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EXCESS RAINFALL AND DIRECT SURFACE RUNOFF MODELLING USING GEO - MORPHOLOGICAL CHARACTERISTICS

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PREFACE

Rainfall-runoff modelling still finds a central place in research in the field of hydrology. The phenomenon of watershed runoff is very complex. Our understanding of the physical principles and mathematical formulations to represent them is not yet adequate. Though the instrumentation is being done at a very fast speed, yet, there are vast expanses of land, especially those constituting small to medium sized catchments, which does not have adequate facility for the observation of hydrological variables. This has led to the modelling of ungauged catchments where a very limited amount of information is generally available. Indirect inferences through regionalisation are sought for such types of catchments. Many times this task of regionalising the hydrological parameters becomes very tedious and in certain cases even imposible. Recently the concept of geomorphological instantaneous unit hydrograph (GIUH) has been introduced by many investigators wherein the characteristics of the instantaneous unit hydrograph are related to the geomorphological characteristics.

The research in the field of fluvial geomorphology has recently picked up and offers some great opportunities in solving many of the problems facing the hydrologists today. A very complicated analysis is required for accurate inferences based on the geomorphological theory. Many investigators have simplified its application to different levels. Also, there have been attempts to relate the parameters of the conventional conceptual models of instantaneous unit hydrograph to the geomorphological characteristics of the catchment.

In the present study the parameters of the Clark model, which is a conceptual rainfall-runoff model for the simulation of flood hydrograph of a small to medium sized catchments, are evaluated using geomorphological characteristics of the catchment. The necessity of extensive observed runoff data for the calibration of the Clark model parameters is avoided. This study has been carried out by Shri Hemant Chowdhary, Scientist 'B' under the guidance of Shri R D Singh, Scientist 'E' of the Surface Water Analysis and Modelling Division of the National Institute of Hydrology, Roorkee. It is expected that this report, on one hand, would be greatly appreciated by the practicing hydrologists, and on the other hand, introduce a new idea for research and its application in the field of fluvial geomorphology. A continuous effort in this regard may result in a better understanding and an easy modelling procedure for rainfall-runoff process using geomorphological approach.

(S M Seth)

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ABTRACT

The computations of flood hydrographs have always been one of the major concerns of the water resources engineers and scientists. For the purpose of rainfall-runoff process simulation, mathematical modelling is often resorted to. Continued research in this field has resulted in numerous types of rainfall-runoff models. For simulation and design flood evaluation, conceptual models and physically based madels are widely used. The linearity principle of unit hydrograph theory has been widely applied for the simulation of rainfall-runoff process, particularly for small and medium sized catchments. Derivation of unit hydrograph has been extensively investigated by many researchers since Sherman gave the principle of unit graph in 1932. For the gauged catchments the unit hydrographs can be derived by analysing the historical rainfall-runoff records. However, for ungauged catchments some indirect approaches have been used for the derivation of the unit hydrographs. Due to scarcity of data, particularly for small and medium sized catchments, physically based models are very difficult to be implemented. Greater emphasis is now being given to the concept of models based on geomorphological characteristics. Geomorphological instantaneous unit hydrograph is one among the various approaches available for the simulation of flood events, especially for the ungauged catchments. Many investigators have tried to relate the parameters of the conceptual models to the geomorphological characteristics of the catchments.

In this study a hybrid approach is developed integrating the

Clark model and the geomorphological instantaneous unit hydrograph approach. This approach enables the estimation of Clark model parameters using the geomorphological characteristics and storm pattern. It avoids the use of extensive rainfall-runoff records, which are many times not available, for the calibration of the Clark model parameters. Various event based conceptual models and the models for ungauged catchments have been reviewed. The developed approach is illustrated by applying it for the simulation of the historical flood events of Kolar sub-basin of river Narmada locatedin Madhya Pradesh state. The description of the study area alongwith the availability of the data for the present study has also been presented. The methodology is also presented in full detail. Analysis has been carried out by using the computer software developed for this approach. In general, the reproduction of observed flood events using this approach is good for all the events considered in the study. Further investigations and field applications are needed to improve upon the present form of the model structure by incorporating the latest developments in the field of fluvial geomorphology.

1.0 INTRODUCTION

Simulation of rainfall-runoff process for ungauged catchments is one of the important areas of research in the sphere of rainfall-runoff modelling. There are a number of well established techniques like unit hydrograph, conceptual or physically based modelling which are employed for the purpose of rainfall-runoff process simulation for the catchments. All such techniques require a certain amount of historical data for establishing various parameters. However, due to very sparse gauging network available in most of the Indian catchments, particularly for small catchments it becomes very difficult for such techniques to be directly applicable. In such situations of very poor data availability, the options available are, either to go for regionalization of parameters based on the data available for the gauged catchments in nearby hydro-meteorologically similar regions or by using the morphological details available for the ungauged catchments for modelling their hydrological response. Regionalisation of the parameters is, however, a very tedious task to accomplish since the hydrological behaviour of many nearby catchments have to be ascertained before being confident about the values of the parameters. On the other hand the geomorphological approach has many advantages over the regionalization techniques as it avoids the requirement of data and computations in the neighbouring gauged catchments in the region.

As a first step in the direction of using geomorphologic characteristics with the conviction that the search for a theoretical coupling of quantitative geomorphology and hydrology is an area which will provide some of the most exiting and basic developments of hydrology in the future the concept of Geomorphologic Instantaneous Unit Hydrograph (GIUH) was introduced. This technique, though appears to be simple and tempting to the practitioners for its use in areas of insufficient or inexistent hydrologic data, is having a difficulty of the dependence on the dynamic parameter "velocity" whose estimation is very much subjective.

A new approach, in which the conceptual modelling of instantaneous unit hydrograph (IUH) is combined with the geomorphologic instantaneous unit hydrograph approach, is developed in the present study for the simulation of the flood hydrographs specially for the small to medium sized catchments which are ungauged. By this way, the estimation of parameters of the conceptual model of IUH is not required to be carried out through the tedious regionalisation process. This hybrid approach is developed by linking the Clark's model parameters with the geomorphological instantaneous unit hydrograph. In this approach a simple procedure for estimation of velocity using the channel cross-section, channel roughness and the storm characteristics is adopted. The methodology is tested by simulating the six storm events of Kolar sub-basin of river Narmada in the Central parts of India.

2.0 REVIEW

2.1 GENERAL

The problem of transformation of rainfall into runoff has been a very active area of research throughout the evolution of the subject of hydrology. Through their intuition, many investigators have tried to relate the runoff with the different characteristics which affect it. The simplest theory proposes to multiply the rainfall with some factor (called the runoff coefficient) to get the runoff. A better way to transform rainfall into runoff is to apply conceptual models in which the various interrelated hydrological processes are conceptualized. More sophisticated procedures are also evolved which are based on the physical concept of the process and try to model this hydrological phenomenon on the basis of physical laws governing them. Never it is inferred that a particular model is the best for rainfall-runoff transformation. Actually, many more factors, besides the accuracy, e.g., the availability of data, computing facility, time, resources etc. govern the applicability of a model. The search for suitable models for different conditions still continues and thus more and more mathematical models are being suggested.

2.1 REVIEW OF EVENT BASED CONCEPTUAL MODELS

The approaches utilized to develop linear conceptual models of rainfall-runoff relationship may be classified into three groups. The first group employs a differential equation that supposedly governs the operation of a specified system (Kulandaiswamy, 1964; Chow 1964; Shen, 1965; Chaudhry, 1976; Jackson, 1968; Chow and Kulandaiswamy, 1971, 1982; V.P.Singh and Mc Cann, 1979; Mc Cann and V.P. Singh, 1980, 1981; Te and Kay, 1983). The second group utilizes an arrangement of the so-called conceptual elements, including linear channels and linear reservoirs (Nash, 1957; Dooge, 1959, 1977; Chow, 1964; S Bravo et.al., 1970; Maddaus and Eagleson, 1969; Harley, 1967; O'Meara, 1968; V.P. Singh and Mc Cann, 1980a). The third group makes some hypothesis about rainfall-runoff relationship more or less on intuitive grounds (Lienhard, 1964, 1972).

In the second category of the conceptual models Clark (1945) suggested that the unit hydrograph for a watershed due to instantaneous rainfall can be determined by routing its Time-Area-Concentration (TAC) curve through a single linear reservoir. Physcially, it is equivalent to Zoch (1934) Model, in which the concept of instantaneous unit hydrograph (IUH) is replaced by one of unit hydrograph. O'Kelly (1955) defined the TAC curve by an isosceles triangle and routed it through a linear reservoir to produce the instantaneous unit hydrograph for the watershed. Thus, O'Kelly model is equivalent to Clark's model except for the definition of TAC curve.

Nash (1957) developed a model based on a cascade of equal linear reservoirs for derivation of the IUH for a natural watershed. This is one of the most popular and frequently used models in applied hydrology.

Dooge (1959) developed a general unit hydrograph theory, which embraced all previous models as its special cases. The three elements : TAC curves, linear channel and linear reservoir were included in the theory. The basic premise of the Dooge model is that a watershed can be represented by some combination of linear channels and reservoirs. The watershed is drained by a network of channels composed of a complex network of linear channels and linear reservoirs placed in series.

2.2 MODELS FOR UNGAUGED WATERSHEDS

The parameters of the models reviewed in previous section are generally calibrated based on the analysis of rainfall-runoff data for gauged catchments. However, these models can not be calibrated for those catchments which lack such data. Consequently, the parameters of those models for ungauged catchments may be determined from the regional relationships developed by correlating the model parameters with physically measurable catchment characteristics of the gauged catchments. optimization is one of the most widely used techniques available to calibrate the model for gauged catchments. Frequently the model parameters are optimized for some selected rainfall-runoff events over a given watershed, using a suitable optimization procedure. The optimized parameter values are then utilized in the model to predict runoff for the rainfall events of interest not used in the optimization. This approach is obviously not applicable to ungauged watersheds. Further, it has other shortcomings as the optimized parameters can best represent the watershed only for the events used in the optimization. The optimized values change with the change in the event. Also, the extensive amount of data required for optimization is normally lacking and thus prove prohibitive in the widespread use of model applicability.

The other approach attempts to establish relationships between model parameters and physically measurable watershed characteristics. These relationships are then assumed to hold for ungauged watersheds having similar hydrologic characteristics. Rainfall-runoff relationships for ungauged watersheds have been developed along two complimentary lines : (1) Empirical equations have been developed to relate some individual runoff hydrograph characteristics to watershed characteristics (2) Procedures have been developed to synthesize the entire runoff hydrograph from watershed characteristics. Some of these models are reviewed here under.

Bernard (1935) model is perhaps the first attempt to synthesize the unit hydrograph (UH) from watershed characteristics. It assumes that the peak of the UH is immensely proportional to the time of concentration, which in turn is assumed to be proportional to a watershed factor. A distribution graph establishes relation between the effective percentage area contributing and the watershed factor for different days of the storm.

Snyder (1938) established a set of formulas relating the physical geometry of the watershed to three basic parameters of the unit hydrograph. Mc Carthy (1938) related three parameters of 6-hour UH, including the time of rise, the peak discharge, and the base length, to watershed characteristics such as area, overland slopes expressed as the average slope of the hypsometric curve and stream pattern. Taylor and Schwarz (1952), in addition to the watershed characteristics employed by Snyder (1938), introduced the average slope of the main channel. The method of hydrograph synthesis employed by the Soil Conservation Service (SCS) (1971), U.S. Deptt. of Agriculture, uses an average dimensionless hydrograph derived from an analysis of a large number of natural UHs for watersheds varying widely in size and geographical locations.

As mentioned earlier, the Clark model involves determination of the TAC diagram and the storage coefficient. This storage coefficient has been related with the catchment characteristics as

$$K = b\ L\ (A/S_c)^{0.5}$$

where, K => storage coefficient
L => channel length in miles
S => mean channel slope
A => area in sq. miles
and C => a constant varying from 0.04 to 0.075 for 11 rivers in California and Virginia.

The time of concentration was considered to equal the time interval between the end of rain and the point of contraflexure of the hydrograph recession limb. This time base was measured from the recorded floods and not related to watershed characteristics.

Nash (1960) model has two parameters n and K. Nash showed that these parameters were related to the first and second moments of the IUH about the origin as :

$m = nk$

and $m = 1/n$

These moments were then correlated empirically with watershed characteristics as :

$$m = 27.6\ A^{0.3}\ S^{-0.3}$$

and $$m = 0.41\ L^{-0.1}$$

where; S => the overland slope in parts per 10,000, calculated as the mean of a grid sample of slopes.

A => area of watershed in sq.miles

and L => channel length in miles.

