus, conceptually, the reservoir may be considered to be located at the watershed outlet.

3.3 Nash Model

Nash (1957) proposed the following conceptual model of a catchment to develop an equation for IUH. The catchment is assumed to be made up of a series of n identical linear reservoirs each having the same storage constant K .The first reservoir receives a unit volume equal to 1 cm of effective rainfall from the catchment instantaneously .This inflow is routed through the first reservoir to get the outflow hydrograph .The outflow from the first hydrograph is considered as the input to the second; the outflow from the second reservoir is the input to the third and so on for all the n reservoirs. The conceptual cascade of reservoir as above and the shape of the outflow hydrographs from each reservoir of the cascade is shown in the fig. (3.3). The outflow hydrograph from the nth reservoir is taken as the IUH of the catchment.

Basic Concepts and Equations

From the equation of continuity -

$$\frac{ds}{dt} = I(t) - Q(t) \tag{3-9}$$

For a linear reservoir -

$$S(t) = k_1 Q(t) \tag{3-10}$$

where K_1 is the storage coefficient and it is a constant that confers to the reservoir the property of linearity.

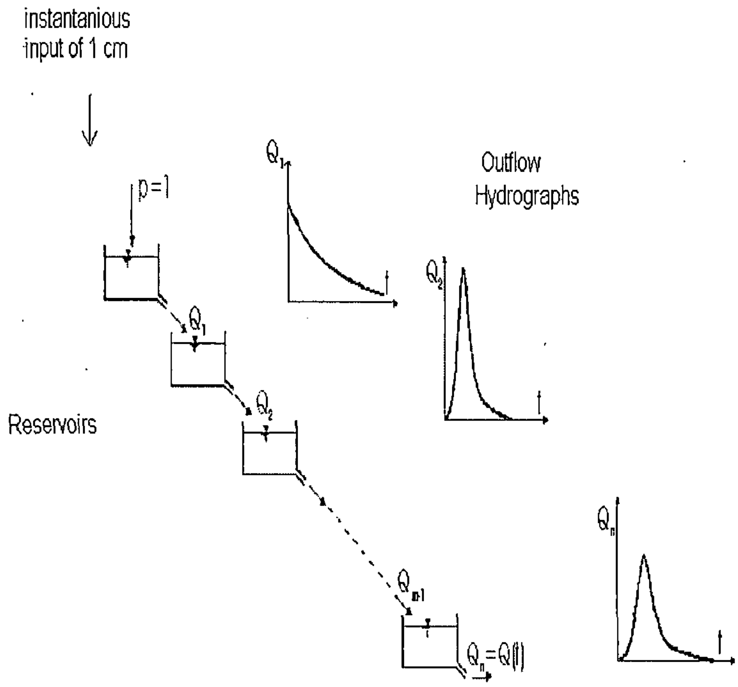

Fig. 3.5: The Nash concept for deriving the instantaneous unit hydrograph

In effect, replacing (3-11) in (3-10) –

$$k_1 \frac{dQ}{dt} + Q = I \qquad (3\text{-}11)$$

Eq 3-12 can also be written as -

$$\frac{dQ}{dt} + \frac{1}{k_1}.Q(t) = \frac{1}{k_1}..I(t) \qquad (3\text{-}12)$$

Considering the equation of convolution the expression of Q (t) is:

$$Q(t) = \int_{0}^{t} I(t).u(t-\tau).dt \tag{3-13}$$

The integration of equation (3-13) is obtained by multiplying both terms from the right and left side with $e^{t/K}$ and one obtains:

$$u(t-\tau) = \frac{1}{k_1}.e^{\left(\frac{-1}{k_1}(t-\tau)\right)} \tag{3-14}$$

Thus the output from the first reservoir becomes the input in the second one and so on. Therefore, replacing t - τ by τ yields:

$$I(t) = \frac{1}{k_1}.e^{\left(\frac{-t}{k_1}\right)} \tag{3-15}$$

If IUH of the second reservoir $= \frac{1}{k_2}.e^{\frac{-t}{k_2}}$

Where k_2 is the storage coefficient of second reservoir, then output from the two linear reservoirs put in series can be computed using convolution equation.

$$u(t) = \int_{\tau=0}^{\tau=t} \frac{1}{k_1}.e^{\frac{-\tau}{k_1}}.\frac{1}{k_2}.e^{-\left(\frac{t-\tau}{k_2}\right)} d\tau \tag{3-16}$$

$$= \int_0^t \frac{1}{k_1} . e^{\frac{-\tau}{k_1}} . \frac{1}{k_2} e^{-\frac{t}{k_2}} e^{\frac{\tau}{k_2}} d\tau$$

$$= \frac{1}{k_1 k_2} . e^{\frac{-t}{k_2}} \int_0^t . e . ^{\frac{(k_1 - k_2)\tau}{k_1 k_2}} d\tau$$

$$= \frac{1}{k_1 k_2} . e^{\frac{-t}{k_2 \{ \}}} \left[e^{\frac{t(k_1 - k_2)}{k_1 k_2}} - 1 \right] \frac{1}{k_1 - k_2} * k_1 k_2$$

On simplification we obtain,

$$U(t) = \frac{1}{k_1 - k_2} \left(e^{-\frac{t}{k_1}} - e^{\frac{-t}{k_2}} \right) \qquad (3\text{-}17)$$

When storage coefficients of all linear reservoirs of the system are same i.e., $k_1 = k_2 .... = k$ then IUH of system is derived by replacing $k_1 = k_2 = k$ in eq. (3-17)

$$u(t) = \int_0^t \frac{1}{k} e^{\frac{-\tau}{k}} . \frac{1}{k} e^{\frac{-(t-\tau)}{k}} d\tau \qquad (3\text{-}18)$$

$$u(t) = \frac{t}{k^2} e^{\frac{-t}{k}} \qquad (3\text{-}19)$$

When three reservoirs are placed in series then similarly-

$$u(t) = \frac{t^2}{k^3} e^{\frac{-t}{k}} \qquad (3\text{-}20)$$

When n numbers of reservoirs are placed in series, then

$$u(t) = \frac{t^{n-1}}{k^n} \frac{1}{(n-1)!} e^{\frac{-t}{k}} \tag{3-21}$$

$$u(t) = \frac{1}{k}\left(\frac{t}{k}\right)^{n-1} \frac{1}{(n-1)!} e^{\frac{-t}{k}} \tag{3-22}$$

Where n is an integer value. To include non-integer n values, Nash (1957) substituted the factorial by Gamma function and defined the model as

$$u(t) = \frac{1}{k}\left(\frac{t}{k}\right)^{n-1} \frac{1}{\Gamma n} e^{\frac{-t}{k}} \tag{3-23}$$

The unit hydrograph of duration T is computed as (Nash, 1957),

$$U(t) = \frac{1}{T}\left[I(n, t/k) - I(n, (t-T)/k)\right] \tag{3-24}$$

Where, *I(n, t/k)* is the incomplete gamma function of order *n* at *(t/k)*.

3.4) Estimation of Model Parameters:

Both SCS dimensionless unit hydrograph and Clark model use time of concentration as a parameter. For SCS dimensionless unit hydrograph initial value of t_C

for each sub-watershed was calculated using a time of concentration formula proposed by Kirpitch (1940):

$$t_C = 0.06628[L/\ S^{0.5}]^{0.77} \tag{3-25}$$

where t_c is in hours, L is length of travel in Km from the most remote point on the drainage basin along the drainage channel to the basin outlet, and S is the slope in Km per km determined by the differences in elevation of the most remotest point and that of the outlet. The value of t_c is further calibrated using some of the storm events by optimization.

For time area relationship, t_c can be estimated via calibration. The basin storage coefficient, R, is an index of the temporary storage of precipitation excess in the watershed as it drains to the outlet point. It, too, can be estimated via calibration if gaged precipitation and stream flow data are available. Though R has units of time, there is only a qualitative meaning for it in the physical sense. Clark (1945) indicated that R can be computed as the flow at the inflection point on the falling limb of the hydrograph divided by the time derivative of flow

In case of Nash model, the parameter n and k have been estimated using method of moments. For a given ERH and the corresponding DRH, Nash (1960) has given the expressions to determine the IUH parameters n and k. In the present case, the parameters n and k can be determined using the moments of single representative ERH and DRH obtained from processing of multistorm data. The first moment M_1 represents the lag time of centroid of IUH. This is the same as the difference of the moments of DRH and ERH from the centroid.

The first moment of IUH about origin, i.e., when t=0,

$$M_1 = M_{DRH1} - M_{ERH1} = nk \tag{3-26}$$

where $M_{DRH\,1}$ and $M_{ERH\,1}$ are first moment of the DRH and ERH about origin.

The second moment of IUH about the centroid is given as,

$$M_2 = M_{DRH2} - M_{ERH2} = nk^2 \tag{3-27}$$

Where $M_{DRH\,2}$ and $M_{ERH\,2}$ are second moment of the DRH and ERH about Centroid. Solving equation (3-21) and (3-22),

$$k = \frac{M_{DRH2} - M_{ERH2}}{M_{DRH1} - M_{ERH1}} = \frac{M_2}{M_1} \tag{3-28}$$

$$n = \frac{(M_{DRH1} - M_{ERH1})^2}{M_{DRH2} - M_{ERH2}} = \frac{M_1^{\,2}}{M_2} \tag{3-29}$$

Using equation (3-29) and (3-30) values of n and k can be calculated.

Use of method of moments results in different values of n and k for different storm events. However for practical applications, a representative value of n and k is often required. In the present study representative values are obtained by 1) taking average of values of n and computed for each of the storm event and 2) processing multistorm data into single representative ERH and DRH described as under.

Processing of Multistorm Data

Multistorm data are processed to get a single representative ERH and corresponding DRH. The peaks of the DRHs are made to coincide or align. For this, the time axes of all DRHs are shifted so that their peaks coincide. In order that the linearity is preserved, similar shifts are also exercised for ERHs corresponding to the respective DRHs. Since all DRHs are equally important irrespective of the high or low peaks, these should be given equal weight while obtaining the representative storm. Hence, the shifted DRHs are normalized by their respective peaks. In this way, the peaks of all storms are made equal or have equal weights. Thus, the high or low peaks are treated equivalently. In order that the linear property of the system is maintained, the shifted ERHs are also normalized by the peaks of the corresponding DRHs.

