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FLOOD HYDROGRAPH SIMULATION IN KOLAR SUB-BASIN USING EVENT BASED DISTRIBUTED RAINFALL-RUNOFF MODEL

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PREFACE

An accurate estimation of flood hydrograph for a given rainfall event is one of the important aspects to be considered for planning, design and operation of the water resources projects. With the advancement in computational facilities and considerable improvement in the hydrological data base, significant progress has been made in the area of rainfall-runoff modelling. Number of event based models have been developed and applied to simulate the flood hydrographs for the various sizes and types of catchments. Whenever a catchment receives rainfall which is fairly uniform in space, its response as a flood hydrograph can be simulated using lumped event based models such as Nash and Clark models. However, if the highly non-uniform rainfall occurs over a catchment the application of such models may not be able to provide satisfactory results. In such a situation distributed event based models are capable of simulating the flood hydrographs within the desirable accuracy.

In this study a distributed event based rainfall-runoff model, developed at National Institute of Hydrology, is successfully applied to simulate the flood hydrographs of Kolar sub-basin of Narmada basin. The model has a very simple structure for representing the spatial and temporal variability of the rainfall over the catchment. The model may be applied to estimate the design flood hydrograph provided the design storm alongwith its temporal and spatial distributions is supplied as input. The model may also be used for real time flood forecasting after making suitable modifications in the software. Other applications of the model include estimation of flood hydrograph for ungauged catchments and filling up of the short term missing runoff records. The study has been carried out by Hemant Chowdhary, Scientist 'B' and R.D. Singh, Scientist 'E' of Surface Water Analysis & Modelling Division of National Institute of Hydrology, Roorkee. The report presents the calibration and validation of the distributed event based model for the Kolar-sub basin. Furthermore the advantages and limitations of the model are also brought out . The step by step procedure for calibration and validation of the model is described . It is hoped that after going through this report, the field engineers would be able to apply this model for solving the field problems related to rainfall-runoff transformations.

(S M Seth)
DIRECTOR

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ABSTRACT

Generally the event based lumped models consider uniform rainfall throughout the catchment area i.e.; an average of the spatially distributed rainfall and thus are not able to include the spatial variability of rainfall which is normally present in medium sized or bigger catchments. An event based distributed rainfall-runoff model has been developed at the National Institute of Hydrology.This model considers the spatial variability of the rainfall and is very useful in simulating the excess rainfall-dircet surface runoff process for those catchments which experience non-uniform rainfall. Such catchments may be eg.; mountainous catchments or larger sized catchments over which rainfall ceases to be uniformly distributed over the whole area.

The structure of the model is very simple, wherein the whole catchment is divided into a number of isochronal areas. The direct surface runoff generated from each of these areas is suitably lagged in time according to the time of travels for individual areas. These are then superimposed in order to have the translated direct surface runoff hydrograph. This hydrograph is then routed through a single linear reservoir having a storage coefficient. This model consists of two parameters viz.; the Time of Concentration and the Storage Coefficient. These parameters are calibrated using trial and error method together with Rosenbrock optimisation technique. During the calibration, the model is run for various trial values of time of concentration and corresponding storage coefficient is determined using Rosenbrock optimisation technique. The set of those values of time of concentration and runoff coefficient is chosen which gives the better reproduction of the observed hydrograph.

In this study the model has been applied for Kolar sub-basin of Narmada river system. The model simulation results suggest a fairly satisfactory degree of acceptance of the model. The model may further be strengthened by incorporating other methods of computing the excess rainfall. The data requirement for the model is not enormous and its preparation is very handy. The model may be employed for design flood studies, filling of short term gaps in runoff records, reservoir inflow forecasting studies, extension of runoff records and real time flood forecating etc.. Its application on many more Indian catchments, would demonstrate its applicability and advantages with greater confidence.

1.0 INTRODUCTION

1.1 General

Better management of available water resources is becoming the need of the hour. Since both the population around the globe and the general living standard, are rising, the demand of water also is constantly rising, notwithstanding the already present gap between the demand and the supply. In such a situation which is becoming worse day by day, atleast in the developing countries it is needless to emphasis the need for the better management of available water resources and to explore new sources of water.

The first requirement for management of water resources is the estimation of the available water in various water bodies. Water flowing down the streams is considered as the main source of fresh water if cost of fetching the water is taken into account. Even in this era of modern technology it is not easy and cheap to estimate the amount of water flowing down the natural streams. More so, when this is to be estimated on a fairly continuous basis. Though in the developing countries, discharge measurements are made atleast once daily, at a few locations on the big rivers and major tributaries, it is not enough to get the complete picture of the available water resource and its distribution in time at all the desired locations. In order to know the quantity of water flowing at many more locations, without incurring big amounts, these indirect approaches in the estimation of runoff in natural streams by knowing the amount of rainfall in their catchment areas. The runoff process is simulated by inputting the rainfall and some other characteristics of the catchment in a suitable mathematical model. This is generally called as rainfall runoff transformation.

Rainfall-runoff modelling of a catchment, thus forms an important part of many hydrological studies. There are scores of types of mathematical models available for this purpose. Black box models to completely physically based models covers the range of deterministic type of models. Black-box models are usually emperical and do not considers the nature of the hydrological system. On the other hand physically based model structures which require extensive database and good computational facilities for their calibration and validation. However, for the Indian conditions at present,the scope of the applications of the physically based models is some limited. In between the two types is the conceptual mathematical models and are best suited for the task and may provide solutions for a number of hydrological problems within the desirable accuracy. There are many conceptual mathematical models available all of which have there own characteristics. The models can also be classified as the event based types and the continuous simulation types of models. In the event based type of models the flow hydrographs of individual storms are simulated whereas continuous models simulate the flow hydrograph continuously for a longer period of time viz., season, year(s) etc..

Characteristics and limitation of both the types of models are quite different. The continuous hydrological models are developed based on the continuous hydrological water balance at each time step for a longer period of time and are used for extending the runoff records, water availability studies, reservoir inflow forecasting, study of the effects of land use changes on the hydrological regimes, consumptive use of surface and ground water and water quality modelling studies etc.. The event based models usually simulate the excess rainfall-direct runoff process and can be applied for filling the short term gaps in the runoff records, the real time flood forecasting, design flood estimation and estimating the flood hydrographs for ungauged catchments etc..

One such event based rainfall runoff model has been developed at the National Institute of Hydrology, India. This model is capable of representing the spatial variability in the rainfall pattern together with the influence of topography, besides other governing factors. The model has been applied for a sub-basin of Kolar river in Madhya Pradesh State of India. The results obtained could demonstrate the scope of applicability and limitations of the model.

1.2 Scope of the Report

A model to simulate individual storm event has been developed at the National Institute of Hydrology, India. The performance of the model has to be ascertained before it may be applied for simulating the flow hydrographs in different regions. This aspect is looked into by applying the model for a catchment in the Kolar basin of the Narmada river system in Central India. This report is the presentation of the case study conducted based on the data available for the period between 1983 and 1987. On one side, this report would ascertain the applicability of the model while on other side it would demonstrated the use of the model and various stages of the model application. Different stages of preparation of data for the model has been well illustrated in the report.

The main reason for selecting this sub-basin for the study is that all the required input for the model as well as catchment boundary map showing the topographic contours and locations of raingauge stations on the scale of 1:50,000 were readily available. Furthermore, the event based rainfall-runoff simulation study of Kolar sub-basin also become necessary as a dam is under construction within the sub-basin at the Kolar river and number of hydrological problems are likely to come up after the construction of the dam.

2.0 A BRIEF DESCRIPTION OF EVENT BASED DISTRIBUTED RAINFALL-RUNOFF MODEL

2.1 General

Lumped models such as Clark Model (Clark, 1945), Laurenson Model (Laurenson, 1964) and others are not capable of properly representing the transformation of spatially varying rainfall excess into the direct surface runoff. The distributed modelling approach is better suited for this purpose and also for reflecting the combined effects of translation and attenuation. Based on this approach, Mein et al (1974), Boyd et al (1979) and others developed distributed models which consider the excess rainfall direct runoff processes occurring within the boundaries of the catchment divided into sub-catchments. Here a distributed rainfall-runoff model for use in simulation of rainfall-runoff process considering the catchment divided into various isochronal zones to represent the spatially varying rainfall, is presented.

The main features of the model are described in the following text.

2.2 MODEL STRUCTURE

The schematic representation of the distributed model considering the catchment distributed into various isochronous zones is given in Fig. 2.1.

The time of concentration is the most important parameter used for rainfall runoff modelling of catchments having marked spatial variation in the rainfall distribution. It is usually defined in terms of either the physical characteristics of a catchment or the excess rainfall hyetograph and direct surface runoff hydrograph. There are two commonly accepted definitions of

TRH_1, TRH_2.....TRH_n — Average rainfall over different isochronous zones (mm)

A_1, A_2...... A_n — Area of different isochronous zones (Km^2)

ERH_1, ERH_2... ERH_n — Excess rainfall volume hyetograph contributed from different isochronous zones (m^3)

Δt ———— Sampling interval (hrs)

DRH ———— Direct surface runoff hydrograph (m^3/s)

BASE ———— Base flow hydrograph (m^3/s)

DIS ———— Discharge hydrograph (m^3/m)