In early years, in India, the design discharges for very small and medium catchments were used to be calculated by well known empirical formulae viz. Dickens, Ryves, Inglis, Ali Nawaz Jung, etc. Later on, to evolve a method of estimation of design flood peak of desired frequency, the unit hydrograph approach has been adopted by the Central Water Commission. For this purpose, the country has been divided into 7 major zones which are sub-divided into 26 hydrometeorologically homogeneous subzones. For most of these sub-zones, Central Water Commission has already developed regional formulae for the derivation of the synthetic unit hydrograph. Computation of unit hydrograph characteristics such as peak (Q_p), time to peak (t_p), W_{50}, W_{75}, WR_{50}, WR_{75}, time base (t_b) etc. on the basis of physiographic features has been done. These regional formulae enable computation of unit hydrograph for ungauged catchments of the sub-zones.

A few regional unit hydrograph studies have also been carried out for some of the sub-zones by various other organisations besides Central Water Commission. Singh (1984) developed regional unit hydrograph relationship for lower Godavari sub-zone (3f) relating the parameters of Nash and Clark models with the physiographic characteristics of five gauged catchments in the sub-zone.

National Institute of Hydrology (1985) has carried out a regional unit hydrograph study for Narmada basin based on Clark's approach. In this study the parameters of the Clark model have been derived for each of the sub-basin of Narmada basin using HEC-I package. A regional relationship has been developed in the graphical form relating average value of $(t_c + R)$ for each sub-basin with their respective catchment area. A regional value of $R/(t_c + R)$ along with their graphical relationship has been used to estimate the parameters of the Clark model for ungauged catchment of the Narmada basin.

Huq. et.al. (1982) developed synthetic unit hydrograph relationships using the data of the catchments in Gangetic plains, Mahanadi basin, Krishna basin and Bhramaputra basin. These relationships have been developed relating the parameters of the representative unit hydrograph for gauged catchment with a suitable combination of the physical characteristics of the catchment using regression analysis.

Mathur and Vijay Kumar (1987) related the physical parameters of twenty small and medium catchments in order to arrive at the most effective combination of the physical parameters for the development of the regional unit hydrograph relationships.

Although number of such relations are developed with the hope that they will yield satisfactory results when applied to the ungauged basin, these approaches have following limitations :

(i) The catchment for which data is used in a regional study have to be similar in hydrological and meteorological characteristics. However, it is usually difficult to locate catchments strictly satisfying these requirements.

(ii) While establishing such relations, the inherent limitations of the unit hydrograph theory are also being carried out with it. As a result the prevailing method of predicting the discharge hydrograph for a design storm by using the average unit hydrograph will not be appropriate, since the average unit hydrograph does not necessarily reproduce the actual response due to such inherent limitations.

(iii)The relationship evolved are based upon the gauged observations in number of catchments in the region. It is practically very difficult to always have gauged catchments available in adequate numbers in a region to enable the development of such relationships.

Boyd (1978, 1982) developed the linear watershed bounded network (LWBN) model for synthesis of the IUH employing geomorphologic and hydrologic properties of the watershed. The model divides a watershed into sub-areas bounded by watershed lines using large-scale topographic maps. The model has a large number of lumped storage parameters. Most of these parameters are deduced from geomorphologic properties.

Rodriguez-Iturbe and Valdes (1979) developed an approach for derivation of the IUH by explicitly incorporating the characteristics of drainage basin composition (Horton, 1945; Strahler, 1964; Smart, 1972). The approach coupled the empirical laws of geomorphology with the principles of linear hydrologic systems. Rodriguez-Iturbe and his associates have since extended this approach by explicitly incorporating climatic characteristics and have studied several aspects including hydrologic similarity. Gupta, Waymire and C.T.Wang (1980) examined this approach, and reformulated, simplified and made it more general.

The effect of climatic variation is incorporated by having a dynamic parameter velocity in the formulation of Geomorphological IUH (GIUH). This is a parameter that must be subjectively evaluated. It is shown (Rodriguez-Iturbe, et.al., 1979) that this dynamic parameter "velocity" of the GIUH can be taken as the velocity at the peak discharge time for a given rainfall-runoff event in a basin. This transforms the time invariant IUH throughout the event into a time invariant IUH in each storm occurrence.

In the derivation of GIUH one of the greatest difficulties involved is the estimation of peak velocity. This is a parameter that must be evaluated for each flood event.

Rodriguez et.al. (1982) rationalised that velocity must be a function of the effective rainfall intensity and duration and proceeded to eliminate velocity from the results. It leads to the development of geomorphoclimatic instantaneous unit hydrograph. The governing equations consists of the terms such as the mean effective rainfall intensity, Manning's roughness coefficient, average width, and slope of the highest order stream.

Janusz Zelazinski (1986) gave a procedure for estimating the flow velocity. It involves the development of the relationship between the velocity and corresponding peak discharge. A methodology based on trial and error procedures has been suggested for estimating the maximum value of the velocity for each flood event.

Panigrahi (1991) estimated the velocity using the Manning's equation. The methodology involves the estimation of equilibrium discharges and subsequently the estimation of the velocity corresponding to it using Manning's equation. It requires the intensity of each rainfall block for the event for the computation of equilibrium discharge. The channel cross-section at the gauging site, longitudinal slope and Manning's roughness are also required during the computation of the velocity. The methodology has been applied to estimate the velocity to derive the Nash model parameters using GIUH approach for the Kolar sub-basin of Narmada basin.

Development of GIUH has potential applications for the estimation of runoff, flood forecasting and design flood estimation particulars for the ungauged catchments or for the catchments with limited data. Most of the studies available in literature regarding the GIUH approach are synthetic in nature and are in the early stage of research and development. Very few studies are available where its practical applications have been demonstrated. As GIUH approach has many advantages over the traditional method of developing the regional unit hydrograph for the simulation of flood events in the ungauged catchment, it would be appropriate to verify the application of GIUH approach for simulating the flood response of a gauged catchment.

In this study a hybrid approach is developed linking the GIUH equations derived by Rodriquez and the parameters of the Clark model. It enables the estimation of parameters of Clark model using the geomorphological characteristics, channel cross section and storm characteristics.

3.0 STATEMENT OF THE PROBLEM

There are two parameters of the Clark's rainfall-runoff conceptual model viz., the time of concentration (T_c) and the storage coefficient (R). The usual procedure to estimate these parameters, for the gauged catchments is by making use of the rainfall-runoff records of several storm events. The computed and observed hydrograph ordinates are compared and the sum of the square of errors is minimised in the successive iterations while improving the values of the parameters using a non-linear optimisation technique.

In case of ungauged catchments where runoff data is not available, regionalisation of parameters is often resorted to. However, such a procedure becomes very tedious and many times very difficult in the absence of enough data available for the nearby catchments.

In this study a new approach has been attempted which makes use of geomorphologic details of the catchment while establishing the parameters of Clark's model for the ungauged catchments. The following aspects have been covered in the present study :

(i) estimation of the excess rainfall.

(ii) development of the methodology linking the Clark model's parameter with the GIUH approach.

(iii) application and testing of the methodology for the Kolar sub-basin.

4.0 DESCRIPTION OF THE STUDY AREA

4.1 GENERAL

The study area is a sub-basin "Kolar" of Narmada river system in the central parts of India. The Kolar river originates in the Vindhayachal mountain range at an elevation of 550 m above mean sea level (msl) in the district of Sehore of Madhya Pradesh (M.P.) state in Central India. It is a tributary from northern side of the Narmada river which flows from east to west side and drains in the Arabian sea. The catchment area of this sub-basin lies between the latitudes $22^{\circ}40'$ to $23^{\circ}08'$ and longitudes $77^{\circ}01'$ to $77^{\circ}29'$. The catchment has an elongated shape which is oriented in east-west direction in its upper part and north-south direction in the lower part. The Kolar river also, during its 100 kms. course, first flows towards east and then towards south before joining the main river Narmada near Neelkanth. The Kolar river has an elaborate drainage network which drains a total area of 1350 sq.kms.. However, this case study is done only for an area of 875 sq. kms. which drains through a gauge-discharge measurement site near Satrana. The entire sub-basin lies in two districts, Sehore and Raisen, of Madhya Pradesh State. A map showing locations of various rainfall stations and stage discharge gauging site is given in Fig. 4.1.

During the period pertaining to the data used for the case study a dam was nearing completion near the village Lawakeri. This multipurpose Kolar dam would provide drinking water to the city of Bhopal which is at a distance of 30 km. north and irrigation to farmers. A barrage was being constructed near Jholiapur from where two canals will take off to irrigate the fields. Construction of these lined canals was in progress and was to be operational soon.

4.2 HYDROLOGY OF KOLAR SUB-BASIN

Kolar sub-basin is divided into two distinct topographical regions. The first is the upper fourth-fifth part having elevations ranging from 350 m to 600 m and predominantly covered by deciduous forest which is dense and open. The boundaries of the catchment are mild sloped at the northern end of the basin. The river debauches to plains from this area upstream of Jholiapur through ramp shaped southward sloping topography. The soils are skeleton to shallow in depth except near channels where they are relatively deeper. The rock outcrops are also easily visible at many places. In this area, the rocks are weathered, and deep fissures can be seen. The channel beds are rocky and graveled. The thin soils get saturated even during low intensity rains and water moves through the fissures rapidly. Agricultural activity is carried out in relatively large areas in the north western part and in small pockets elsewhere in which the main crops are wheat and grains. In general the response of this upper part of the basin to the rains appears to be quick.

The second region is the lower one fifth of the sub-basin consisting of flat bottomed valley narrowing towards the outlet and having elevations ranging from about 300 m to 350 m and is predominantly cultivatable area. The soils are deep in the area and have flat slopes. The places where agricultural activity is carried out have bunded fields in which water is impounded during

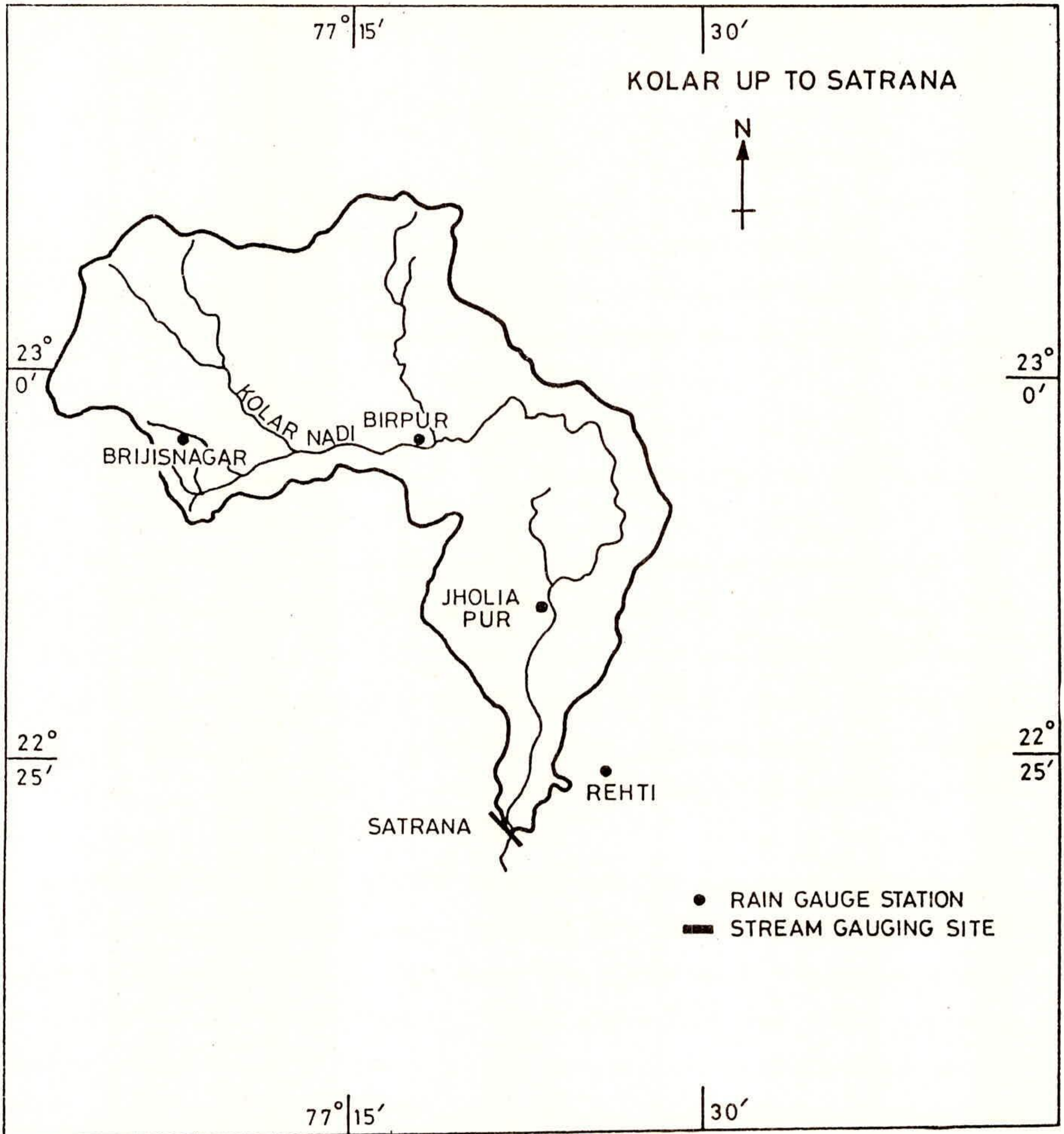

FIG. 4·1 THE KOLAR BASIN UP TO SATRANA GAUGING SITE

the monsoon period. The response of this area to the rainfall is likely to be quite slow. Some parts of this area are covered under the command of Kolar dam.