The ordinates of these modified ERHs and DRHs are added separately and an average ERH and the corresponding DRH are obtained, which are termed "representative ERH and DRH or representative storm." This linear transform that is used to get the representative storm does not contradict or disturb the linear property of the ERH–DRH. The linear transform used to preprocess the ERH and DRH data of multistorm is written as,

$$\mathrm{NRRE}_{j+\gamma_i} = \frac{1}{M}\sum_{i=1}^{M} \frac{\mathrm{RE}_{i,j}}{(\mathrm{DR}_{\mathrm{peak}})_i} \tag{3-30}$$

$$\mathrm{NRDR}_{j+\gamma_i} = \frac{1}{M}\sum_{i=1}^{M} \frac{\mathrm{DR}_{i,j}}{(\mathrm{DR}_{\mathrm{peak}})_i} \tag{3-31}$$

where DR = ordinate of direct runoff (LT^{-1}); RE = intensity of effective rainfall (LT^{-1}); i = index for storms (dimensionless); j=index for ordinate of DRH (dimensionless); M = number of storms (dimensionless); NRRE=normalized representative effective rainfall intensity (dimensionless); NRDR = normalized representative direct runoff (dimensionless); and γ_i = shifting in terms of number of time steps in both DRH and ERH of ith storm (dimensionless), its value is selected for each storm so that the peaks of all the DRHs are coincident.

In this way, the proportionate ordinates of the DRHs are averaged since the aligned DRHs are made to have the equal peaks in the peak aligning technique; the IUH obtained using the normalized representative ERH and DRH could be more appropriate for the design of the hydraulic structures.

CHAPTER IV

THE STUDY AREA AND DATA AVAILABILITY

For the present study, watershed defined at railway bridge no. 719 on the Jolarpet-Bangalore section of the Indian railways (southern section) is taken up. The study watershed lies between longitude 78^0 15' and 78^0 20' of east and latitude 12^0 52'to 12^0 55' north. Total catchment area of the watershed at railway bridge no. 719 is 14.376 km^2. Fig. 4.1 shows the study watershed with drainage lines.

The study watershed is covered in survey of India topographic sheet no. 57L/5.Catchment information such as drainage, contours and spot heights have been taken from SOI toposheets. The soil in the watershed is mainly red in color. The watershed is having a mild relief of 136 m. The study watershed falls under humid climate zone. Mostly dry land farming is practiced in the watershed.

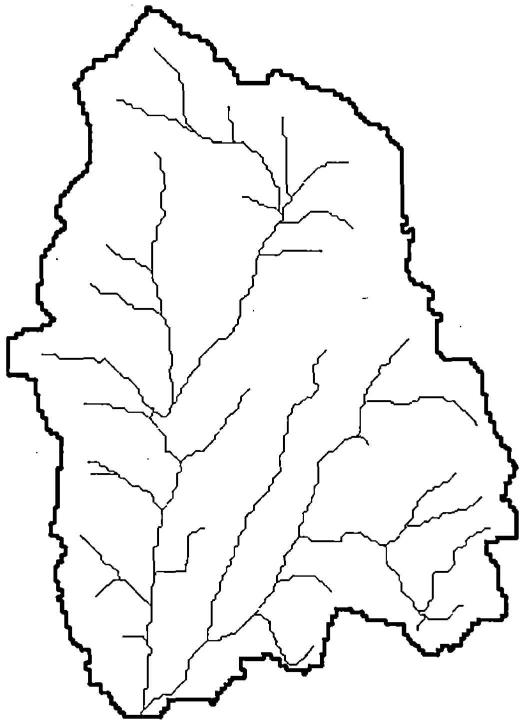

Fig.4.1 Study watershed with drains

4.1 DATA AVAILABILITY

Storm data at half Hourly interval for rainfall and corresponding runoff for the years 1964-1966 was available. Rainfall events of 24.07.1964, 26.07.1964, 3.09.1964, 11.08.1965, 16.09.1966 and 19.09.1966 were considered for the simulation of runoff hydrographs. Available storm data were processed to separate base flow and direct surface runoff to carryout unit hydrograph analysis. The excess rainfall was computed using phi index method. Information about storm runoff, rainfall and excess rainfall computed for each storm is given in table 4.1 – 4.6.

Table 4.1: Storm event dated: 24 July 1964

Time	Discharge	Time	Rainfall	Phi index	Rainfall excess
(HR)	(Cumec)	(HR)	(mm)	(mm/HR)	(mm/ 0.5HR)
0.0	1.16	0.0-0.5	7.55	13.18	0.96
0.5	1.53	0.5-1.0	7.00		0.41
1.0	2.04	1.0-1.5	9.00		2.41
1.5	3.65	1.5-2.0	2.45		5.41
2.0	4.56	2.0-2.5	6.00		8.41
2.5	8.10	2.5-3.0	12.00		
3.0	9.77	3.0-3.5	15.00		
3.5	12.89	3.5-4.0	4.00		
4.0	21.24				
4.5	25.25				
5.0	21.24				
5.5	16.99				
6.0	12.60				
6.5	8.69				
7.0	5.58				
7.5	4.42				
8.0	2.41				
8.5	1.16				

Table 4.2: Storm event dated: 26 July 1964

Time (Hr)	Discharge (Cumec)	Time (Hr)	Rainfall (Mm)	P Index (mm/Hr)	Rainfall Excess (mm/ 0.5hr)
0.0	1.20	0.0-0.5	6.00	6.63	2.685
0.5	2.50	0.5-1.0	8.00		4.685
1.0	4.60	1.0-1.5	3.84		0.525
1.5	6.60	1.5-2.0	2.36		2.975
2.0	8.50	2.0-2.5	6.29		5.585
2.5	11.80	2.5-3.0	8.90		9.685
3.0	13.93	3.0-3.5	13.00		
3.5	18.21				
4.0	25.50				
4.5	33.84				
5.0	28.20				
5.5	22.14				
6.0	19.09				
6.5	16.09				
7.0	8.61				
7.5	6.88				
8.0	5.52				
8.5	3.65				
9.0	2.57				
9.5	1.2				

Table 4.3: Storm event dated 3 Sep 1964

Time (HR)	Discharge (Cumec)	Time (HR)	Rainfall (mm)	p index (mm/HR)	Rainfall excess (mm/ 0.5HR)
0.0	1.13	0.0-0.5	2.36	13.845	1.158
0.5	2.13	0.5-1.0	1.96		2.708
1.0	3.96	1.0-1.5	8.08		4.488
1.5	5.09	1.5-2.0	9.63		
2.0	6.79	2.0-2.5	11.41		
2.5	8.49				
3.0	11.61				
3.5	10.76				
4.0	8.49				
4.5	6.85				
5.0	5.81				
5.5	4.43				
6.0	3.26				
6.5	2.65				
7.0	2.41				
7.5	1.70				
8.0	1.13				

Table 4.4: Storm event dated: 11 Aug 1965

Time (HR)	Discharge (Cumec)	Time (HR)	Rainfall (mm)	p index (mm/HR)	Rainfall excess (mm/ 0.5HR)
0.0	1.10	0.0-0.5	39.37	42.95	17.9
0.5	7.50	0.5-1.0	28.35		6.88
1.0	19.31	1.0-1.5	0.00		0.00
1.5	27.1	1.5-2.0	2.36		0.00
2.0	41.06	2.0-2.5			
2.5	31.86				
3.0	28.32				
3.5	21.83				
4.0	13.48				
4.5	10.22				
5.0	6.93				
5.5	2.09				
6.0	1.47				
6.5	1.10				

Table 4.5: Storm event dated: 16 Sep1966

Time (HR)	Discharge (Cumec)	Time (HR)	Rainfall (mm)	p index (mm/HR)	Rainfall excess (mm/ 0.5HR)
0.0	15.43	0.0-0.5	24	15.32	16.34
0.5	16.62	0.5-1.0	35		27.34
1.0	37.52	1.0-1.5	11		3.34
1.5	97.37	1.5-2.0			
2.0	86.47	2.0-2.5			
2.5	71.91				
3.0	57.35				
3.5	50.07				
4.0	40.95				
4.5	37.52				
5.0	31.86				
5.5	29.45				
6.0	25.97				
6.5	23.64				
7.0	22.37				
7.5	22.37				

Table 4.6: Storm event dated: 19 Sep 1966

Time (HR)	Discharge (Cumec)	Time (HR)	Rainfall (mm)	p index (mm/HR)	Rainfall excess (mm/ 0.5HR)
0.0	7.60	0.0-0.5	16.40	19.12	6.84
0.5	19.40	0.5-1.0	34.64		25.08
1.0	32.86	1.0-1.5			
1.5	46.42	1.5-2.0			
2.0	41.95	2.0-2.5			
2.5	37.52				
3.0	31.86				
3.5	28.32				
4.0	25.97				
4.5	22.30				
5.0	19.80				
5.5	18.50				
6.0	15.03				
6.5	14.50				
7.0	13.00				
7.5	12.03				
8.0	12.00				
8.5	10.50				
9.0	10.00				

CHAPTER V

GENERATION OF DIGITAL GEO DATABASE AND GEOPROCESSING

For the present study, HEC-HMS model has been used for modeling. Another model HEC-GeoHMS is used to create input files for HEC-HMS model. HEC-GeoHMS is an extension of Arc-View GIS. Using GIS capability a DEM (digital elevation model) has been prepared. HEC-GeoHMS operates on that DEM to derive sub basin delineation and to prepare a number of hydrologic inputs. HEC-HMS accepts these hydrologic inputs as a starting point for hydrologic modeling. The relation between GIS, HEC-GeoHMS and HEC-HMS is shown in fig (5.1).