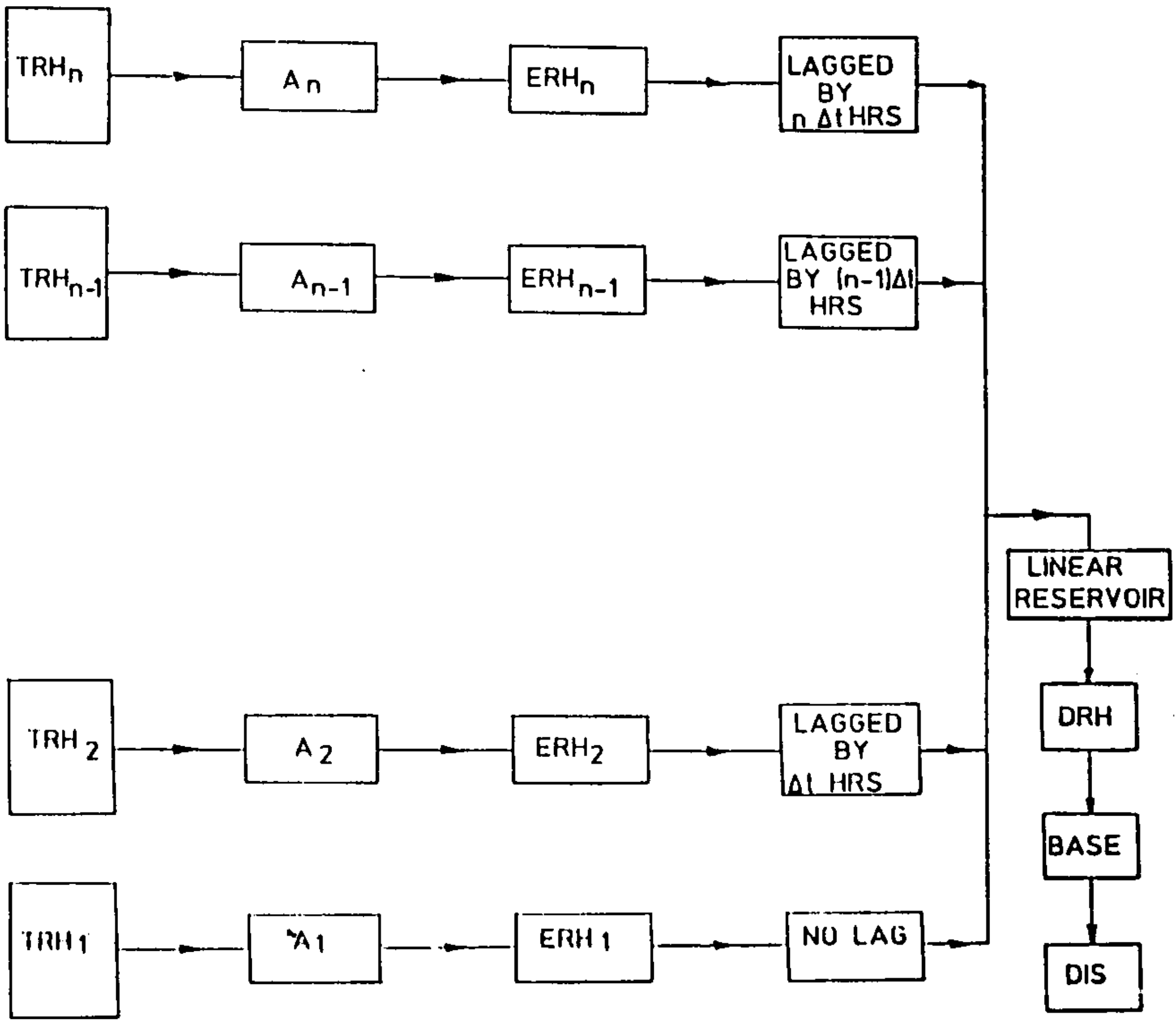

FIG. 2·1 - SCHEMATIC REPRESENTATION OF DISTRIBUTED MODEL

t_c. First, that t_c is the time required for water to travel from the most remote portion of the catchment to its outlet or design point. The methods of estimation based on this definition use catchment characteristics and a precipitation index. A number of empirical formulae based on this definition are available (Singh ,1988). The second definition of t_c is based on a rainfall hyetograph and the resulting runoff hydrograph. The excess rainfall and direct surface runoff hydrograph are computed from these observations. The time of concentration is the time from the end of excess rainfall to the point of inflection on direct runoff hydrograph recession (Singh ,1988).Neither definition yields a true value or even a reproducible value of t_c due to the difficulty of uniquely defining and then measuring the factors affecting it.Therefore,in each specific study some trial and error or an optimization technique has to be adopted to arrive at representative value of time of concentration based on observed records and physical characteristics of the catchment.

The isochronal map is prepared based on the time of concentration computed using one of the empirical approaches referred above. The contours of equal time of travel are plotted on the map taking the time interval same as that of the sampling interval of available rainfall runoff records.The isochronal map for a typical catchment is shown in Fig.2.2. The histogram of Time-Area diagram for the catchment is illustrated in Fig.2.3.

The average rainfall over each isochronal zone and the discharge hydrograph observed at the catchment outlet are the required hydrometeorological inputs for the model. The excess rainfall hyetograph computed using uniform loss rate procedure for each isochronal zone is routed through a single linear reservoir with storage coefficient (R). Thus the time of concentration (t_c)

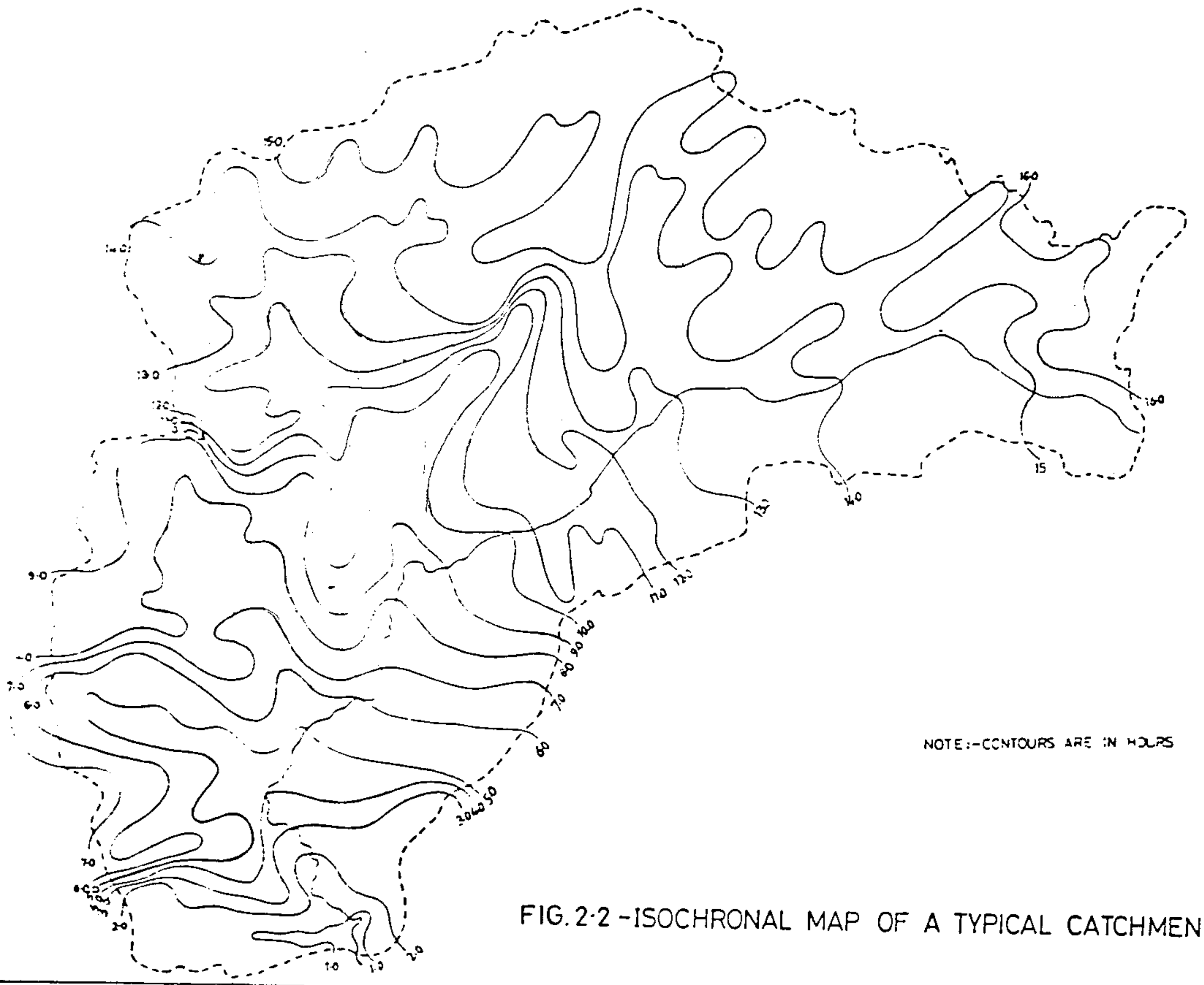

FIG. 2·2 -ISOCHRONAL MAP OF A TYPICAL CATCHMENT

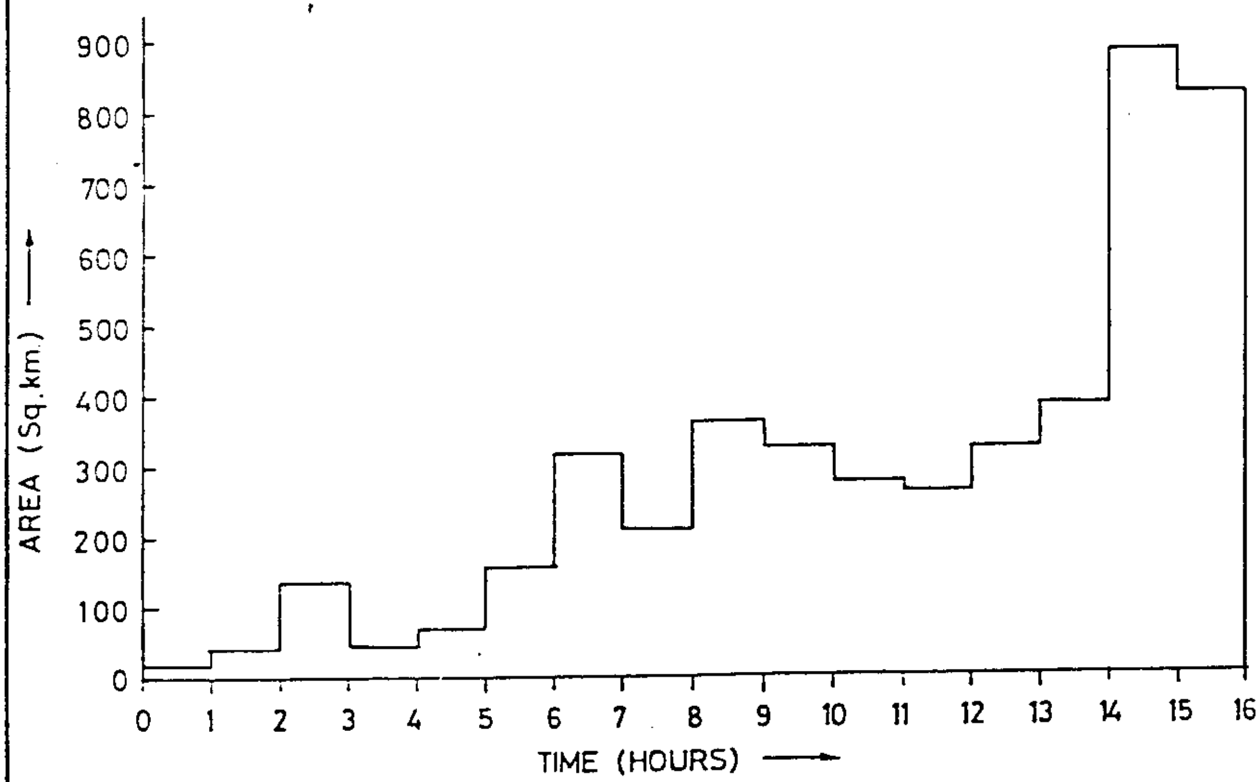

FIG. 2·3– HISTOGRAM OF TIME-AREA DIAGRAM FOR THE TYPICAL MOUNTAINOUS CATCHMENT and storage coefficient of linear reservoir (R) are the two parameters of the model which may be calibrated from the observed rainfall-runoff records. The calibrated model may now be used to simulate rainfall-runoff process for a given storm event for the sub-basin.