5.0 DATA AVAILABILITY FOR THE STUDY

5.1 TOPOGRAPHIC DATA

The topographic map of Kolar sub-basin was prepared using the Survey of India toposheets of 1:50,000 scale. This map is used for the preparation of catchment map alongwith contours and river network.

5.2 RAINFALL AND DISCHARGE DATA

At the time of carrying out the present study the rainfall and runoff data for the period 1983 to 1986 was available. Six events were selected from the data of 1983 to 1986. Hourly rainfall values at four rainfall stations namely Rehti, Jholiapur, Birpur and Brijeshnagar were obtained from the records of recording type rainfall stations at these placed.

The Satrana gauging site located at the outlet of this basin was established in 1983. The gauge-discharge measurements are made at a bridge on Rehti-Nasrullganj road where an automatic gauge recorder (AGR) has been installed. The flow velolcity is measured using current meter. At the Satrana gauging site, hourly gauge observations and daily discharge measurements were available for the monsoon months during 1983-86. Based on the rating curves for this period the hourly discharges were calculated and the values pertaining to the six events were taken for analysis.

6.0 METHODOLOGY

6.1 COMPUTATION OF EXCESS RAINFALL

When the rainfall occurs over the catchment not all the rain contribute to the direct surface runoff. A part of the rainfall is abstracted as interception, evapotranspiration, surface depression storage and infiltration. The remainder of the rainfall termed as excess rainfall contributes to the direct surface runoff. Thus the computation of excess rainfall is required for the estimation of direct surface runoff by separating the hydrological abstractions from the rainfall hyetographs. Although number of techniques are available for the computation of excess rainfall but the ϕ-index method is one of the simple and most commonly used technique. Among the other techniques SCS curve number method is being widely used for the estimation of the excess rainfall particularly when the catchment is ungauged. In the present analysis the ϕ-index and SCS curve number methods (Chow et. al., 1988) are used to estimate the excess rainfall hyetograph pattern. For both the methods the volume of the excess rainfall for a given storm event is assumed to be known. It is computed as the volume of direct surface runoff hydrograph for a given event. The direct surface runoff hydrograph is computed by separating the baseflow from the observed hydrograph ordinates. Here the observed direct surface runoff is used only for the estimation of excess rainfall hyetograph and is not used further for the derivation of instantaneous unit hydrograph. However, the use of the observed direct surface runoff for the estimation of excess rainfall has to be avoided for the ungauged catchment as no runoff records would be available for such catchments. In such situations the values of ϕ-index can be estimated by analysing the rainfall-runoff records of flood events of the same period of the neighbouring catchments having similar hydro-meteorological characteristics. For the application of SCS method it is however expected that the land use, soil type, treatment class, hydrologic condition and antecedent soil moisture condition would be known for the estimation of runoff curve number which is utilised for the estimation of excess rainfall in the method.

6.2 PREPARATION OF TIME-AREA DIAGRAM

Time of travel through the streams ,t , is considered proportional to $L / \sqrt{S}$

or $$t = K L / \sqrt{S} \qquad ...(1)$$

where:

- t => time of travel
- L => length of the stream
- S => slope of the stream

and K => proportionality constant.

Using eq.(1) we may have a relationship between the average slope of the main stream and its individual segment slopes as:

$$K L / \sqrt{S_A} = K L_1 / \sqrt{S_1} + K L_2 / \sqrt{S_2} + K L_3 / \sqrt{S_3} + \qquad ...(2)$$

where,

L => the total length of main stream

L_1, L_2 => the lengths of each individual segments

S_A => average slope of main stream

S_1, S_2 => average slope of individual segment slopes.

Eq.(2) may be rewritten as:

$$L/\sqrt{S_A} = L_1/\sqrt{S_1} + L_2/\sqrt{S_2} + L_3/\sqrt{S_3} + \qquad \ldots(3)$$

Substituting the values for the various segments we get the value of average slope S_A of the basin.

Time of travel or time of concentration is also given by the Kirpich's formula as:

$$t_c = 0.06628\ L^{0.77}\ H^{-0.305} \qquad \ldots(4)$$

where,

t_c => concentration time in hours.

L => length of stream in kms.

H => average slope of the stream.

Substituting values of L and H in eq.(4) we get the value of time of concentration t_c for the catchment.

This value of t_c may be substituted in eq.(1) and then may be rearranged in the form :

$$K = t_c\sqrt{S_A}\ /\ L \qquad \ldots(5)$$

substituting the known values of t_c, L and S_A in eq.(5), the value of K may be computed.

Knowing now the value of constant of proportionality K we may use eq.(1) to calculate time of travel between any two points in the catchment. Starting from the basin outlet the time of travel of various points over the catchment is thus progressively calculated.

All the values of the time of travels for different points are then denoted on the map at their respective locations. Curves of specified time of concentration called the "Isochrones" are then drawn through these points by making use of linear interpolation and consideration of elevation contour pattern and stream layout.

6.3 DERIVATION OF CLARK MODEL IUH AND D-HOUR UNIH HYDROGRAPH

The Clark model concept suggests that the IUH can be derived by routing the unit inflow in the form of time-area diagram, which is constructed from the isochronal map, through a single reservoir. For the derivation of IUH the Clark model uses two parameters, time of concentration (T_C) in hours, which is the base length of the time-area diagram, and storage coefficient (R) ,in hours,of a single linear reservoir in addition to the time-area diagram.

The governing equation of IUH using this model is given as :

$$u_i = C \; I_i + (1-C) \; u_{i-1} \qquad \ldots(6)$$

where;

u_i => ith ordinate of the IUH

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

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

Δt => computational interval in hours

I_i => the ith ordinate of the time-area diagram

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

$$U_i = 1/n \,\{0.5\, u_{i-n} + u_{i-n} + u_{i-n+1} + \ldots\ldots\ u_{i-1} + 0.5\, u_i\} \qquad \ldots(7)$$

where;

U_i => ith ordinate of unit hydrograph of duration D hour and at computational interval Δt hours

n => no. of computational intervals in duration D $= D \,/\, \Delta t$

u_i => ith ordinate of the IUH

6.4 USE OF GEOMORPHOLOGICAL CHARACTERISTICS

Rodriquez-Iturbe and Valdes (1979) first introduced the concept of geomorphologic instantaneous unit hydrograph, which led to the renewal of research in hydrogeomorphology.

The expression derived by Rodriquez-Iturbe and Valdes (1979) yields full analytical, but complicated, expressions for the instantaneous unit hydrograph. Rodriquez-Iturbe and Valdes (1979) suggested that it is adequate to assume a triangular instantaneous unit hydrograph and only specify the expressions for the time to peak and peak value of the IUH. Those expressions are obtained by regression of the peak as well as time to peak of IUH, derived from the analytic solutions for a wide range of parameters with that of the geomorphologic characteristics and flow velocities. The expressions are given as:

$$q_p = 1.31 \; R_L^{0.43} \; V / L_\Omega \qquad \ldots(8)$$

$$t_p = 0.44 \; (L_\Omega / V) \; (R_B / R_A)^{0.55} \; (R_L)^{-0.38} \qquad \ldots(9)$$

where;

L_Ω => the length in kilometers of the highest-order stream

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

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

t_p => the time to peak, in hours

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

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

6.5 DEVELOPMENT OF RELATIONSHIP BETWEEN THE INTENSITY OF THE EXCESS RAINFALL AND THE VELOCITY

For the dynamic parameter velocity (V) , Rodriquez et. al.

(1979) in their studies assumed that the flow velocity at any given moment during the storm can be taken as constant throughout the basin. The characteristic velocity for the basin as a whole changes throughout as the storm progresses. For the derivation of GIUH, this can be taken as the velocity at the peak discharge time for a given rainfall-runoff event in a basin. However, for ungauged catchments the peak discharge is not known and so this criteria for estimation of velocity cannot be applied. In such a situation the velocity may be estimated using the relationship developed between the velocity and the excess rainfall. The steps involved in developing the relationship between the velocity and excess rainfall are as follows :

(i) Compute cross sectional area (A), Wetted Perimeter (P) and hydraulic radius (R) on the basis of X-sectional details corresponding to different depths.

(ii) Assume the frictional slope to be equal to the bed slope of the channel.

(iii) Choose an appropriate value of Manning's roughness coefficient (n) from the values given in literature (Chow 1964) for different surface conditionsof the channel.

(iv) Compute the discharge (Q) using the Manning's formulae corresponding to each depth.

(v) Plot depth v/s discharge and depth v/s area curves.

(vi) Compute the equilibrium discharge (Q_e) corresponding to an excess rainfall intensity (i in mm/hr) using the relation :

$$Q_e = 0.2778 \; i \; A_c \qquad \ldots(10)$$

where ;

A_c => catchment area in Sq. Kms..

(vii) Compute the depth corresponding to the equilibrium discharge (Q_e) using the depth v/s discharge curve.

(viii) Compute the area corresponding to the depth computed at step (vii) using the depth v/s area curve.

(ix) Compute the velocity V by dividing the discharge (Q_e) by the area computed at step (viii).

(x) Repeat steps (vi) to (ix) to find velocity with respect to different intensities (e.g., 1,2,3mm/hr. etc.) of rainfall excess.

(xi) Develop the relationship between velocity and rainfall excess intensity obtained at step (x) in the form : $v = a\, i^b$, using method of least square.

6.6 ESTIMATION OF CLARK MODEL PARAMETERS THROUGH USE OF GEOMORPHOLOGIC CHARACTERISTICS

A new approach is developed in this study for the estimation of the parameters of the Clark model through use of geomorphological characteristics. The step-by step explanation of the procedure is given hereunder :

(i) Excess rainfall hyetograph is computed either by uniform loss rate procedure or by SCS curve number method.

(ii) For a given storm the first estimate of the velocity using the highest rainfall excess is made by using the relatationship between velocity and intensity of rainfall excess (as developed in section 6.5).

(iii) Compute the time of concentration (T_c) using the equation :

$$T_c = 0.2778 \; L \; / \; V \qquad \ldots(11)$$

where;

L => length of the main channel

V => the velocity in m/sec.

(iv) Compute the product $(PR)_g$ of the peak discharge (Q_{pg}) and time to peak discharge (T_{pg}) of IUH given by equations (8) and (9) as :

$$(PR)_g = Q_{pg} * T_{pg} = 0.5764 \; (R_B / R_A)^{0.55} \; (R_L)^{0.05} \qquad \ldots(12)$$

(v) Assume two trial values of the storage coefficient of GIUH based Clark model as R_1 and R_2 . Compute the ordinates of two instantaneous unit hydrographs by Clark model using time of concentration T_c as obtained in step (iii) and two storage coefficients R_1 and R_2 respectively with the help of equation (6).

(vi) Find out the products PR_{c1} and PR_{c2} by multiplying time to peak T_{pc} and peak flow Q_{pc} of the instantaneous unit hydrographs obtained for Clark model for the storage coefficients R_1 and R_2 respectively.

(vii) Find out the value of objective function , using the relation :

$$FCN1 = (PR_g - PR_{c1})^2 \qquad \ldots(13)$$

$$FCN2 = (PR_g - PR_{c2})^2 \qquad \ldots(14)$$

(viii) Compute the first numerical derivative FPN of the objective function FCN with respect to parameter R as :

$$FPN = \frac{FCN1 - FCN2}{R_1 - R_2} \qquad \text{...(15)}$$

(ix) Compute the next trial value of R using the following governing equations of Newton-Raphson's method :

$$\Delta R = \frac{FCN1}{FPN} \qquad \text{...(16)}$$

and $RNEW = R1 + \Delta R$ ...(17)

(x) For the next trial consider $R_1 = R_2$ and R_2 = RNEW and repeat steps (v) and (ix) till one of the following criteria of convergence is achieved.