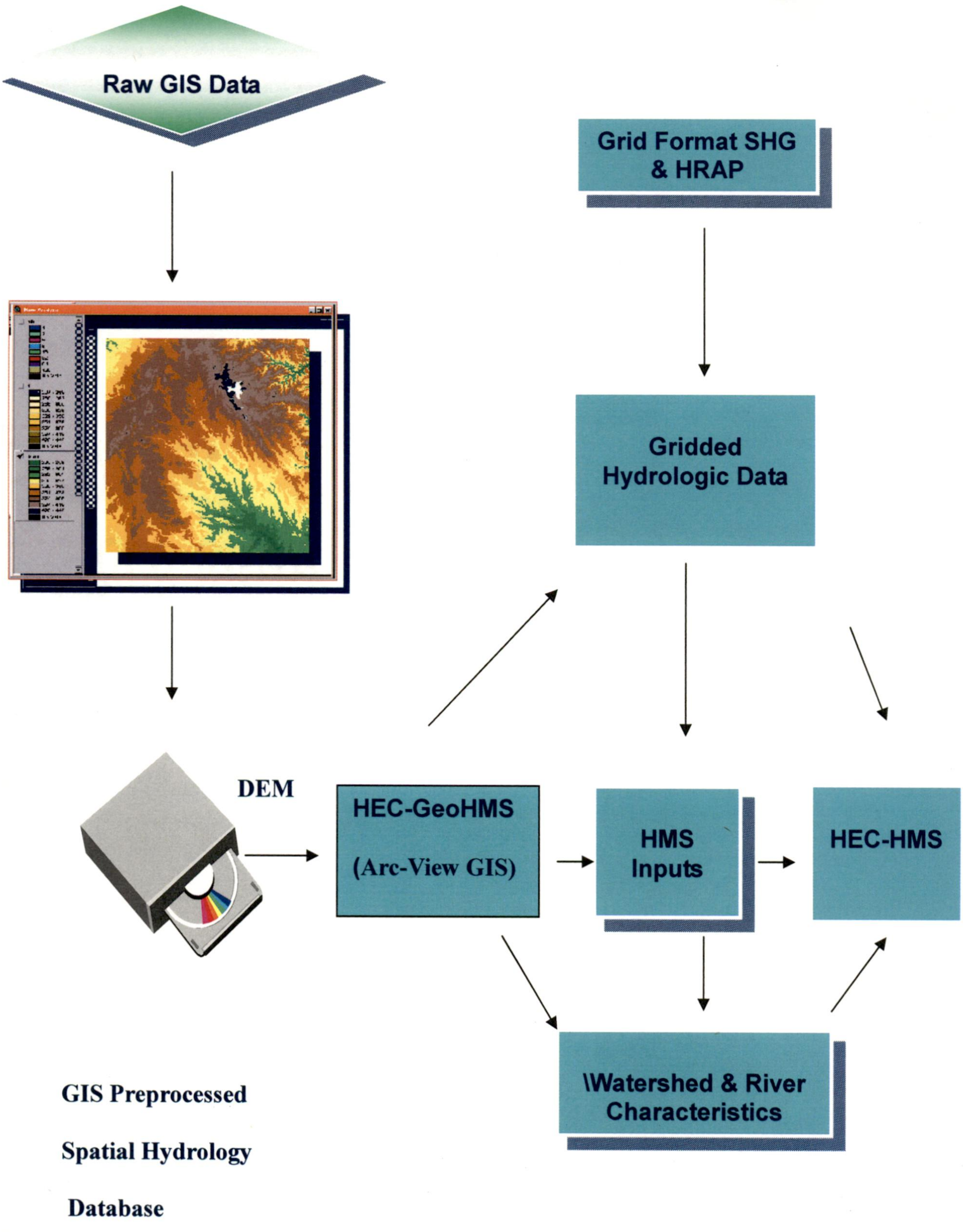

Fig. 5.1: Relation among GIS, HEC-GeoHMS and HEC-HMS.

5.1 STEPS FOR HEC-GEOHMS PROCESSING

Following are the different steps of HEC-GeoHMS, used to create input network files for the HEC-HMS model. The two main steps of the HEC-GeoHMS processing are

1) Terrain preprocessing

2) Hydrologic processing

DEM, prepared using Arc-View GIS, has been reconditioned for terrain preprocessing using TERRAIN RECONDITIONING option. This function creates a gradual transition from the over bank to the stream centerline in the DEM for water to enter the stream. The result is Agree DEM shown in fig (5.2).

5.2.1. Terrain Preprocessing

a) Fill sink:

This option is used to make DEM depressionless. 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 because pits are considered as errors in the DEM due to interpolating grids. Fig (5.3) shows depressionless DEM.

b) Flow directions:

The step is to compute the flow direction of the steepest descent for each cell. The result of flow direction operation is shown in fig (5.4).

c) Flow accumulation:

This step determines the number of up stream cells draining to a given cell. The result of flow accumulation is shown in fig (5.5).

d) Stream definition:

This step classifies all cells with flow accumulation greater than the user-defined threshold value 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.

e) Stream segmentation:

This step divides streams 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 stream segments as shown in fig (5.6).

f) Watershed delineation:

This step delineates a sub basin or watershed for every stream segment.

g) Watershed polygon processing:

This step converts sub basin in the grid representation into a vector representation. Fig (5.7) shows the results of this operation.

h) Stream segment processing: this step converts streams in the grid representation into a vector representation. The stream processing operation vectorized grid based streams into line vectors as shown in the fig (5.8)

i) Watershed aggregation:

This step aggregates the upstream sub basin at every stream confluence. This step is performed to improve computational performance for interactively delineating sub basin and to enhance data extraction. This step does not have any hydrologic significance.