2.3 Data Requirement

The model considers the rainfall input distributed over the catchment into various isochronal zones. Hence the drawing of the isochronal map is one of the important aspects of the input data preparation for the model. In order to draw the isochronal map the topographic map of the sub-basin having the elevation contours at shorter interval is needed. For this purpose the topographic map of the sub-basin on the scale of 1:50,000 supplied by the Survey of India may be preferred.

Based on the topographical features of the catchment and making use of any suitable approach as referred in section 2.1 the isochronal map is prepared for equal time of travel considering an interval same as the time interval at which the rainfall information is available. It is preferred that the information is available at an hourly interval so as to attain a fair degree of accuracy. Having the isochronal map, the cumulative areas of various isochronal regions are obtained. These cumulative areas can be specified as the ordinates of time-area diagrams corresponding to each time of travel as fraction of time of concentration.

In a catchment the rainfall is usually observed by some ordinary raingauges (ORGs) and some self recording raingauge (SRRGs) stations. The ordinary raingauge stations, which are generally available in larger number than self recording raingauge stations, provide daily rainfall data. On the other hand at SRRG stations the rainfall data are available in the form of daily charts. These charts are utilized to extract the rainfall at shorter time intervals, say hourly interval. As the model requires the rainfall at shorter duration (say one hour), the daily rainfall observed at ORG stations are required to be distributed. One way of distributing the daily rainfall values to the hourly (or other intervals) rainfall values is using the distributed coefficients computed based on the representative SRRG rainfall data for the respective days.

For this model the rainfall information in terms of either the average rainfall at shorter interval (say one hour) on various isochronal areas or the station rainfall (at say hourly interval) alongwith the corresponding Thiessan weights of the stations for various isochronal areas are required as input. The procedure used in the study to compute the average hourly rainfall over each isochronal areas by the Thiessan weight method is discussed in section 2.4. While computing the average rainfall, the observed rainfall at ORG and SRRG stations for that event are utilized.

The observed discharge at the outlet of the catchment is also required at the same interval as the rainfall for the purpose of calibration of the model. The point of recession in terms of the discharge has to be specified for the calculation of the base flows.

Various parameters used in the Rosenbrock optimization technique are to be supplied alongwith the trial value of the storage coefficient (R). Also, some information for the automatic formating of the graphs is supplied in the end. For a specific value of time of concentration (t_c), the model can be run and an optimum value of R can be obtained for that run. Number of trial runs are usually made to arrive at the optimum values of t_c and R.

2.4 Model Calibration

The steps involved in model calibration from the historical rainfall-runoff records are as follows:

(i) Compute the direct surface runoff after separating the baseflow from observed discharge hydrograph using straight line technique for base flow separation.

(ii) Compute the initial estimate of the time of concentration (t_c) using one of the empirical formula (Singh, 1988).

(iii) Prepare isochronal map taking the time contour interval same as the sampling interval of the historical rainfall-runoff record for different flood events.

(iv) Plot ratio of cumulative area of consecutive isochrones and total area against (t/t_c), where t (time in hrs) = integral multiple of time interval and t_c is time of concentration in hours.

(v) Draw Thiessan polygons on the catchment map and find the Thiessan weights for each raingauge station with respect to each isochronal area separately.

(vi) Distribute the daily rainfall values available at ORG stations with the help of hourly values available at nearby SRRG stations. For this :

* Prepare the mass curve from the average daily rainfall values.
* Prepare the mass curve from the available hourly rainfall values obtained from Self Recording Rain gauge (SRRG) Stations records.

* Compare the slope of the mass curve of average daily rainfall values with that of the SRRG stations and select the most representative SRRG station having similar slope of mass curve.
* Distribute average daily rainfall values into hourly rainfall values based on appropriate SRRG station.

(vii) Compute the average rainfall for each time step (at hourly interval) for every isochronal area based on the Theissan weights calculated in step (v).

(viii) Compute the average rainfall for each time step for cumulative areas based on the averages obtained in step (vii) for individual areas.

(ix) Plot the average rainfall obtained for cumulative area against the ratio of cumulative area and the total area.

(x) For the trial value of time of concentration find out out the number of isochrones by dividing the former by the time interval. Then for each area enclosed between two consecutive isochrones :

* Estimate the area enclosed using the plot developed at step (iv).
* Estimate the average hourly rainfall using the plot developed at step (viii)

(xi) Compute the excess rainfall hyetograph for each isochronal zone using an uniform loss rate procedure to the average hourly rainfall values obtained from step (viii).

For a known value of uniform loss rate (Φ -index), the total excess rainfall volume at the catchment outlet is obtained by adding the excess rainfall volume contributed by different isochronous zones after lagging them by their respective time of travel. The required uniform loss rate is the one which makes total excess rainfall volume equal to the volume of direct surface runoff hydrograph.

(xii) Compute the simulated direct surface runoff hydrograph routing the total excess rainfall hyetograph obtained at step (ix) through a single linear reservoir with storage co-efficient (R) (Singh, 1988).

(xiii) Evaluate the objective function, F, as

$$F = \sum_{i=1}^{n} (y_i - \hat{y}_i)^2 \qquad \text{(a)}$$

Where y_i is the ith ordinate of observed direct surface runoff and $\hat{y}_i$ is the ith ordinate of simulated direct surface runoff at step (x) and n is the total number of ordinates.

(xiv) Minimise the objective function F using Rosenbrock optimisation technique to estimate an optimum value of the storage co-efficient (R)(Rosenbrock,1960).

(xv) Select another trial value of time of concentration (t_c) and repeat step (x) to (xiv) to find out another optimum value of the storage co-efficient (R).

(xvi) Based on the above procedure estimate the optimum values of t_c and R so that the objective function, F is minimized or the efficiency is maximized.

2.5 : Model Validation

The calibrated model may be used to reproduce the direct surface runoff hydrographs of some of the independent flood event not used for calibration. The reproduction may be judged based on the error functions as enumerated in section 2.5.1.

2.5.1: Error Functions

The following error functions may be evaluated using the calibrated model parameters.

(i) Percentage Error in Peak

It is the percentage ratio of the absolute difference between observed and computed hydrograph peak and observed peak.

(ii) Percentage Error in Time to Peak

It is the percentage ratio of the absolute difference between observed and computed hydrographs time to peak and observed time to peak.

(iii) Average Absolute Error

This error function is defined as the average of the absolute value of the differences between observed and computed hydrographs.

(iv) Average Percent Absolute Error

It is defined as the average of absolute value of the differences between observed and computed hydrographs.

(iv) Average Percentage Absolute Error

It is defined as the average of absolute value of percentage difference between computed and observed hydrograph ordinates.

(v) Standard Error

This error function is defined as the root mean squared sum of the difference between observed and computed hydrograph.

(vi) Efficiency

Efficiency of the model in reproducing an event is defined mathematically, as:

$$EFF = \frac{F_0 - F_1}{F_0} \times 100 \qquad (b)$$

$$F_0 = \sum_{i=1}^{n} (y_i - \bar{y})^2 \qquad (c)$$

$$F_1 = \sum_{i=1}^{n} (y_i - \hat{y})^2 \qquad (d)$$

where EFF = Efficiency of the method , y_i = ith ordinate of observed hydrograph , $\bar{y}$ = mean of the observed hydrograph ordinates, $\hat{y}_i$ = ith ordinate of computed hydrograph and n = no. of discharge hydrograph ordinates.

2.6 Presentation of Output

The final results of the optimisation of the storage coefficient are given. The total observed flows are separated into direct surface and base flow components and the excess rainfall is tabulated alongwith the values of the infiltration capacity obtained by constant loss rate method. Finally, the observed and computed hydrograph ordinates are tabulated side by side for easy comparison.

A graphical facility supported by Fortran compiler version 5.1 has been added for drawing of the graphs of computed and observed hydrograph. These graphs may be displayed on the screen for an efficient and easy comparison and also may be drawn on paper sheet whenever so desired.

2.7 Advantages and Limitations

The Event Based Distributed Rainfall-Runoff Model considers the spatial and temporal rainfall variability within the catchment. Therefore the model can be applied to simulate the catchment response within the desirable accuracy particularly for mountainous rain fed catchments. The model structure is so simple that its parameters can be easily calibrated after taking a few trial runs. Before running the program the user has to process different kinds of data required as input to the model.

The model can simulate the excess rainfall and direct surface runoff for different flood events considering one event at a time. The excess rainfall volume and its temporal variations are computed using the uniform loss rate separation procedure. The straight line base flow separation technique is used to compute the direct surface runoff hydrograph. These computations are performed with the help of two different subroutines. If user wants to compute the excess rainfall and direct surface runoff using some other methods, then the separate subroutines developed based on those methods, may be included in the software.Accordingly, necessary changes may be incorporated in the main program for providing the options for different methods to compute the excess rainfall and direct surface runoff.

In the present stage of model development the infiltration loss rate is found out by comparing the observed discharge with the excess rainfall. This aspect of the model requires further improvement with the incorporation of suitable independent method for estimating the variable loss rate which exist in the real situation. Secondly the computer program is able to take only those trial values of time of concentration which are an integral multiple of time interval at which the model is run. It would be better to modify the computer program so that any value of time of concentration may accepted. Thirdly, the Rosenbrock optimisation technique is not suitable when the objective function is multi-modal. Some other technique e.g; Marquardt algorithm may be applied so that absolute minimum objective function may be computed even when it is not uni-modal. The concept of linear channels draining simultaneously from the various isochronal areas is considered in the model structure. It would be better to consider various isochronal areas as linear reservoirs connected to a common linear reservoir through linear channels of different lengths depending upon the placement of the isochronal areas.

The software is developed in FORTRAN-77 language and implemented and tested on IBM compatible personal computers (PC) having Microsoft FORTRAN compiler version 5.1 with graphics library. In case a user wants to run the program for different version of FORTRAN compiler than the Microsoft FORTRAN 77 version 5.1 then it has to be suitably modified in order to meet the compiler's requirement.

The software is user friendly and provides the graphical output of the observed and simulated direct surface runoff hydrographs over the Visual Display Unit (VDU). Such graphical representations of the output help the users during the calibration and validation of the model.

3.0 GENERAL DESCRIPTION OF KOLAR BASIN AND DATA AVAILABILITY

3.1 General

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 22.40 to 23.08 and longitudes 77.01 to 77.29. The catchment has an elongated shape which is oriented in east-west direction in its upper part and norther-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.km. However, this case study is done only for an area of 875 sq km 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 of showing locations of various rainfall stations, stage discharge gauging sites and other hydraulic structures is given in Plate I.