(a) FCN2 = 0.000001

(b) No. of trials exceeds 1000

(c) ABS(ΔR)/R1 = 0.000001

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

(xii) Compute the instantaneous unit hydrograph (IUH) using the GIUH based Clark Model with the help of final values of storage coefficient (R), Time of concentration (Tc) as obtained in the step (xi) and time-area diagram.

(xiii) Compute the D-hour unit hydrograph (UH) using the relationship between IUH and UH of D-hour as given by equation (7).

(xiv) Estimate the velocity V_1 using the relationship:

$V_1 = V * (Q_{pc}/Q_{pg})$

(xv) Consider velocity $V = V_1$ and repeat steps (ii) to (xiv) till the % error in Q_{pc} with respect to Q_{pg} is less than or equal to 5%.

(xvi) The final values of parameters, T_c and R obtained after step (xvi) are then the required parameters of Time of concentration (T_c) and storage coefficient (R) to be considered for the simulation of the given storm events.

(xvii) Use the final set of parameters T_c and R alongwith the time area diagram to compute the IUH for GIUH based Clark Model methodology discussed in section 6.3.

(xviii) Derive the unit hydrograph of desired duration D-hour (equal to each rainfall block duration) using eq. (7).

(xix) Compute the direct surface runoff for a storm event whose excess rainfall values are known at D-hour interval using the convolution based on the D-hour unit hydrograph. The convoluted hydrograph ordinates are given as :

$$Q(t) = \Delta t \sum_{i}^{n} [U\ (D,t-(i-1)\Delta t)] * I_i \qquad ...(18)$$

where,

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

I_i => rainfall intensity at ith interval (i.e., at time = $\Delta t*i$)

n => no. of rainfall blocks

Δt => computational time interval

(xx) Plot the observed and computed direct surface runoff hydrographs and compare the simulation results.

7.0 ANALYSIS

7.1 DATA PREPARATION

7.1.1 Preparation of contoured map of watershed:

The catchment area of the sub basin up to Satrana is covered by Survey of India toposheets nos. 55 E/4, 55 E/8, 55 F/1, 55 F/5, and 55 F/6 on 1:50,000 scale. The watershed divide is marked for the gauge discharge site at Satrana. The contours at an equal interval are also marked on the map. At some places where contours are not very clear are left out which would not create any appreciable error in the estimation of time of concentration.

7.1.2 Preparation of time-area diagram:

Starting from the outlet of the catchment i.e., from the gauge-discharge site at Satrana, the time of travels are computed and marked at every intersection point between the contour and the stream by making use of the methodology given in section 6.2.

Substituting the values of L and S for the various segments in eq.(3), we get the average slope of the basin as:

$$S_A = 0.024988$$

Substituting values of L and H in eq.(4) we get time of concentration for the catchment as:

$$t_c = 5.107 \text{ hrs.}$$

or $$t_c = 306.46 \text{ minutes}$$

substituting values of t_c, L and S_A in (5), we get:

$$K = 0.51536 \text{ min./km.}$$

Knowing now the value of constant of proportionality K we may use eq.(1) to calculate time of travel for each point of intersection of a contour and a stream to the sub-basin outlet as:

$$t_c = K\, L / \sqrt{S}$$

$$t = \frac{0.51636 \left(\frac{\text{min.}}{\text{Km}}\right) * L \left(\frac{\text{cms.} * 50000}{100 * 1000}\right)}{\sqrt{\dfrac{(h_1 - h_2)\ (\text{mts.})}{L \left(\frac{\text{cms.}*50000}{100}\right)}}}$$

$$t = \frac{11.523280\ L^{3/2}}{\sqrt{(h_1 - h_2)}} \qquad ...(19)$$

where,

t => time of travel in minutes

L => length of stream on map in cms.

$h_1 - h_2$ => contour interval in metres.

Eq.(19) may be used to calculate the time of travel of various points over the catchment progressively from the

Table 7.1 : Time of concentration and Isochronal Areas.

Time of Travel (min.)	Ratio t/T_c	Inter Isochro. Area(a) Sq.Kms.	Cummu. Isochro. Area(A) Sq.Kms.	% Inter Isochr. Area
0	0.000			
		6.85	6.85	0.008
30	0.083			
		14.70	21.55	0.017
60	0.166			
		25.47	47.02	0.029
90	0.250			
		51.40	98.42	0.059
120	0.333			
		84.58	183.00	0.097
150	0.416			
		35.60	218.60	0.041
180	0.500			
		46.75	265.35	0.053
210	0.583			
		65.15	330.50	0.074
240	0.666			
		113.25	443.75	0.129
270	0.750			
		158.58	602.33	0.181
300	0.833			
		154.57	756.90	0.177
330	0.916			
		118.10	875.00	0.135
360	1.000			

Table 7.2(a) : Comparison of excess rainfall hyetograph ordinates computed for Case I & II for Event 1.

Time in hrs.	Obser--ved Total Rain (mm)	Comp. Rainfall Excess (mm)	
		Case I =3.14	CaseII CN=81
1	01.20	00.00	00.00
2	02.34	00.00	00.00
3	04.11	00.96	00.00
4	26.50	23.36	06.38
5	29.33	26.19	18.25
6	21.94	18.79	16.88
7	21.02	17.87	17.54
8	18.86	15.72	16.46
9	19.64	16.49	17.65
10	20.56	17.42	18.87
11	19.25	16.10	17.94
12	29.12	25.98	27.51
13	29.78	26.63	28.47
14	24.69	21.55	23.79
15	12.72	09.57	12.31
16	07.55	04.41	07.32
17	06.01	02.87	05.83
18	03.00	00.00	02.91
19	04.26	01.18	04.14
20	02.56	00.00	02.49
21	00.28	00.00	00.27

Table 7.2(b) : Comparison of excess rainfall hyetograph ordinates computed for Case I & II for Event 2.

Time in hrs.	Obser--ved Total Rain (mm)	Comp. Rainfall Excess (mm)	
		Case I =3.21	CaseII CN=80
1	02.07	00.00	00.00
2	01.67	00.00	00.00
3	05.97	02.76	00.00
4	08.39	05.18	00.49
5	06.99	03.78	01.68
6	09.97	06.76	03.92
7	13.28	10.07	07.14
8	09.47	06.26	06.02
9	15.12	11.91	10.71
10	10.36	07.15	07.92
11	12.16	08.95	09.74
12	13.14	09.93	10.95
13	05.07	01.86	04.32
14	02.67	00.00	02.29
15	03.95	00.74	03.41
16	00.15	00.00	00.13
17	00.68	00.00	00.59
18	03.01	00.00	02.62
19	03.52	00.31	03.08
20	00.77	00.00	00.68

Table 7.2(c) : Comparison of excess rainfall hyetograph ordinates computed for Case I & II for Event 3.

Time in hrs.	Obser- -ved Total Rain (mm)	Comp. Rainfall Excess (mm)	
		Case I =9.79	CaseII CN=60
1	00.64	00.00	00.00
2	05.08	00.00	00.00
3	14.13	04.34	00.00
4	15.68	05.89	00.03
5	09.78	00.00	00.77
6	03.61	00.00	00.53
7	05.31	00.00	00.99
8	09.91	00.13	02.47
9	18.19	08.40	06.31
10	13.50	03.71	05.90
11	26.06	16.27	13.66
12	23.47	13.68	14.29
13	08.57	00.00	05.59
14	00.01	00.00	00.01
15	00.29	00.00	00.19
16	00.19	00.00	00.13
17	00.06	00.00	00.04
18	00.51	00.00	00.34
19	01.50	00.00	01.00
20	00.29	00.00	00.19

Table 7.2(d) : Comparison of excess rainfall hyetograph ordinates computed for Case I & II for Event 4.

Time in hrs.	Obser- -ved Total Rain (mm)	Comp. Rainfall Excess (mm)	
		Case I =9.73	CaseII CN=54
1	07.11	00.00	00.00
2	20.88	11.14	00.00
3	08.89	00.00	00.00
4	05.30	00.00	00.00
5	11.44	01.70	00.55
6	11.08	01.35	01.55
7	06.26	00.00	01.26
8	05.97	00.00	01.44
9	14.45	04.72	04.34
10	25.36	15.63	10.08
11	18.89	09.15	09.15
12	11.01	01.28	05.87
13	08.38	00.00	04.70
14	03.28	00.00	01.89
15	01.64	00.00	00.96
16	03.72	00.00	02.19
17	01.66	00.00	00.99

Table 7.2(e) : Comparison of excess rainfall hyetograph ordinates computed for Case I & II for Event 5.

Time in hrs.	Obser- -ved Total Rain (mm)	Comp. Rainfall Excess (mm)	
		Case I =2.90	CaseII CN=85
1	01.49	00.00	00.00
2	03.37	00.46	00.00
3	02.58	00.00	00.00
4	04.73	01.83	00.24
5	04.52	01.62	00.96
6	06.00	03.09	02.14
7	09.83	06.92	04.98
8	17.99	15.08	12.00
9	06.05	03.14	04.56
10	07.12	04.21	05.60
11	10.45	07.55	08.58
12	16.12	13.21	13.85
13	09.07	06.17	08.03
14	05.36	02.45	04.81

Table 7.2(f) : Comparison of excess rainfall hyetograph ordinates computed for Case I & II of Event 6.

Time in hrs.	Obser- -ved Total Rain (mm)	Comp. Rainfall Excess (mm)	
		Case I =9.94	CaseII CN=87
1	00.00	00.00	00.00
2	12.71	02.77	00.56
3	24.61	14.67	12.21
4	05.54	00.00	03.89
5	01.09	00.00	00.79

downstream end i.e.; from the outlet of the sub-basin.

All the values of the time of travels for different points are then denoted on the map at their respective locations. Curves of specified time of concentration called the "Isochrones" are then drawn through these points by making use of linear interpolation and consideration of stream layout. The areas between various isochrones at an interval of 30 minutes and corresponding cumulative areas are given in Table 7.1.

7.1.3 Computation of Excess Rainfall hyetograph.

7.1.3.a By ϕ-index method

A uniform value of loss rate (ϕ-index) has been computed by trial and error method to make the volume of excess rainfall equal to the volume of direct surface runoff.

Straight line base seperation flow technique is followed to compute the direct surface runoff ordinates from the total runoff ordinates. The excess rainfall ordinates alongwith ϕ-index values computed by ϕ-index method are given in Table 7.2(a) to 7.2(f) for all the six events individually.

7.1.3.b By SCS curve number method

In SCS curve number method the cummulative values of total rainfall blocks are used in the governing equation of SCS method to compute the cummulative values of the rainfall excess for an assumed trial value of the curve number (CN). The final values of

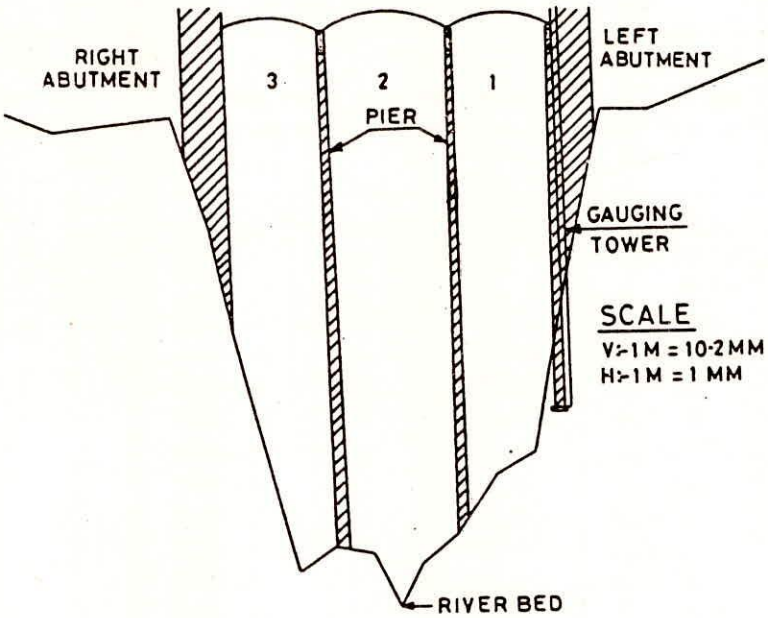

FIG. 7·1 CROSS SECTION OF CHANNEL AT SATRANA

Table 7.3 : Cross sectional details and hydraulic properties at the gauging site.