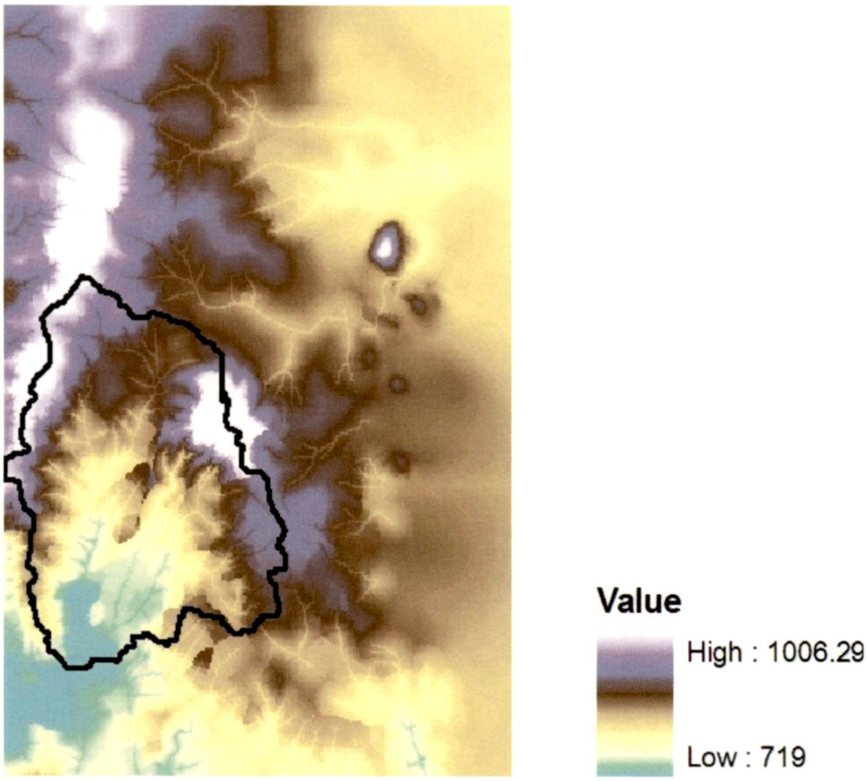

Fig. 5.2: Reconditioned DEM

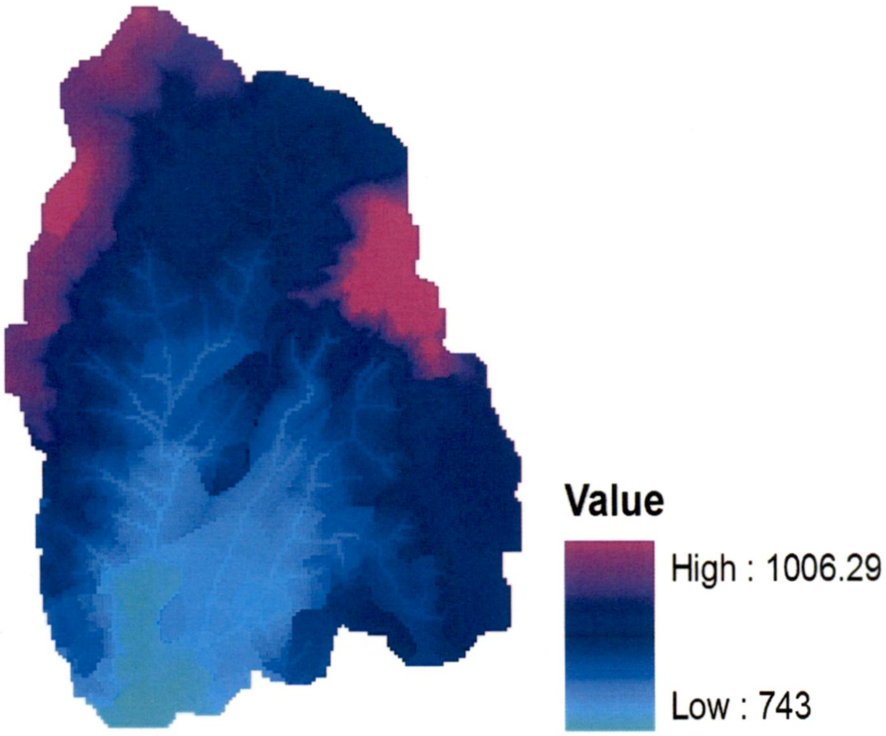

Fig. 5.3: Depressionless DEM

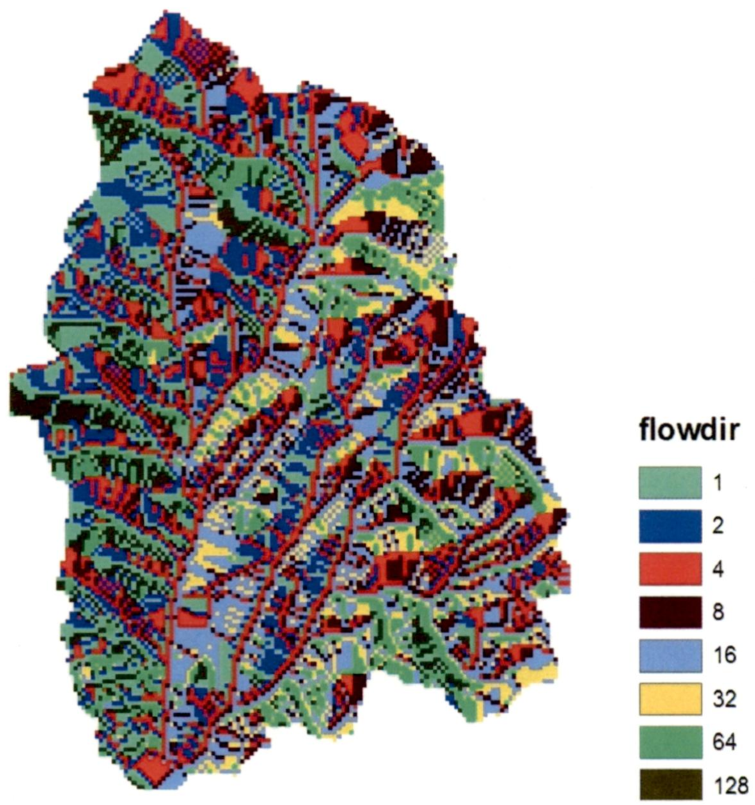

Fig.5.4: Flow direction operation result

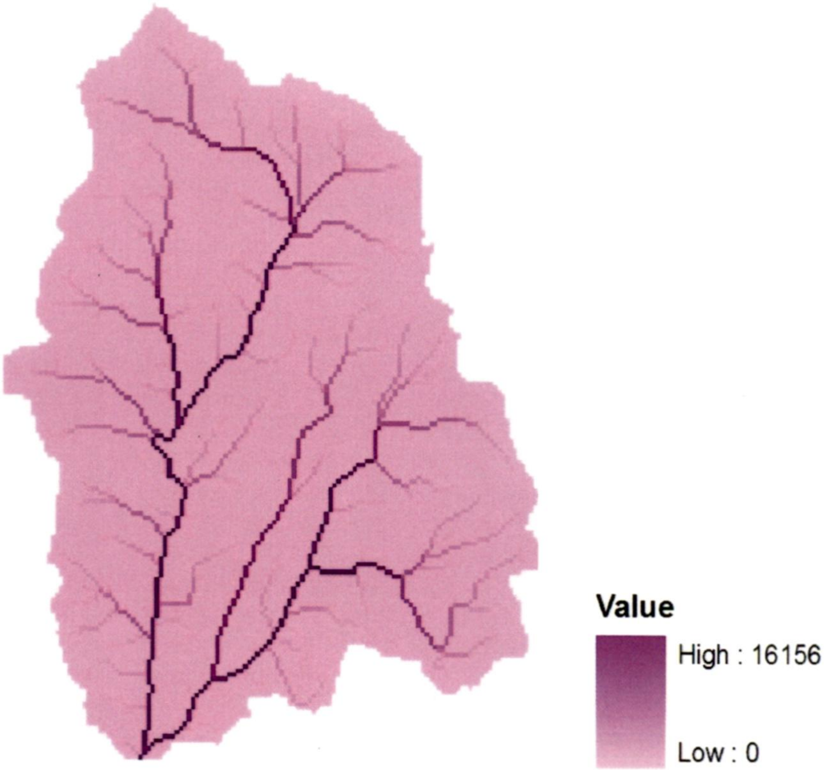

Fig.5.5: Slow accumulation operation result

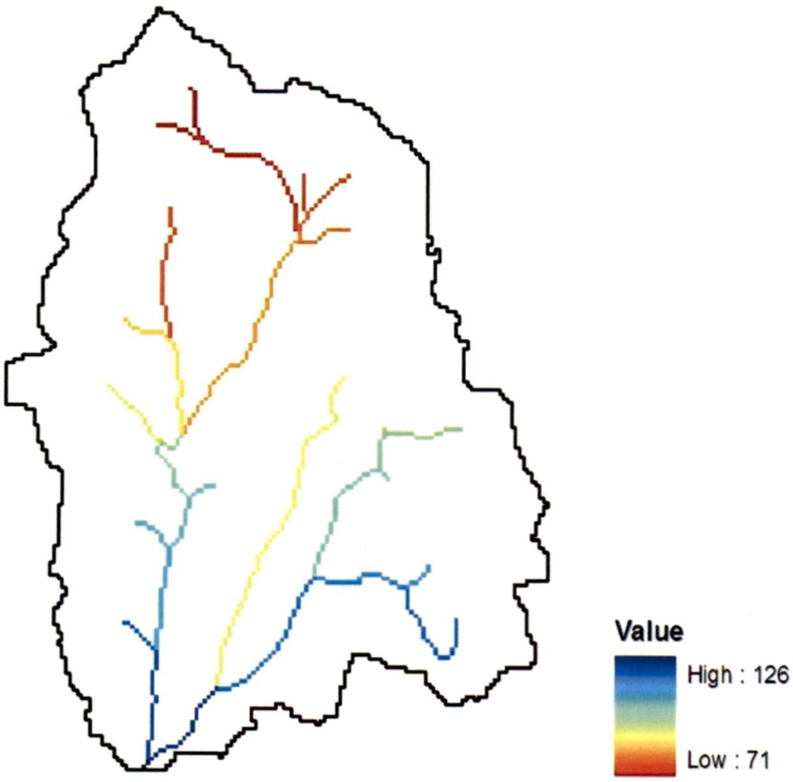

Fig.5.6. Stream segmentation operation result

Fig. 5.7: Watershed polygon processing result

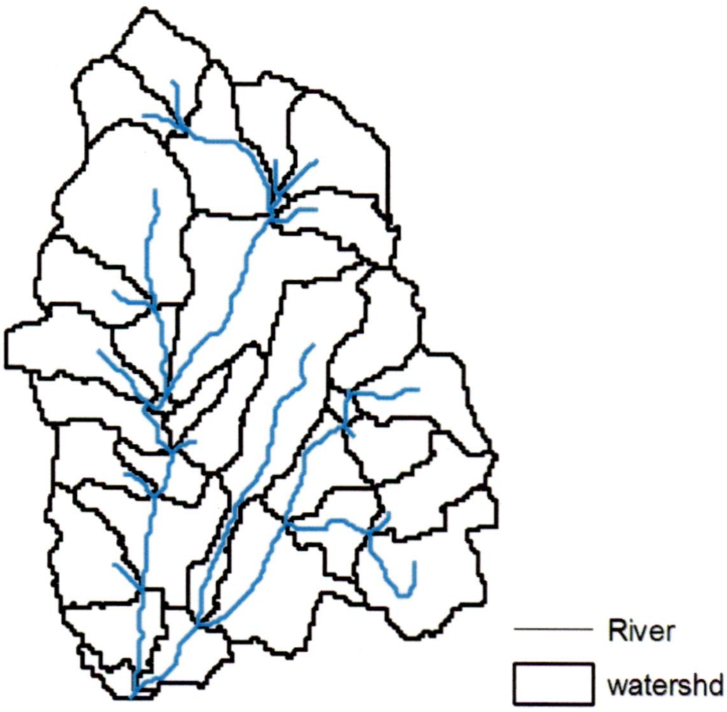

Fig. 5.8: Watershed stream segment processing results

5.2.2 Hydrologic processing

a) Basin Processing

- **Basin merge:** This function merges multiple sub basins. In the present case multiple sub basins has been merged in a single basin and in two sub basins. Results are shown in fig (5.9) and fig (5.10).

HEC-GeoHMS computes several topographic characteristics of streams and watershed. These characteristics are useful for comparing of basins and for estimating hydrologic parameters.

b) Basin characteristics

- River length
- River slope
- Basin centriod
- Longest flow path
- Centroidal flow path

Applying these steps streams and watershed physical characteristics are determined. And these characteristics are stored in attribute tables, which can be exported for use with a spread sheet or other program.

- Results of River length and River slope operations are stored in the attribute tables.

- **Basin centriod:** This function estimates the centroid of the basin. Results are shown in fig (5.11) and fig (5.12).

- **Longest flow path:** The flow path method draws the longest flow path for the sub basin and approximates the centroid as the midpoint on the longest flow path. Results can be seen in fig (5.13) and fig (5.14).

- **Centroidal flow path:** This operation computes the centroidal flow path length by projecting the centroid onto the longest flow path. The centroid flow path is measured from the projection point on the longest flow path to the sub basin outlet. Result is shown in fig (5.15) and fig (5.16).

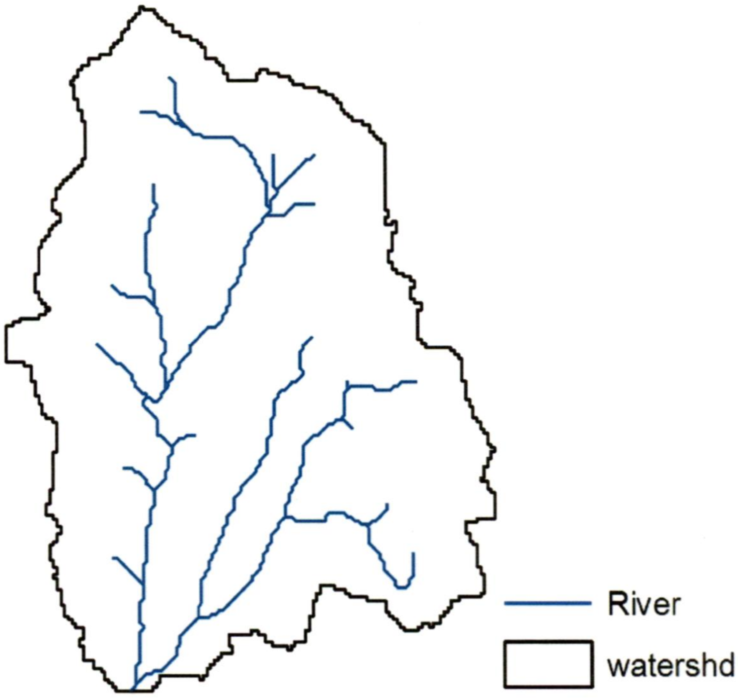

Fig 5.9: Basin merge result (single basin)

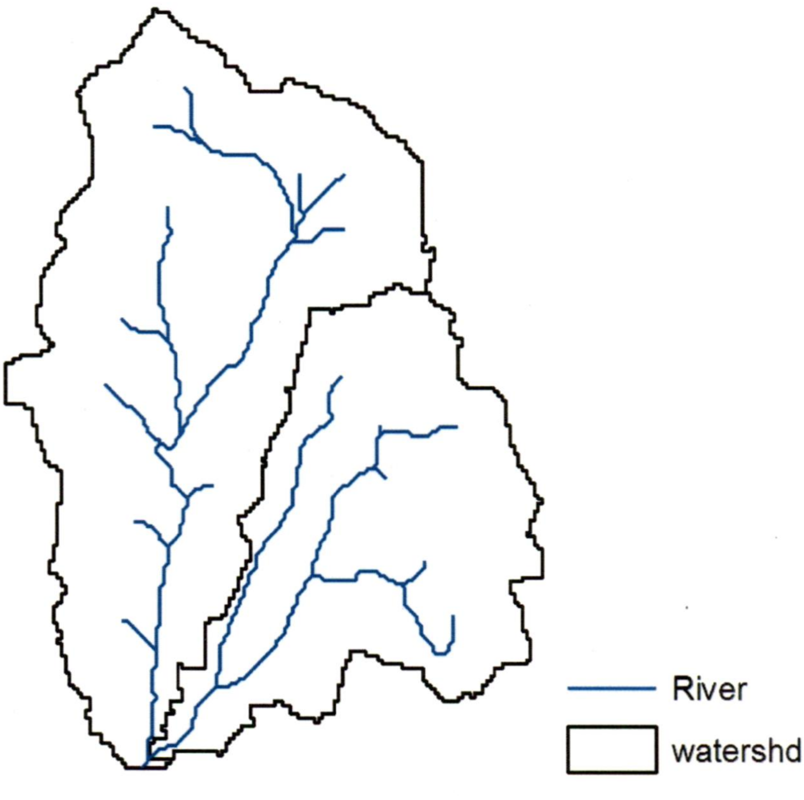

Fig 5.10: Basin merge result (two sub basin)