During the period pertaining to the data used in 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.

3.2 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 (adjacent to Ichhawar) and in small pockets elsewhere in which the main crops are wheat and grains. The general response to rain of this upper part of basin appears to be quick.

The second 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 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.

3.3 Data Availability

Topographic data:

The topographic map of Kolar 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.

Rainfall and discharge data:

At the time of carrying out the present study 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-Nasrullagnj road where an automatic gauge recorder (AGR) has been installed. The flow velolcity is meausred 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.

4.0 APPLICATION OF THE MODEL FOR THE KOLAR SUB-BASIN

4.1 General

The model, being event based, is capable of simulating discrete events of rainfall occurrences. The model has two major parameters as (i) the time of concentration (t_c) and (ii) the storage coefficient (R) of the linear reservoir through which all the flows are routed. These parameters have to be established before the model may be used to simulate the catchment's response to the rainfall. For this, the calibration of the time of concentration (t_c) and storage coefficient (R) is done for each event separately.

Value of R is obtained by Rosenbrock optimization technique for different values of time of concentration (t_c). The set of values of R and t_c which gives maximum efficiency is selected for that particular event. Similar sets of values of R and t_c are obtained for each event. Since, the expressions (t_c+R) and R/(t_c+R) are independent of the event an average value of both of them are obtained by averaging the individual values from different storms. Substitution of (t_c+R) in the later gives the value of R. This value of R when substituted in the first expression yields the value of t_c.

This final set of values of t_c and R is then used for simulation of events to validate the model. In case the validation is found satisfactory then these parameters may be recommended for their use in transforming rainfall into corresponding runoff.

One more factor which needs to be determined is the ϕ-index. Since, the value of ϕ-index in dependent on the varying factors like antecedent moisture conditions and land use besides on the more stable factors like soil type, it becomes difficult to fix-up the value of ϕ-index for a particular event. It is suggested that some indirect method may be employed for ascertaining the value of ϕ-index before proceeding for simulation of rainfall-runoff process.

4.2 Data Preparation

(i) 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 F/6, 55 F/5, 55 E/4, 55 F/1, and 55 E/8 on 1:50,000 scale. The watershed divide is marked for the gauge discharge site at Satrana. The contours at a same interval are also marked on the map. At some places where contours are not very clear are left out which would not create any appriciable error in the estimation of time of concentration.

(ii) 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 Kirpich's formula as given below:

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:

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

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:

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

Substituting the values for the various segments we get

$$S_A = 0.024988 \qquad (5)$$

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

$$t_c = 0.06628\, L^{0.77}\, H^{-0.305} \qquad (6)$$

where:

t_c = concentration time in hours.

L = length of stream in kms.

H = slope of the stream.

Substituting values of l and H in eq.(6) we get:

time of concentration for the catchment,

t_c = 5.107 hrs.

or t_c = 306.46 minutes (7)

From eq.(2) and (7) we may write

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

or $$K = t_c \sqrt{S_A} \,/\, L \qquad (8)$$

substituting values of t_c, L and S_A in (8), we get:

$$K = 0.51536 \text{ min./km.} \qquad (9)$$

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

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

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

$$t = \frac{11.523280\ L^{9/2}}{\sqrt{(h_1 - h_2)}} \qquad (10)$$

where :

- t = time of travel in minutes
- L = length of stream on map in cms.
- $h_1 - h_2$ = contour interval in metres.

Eq.(10) may be used to calculate time of travel of various points over the catchment progressively from the downstream end i.e.; 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 map of the sub-basin showing the isochrones at an interval of 30 minutes is given on Plate II. The areas between various isochrones and cumulative areas are also given in Table 4.1.

(iii) Average rainfall over various isochronal areas:

The location of rainfall stations is marked on the map and then Thiessan polygons are drawn. There are four rainfall stations whose data are available. They are Rehti, Jholiapur, Birpur and Brijeshnagar. Rehti is located down south outside the sub-basin. Areas of various polygons are found with the help of Digital planimeter both on the basis of 30 minutes and 60 minutes time intervals. Based on the Thiessen Polygons' areas for each isochronal area, the weights of all the four rainfall stations are computed. These weights are given in Table 4.2 and 4.3 for isochronal areas based on 30 minutes and 60 minutes time intervals respectively. Computation of average rainfall over various isochronal areas is done while running the program.

4.3 CALIBRATION OF THE MODEL PARAMETERS

There are two major parameters used in this model the values of which are to be ascertained by calibration. For being confident enough that the model would correctly simulate the rainfall-runoff process it is required to validate the model thus calibrated against the known response to the rainfall input. Only after the

Table 4.1 : Areas and Cumulative Isochronal Areas.

Time of Travel	Inter Isochronal Area (Sq.km.)	Cumulative Areas (Sq.km.)
0-30	6.85	6.85
30-60	14.70	21.55
60-90	25.47	47.02
90-120	51.40	98.42
120-150	84.58	183.00
150-180	35.60	218.60
180-210	46.75	265.35
210-240	65.15	330.50
240-270	113.25	443.75
270-300	158.58	602.33
300-330	154.57	756.90
330-360	118.10	875.00

Table 4.2 : Thiessan Weights on the basis of 30 minutes isochrones

Time of Travel	Rehti	Jholiapur	Birpur	Brijeshnagar
0-30	1.00	0.00	0.00	0.00
30-60	1.00	0.00	0.00	0.00
60-90	0.33	0.67	0.00	0.00
90-120	0.00	1.00	0.00	0.00
120-150	0.00	1.00	0.00	0.00
150-180	0.00	1.00	0.00	0.00
180-210	0.00	0.20	0.80	0.00
210-240	0.00	0.00	1.00	0.00
240-270	0.00	0.00	0.98	0.02
270-300	0.00	0.00	0.42	0.58
300-330	0.00	0.00	0.30	0.70
330-360	0.00	0.00	0.00	1.00

Table 4.3 : Thiessan Weights on the basis of 60 minutes isochrones

Time of Travel	Rehti	Jholiapur	Birpur	Brijeshnagar
0-60	1.00	0.00	0.00	0.00
60-120	0.22	0.78	0.00	0.00
120-180	0.00	1.00	0.00	0.00
180-240	0.00	0.15	0.85	0.00
240-300	0.00	0.00	0.65	0.35
300-360	0.00	0.00	0.25	0.75

calibration and validation of these model parameters, the model may be used for simulation purposes. The model parameters to be calibrated are (i) the storage coefficient (R) and (ii) the time of concentration t_c.

The data of six events during the period 1983-87 is available. The dates of the events are event 1 : 28 August 1983, event 2 : 10 August 1984, event 3 : 31 July 1985, event 4 : 13 August 1985, event 5 : 15 August 1986 and event 6 : 27 August 1987. From these six events the first four are chosen for calibrating the model parameters. The data of the last two events are used for validation of the model.

The value of R is obtained by Rosenbrock optimization technique to minimize the objective function computed as the sum of the squares of deviations between the observed and the computed hydrograph ordinates for each trial value of t_c. In order to arrive at optimum values of the parameters t_c and R the model is run for a few trial times of concentration (t_c). The plots between observed and simulated discharge values for the events 1, 2, 3 and 4 for various trial values of t_c are given in Fig. 4.2, 4.3, 4.4 and 4.5 respectively. Those values of R and tc are chosen for each of the four events which gives the minimum objective function or the maximum efficiency. The summary of the results obtained for the first four events are tabulated in Table 4.4. The observed hydrograph ordinates and the computed hydrograph ordinates for different trial values of time of concentrations (t_c) for the Events 1, 2 ,3 and 4 are given in Tables 4.5, 4.6, 4.7 and 4.8 respectively. The set of values of t_c and R which gives the maximum efficiency is given in Table 4.9 for these four events.

$$\text{The average value of } (t_c+R) = 8.485 \text{ h} \qquad (11)$$

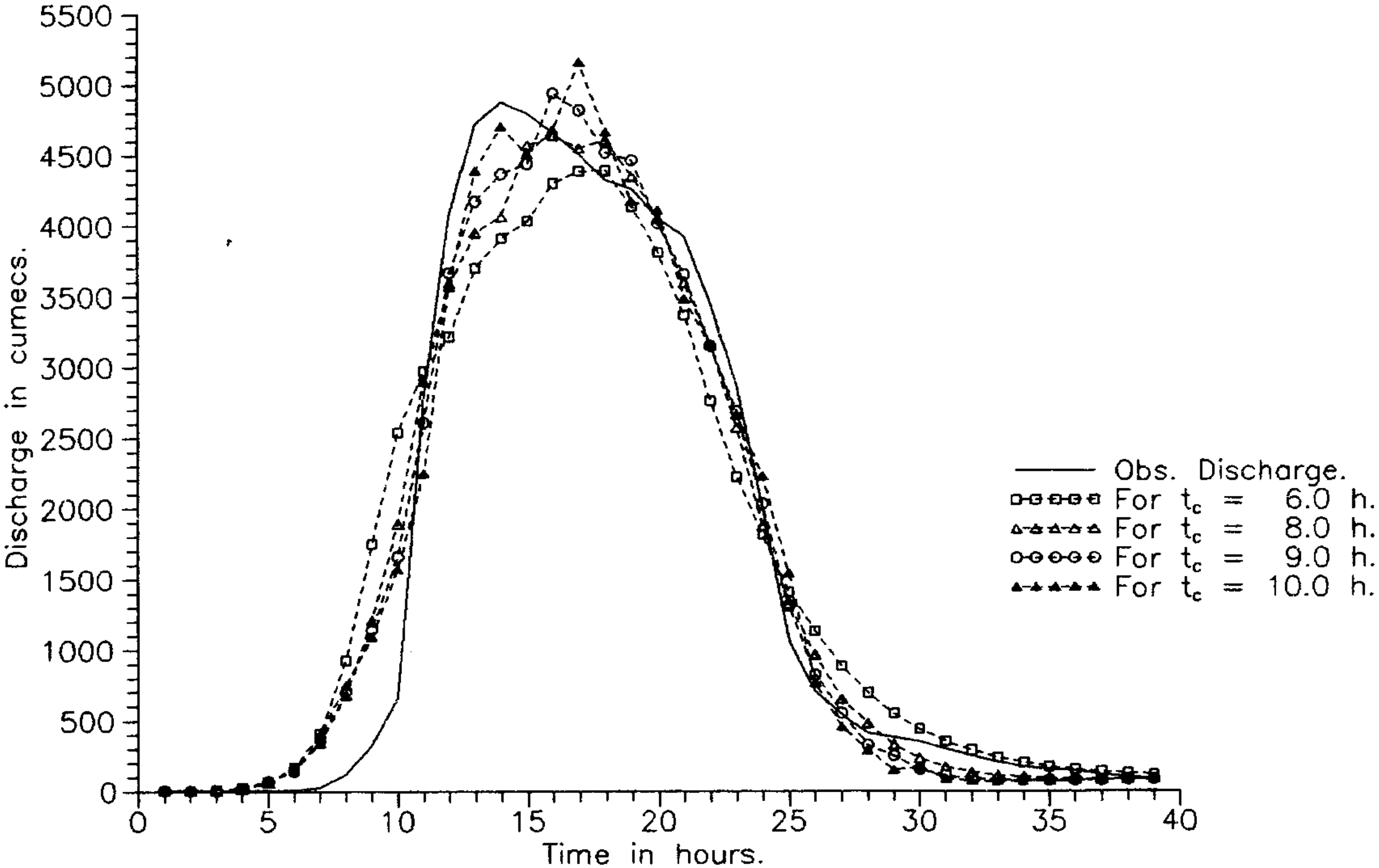

Fig. 4.2 : Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 1 (Time interval = 1.0h).