Sl. No.	Depth of flow (m)	X-Sect. Area (m**2)	Wetted Peri. (m)	Hydr. Radius (m)	Discharge (cumecs)	Vel. (m/sec)
1	1.176	7.06	12.22	0.57	8.92	1.264
2	1.421	10.49	16.26	0.64	14.25	1.360
3	1.666	19.67	40.09	0.49	22.29	1.130
4	1.911	29.93	44.49	0.67	41.87	1.398
5	3.166	92.81	57.02	1.63	233.93	2.520
6	3.823	134.27	70.96	1.89	374.11	2.786
7	5.883	287.74	80.87	3.56	1220.92	4.243
8	7.844	451.48	90.7	4.98	2396.68	5.310
9	9.736	626.96	100.92	6.21	3859.14	6.155
10	12.256	891.20	116.51	7.64	6300.34	7.070
11	13.236	1078.87	167.55	6.43	6799.93	6.303
12	14.316	1344.55	225.59	5.96	8048.41	5.986

Table 7.4 : Equilibrium discharge and velocity corresponding to different rainfall excess intensities.

Sl. No.	Rain. Excess Inten. mm/hr.	Equi. Discha- -rge (cumecs)	Depth of flow (m)	X-sec. Area (m**2)	Vel. (m/sec)
1	1.0	244.4	3.32	102	2.390
2	1.3	317.8	3.70	125	2.540
3	1.6	391.1	3.90	138	2.830
4	1.9	464.4	4.15	155	2.990
5	2.0	488.9	4.20	160	3.056
6	3.0	733.3	4.90	215	3.411
7	4.0	977.8	5.45	253	3.865
8	5.0	1222.3	5.90	288	4.240
9	6.0	1466.7	6.40	330	4.444
10	7.0	1711.2	6.80	365	4.680
11	8.0	1955.7	7.20	397	4.926
12	9.0	2200.1	7.55	427	5.150
13	10.0	2444.6	7.90	460	5.314
14	11.0	2689.1	8.25	490	5.487
15	12.0	2933.5	8.60	522	5.619
16	13.0	3178.0	8.92	552	5.757
17	14.0	3422.4	9.20	577	5.931
18	15.0	3666.9	9.50	604	6.071
19	20.0	4889.2	10.80	735	6.652
20	25.0	6111.6	12.10	872	7.008
21	26.0	6356.0	12.31	905	7.023
22	27.0	6600.0	12.90	1010	6.534
23	28.0	6845.0	13.31	1090	6.279
24	30.0	7333.9	13.82	1210	6.060

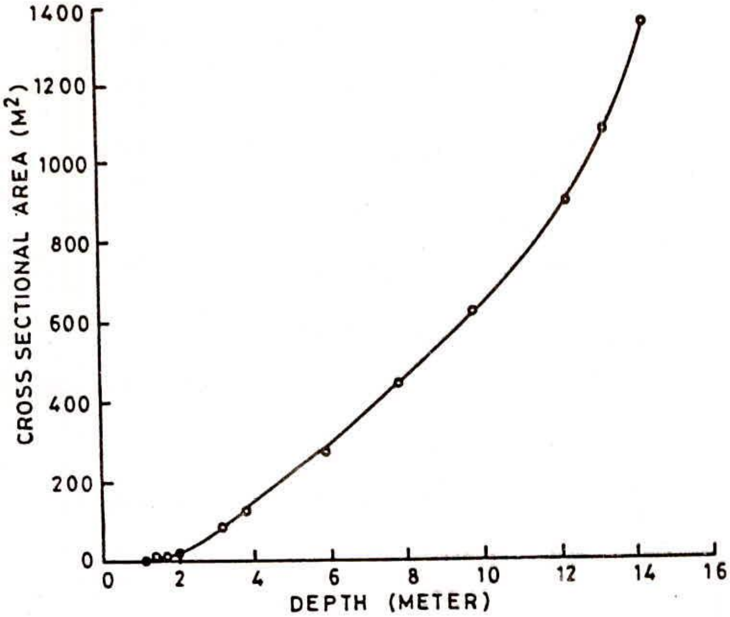

FIG. 7·2 PLOT SHOWING THE VARIATION OF CROSS SECTIONAL AREA (A) WITH DEPTH OF FLOW

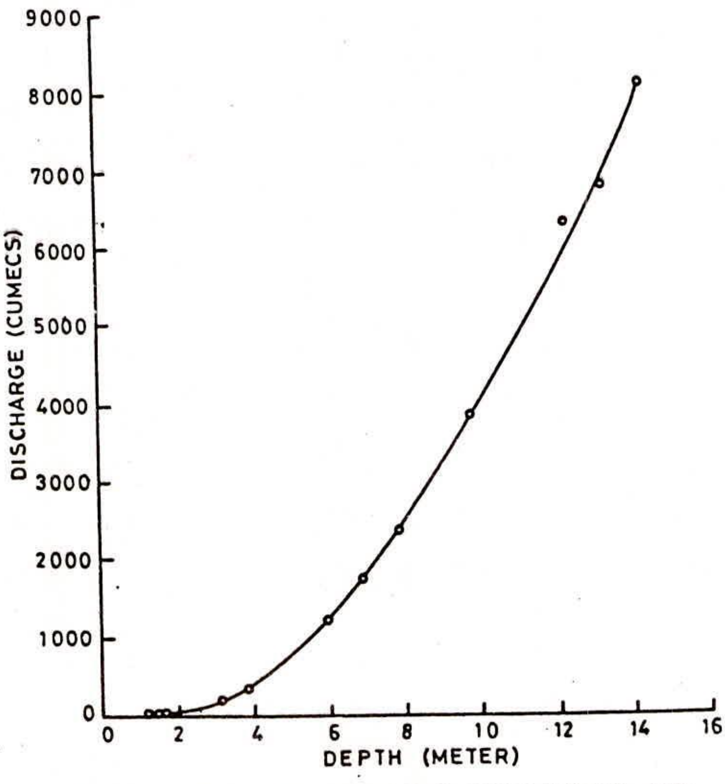

FIG. 7·3 PLOT SHOWING THE VARIATION OF DISCHARGE WITH DEPTH OF FLOW the curve number (CN) is obtained by trial and error method such that the total volume of the excess rainfall is equal to the volume of the direct surface runoff. The excess rainfall ordinates are obtained by subtracting the two consecutive values of the cummulative values computed above. The excess rainfall ordinates alongwith the values of SCS curve number (CN) is given in Table 7.2(a) to (f) for all the six events respectively.

7.1.4 Development of relationship between velocity and intensity of the excess rainfall

The cross-sectional details of the Kolar river at Satarana gauging site are given in Fig. 7.1. The cross-sectional areas and wetted perimeters are calculated at different depths and the same are given in Table 7.3. An average bed slope S = 0.00002 is obtained from the longitudinal section at the gauging site. The value of the Manning's roughness (n) derived from the literature (Chow, 1964) based on the channel surface characteristics is 0.00028. The discharge corresponding to different depths is computed using the Manning's formula. The velocity is computed by dividing the discharge by the X-sectional area. The the mean hydraulic radius (R), discharges and corresponding velocities at different depths are given in Table 7.3. A plot between the depth of water and X-sectional area is made and is given as Fig.7.2. A second plot showing depths and corresponding discharges is shown in Fig.7.3.

The equilibrium discharges corresponding to the different depths is computed by using equation (10). The depths are then derived corresponding to these equilibrium discharges using the plot between depth and discharge (Fig.7.3). Corresponding to these depths the X-sectional areas are derived using the plot between depth and X-sectional area (Fig.7.2).

The values of the equilibrium velocities are computed by dividing the equilibrium discharges by the corresponding X-sectional area. For different rainfall intensities, the computed equilibrium discharges, depths of flow, X-sectional areas and velocities are tabulated in Table 7.4.

Knowing the set of values of intensities of excess rainfall and the corresponding velocities a relationship of the form $v=a\ i^b$ is developed by using least square approach. The relationship obtained in this way is given as :

$$v = 2.387\ i^{0.34} \qquad \ldots(20)$$

The correlation coefficient for the above relation is given as 0.99.

7.2 MODEL APPLICATION

The methodology given in section 6.5 is applied for the six events of dates 28.3.83, 10.8.84, 31.7.85, 13.8.85, 15.8.86 and 27.8.87. These events would be called event no. 1, 2, 3,---,6 respectively, hereinafter in this study. The analysis is done taking two cases as :

Case I : In this case the ϕ-index method is used for computing the excess rainfall ordinates.

Table 7.5 : Summary of the velocities and GIUH based Clark model parameters.

Event No.	Summary of results							
	Case I				Case II			
	Velocity	T_c	r	$r/(T_c+r)$	velocity	T_c	r	$r/(T_c+r)$
1	5.762	2.777	4.088	0.595	5.762	2.777	4.088	0.595
2	5.542	2.888	4.184	0.591	5.386	2.971	4.252	0.589
3	5.658	2.828	4.134	0.593	4.786	3.344	4.858	0.592
4	5.504	2.907	4.201	0.591	4.894	3.270	4.786	0.594
5	5.373	2.978	4.256	0.588	4.586	3.489	4.966	0.587
6	4.011	3.990	5.689	0.588	5.589	2.863	4.164	0.592

Table 7.6(a) : Comparison of GIUH characteristics for Case I & II.

Event No.	Characteristics of GIUH					
	Peak Flow (cumecs)		Time to Peak (hrs)		Base Length (hrs)	
	Case I	Case II	Case I	Case II	Case I	Case II
1	47.50	47.50	2.92	2.92	10.29	10.29
2	45.68	44.40	3.03	3.12	10.70	11.01
3	46.64	39.45	2.97	3.51	10.48	12.39
4	45.37	40.34	3.05	3.43	10.78	12.12
5	44.29	37.81	3.13	3.66	11.04	12.93
6	33.06	46.07	4.19	3.01	14.79	10.61

Table 7.6(b) : Comparison of GIUH based Clark IUH characteristics for Case I & II.

Event No.	Characteristics of GIUH based Clark IUH					
	Peak Flow (cumecs)		Time to Peak (hrs)		Base Length (hrs)	
	Case I	Case II	Case I	Case II	Case I	Case II
1	46.19	46.19	3.0	3.0	30.50	30.50
2	46.21	46.20	3.0	3.0	31.50	32.00
3	46.19	39.59	3.0	3.5	31.00	36.50
4	46.20	39.60	3.0	3.5	31.50	36.00
5	46.20	39.59	3.0	3.5	32.00	37.00
6	34.64	46.20	4.0	3.0	42.50	31.50

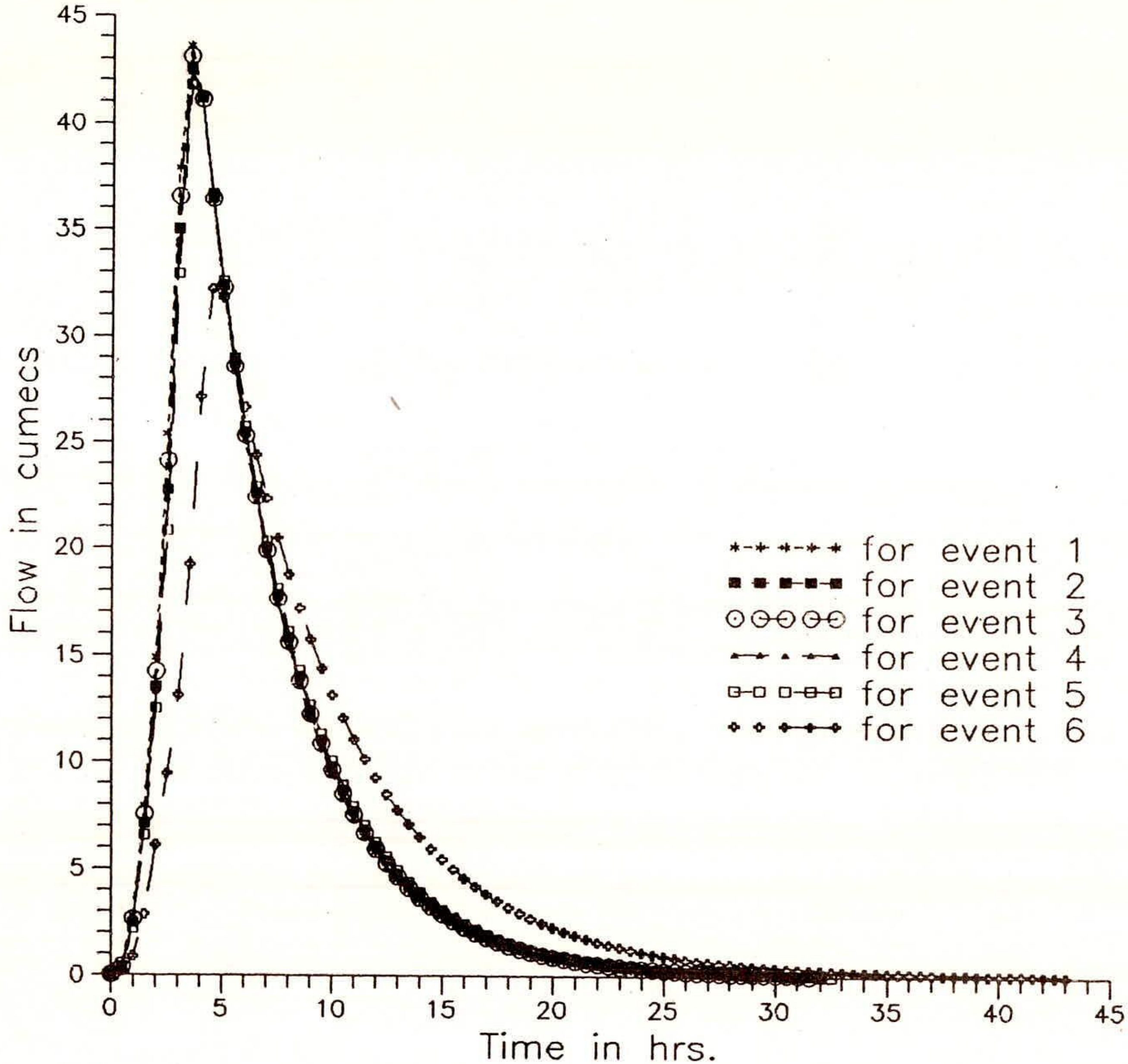

Fig. 7.4(a) 1 hr. unit hydrographs derived by the model for different events (Case I).