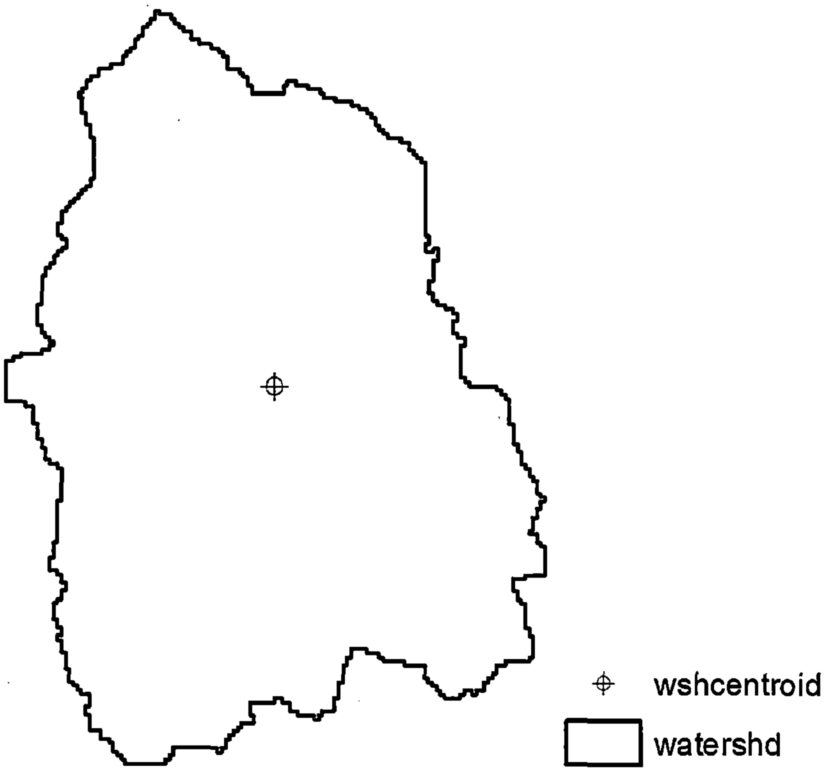

Fig 5.11: Single basin centroid result

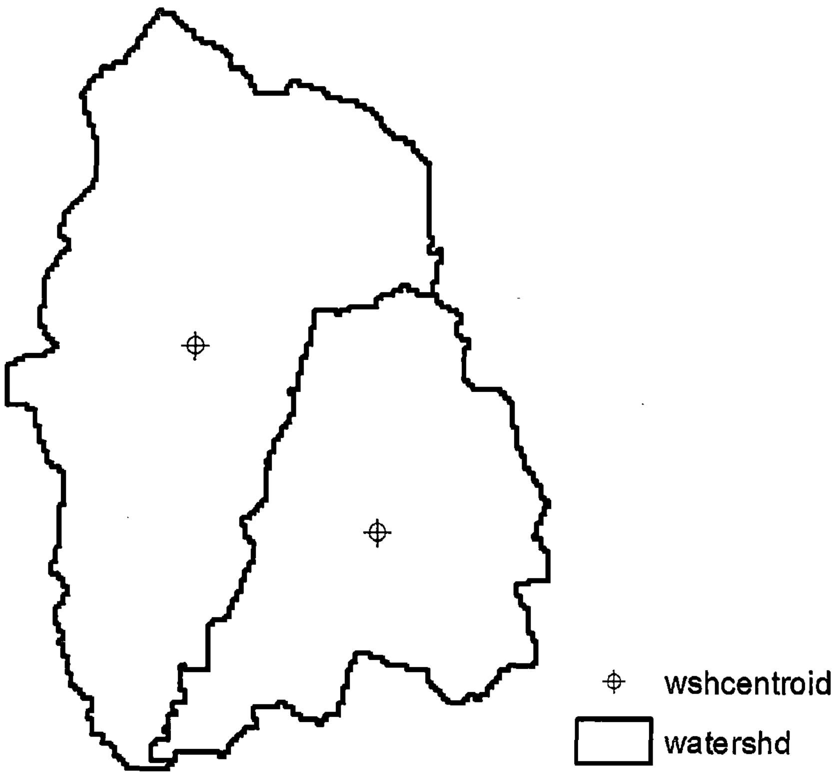

Fig 5.12: Two sub basin centroid result

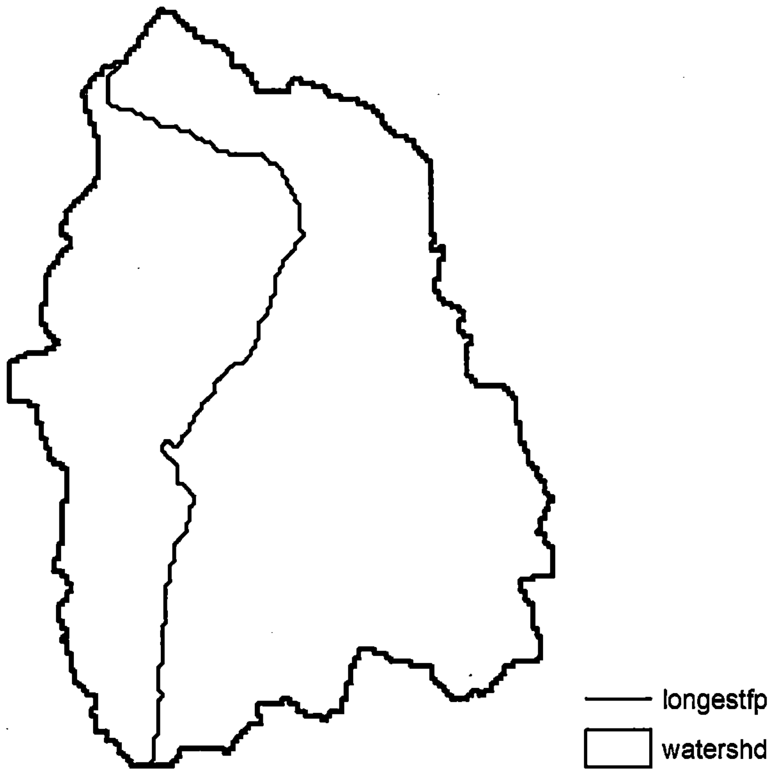

Fig 5.13: Longest flow path result for single basin

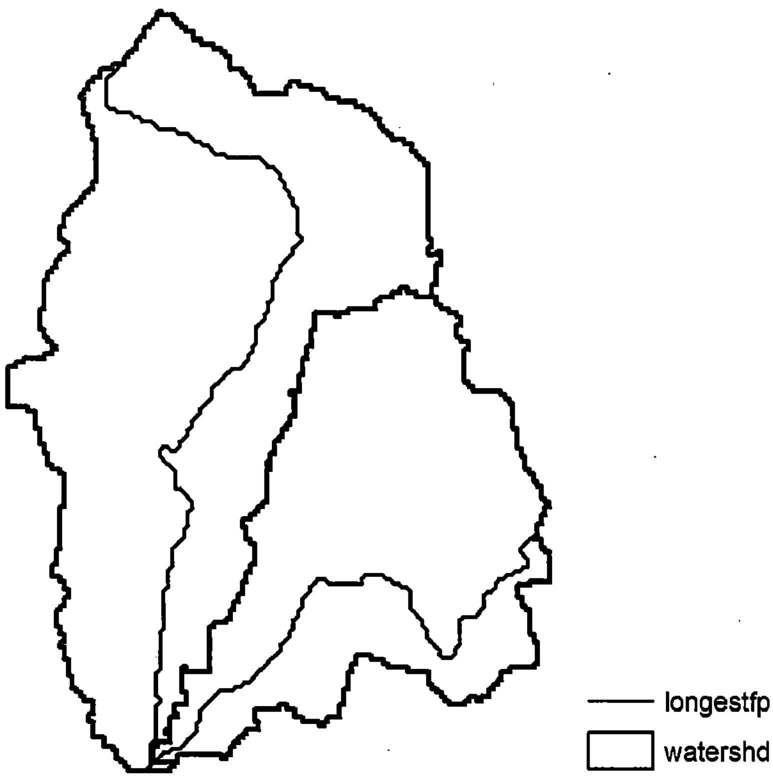

Fig 5.14: Longest flow path result for two sub basins

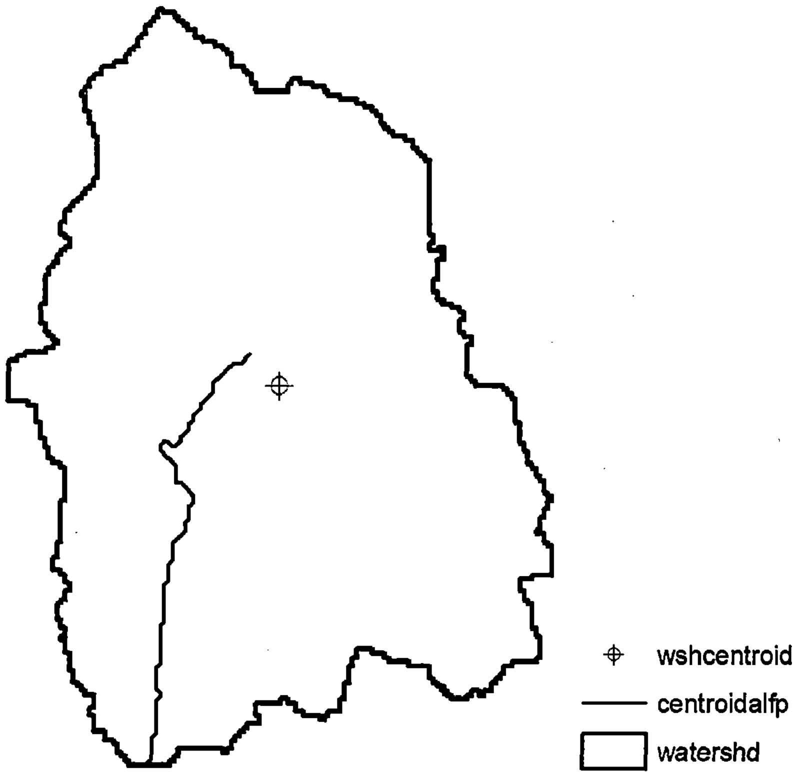

Fig 5.15: Centroidal flow path result for single basin

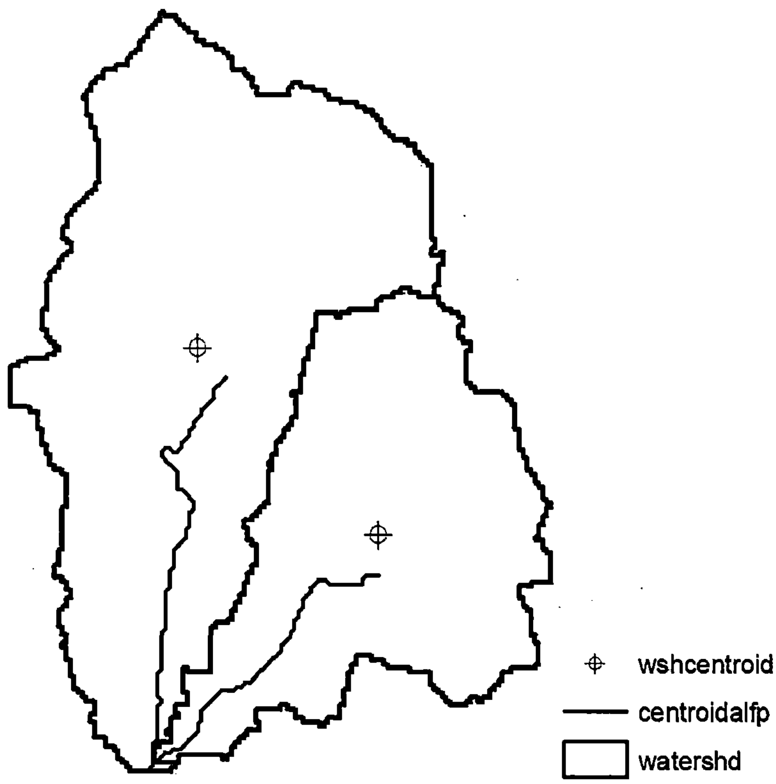

Fig 5.16: Centroidal flow path result for two sub basins

b) HMS: This menu performs a number of tasks related to HMS. It includes assigning default names for the reaches and sub basins, unit conversions, checking, and creation of the basin schematic, and HMS files generation. Following are operations performed. Results are shown on fig (5.17).