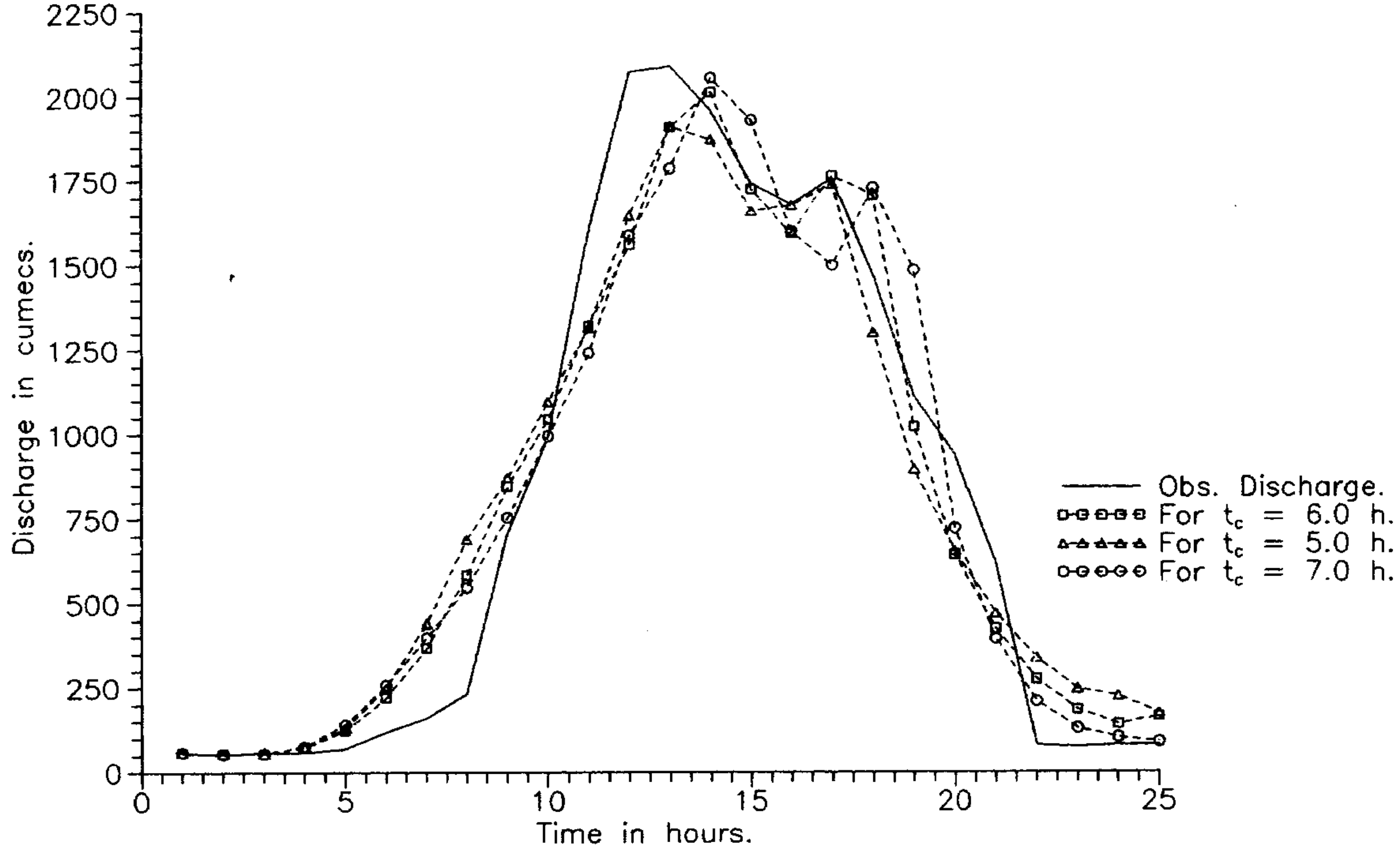

Fig. 4.3 : Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 2 (Time interval = 1.0h).

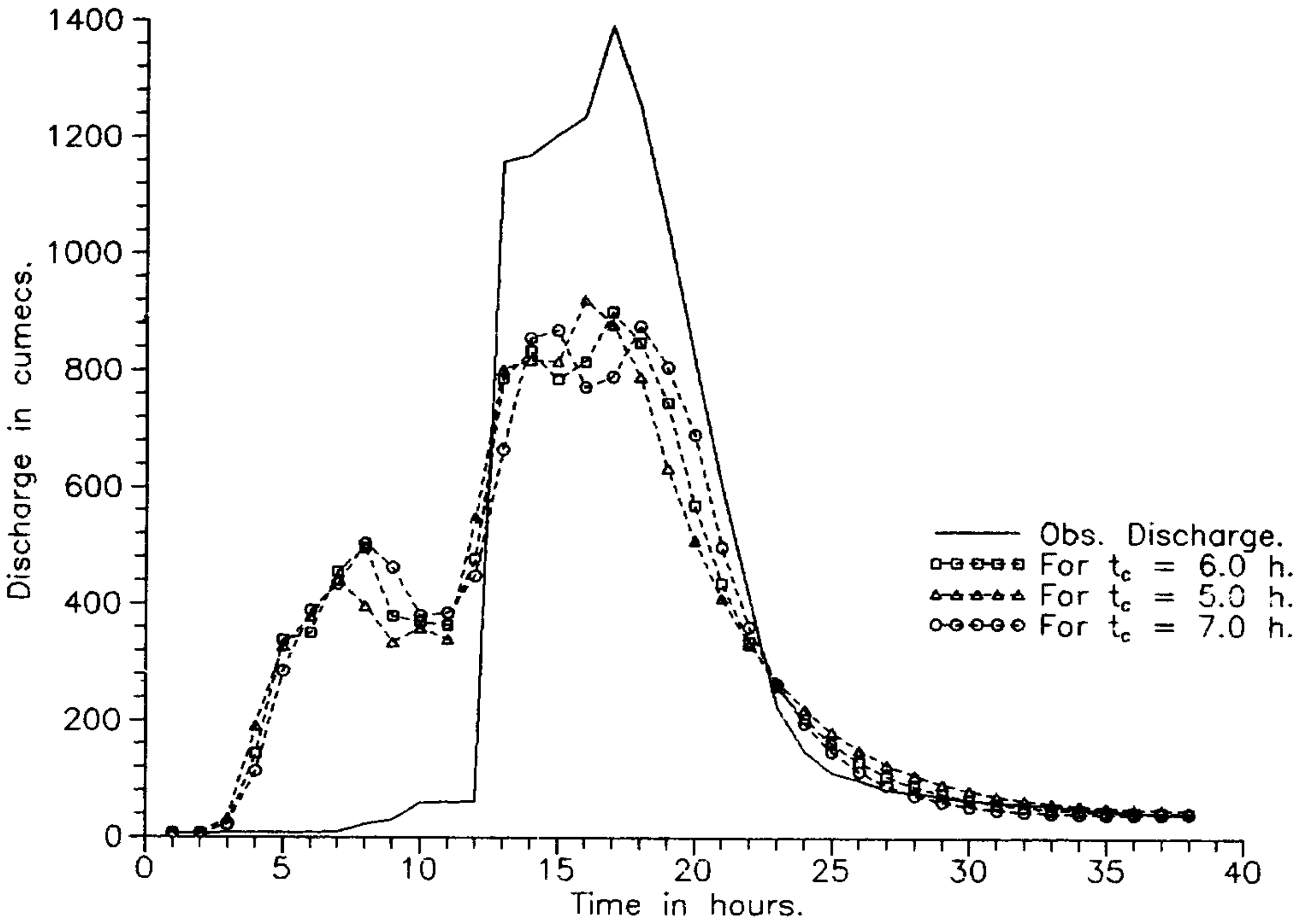

Fig. 4.4 . Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 3 (Time interval = 1.0h).