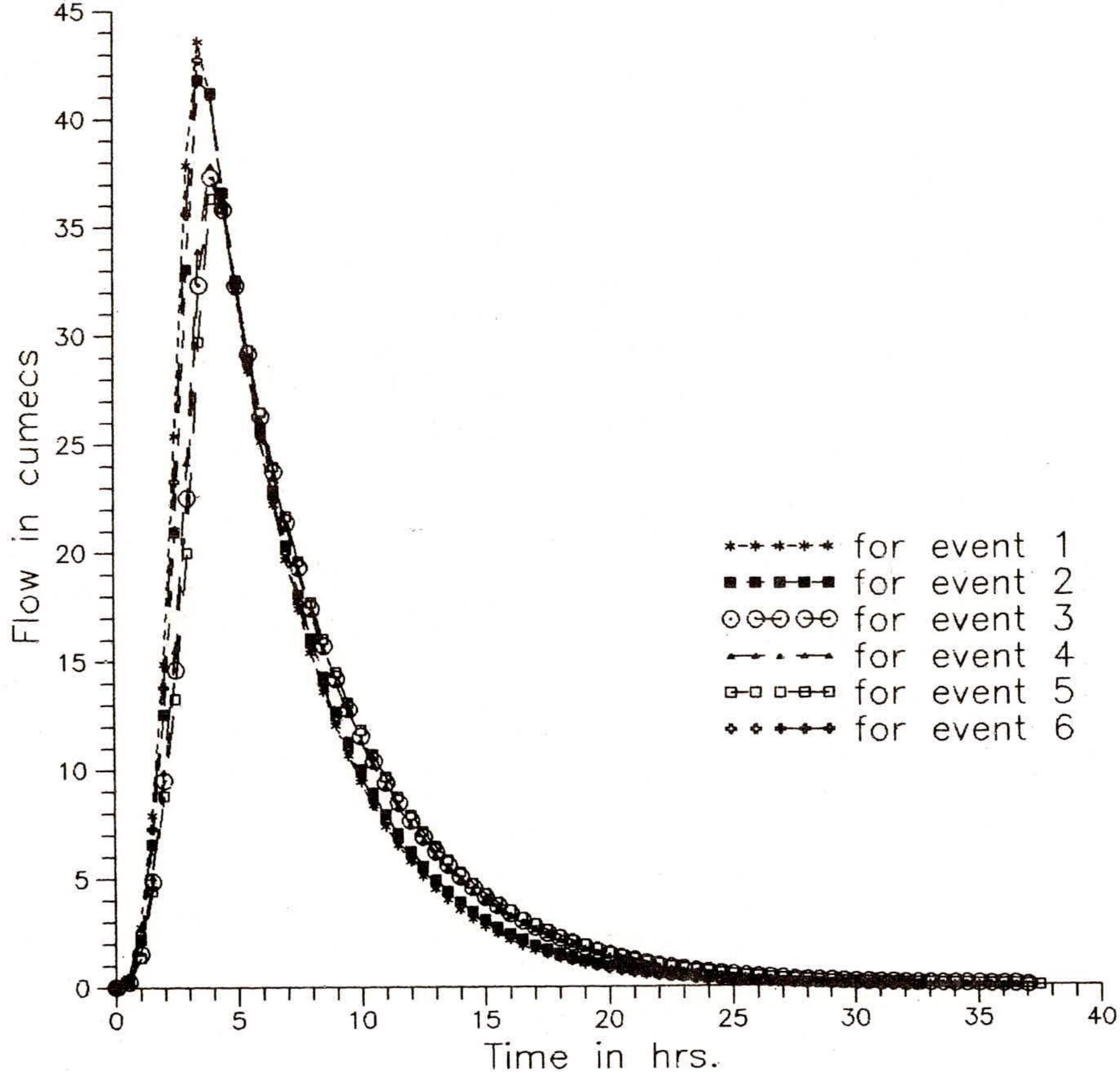

Fig. 7.4(b) 1 hr. unit hydrographs derived by the model for different events (Case II).

Table 7.7 : Comparison of Peak flow and Time to Peak Flow of Observed and Computed Direct Surface Runoff for Case I & II.

Event No.	Peak Flow (cumecs.)					Time to Peak (hrs.)				
	Observed	Case I		Case II		Obser-ved	Case I		Case II	
		Computed	%error	Computed	%error		Comp.	%error	Comp.	%error
1	4847.90	4469.68	7.8	4873.77	0.5	13.00	15.00	15.4	16.00	23.1
2	2024.42	1739.79	14.1	1664.75	17.8	12.00	14.00	16.7	14.00	16.7
3	1273.78	1572.92	23.2	1461.43	14.9	14.00	14.00	00.0	15.00	8.3
4	1370.73	1240.44	9.5	1071.61	21.8	16.00	13.00	18.7	15.00	12.5
5	1948.31	1585.36	18.6	1554.00	20.2	13.00	15.00	15.4	15.00	15.4
6	874.83	540.27	38.2	669.63	23.4	5.00	7.00	40.0	6.00	20.0

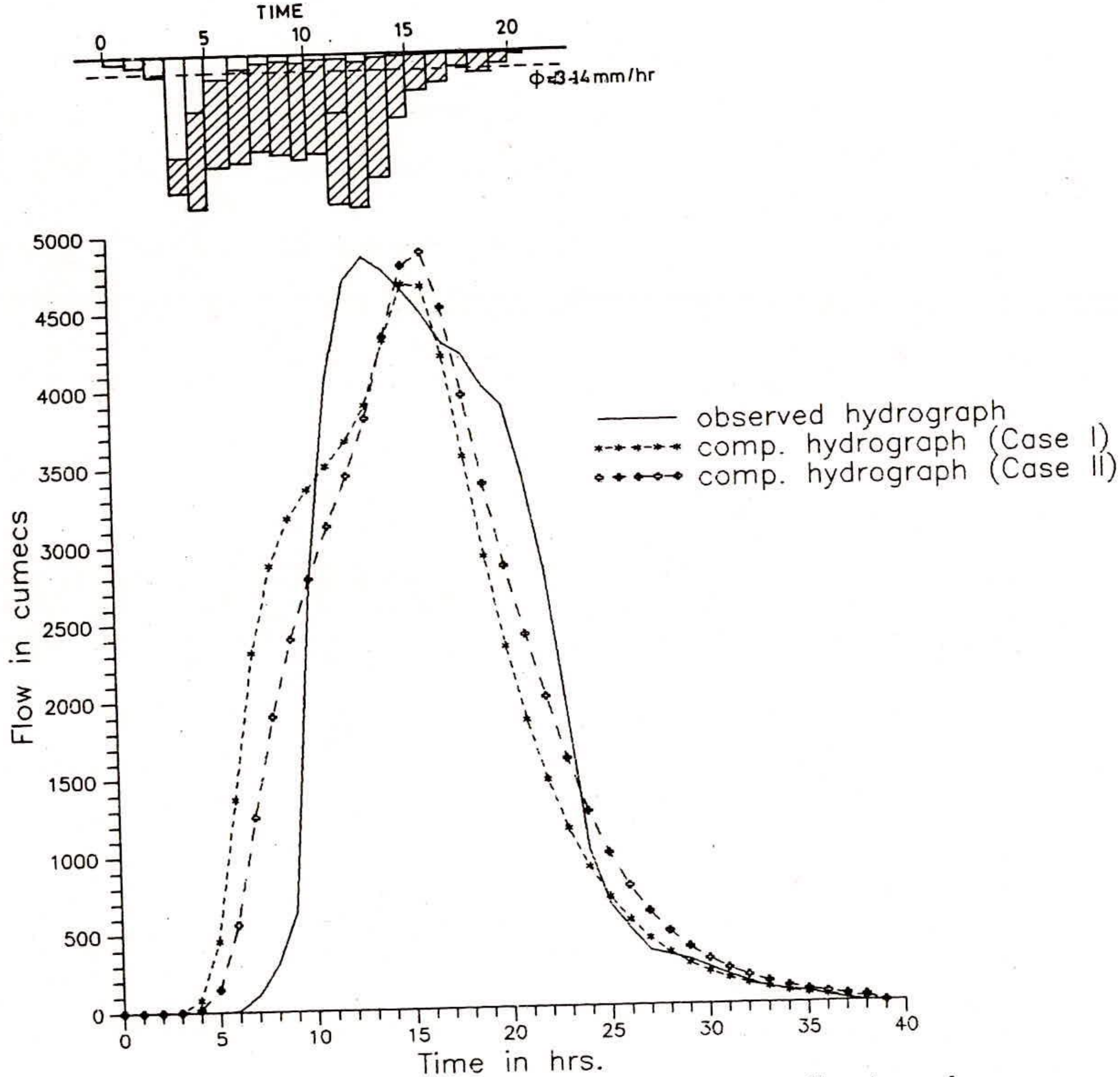

Fig. 7.5(a) : Comparison of observed and computed direct surface runoff for event 1 (Cases I & II).