- Reach Autoname
- Basin Autoname
- Map to HMS units
- HMS data check
- HEC-HMS basin schematic
- HMS Legend
- Add coordinate
- Standard HMS processes
- Background map file
- Lumped basin model
- HMS project setup

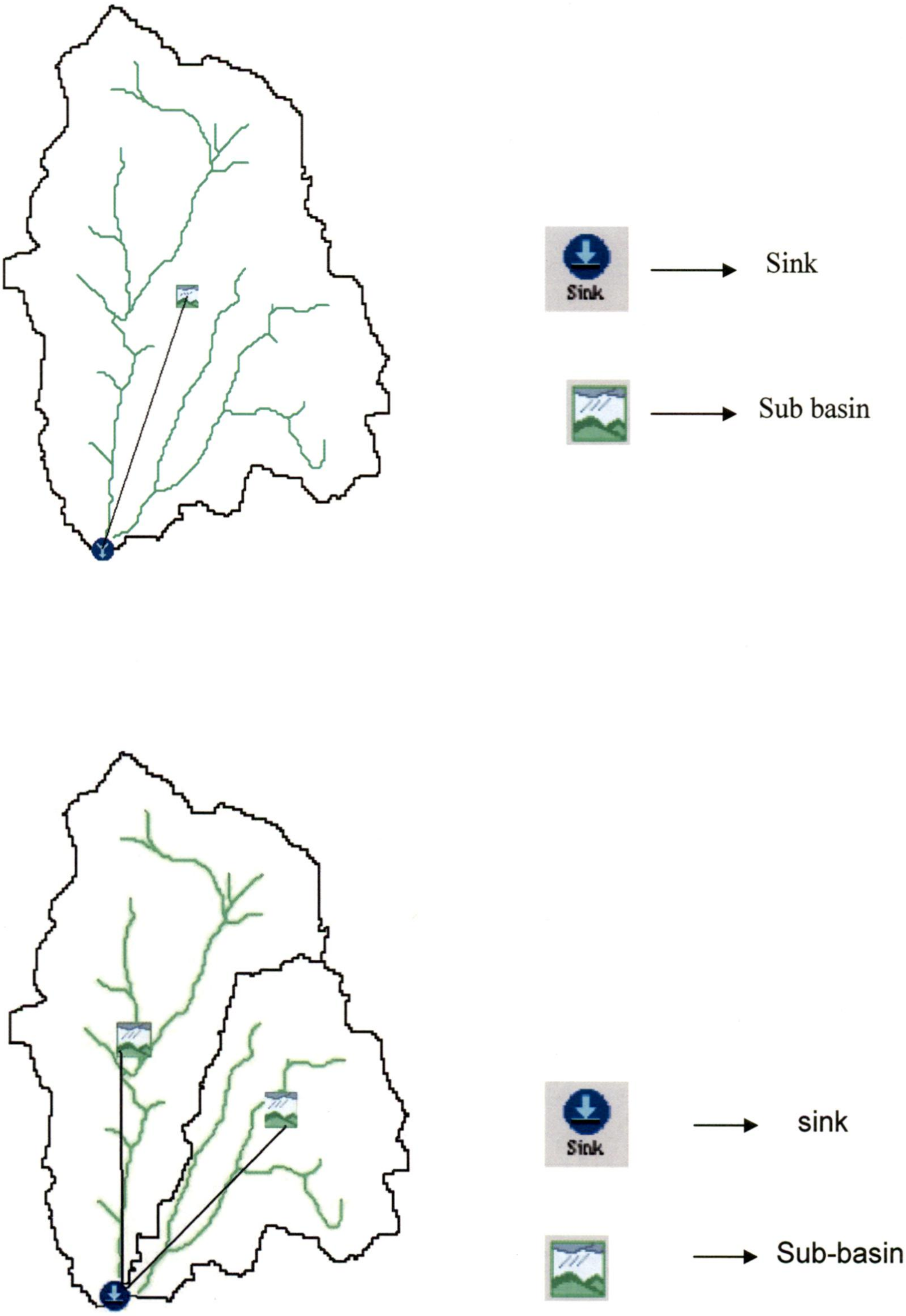

Fig. 5.17: HMS Schematic for single and two sub-basins with symbols

CHAPTER VI

ANALYSIS AND DISCUSSION OF RESULTS

6.1 PARAMETER ESTIMATION

Synthetic hydrographs used in this study requires evaluation of model parameters. The SCS dimensionless unit hydrograph method consists of a single parameter t_l (time lag) which can be related to basin time of concentration. Clark model uses two parameters, time of concentration and storage coefficient. Using basin physical properties such as longest length and slope, initial estimate for time of concentration were made using Kirpitch formula (3-25) described previously. Other parameter, storage coefficient in Clark model can be computed using analysis of recession characteristics of observed hydrographs. However in the present study, final values of time of concentration and storage coefficient were evaluated using optimized in HEC-HMS. To optimize model parameters for SCS dimensionless unit hydrograph and Clark models, flood events occurred on 24.07.1964, 26.07.1964 and 11.08.1965 were used. Optimized values of model parameters for SCS dimensionless unit hydrograph and Clark model that gave minimum value of peak weighted root mean square error (eq. 6.2) are given in table 6.1.

For avaible six storm events parameters N and K for Nash model were computed for each event, with the method of moments (eq.3-28, 3-29). Values of N and K for all six events are shown in table 6.2. Average values of N and K is then computed to simulate different storm events.

Table 6.1 Estimated values of model parameters

SCS Dimensionless UH	Clark model	
Lag time	Time of concentration	storage coefficient
1.33	1.50	1.40

As described earlier, representative value of parameters N and K were also estimated using single representative ERH and DRH obtained using multistorm processing described in section (3.4). Using eq. (3-19) and (3-20).Available multistorm data are processed to get a single representative ERH and DRH. Table 6.3 shows ordinates of RDRH and table 6.4 shows the coordinates of RERH. Now using the method of moments, representative values of parameter N and K were computed as N=3.82 and K=0.69 hrs.

Table: 6.2 Values of n and k with Method of Moments

Events	No. of reservoirs (n)	Storage coefficient (k)
24-Jul-64	5.190	0.56
26-Jul-64	3.882	0.66
3-Sep-64	1.360	1.24
11-Aug-65	4.400	0.53
16-Sep-66	3.134	0.70
19-Sep-66	1.970	1.24
average	3.320	0.82

Table: 6.3 Representative Normalized RDRH

Time (HR)	Discharge (Cumecs)	Time (HR)	Discharge (Cumecs)	Time (HR)	Discharge (Cumecs)
0.00	0.000	4.25	0.821	8.50	0.097
0.25	0.043	4.50	1.000	8.75	0.075
0.50	0.003	4.75	0.925	9.00	0.055
0.75	0.007	5.00	0.849	9.25	0.042
1.00	0.013	5.25	0.769	9.50	0.029

1.25	0.024	5.50	0.688	9.75	0.023
1.50	0.035	5.75	0.610	10.00	0.018
1.75	0.056	6.00	0.532	10.25	0.014
2.00	0.077	6.25	0.471	10.50	0.011
2.25	0.112	6.50	0.408	10.75	0.011
2.50	0.146	6.75	0.346	11.00	0.011
2.75	0.181	7.00	0.282	11.25	0.007
3.00	0.215	7.25	0.244	11.50	0.003
3.25	0.301	7.50	0.208	11.75	0.002
3.50	0.387	7.75	0.171	12.00	0.000
3.75	0.514	8.00	0.133		
4.00	0.643	8.25	0.115		

Table: 6.4 Representative Normalized RERH

Time(HR)	RERH(MM)	Time(HR)	RERH(MM)
0.00	0.013	2.25	0.233
0.25	0.013	2.50	0.349
0.50	0.033	2.75	0.349
0.75	0.033	3.00	0.282
1.00	0.081	3.25	0.282
1.25	0.081	3.50	0.327
1.50	0.117	3.75	0.327
1.75	0.117	4.00	0.014
2.00	0.233	4.25	0.014

6.2 Criterion of Evaluation of Model Results

Results obtained using different models are evaluated based on visual comparison as well as on the basis of three different statistical criteria. The statistical criteria used are detailed below.

6.2.1 Nash -Sutcliffe Efficiency

The Nash-Sutcliffe model efficiency coefficient is used to assess the predictive power of hydrological models. It is defined as:

$$\eta = \frac{\left[\sum_{i=1}^{N} \left(Q_{O,i} - \overline{Q_{O,i}}\right)^2 - \sum_{i=1}^{N} \left(Q_{O,i} - Q_{e,i}\right)^2\right]}{\sum_{i=1}^{N} \left(Q_{o,i} - \overline{Q_{o,i}}\right)^2} * 100 \qquad (6.1)$$

where $Q_{o,i}$ is ith observed data, and $\overline{Q_{o,i}}$ is averaged value of observed data and $Q_{e,i}$ is the estimated value.

Nash-Sutcliffe efficiencies can range from -∞ to 1. An efficiency of 1 (E=1) or $\eta = 100\%$ corresponds to a perfect match of modeled discharge to the observed data. An efficiency of 0 (E=0) indicates that the model predictions are as accurate as the mean of the observed data, whereas an efficiency less than zero ($-\infty < E < 0$) occurs when the observed mean is a better predictor than the model.

Essentially, the closer the model efficiency to 1, the more accurate the model is. It should be noted that Nash-Sutcliffe efficiencies can also be used to quantitatively describe the accuracy of model outputs other than discharge. This method can be used to describe the predicative accuracy of other models as long as there is observed data to compare the model results to.