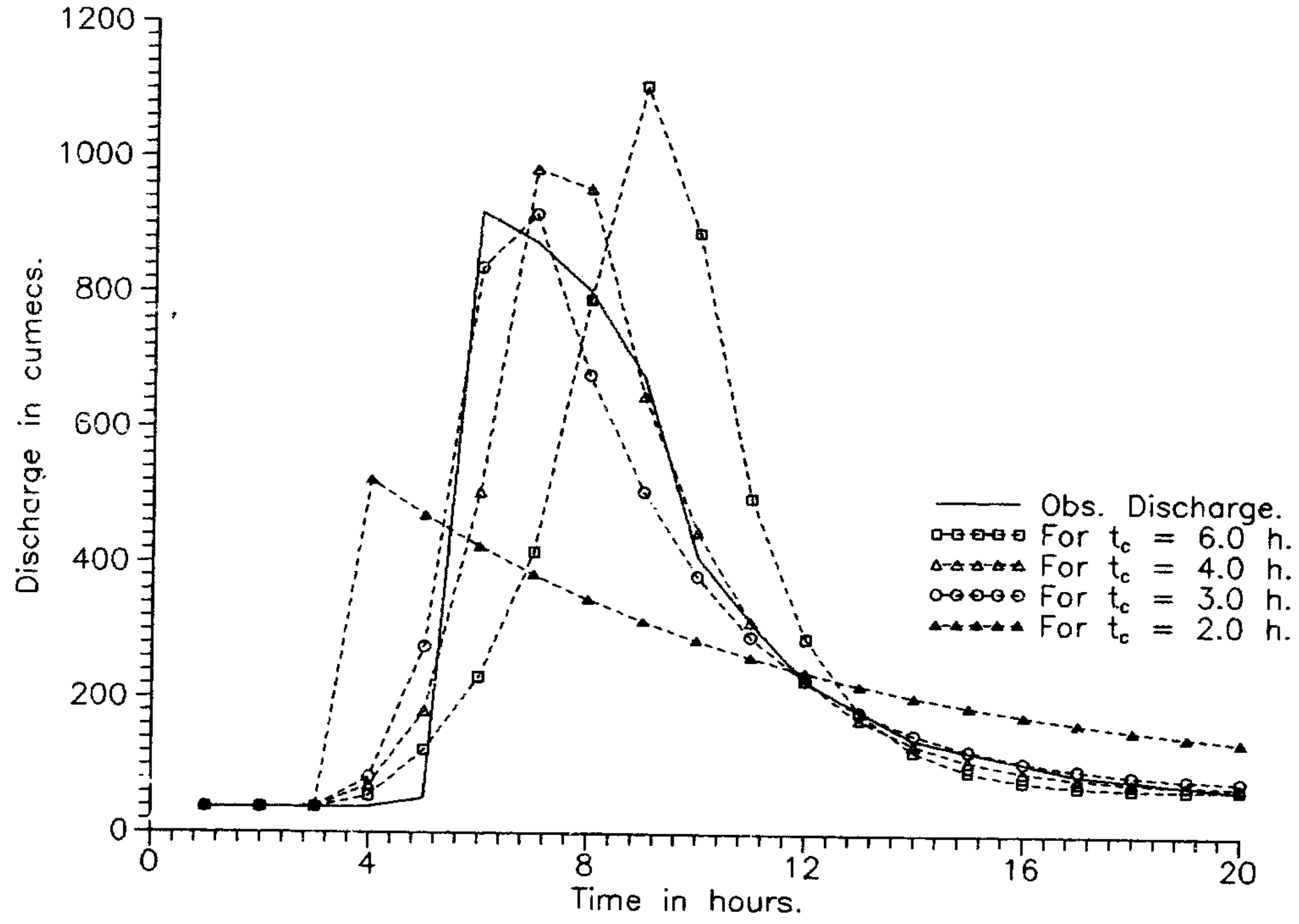

Fig. 4.5 : Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 4 (Time interval = 1.0h).

Table 4.4 : Summary of the results of calibration runs for Events 1, 2, 3 and 4.
(Time interval = 1.0h)

Event No.	t_c	R	ϕ	Obs. Peak Peak Flow (cumecs)	Comp.Peak Flow (cumecs)	% Error in Peak Flow	Obs.Time to Peak (h)	Comp. to Peak (h)	% Error in Time to Peak	Ave. % Absolute Error	Efficiency
1	6.0	3.81	3.21	4879.2	4394.3	9.9	13.0	17.0	30.8	155.4	91.34
	5.0	4.60	3.13		4331.7	11.2		16.0	23.1	204.0	87.80
	7.0	3.04	3.17		4550.3	6.7		16.0	23.1	134.5	94.17
	8.0	2.23	3.25		4639.1	4.9		15.0	15.4	128.2	96.36
	9.0	1.45	3.22		4944.2	1.3		15.0	15.4	132.7	97.25
	10.0	.76	3.21		5167.6	5.9		16.0	23.1	145.3	96.92
2	6.0	1.79	3.56	2092.4	2015.2	3.7	12.0	13.0	8.3	48.7	94.17
	5.0	2.44	3.50		1913.4	8.6		12.0	.0	65.3	93.33
	7.0	1.20	3.53		2058.9	1.6		13.0	8.3	41.0	92.47
3	6.0	3.59	10.74	1392.1	900.0	35.3	16.0	16.0	.0	506.7	70.31
	5.0	4.37	10.57		920.7	33.9		15.0	6.3	519.5	69.69
	7.0	2.95	10.64		875.6	37.1		17.0	6.3	494.5	69.00
4	6.0	1.63	-5.34	919.3	1105.8	20.3	5.0	8.0	60.0	34.4	34.46
	5.0	1.95	-5.34		1076.3	17.1		7.0	40.0	29.5	65.88
	4.0	2.46	-5.35		982.6	6.9		6.0	20.0	24.3	86.98
	3.0	3.13	-5.32		915.8	.4		6.0	20.0	34.1	93.81
	2.0	8.42	9.22		522.2	43.2		3.0	40.0	146.2	25.92

Table 4.5 : Observed hydrograph ordinates and computed ordinates for various trial runs for Event 1 (Time of interval = 1.0h)

Time in hours	Observed Flow (Cumecs)	Time of Concentration (tc)					
		6.0	5.0	7.0	8.0	9.0	10.0
1.0	8.3	8.3	8.3	8.3	8.3	8.3	8.3
2.0	8.2	8.2	8.2	8.2	8.2	8.2	8.2
3.0	8.0	10.3	10.3	10.3	10.3	10.3	10.3
4.0	7.9	32.5	41.7	27.0	24.3	27.3	33.1
5.0	7.8	72.2	72.9	68.3	66.7	72.2	72.9
6.0	10.3	168.0	204.2	156.8	155.3	148.5	165.3
7.0	27.5	408.6	565.7	367.2	348.7	372.1	406.5
8.0	125.6	929.6	1323.3	777.5	682.7	695.7	754.7
9.0	328.2	1748.7	2093.4	1407.2	1213.2	1143.4	1090.3
10.0	663.7	2537.8	2571.6	2275.8	1892.4	1658.0	1572.2
11.0	2814.3	2972.5	2808.7	3001.1	2901.7	2607.7	2248.0
12.0	4069.3	3219.7	3305.0	3426.8	3575.9	3671.1	3596.0
13.0	4723.8	3705.9	3543.4	3633.9	3951.5	4174.4	4390.1
14.0	4879.2	3912.9	3660.3	4134.8	4068.9	4370.7	4705.5
15.0	4800.0	4037.6	3979.7	4301.3	4573.3	4442.6	4510.9
16.0	4665.0	4303.9	4173.1	4370.5	4639.1	4944.2	4682.3
17.0	4515.0	4390.7	4331.7	4550.3	4554.0	4824.5	5167.6
18.0	4329.1	4394.3	4217.7	4483.8	4610.2	4522.9	4670.0
19.0	4263.2	4140.7	3958.6	4330.7	4354.7	4467.8	4175.8
20.0	4056.6	3812.1	3566.8	3960.2	4051.6	4019.2	4114.8
21.0	3930.7	3369.4	3036.4	3559.2	3599.5	3658.5	3486.1
22.0	3463.1	2762.5	2512.6	3038.5	3163.7	3153.4	3152.6
23.0	2875.4	2222.9	2111.4	2367.4	2571.4	2691.4	2662.7
24.0	2001.2	1816.2	1728.6	1814.9	1877.5	2035.4	2230.1
25.0	1067.1	1413.7	1433.0	1419.9	1328.7	1314.8	1544.1
26.0	709.9	1134.7	1165.0	1044.7	969.9	825.5	767.3
27.0	547.5	887.1	949.9	806.7	650.7	553.4	457.3
28.0	413.7	697.3	777.4	597.6	481.3	325.3	288.9
29.0	383.8	552.1	639.1	448.2	329.8	251.2	150.2
30.0	355.3	440.9	528.4	341.6	234.5	157.0	168.9
31.0	302.4	356.0	439.7	265.7	174.8	112.2	89.1
32.0	254.7	291.3	368.9	211.8	137.7	91.5	74.3
33.0	211.9	242.0	312.3	173.8	115.0	82.4	72.9
34.0	174.0	204.7	267.3	147.0	101.3	79.1	74.2
35.0	156.8	176.4	231.4	128.5	93.4	78.5	76.1
36.0	147.0	155.2	203.0	115.7	89.1	79.3	78.1
37.0	121.3	139.4	180.6	107.1	87.1	80.8	80.2
38.0	98.6	127.7	163.0	101.6	86.6	82.5	82.2
39.0	84.3	119.2	149.2	98.2	87.1	84.4	84.3

Table 4.6 : Observed hydrograph ordinates and computed ordinates for various trial runs for Event 2 (Time of interval = 1.0h)

Time in hours	Observed Flow (Cumecs)	Time of Concentration (t_c)		
		6.0	5.0	7.0
1.0	59.8	59.8	59.8	59.8
2.0	55.3	55.3	55.3	55.3
3.0	58.0	56.5	56.5	56.5
4.0	59.8	74.6	77.7	77.3
5.0	70.3	124.2	131.5	142.6
6.0	118.4	221.2	250.2	258.4
7.0	160.1	368.3	441.8	398.5
8.0	232.7	586.1	691.5	548.2
9.0	705.6	846.2	871.9	754.9
10.0	991.5	1045.8	1097.1	996.9
11.0	1610.7	1321.9	1316.6	1241.8
12.0	2075.6	1561.5	1648.6	1588.9
13.0	2092.4	1910.6	1913.4	1788.0
14.0	1960.5	2015.2	1874.1	2058.9
15.0	1742.4	1725.9	1662.0	1932.3
16.0	1683.1	1597.4	1679.3	1602.7
17.0	1757.5	1767.7	1742.5	1502.0
18.0	1471.7	1708.8	1303.2	1731.3
19.0	1111.9	1023.2	896.3	1487.4
20.0	939.6	645.1	655.3	723.3
21.0	623.5	425.9	471.8	395.5
22.0	80.0	275.5	339.2	210.5
23.0	75.5	185.6	246.4	129.3
24.0	80.3	142.3	225.9	102.4
25.0	80.0	164.5	176.0	89.1

Table 4.7 :
Observed hydrograph ordinates and computed ordinates for various trial runs for Event 3
(Time of interval = 1.0h)

Time in hours	Observed (Cumecs)	Time of Concentration (t_c)		
		6.0	5.0	7.0
1.0	7.6	7.6	7.6	7.6
2.0	7.8	7.8	7.8	7.8
3.0	8.1	22.7	33.9	23.1
4.0	8.1	144.3	191.6	113.8
5.0	8.5	339.0	328.3	285.7
6.0	8.9	350.4	379.4	389.4
7.0	10.3	456.7	441.8	435.5
8.0	23.8	497.1	397.5	504.3
9.0	31.0	379.6	335.4	463.1
10.0	59.8	370.5	359.7	380.8
11.0	60.7	364.6	340.2	385.3
12.0	62.5	476.9	547.6	448.3
13.0	1157.8	787.1	802.5	664.9
14.0	1169.5	832.8	817.8	855.7
15.0	1204.9	785.2	815.0	868.8
16.0	1234.9	815.4	920.7	771.3
17.0	1392.1	900.0	878.7	789.4
18.0	1253.2	848.2	789.8	875.6
19.0	1045.2	744.4	633.0	805.9
20.0	817.4	568.8	508.6	689.7
21.0	603.9	436.4	410.0	497.4
22.0	413.7	336.6	331.8	361.2
23.0	226.3	261.4	269.8	264.7
24.0	148.6	204.9	220.8	196.6
25.0	111.6	162.4	182.0	148.4
26.0	99.8	130.5	151.4	114.5
27.0	84.3	106.7	127.2	90.7
28.0	77.7	88.9	108.2	74.1
29.0	72.4	75.7	93.3	62.6
30.0	65.4	65.9	81.6	54.6
31.0	61.6	58.8	72.6	49.3
32.0	58.9	53.6	65.5	45.7
33.0	56.2	49.9	60.1	43.5
34.0	51.9	47.4	56.0	42.1
35.0	48.6	45.6	52.9	41.4
36.0	45.5	44.6	50.7	41.2
37.0	42.4	44.0	49.1	41.3
38.0	40.3	43.7	48.0	41.7