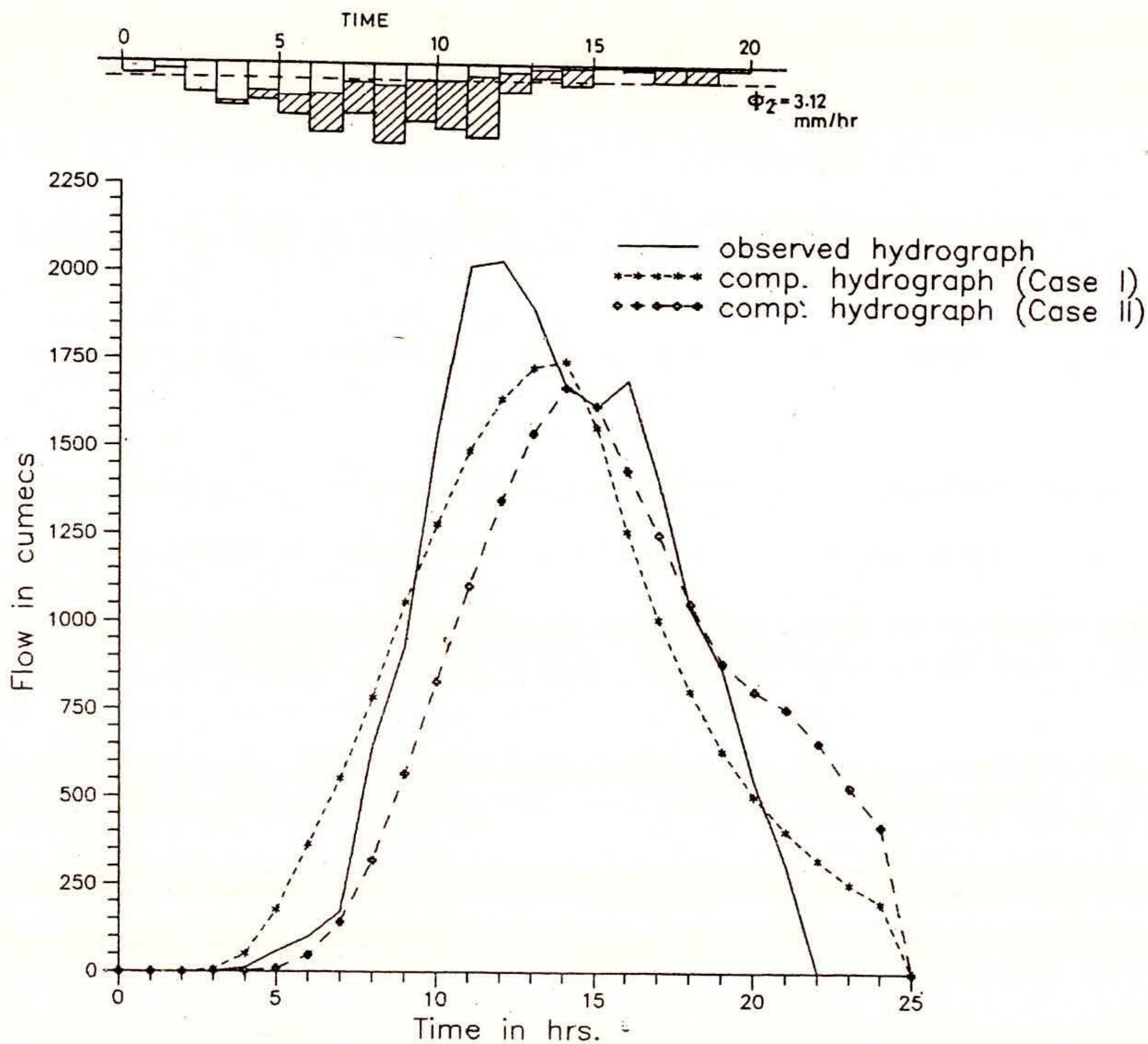

Fig. 7.5(b) : Comparison of observed and computed direct surface runoff for event 2 (Cases I & II).

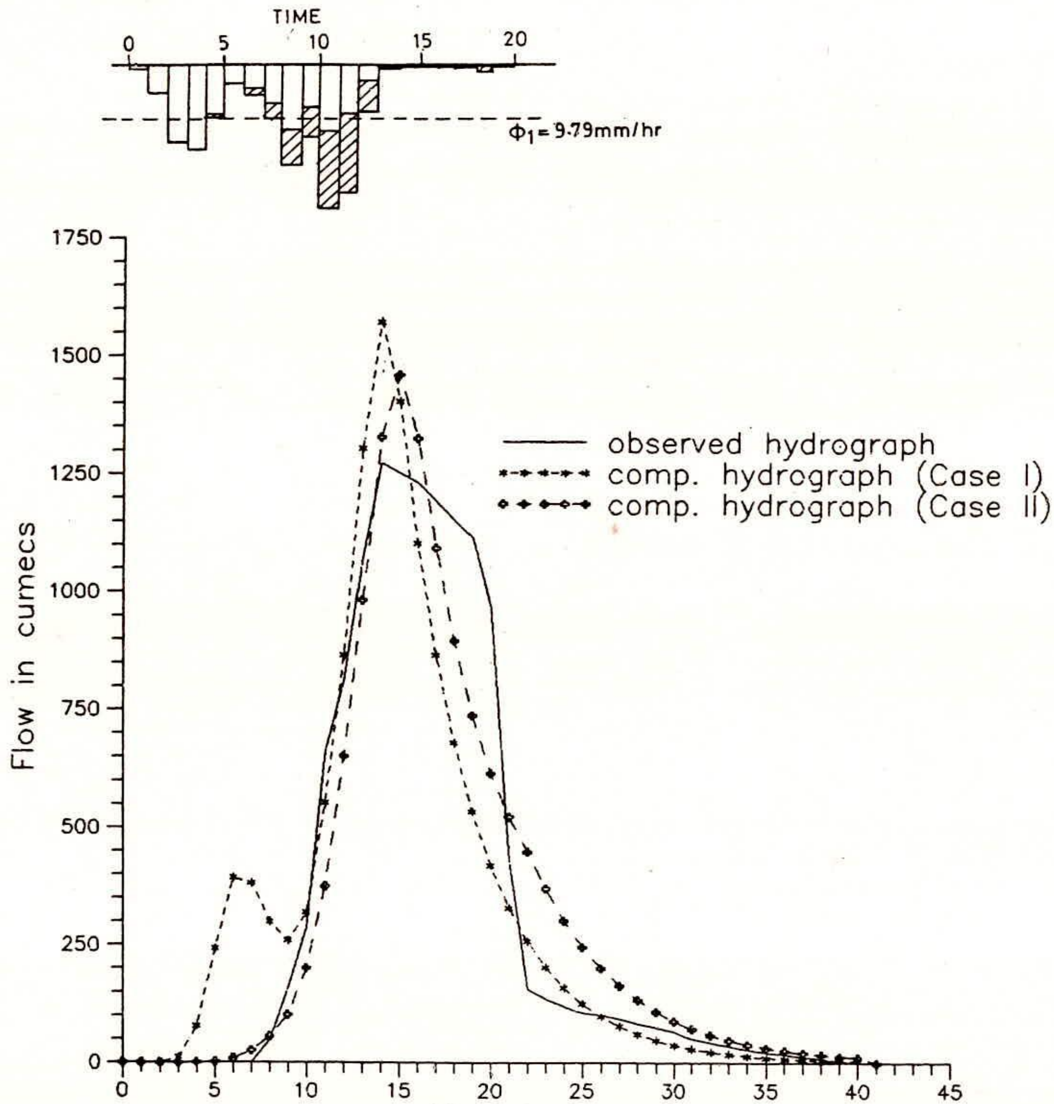

Fig. 7.5(c) : Comparison of observed and computed direct surface runoff for event 3 (Cases I & II).

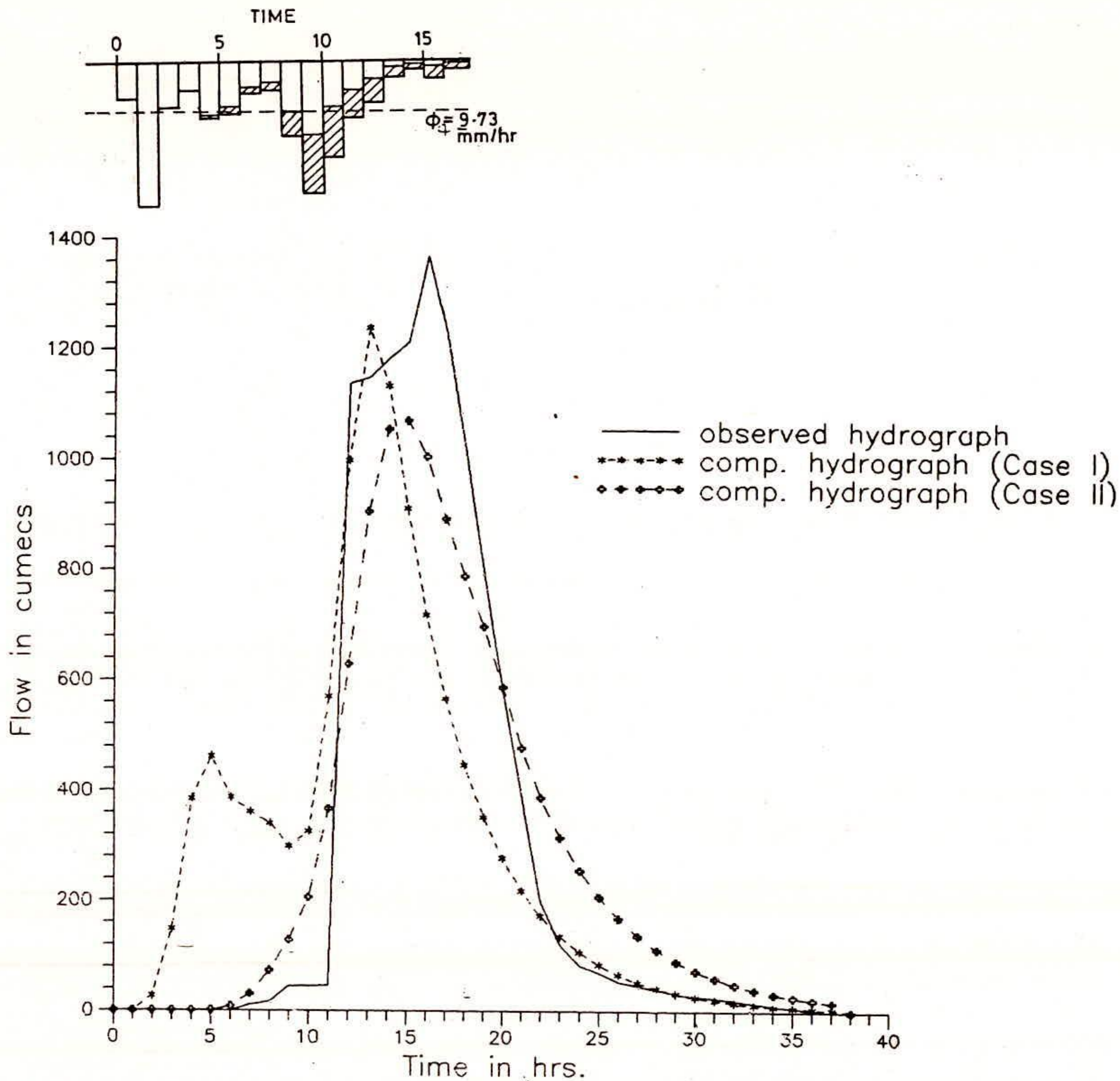

Fig. 7.5(d) : Comparison of observed and computed direct surface runoff for event 4 (Cases I & II).

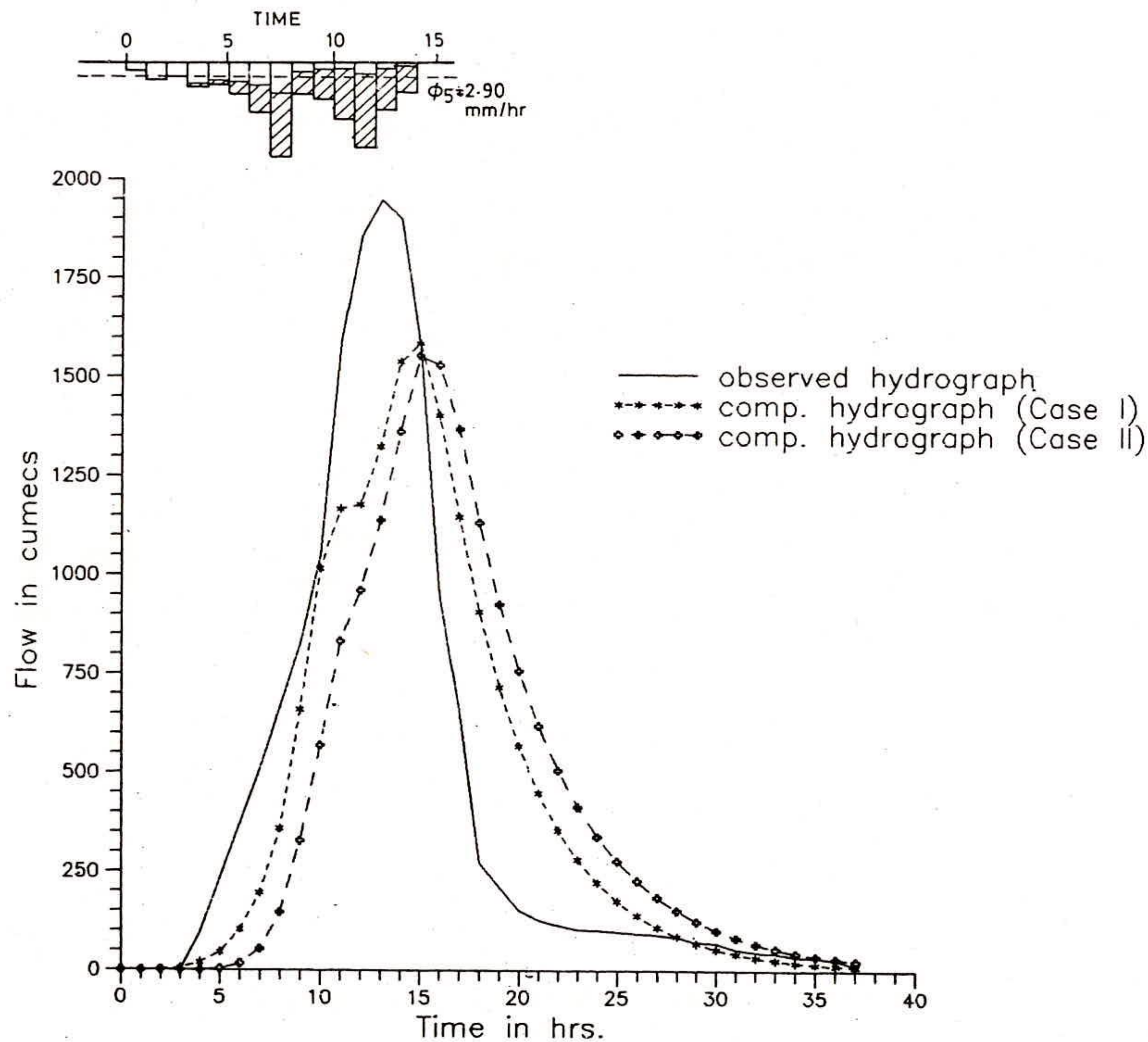

Fig. 7.5(e) : Comparison of observed and computed direct surface runoff for event 5 (Cases I & II).