6.2.2 Peak Weighted RMS Error

The function Peak Weighted RMS Error (USACE, 1998) compares all ordinates, squaring differences, and it weights the squared differences. The weight assigned to each ordinate is proportional to the magnitude of the ordinate. Ordinates greater than the mean of the observed hydrograph are assigned a weight greater than 1.00, and those smaller, a weight less than 1.00. The peak observed ordinate is assigned the maximum weight. The sum of the weighted, squared differences is divided by the number of computed hydrograph ordinates; thus, yielding the mean squared error. Taking the square root yields the root mean squared error. This function is an implicit measure of comparison of the magnitudes of the peaks, volumes, and times of peak of the two hydrographs.

$$\text{PWRMSE} = \left\{ \frac{1}{NQ} \left[\sum_{i=1}^{NQ} \left(q_0(i) - q_s(i)\right)^2 \left(\frac{q_0(i) + q_0(mean)}{2q_0(mean)} \right) \right] \right\}^{0.5} \tag{6.2}$$

where NQ = number of computed hydrograph ordinates; $q_o(t)$ =observed flows; $q_S(t)$ = calculated flows, computed with a selected set of model parameters; $q_o(peak)$ = observed peak; $q_o(mean)$ = mean of observed flows; and $q_S(peak)$= calculated peak

6.2.3 Standard Error

For evaluation of model performance, the standard error of estimates (S_e) is considered,

$$S_e = \left[\sum_{i=1}^{N} \frac{(O_s - O_i)}{(N - m)} \right]^{0.5} \tag{6.3}$$

where O_i = ith observed data; O_s = ith stimulated value; m= number of model parameter; N= total number of observation. It is a better goodness-of-fit measure. A lower value of S_e indicates better model performance, and vice-versa.

6.3 Discussion of Results

Using the parameter evaluated earlier, different storm events were simulated using SCS Dimensionless unit hydrograph for single basin and catchment divided into two sub-basins. In case of Clark model and Nash model the catchment considered as a single unit. Plots of observed and simulated hydrographs for all six storm events considered using all for four modeling statistics are shown in fig 6.1 to 6.6.values of error statistics using all three criteria are given in table 6.5 to 6.7 for Nash-Sutcliffe Efficiency (NSE), PWRMSE and SE respectively. As can be seen from plots of hydrographs shown in figs 6.1 – 6.6 and table 6.5 – 6.7, the SCS method with two sub divisions of watershed give slightly better average NSE of 74.93 as against 73.34 obtained using treating watershed as single unit. Other statistical criteria of PWRMSE and SE also support this statement (see table 6.6 and 6.7).

Table: 6.5 Comparison of the models on the basis of Nash- Sutcliffe Efficiency Test Criterion

Date of Event	Nash Efficiency (%)			
	SCS method (2 SBN)	SCS method (1 SBN)	Clark model (1 SBN)	Nash model (1 SBN)
24-Jul-64	97.84	96.98	97.13	95.4
26-Jul-64	90.35	88.02	93.94	98.2
3-Sep-64	47.59	41.67	65.7	73.9
11-Aug-65	89.1	94.46	85.08	93.6
16-Sep-66	87.95	82.35	95.75	89.4
19-Sep-66	36.75	36.58	62.33	92.6
average	74.93	73.34	83.32	90.52

Table: 6.6 Comparison of the models on the basis of Peak-Weighted RMS Error test Criterion

Events	Peak Weighted RMS Error			
	SCS method (2 SBN)	SCS method (1 SBN)	Clark model (1 SBN)	Nash model (1 SBN)
24-Jul-64	1.26	1.54	1.38	2.47
26-Jul-64	3.61	4.12	2.76	1.37
3-Sep-64	2.69	2.81	2.14	2.29
11-Aug-65	4.57	3.19	5.33	3.31
16-Sep-66	11.37	13.98	6.8	7.78
19-Sep-66	11.18	12.32	8.74	2.82
average	5.78	6.33	4.525	3.34

Table: 6.7 Comparison of the models on the basis of Standard Error Test

Events	Standard Error			
	SCS method (2 SBN)	SCS method (1 SBN)	Clark model (1 SBN)	Nash model (1 SBN)
24-Jul-64	1.12	1.3	1.29	2.21
26-Jul-64	3.02	3.32	2.39	1.25
3-Sep-64	2.38	2.46	0.75	2.85
11-Aug-65	4.22	3.01	4.94	3.17
16-Sep-66	8.8	10.48	5.16	8.12
19-Sep-66	9.44	9.32	7.29	2.6
average	4.83	4.98	3.64	3.37