Table 4.8 : Observed hydrograph ordinates and computed ordinates for various trial runs for Event 4 (Time of interval = 1.0h)

Time in hours	Observed (Cumecs)	Time of Concentration (t_c)				
		6.0	5.0	4.0	3.0	2.0
1.0	38.0	38.0	38.0	38.0	38.0	38.0
2.0	38.0	38.0	38.0	38.0	38.0	38.0
3.0	38.0	38.0	38.0	38.0	38.0	39.6
4.0	38.2	55.0	62.8	71.1	82.4	522.2
5.0	51.0	122.4	150.1	180.1	276.0	470.0
6.0	919.3	231.3	312.9	504.3	836.3	423.7
7.0	874.7	415.7	676.9	982.6	915.8	382.8
8.0	803.4	789.2	1076.3	952.9	677.8	346.7
9.0	676.1	1105.8	940.6	648.1	505.8	314.9
10.0	410.6	887.9	577.4	447.0	381.6	286.7
11.0	315.1	496.8	363.5	314.5	292.1	262.0
12.0	226.3	289.9	237.8	227.4	227.7	240.1
13.0	183.1	180.8	164.2	170.3	181.5	220.9
14.0	140.6	123.7	121.4	133.2	148.4	204.1
15.0	122.7	94.1	96.8	109.1	125.0	189.3
16.0	107.6	79.3	82.9	93.8	108.5	176.3
17.0	90.1	72.2	75.5	84.3	97.0	165.0
18.0	82.0	69.2	71.8	78.5	89.1	155.2
19.0	75.5	68.5	70.3	75.3	83.9	146.6
20.0	67.4	68.9	70.1	73.8	80.6	139.2

Table 4.9 : Set of values of t_c and R for maximum efficiency for events 1, 2, 3 and 4 (Time interval = 1.0h)

Event No.	t_c	R	ϕ	Obs. Peak Peak Flow (cumecs)	Comp.Peak Flow (cumecs)	% Error in Peak Flow	Obs.Time to(Peak (h)	Comp. t to Peak (h)	% Error in Time to Peak	Ave. % Absolute Error	Efficiency
1	9.0	1.45	3.22	4879.2	4944.2	1.3	13.0	15.0	15.4	132.7	97.25
2	5.0	2.44	3.50	2092.4	1913.4	8.6	12.0	12.0	.0	65.3	93.33
3	6.0	3.59	10.74	1392.1	900.0	35.3	16.0	16.0	.0	506.7	70.31
4	3.0	3.13	-5.32	919.3	915.8	.4	5.0	6.0	20.0	34.1	93.81

and the average value of $[R / (t_c+R)] = 0.3132$ (12)

Substituting the value of (t_c+R) in the later we get :

$$R = 0.3132 * 8.485$$

or $$R = 2.657h \quad (13)$$

putting this value of R in eq.(11) we get :

$$t_c = 5.827 \text{ h}$$

For making t_c a whole number we assume :

$$t_c = 6.0 \text{ h}$$

re-substituting this value of t_c in eq.(11) we get :

$$R = 2.485 \text{ h}$$

Thus the calibrated values of R and tc are 2.485 h and 6.0 h respectively.

While calibrating the model the value of infiltration loss rate (ϕ-index) is found by trial and error procedure by equating the excess rainfall to the direct surface runoff volume. The same cannot be done for simulation or validation runs since the direct surface runoff is not known. Therefore, the value of ϕ-index is either chosen based upon the experience or otherwise found through some empirical approach such as SIS method for abstraction (Soil Conservation Service)

4.4 Validation of the Model Parameters

Before the calibrated model can be applied for simulating the rainfall events it must be validated so as to ensure the reliability of the simulated runoff. The validation of the model parameters is done by comparing the simulated runoff with the known runoff for the later two events i.e. events 5 and 6). Nowhere, during the simulation process, the observed discharges are made use of , except while finally comparing it with the simulated flows. However, since at present stage of model development the infiltration loss is estimated by equating the direct surface runoff volume to the excess rainfall, the observed flows are made use of while calculating the excess rainfall. This is done , only as an stop gap procedure till some technique (e.g; SCS method for abstractions)for finding out infiltration loss rate, independent of the observed flows, is included in the present model structure. Hence a uniform loss rate has been calculated for the Events 5 and 6 by equating observed direct surface flow volume to the excess rainfall volume. The values of infiltration loss rate (ϕ-index) for the Events 5 and 6 are thus computed and are equal to 9.802 mm/h and 2.911 mm/h. respectively. These values are utilized while validating the model only on the pretext that some other independent method would be employed in subsequent stages of model development to get the desired values of ϕ-index.

Since, the simulated flows are the direct surface runoff only, we have to add the base flow to this to get the computed total surface runoff. From the data of the six events it could be seen that the base flows are 0.96%, 3.45%, 1.75, 6.08, 5.58 and 4.71% of the peak flow respectively. Also, since it is only a small fraction of the total flow, it is not going to effect significantly. Hence, we take 3.75% of the peak of the simulated direct surface flow as the average to be added to the simulated direct surface runoff to get the simulate total runoff.

The values of R and t_c are taken as those obtained in the calibration process and are equal to 2.485 h and 6.0 h respectively. The summary of the results of the validation process is given in Table 4.10. The observed and the computed hydrograph ordinates for the validation process for the Events 5 and 6 are given in Tables 4.11 and 4.12 respectively. The plots between observed and simulated discharges for these events are given in Fig. 4.6, and 4.7.

Table 4.10 : Summary of the results of validation runs for Events 5 and 6. (Time interval = 1.0h)

Event No.	t_c	R	ϕ	Obs. Peak Flow (cumecs)	Comp. Peak Flow (cumecs)	% Error in Peak Flow	Obs. Time to Peak (h)	Comp. to Peak (h)	% Error in Time Peak	Ave % Absolute Error	Efficiency
5	6.0	2.48	9.80	1333.0	1766.3	32.5	14.0	17.0	21.4	111.0	81.72
6	6.0	2.48	2.91	2029.0	1819.2	10.3	13.0	16.0	23.1	48.6	72.70

Table 4.11: Observed hydrograph ordinates and computed ordinates for validation runs for Event 5 (Time of interval = 1.0h)

Time in hours	Observed Flow (cumecs)	Comp. Flow (cumecs)
1.0	42.3	63.8
2.0	41.8	63.8
3.0	41.5	63.8
4.0	41.2	82.8
5.0	40.8	182.0
6.0	40.0	312.9
7.0	40.0	344.5
8.0	40.0	431.9
9.0	97.5	487.9
10.0	210.0	460.6
11.0	340.1	605.5
12.0	715.6	727.1
13.0	869.2	871.0
14.0	1130.0	1051.5
15.0	1333.0	1046.0
16.0	1313.0	1167.0
17.0	1294.1	1691.4
18.0	1255.7	1766.3
19.0	1217.8	1305.4
20.0	1184.2	889.5

Time in hours	Observed Flow (cumecs)	Comp. Flow (cumecs)
21.0	1036.6	612.9
22.0	499.3	429.0
23.0	226.6	306.6
24.0	208.0	225.3
25.0	193.6	171.2
26.0	182.0	135.2
27.0	178.0	111.3
28.0	170.5	95.4
29.0	163.1	84.8
30.0	155.8	77.8
31.0	148.7	73.1
32.0	135.0	70.0
33.0	128.3	67.9
34.0	123.1	66.6
35.0	116.8	65.7
36.0	111.8	65.1
37.0	109.4	64.6
38.0	106.9	64.4
39.0	103.4	64.2
40.0	100.0	64.1
41.0	97.5	64.0

Table 4.12 : Observed hydrograph ordinates and computed ordinates for validation runs for Event 6 (Time of interval = 1.0h)

Time in hours	Observed Flow (cumecs)	Comp. Flow (cumecs)
1.0	60.6	65.8
2.0	60.9	65.8
3.0	61.0	65.8
4.0	61.1	92.8
5.0	160.0	155.3
6.0	300.0	171.9
7.0	440.0	227.1
8.0	580.0	329.0
9.0	740.0	549.4
10.0	900.0	1008.6
11.0	1125.5	1193.9
12.0	1663.9	1135.2
13.0	1934.5	1279.5
14.0	2029.0	1561.3
15.0	1981.5	1492.6
16.0	1663.9	1667.5
17.0	1036.7	1819.2
18.0	745.2	1555.7
19.0	361.1	1112.4
20.0	299.9	761.8

Time in hours	Observed Flow (cumecs)	Comp. Flow (cumecs)
21.0	244.1	528.6
22.0	220.0	373.6
23.0	210.0	270.4
24.0	200.0	201.9
25.0	200.0	156.3
26.0	198.0	125.9
27.0	195.0	105.8
28.0	193.6	92.4
29.0	190.0	83.5
30.0	180.0	77.5
31.0	178.0	73.6
32.0	165.0	71.0
33.0	160.0	69.2
34.0	158.0	68.1
35.0	151.5	67.3
36.0	150.1	66.8
37.0	145.9	66.4
38.0	135.0	66.2
39.0	121.9	66.1