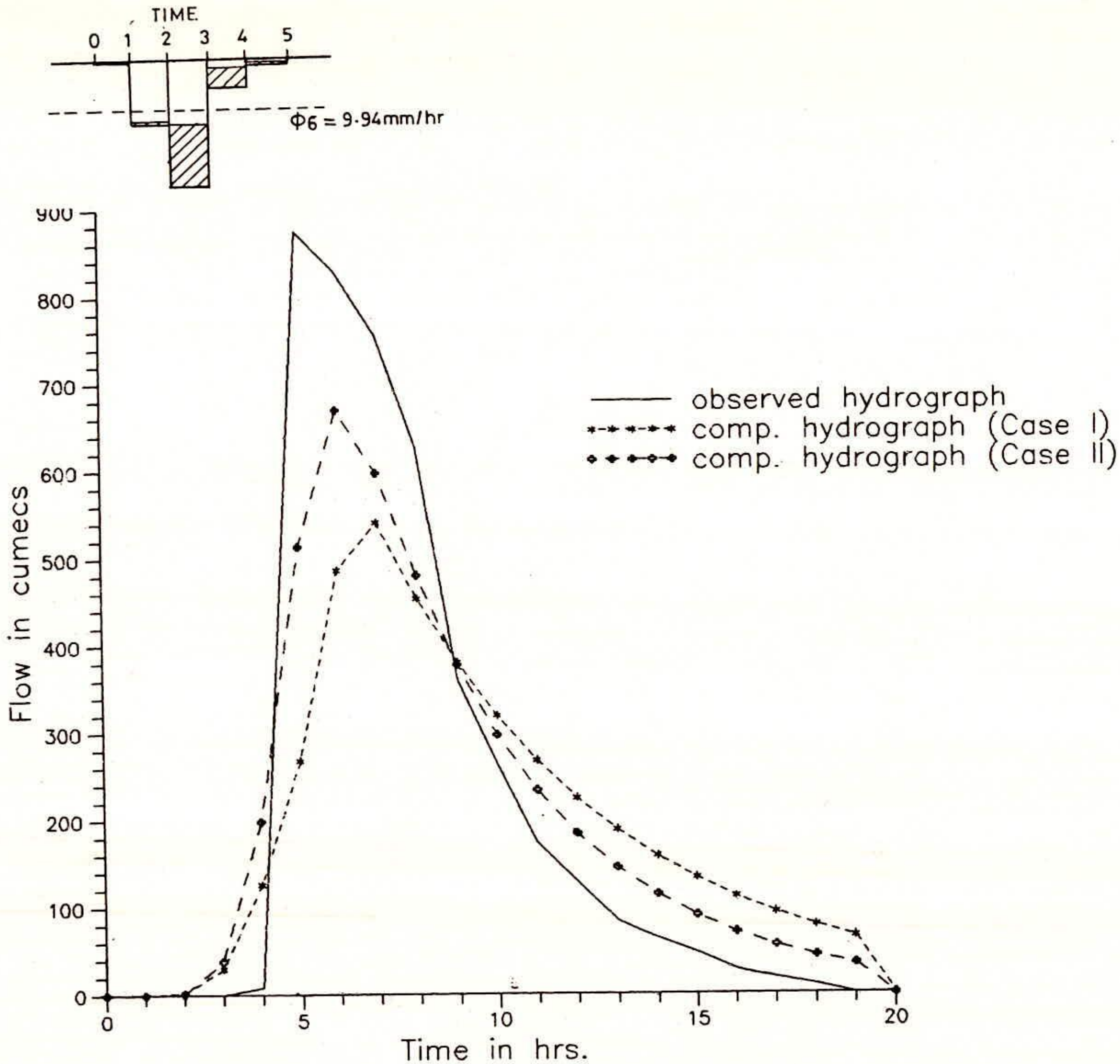

Fig. 7.5(f) : Comparison of observed and computed direct surface runoff for event 6 (Cases I & II).

Case II : In this case the SCS curve number approach is used for computing the excess rainfall ordinates.

The values of the total rainfall ordinates and excess rainfall ordinates computed for events 1 to 6 for cases I & II are given in Table 7.2(a) to (f).

The summary of the velocities and GIUH based Clark model parameters derived for six events using the methodology discussed in section 6.5 for Cases I & II is given in Table 7.5.

Peak flow and time to peak of the instantaneous unit hydrograph by the relationship given by equations (8) & (9) (based on the GIUH approach) and by the GIUH based Clark model for the cases I & II are tabulated in the Table 7.6 (a) & (b) respectively.

One hour unit hydrograph for all the six events are plotted in Fig.7.4 (a) and Fig. 7.4 (b) for the cases I & II respectively.

Table 7.7 gives the values of peak flow and time to peak flow for observed direct surface runoff hydrograph ordinates and those computed by this model for Cases I and II. Fig.7.5 (a) to Fig.7.5 (f) gives the plots of the ordinates of the observed direct surface runoff hydrograph and the ordinates of Cases I & II. These figures also shows the hyetograph of excess rainfall as computed for cases I & II for a better appreciation of the difference in input to the model.

8.0 DISCUSSION OF RESULTS

(i) The computed rainfall excess using ϕ-index method (Case I) and SCS method (Case II) have been compared in Table 7.2 (a) to (f) for the events 1 to 6 respectively. From this table it is evident that ϕ-index method employs a uniform loss rate which is underestimated in the beginning of the storm and overestimated during the later period of the storm. For the SCS method it may be clearly seen by the effective rainfall hyetographs of all the events that the abstractions increase with the increase in the rate of change of cummulative rainfall. This property of the SCS method is well known and may not have a strong physical basis (Chow et. al., 1988; pp-153). However, SCS method results in higher values of loss rate in the beginning and then generally the loss rate gradually reduces as the storm progresses. Thus, the excess rainfall computed by SCS method seems to be more realistic as compared to the ϕ-index method as the loss rate is generally higher in the beginning of the storm and gradually reduces as soil gets saturated.

(ii) The methodology discussed in section 6.6 has been used to estimate the velocity and GIUH based Clark model parameters viz., time of concentred (T_C) and storage coefficient (R) for all the six events. From the Table 7.5 it is observed that the value of the ratio $R/(T_C + R)$ is about 0.59 for all the events for both the Cases I & II. It indicates that this ratio has a specific value of 0.59 for this catchment. This may be attributed to the storage and translation characteristics of the catchment which obviously is expected to be constant if there are no major changes in the catchments characteristics. Further it is also observed from the table that the time of concentration varies in a narrow range between 2.7 hrs. to 4.0 hrs. for different events. Similarly the values of the storage coefficient R are fairly constant (between 4.0 and 5.0) except for event 6 for Case I, where its value is 5.69. Since the values of these parameters have been estimated using the geomorphologic characteristics R_A , R_B and R_L, which are unique for the catchment, the parameters have to be almost constant. However, the storm characteristics which changes from event to event are also involved in the estimation of these parameters therefore the slight variation in the values are obvious.

(iii) In Table 7.6(a) the peak flow of GIUH for Case I and II have been compared. There is a variation of maximum upto 13 cumecs in the peak flow for Cases I & II. The variation may be due to significantly different excess rainfall pattern obtained for Case I and Case II.

Similarly the characteristics of GIUH based Clark IUH have been compared for Case I and Case II in Table 7.6(b).

It is observed from Table 7.6(a) and Table 7.6(b) that the time to peak given by GIUH and GIUH based Clark model approaches differ slightly (i.e., < 0.25 hrs.). This limit of 0.25 hrs. is due to the fact that the computations in this model are done at a time interval of 0.5 hrs. and thus the ordinates are available at the integral multiple of 0.5 hrs. only.

While comparing the baselengths given in Table 7.6 (a) & (b) for GIUH and GIUH based Clark IUH it is observed that the base length for the former case is significantly less than the later. In the case of GIUH approach the base length of the IUH has been computed considering the IUH shape as that of a triangle. On the other hand in the case of GIUH based Clark model approach the IUH shape is derived on the basis of the model parameters T_c and R and the shape of the time-area diagram. This approach gives the IUH whose recession limb decreases gradually and is asymptotic to the time axis. In this case the base length has been computed by truncating the IUH for the unit volume of 0.999 mm. Thus the difference in the base length computed for these two different approaches is due to considering the different shapes of the IUH.

(iv) One hour unit hydrographs are derived using this model for Cases I & II and are shown in Fig.7.4(a) & (b). It is seen that for Case I (Fig.7.4(a)) all the derived unit hydrographs are almost matching except the last one which happens to be a very small event and the discrepancy may be owed to the non-linearity in the system. Also, for case II there is a fair degree of matching between the individual unit hydrographs. The small discrepancy may again be due to varying amount of excess rainfall for the individual events. It is also seen from these two figures that the peak flow and time to peak for the two cases matches perfectly.

(v) Table 7.7 gives the comparison of the characteristics of observed and computed direct surface runoff hydrographs. It is observed that for higher peak flow values the error in the computed peak is less than those having lower peak flows. The difference for the first five cases is less than 20%. The sixth event, which is having only 2 & 4 effective rainfall blocks only for Case I and II respectively, has an error of 38% and 23% respectively which is very high. The peak of event 1 in Case II is matching perfectly, however, the time to peak in this case is off by 3 hours which is very high. However, the importance of using SCS curve number method is that in this a prior estimate of excess rainfall can be done which is not possible by ϕ-index method.

Time to peaks computed for the six events for Case I and II shows a departure of 1 to 3 hours. This difference is also significant as the percentage discrepancy is around 7% to 25% with respect to the observed time to peak. Howevever, it may be emphasised here that a better a priori estimate of excess rainfall hyetograph would give a better matching of the time to peak.

Fig. 7.5(a) to (f) gives the plots of the observed and computed hydrographs (for Cases I & II) for the six events respectively. It is clear from the plots that there is a significant improvement in the match in general when excess rainfall is computed by SCS curve number method.

9.0 CONCLUSION

From this study the following conclusions are drawn :

(1) The parameters of the Clark model could be estimated satisfactorily by using geomorphological characteristics instead of using the observed runoff data, which is not available for the ungauged catchment.

(2) The ratio between storage coefficient (R) and the sum of storage coefficient and the time of concentration (T_c), i.e., $R/(T_c+R)$, has a unique value for a particular catchment. Thus the value of this ratio may be ascertained for a catchment which may then be used for employing simple Clark model also.

(3) Generally, the ϕ-index method of computing the excess rainfall underestimates the infiltration in the beginning and overestimates it in the later portion of the storm. This shifts the entire hydrograph to the left side of the observed hydrograph. While applying the SCS method it is observed that a better reproduction of the flood event is achieved. A better a priori estimate may be made using methods like SCS curve number for which different catchment characteristics like soil type, land use, treatment class, hydrologic conditions and antecedent moisture conditions would be needed as input.

(4) Based on the availability of the rainfall and runoff data for flood events three types of situations generally arise. In case both the rainfall and runoff records are available the unit

hydrograph can be derived easily using these records. However, if only the rainfall record is available the methodology described in this report may be used provided a priori estimate of excess rainfall is available. In case only runoff records are available then also this methodology can be applied after making suitable modifications in the velocity estimation procedure.

(5) For the estimation of the velocity, a relationship is established between the velocity and intensity of excess rainfall for the Kolar sub-basin. The maximum intensity of rainfall excess for each flood event is used in the developed relationship for the computation of first estimate of the velocity. Later on, the final velocity is obtained by trial and error method for the close agreement of instantaneous unit hydrograph based on the GIUH based Clark Model and the geomorphological instanteneous unit hydrograph. The application of this approach has reproduced the flood events very well.

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