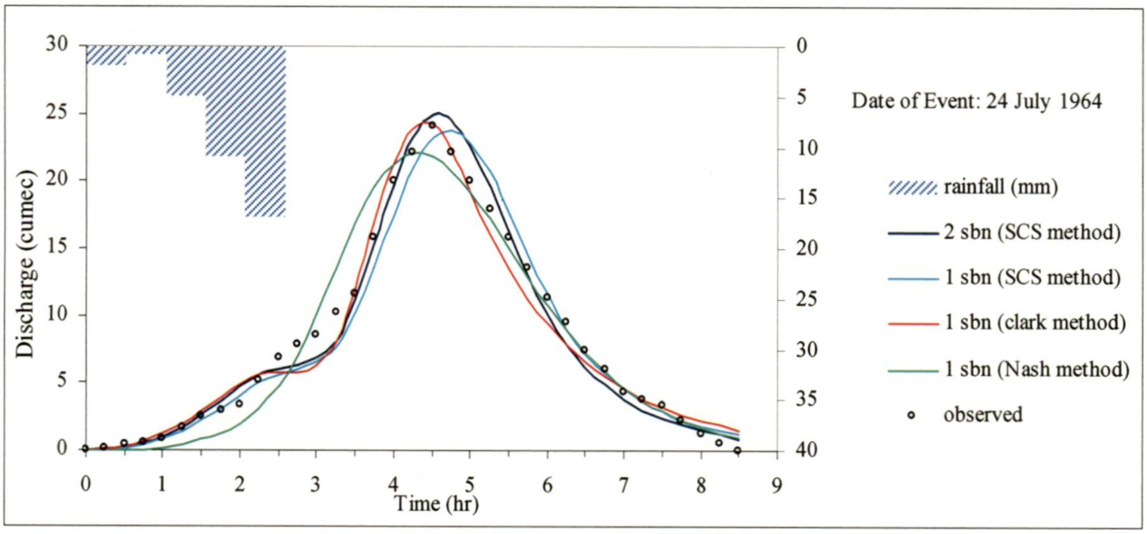

Fig. 6.1: Observed and simulated values for the different models

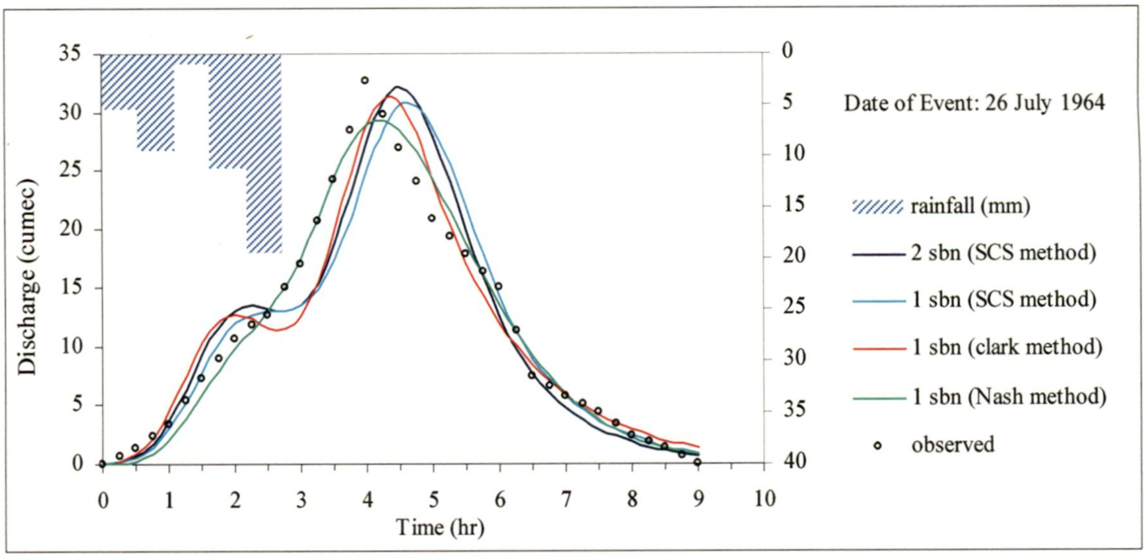

Fig. 6.2: Observed and simulated values for the different models

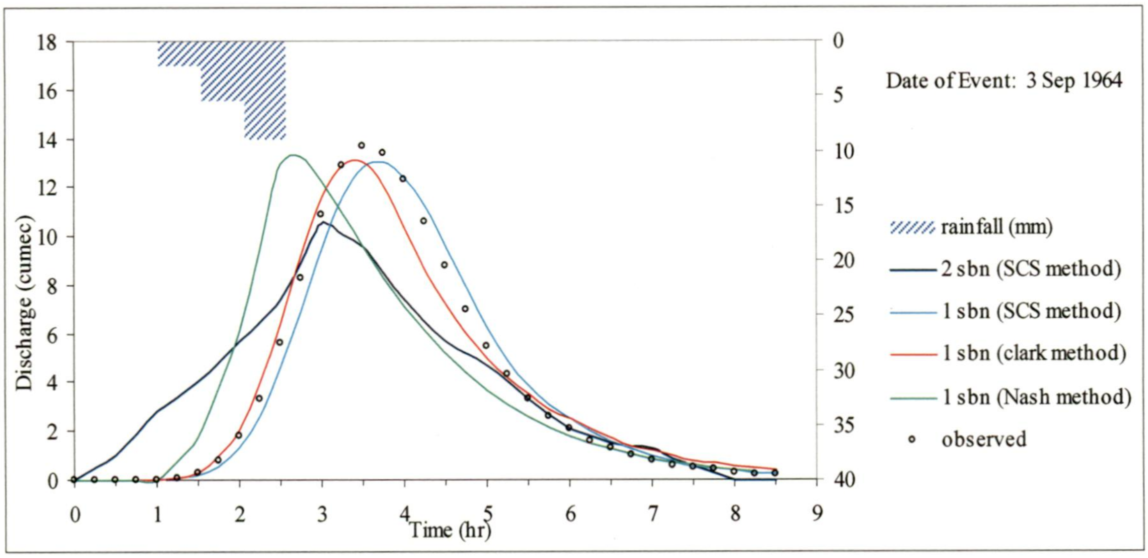

Fig. 6.3: Observed and simulated values for the different models

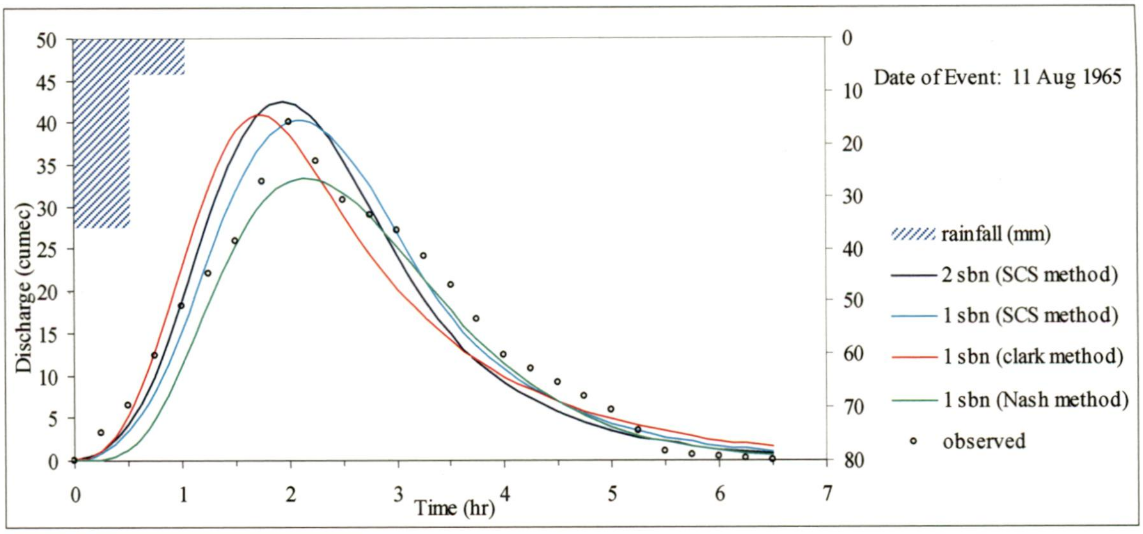

Fig. 6.4: Observed and simulated values for the different models

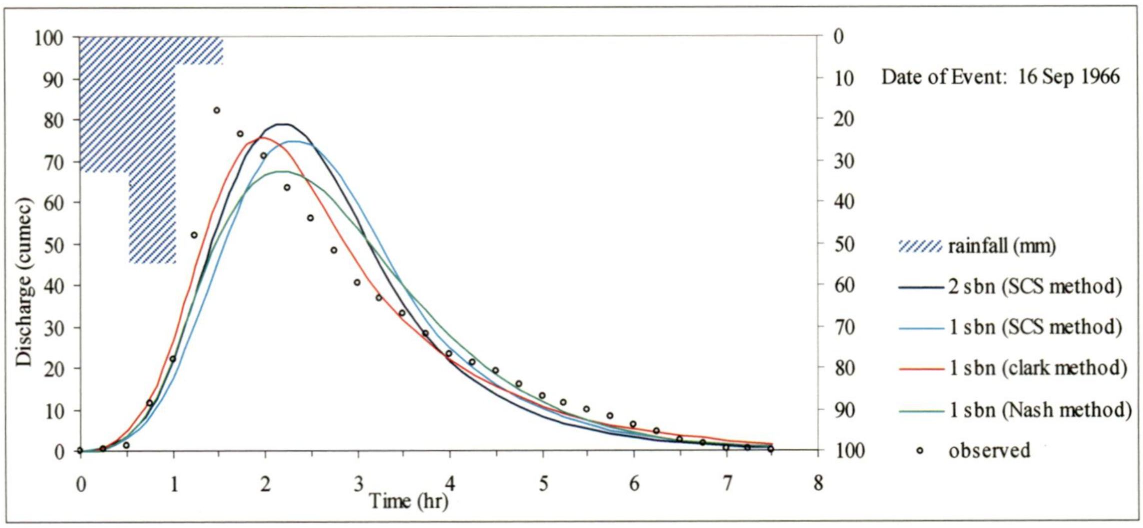

Fig. 6.5: Observed and simulated values for the different models

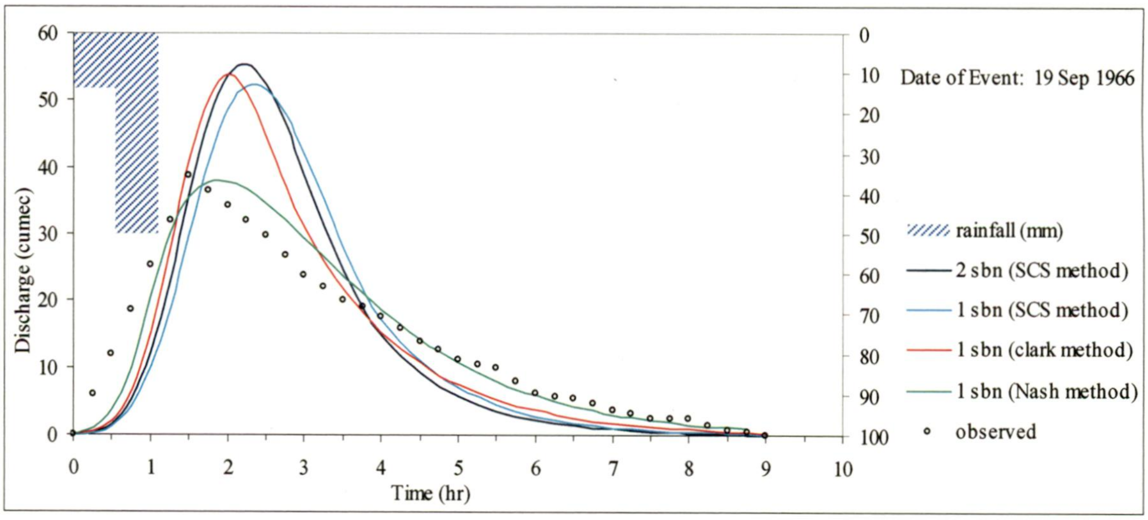

Fig. 6.6: Observed and simulated values for the different models

Comparative analysis of NSE, PWRMSE and SE for all four models presented in tables 6.5-6.7 indicates that on the basis of average values of NSE, PWRMSE and SE; the model of Nash perform better among all with average NSE of 90.52 and lowest average values of PWRMSE and SE . It is worth mentioning here that a lower value of S_e and PWRMSE indicates better model performance. However it is worth mentioning here that the statistical comparison presented in tables 6.5 to 6.7 lists the NSE and PWRMSE and SE obtained using Nash model with variable values of N and K derived using method of moments and in case of SCS and Clark models, the results presented are with average values of model parameters.

To make fair comparison, values for different errors criteria are computed with average values of N and K given in table 6.2 for Nash model. Computed error statistics for NSE, PWRMSE and SE computed using average values of N and K is given in tables 6.8 to 6.10 respectively. Comparative evaluation of different models using average values of model parameters on the basis of average NSE, PWRMSE and SE clearly indicate that the Clark model give highest NSE and lowest values of PWRMSE and SE indicating that Clark model is better choice for simulating flood events in Jolarpet watershed.

Tables 6.8-6.10 are also lists the values of NSE, PWRMSE and SE using values of N and K obtained using analysis of RERH and RDRH. As can be seen from these tables, the procedure of multistorm processing is able to give reasonable estimates of representative values of N ad K. for Nash model as average value of NSE is >75 however, comparison of average values of NSE and PWRMSE indicate that representative values of N and K computed by averaging values of N and K obtained for individual storm events give slightly better results compared to representative value of N and K obtained by multistorm processing which is not in agreement of results reported by Singh (2007).

Table: 6.8 NSE values obtained using average values of N and K values obtained

Event	**Nash Efficiency (%)**	
	Moments of Moments	**Multistorm Process**
24-Jul-64	82.28	82.23
26-Jul-64	96.21	96.78
3-Sep-64	94.51	92.36
11-Aug-65	85.53	87.06
16-Sep-66	66.74	63.53
19-Sep-66	51.42	38.16
Average	79.45	76.69

Table: 6.9 PWRMSE values obtained

Event	Peak Weighted RMS Error	
	Moments of Moments	Multistorm Process
24-Jul-64	3.65	3.57
26-Jul-64	2.16	1.97
3-Sep-64	0.78	0.91
11-Aug-65	5.20	4.91
16-Sep-66	18.67	19.92
19-Sep-66	9.46	10.67
average	6.65	6.99

Table: 6.10 SE values obtained

Event	Standard Error of Estimates	
	Moments of Moments	Multistorm Process
24-Jul-64	3.15	3.2
26-Jul-64	1.89	1.75
3-Sep-64	0.72	0.85
11-Aug-65	4.11	3.89
16-Sep-66	13.56	14.2
19-Sep-66	8.26	9.32
average	5.28	5.54

CHAPTER VII

SUMMARY AND CONCLUSION

Since its inception, the unit hydrograph has been used as a key concept in estimation of storm runoff from a catchment. The unit hydrograph is defined as the watershed response to a unit depth of excess rainfall, uniformly distributed over the entire watershed and applied at a constant rate for a given period of time. A significant contribution to the unit hydrograph theory was given by Clark (1945), who proposed a unit hydrograph which is the result of a combination of a pure translation routing process (plug flow) followed by a pure storage routing process (completely stirred tank reactor). Although Clark does not develop a spatially distributed analysis, the translation part of the routing is based on the time-area diagram of the watershed. The storage part consists of routing the response of the translation through a single linear reservoir located at the watershed outlet. The detention time of the reservoir is selected in order to reproduce the falling limb of observed hydrographs. Note that the actual travel time of a water particle, according to this approach, is the travel time given by the time-area diagram plus the detention time of the reservoir, which is somewhat inconsistent. Some years later, Nash (1957) proposed a unit hydrograph equation which is a gamma distribution, i.e. the response of a cascade of identical linear reservoirs to a unit impulse. It is important to notice that the method proposed by Nash did not model the watershed itself, and was just a fitting technique based on the first and second moments of the calculated and observed hydrographs. In 1972, the Soil Conservation Service (SCS) of the US Department of Agriculture (USDA) proposed a unit hydrograph model based on a single parameter: the lag time between the center of mass of the excess precipitation hyetograph and the peak of the unit hydrograph. The shape of the hydrograph is given by an average pre-computed dimensionless unit hydrograph curve or, as a simplification, by triangular dimensionless unit hydrograph.

The purpose of this research was to determine the most suitable model for simulation of storm runoff for Jolarpet watershed having outlet at railway bridge No. 719. The HEC-GeoHMS interface has been used for terrain processing to prepare input files for use in HEC-HMS model. The storm transformation options for SCS unit hydrograph and Clark unit graph of HEC-HMS have been used. Computations for Nash model have been done using Excel worksheets. In all six storm events have been used for judging performance of different models. In addition a comparison between two procedures, method of moments and multistorm processing, for calculating basin average parameters for Nash model has also been done. Performance of different models used is evaluated using visual comparison and comparison based on statistical criteria such as Nash-Sutcliffe efficiency (NSE), peak weighted root mean square error (PWRMSE) and standard error (SE).

Based on this study, following conclusions are drawn

- The HEC-GeoHMS interface is very useful for quantification and analysis of topographic attributes for a watershed required for rainfall-runoff modeling.

- Visually all models are showing approximate similar results but statistically the difference is clear. Among three models, based on average value of statistical indices used, Clark model with average NSE 83.32 is adjudged best followed by the Nash model with average NSE 79.45. The SCS Dimensionless unit hydrograph model performed poorest.
- Both the methods used for derivation of representative values of Nash model parameters give reasonably good estimates of parameters but representative values obtained from method of moments give slightly better results compared to multi-storm processing.

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