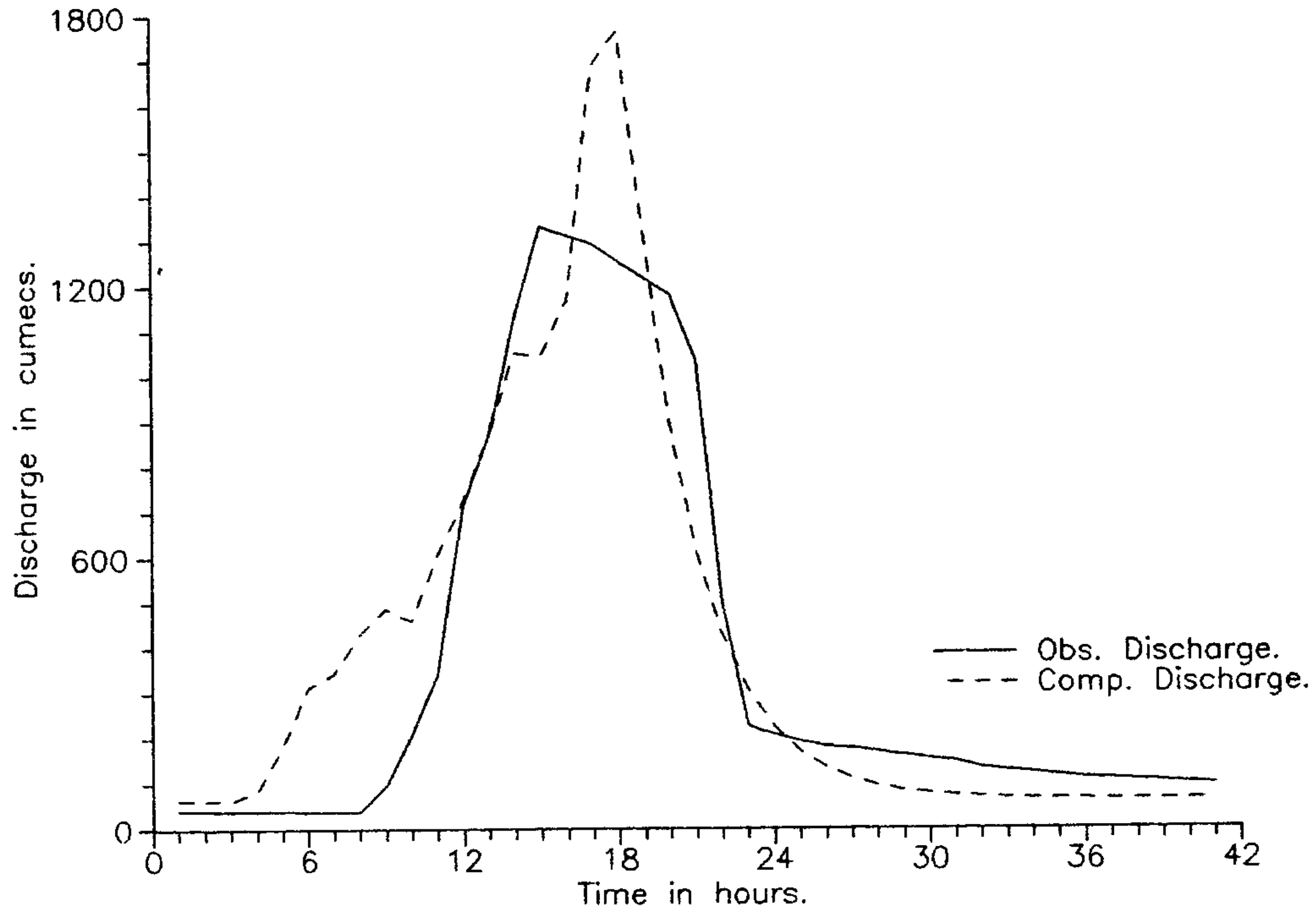

Fig. 4.6 : Validation of the model by comparying observed and computed hydrograph for the Event 5 (Time interval = 1.0h).

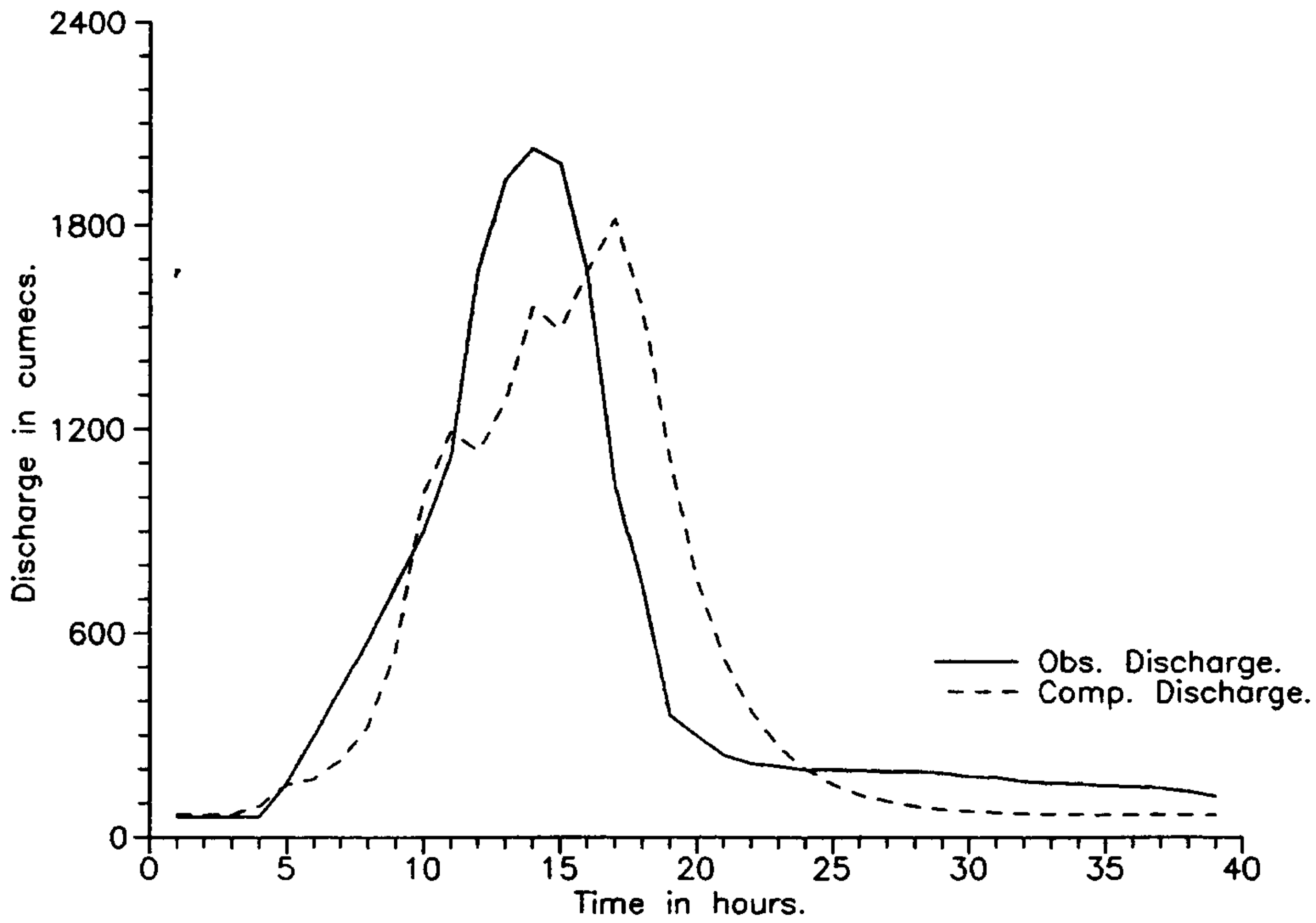

Fig. 4.7 : Validation of the model by comparying observed and computed hydrograph for the Event 6 (Time interval = 1.0h).

5.0 SENSITIVITY ANALYSIS

5.1 General

The rainfall data used for simulation of runoff for medium sized catchments are generally at an hourly interval. However, since the time area diagram has to be obtained by process of interpolation between the points at which the time of travels are calculated and thus may be available at even smaller interval of time. By intuition it may be argued that if isochronal areas are available at finer intervals of time than behavior of the movement of water through the catchment could be better depicted. Although, the information regarding the rainfall remains to be known at a finer interval of time but it is worthwhile to use atleast the isochronal areas at a finer interval of time. This would enable one to atleast include the effect of topographical feature with some higher accuracy though the climatic conditions are not available at such a fine interval of time.

5.2 Effect of number of isochrones in the initial data set

In the present case the rainfall values are available at an hourly interval. The isochronal map is made on an interval of 30 minutes. Since the model has to be run on 30 minutes time interval it is required that the values of rainfall be computed at 30 minutes interval. In the absence of data being available at a smaller interval then 1.0 h it is appropriate to consider the rainfall being falling with a constant intensity within the 1 h interval. Thus to obtain the accumulated rainfall values at 30 minutes time interval the values available at 1.0 h interval are halved and sequentially put at an interval of 30 minutes.

Also, since the comparison with the observed flows has to be made while calibrating the model and that the comparison has to be at an interval of 30 minutes it is necessary to obtain the values of observed flows at 30 minutes interval. It is assumed that since the variation between two ordinates of observed flows is almost linear, the linear interpolation between these ordinates would give the values at 30 minutes interval. Thus rainfall and observed flow values are also obtained at an interval of 30 minutes.

In this manner the rainfall and the discharge data are obtained at a finer interval of time which in this case is 30 minutes. From the isochronal map which was available at 30 minutes interval the Thiessen weights of each raingauge station is obtained for all the isochronal areas. These weights are given in Table 4.2.

The model is now run on the basis of 30 minutes time step and whole procedure of calibration and validation is repeated for obtaining the values of coefficient of linear reservoir (R) and the time of concentration (t_c). The summary of the results of the calibration process is given in Table 5.1. Also, the observed hydrograph ordinates and the computed hydrograph ordinates for various trial times of concentration for the Events 1, 2, 3 and 4 are given in Table 5.2, 5.3, 5.4, and 5.5 respectively. The plots for calibration runs for different trial times of concentration for the Events 1, 2, 3 and 4 are given in Fig. 5.1, 5.2, 5.3 and 5.4 respectively. Those values of R and t_c are chosen which give maximum efficiency or the objective function as minimum. Table 5.6 gives these values of R and t_c for Events 1, 2, 3 and 4.

The average value of (t_c+R) = 8.95 h (14)

and the average value of $[R / (t_c+R)]$ = 0.2610 (15)

Table 5.1 : Summary of the results of calibration runs for Events 1, 2, 3 and 4. (Time interval = 30 MINUTES)

Event No.	t_c	R	ϕ	Obs. Peak Flow (cumecs)	Comp. Peak Flow (cumecs)	% Error in Peak Flow	Obs. Time to Peak (h)	Comp. to Peak (h)	% Error in Time to Peak	Ave. % Absolute Error	Efficiency
1	6.0	90.26	4.06	3.21	4879.2	4366.9	10.5	13.5	16.5	22.2	189.5
	5.5	88.75	4.43	3.20		4322.4	11.4		16.5	22.2	213.8
	6.5	91.67	3.69	3.19		4430.0	9.2		16.0	18.5	170.8
	7.0	92.92	3.31	3.28		4494.5	7.9		16.5	22.2	154.7
	7.5	94.10	2.94	3.26		4545.7	6.8		16.5	22.2	147.0
	8.0	95.04	2.57	3.26		4594.1	5.8		17.0	25.9	142.3
	8.5	95.90	2.20	3.25		4665.8	4.4		15.5	14.8	140.3
	9.0	96.58	1.82	3.25		4821.2	1.2		15.5	14.8	139.3
	9.5	96.90	1.48	3.23		4926.3	1.0		16.0	18.5	141.1
	10.0	96.98	1.14	3.23		5000.8	2.5		16.0	18.5	144.8
	10.5	96.86	.82	3.22		5082.1	4.2		16.5	22.2	151.4
2	6.0	95.24	2.10	3.46	2092.4	1972.2	5.7	12.5	13.0	4.0	47.5
	5.5	94.57	2.42	3.45		1916.1	8.4		13.0	4.0	54.3
	6.5	95.39	1.77	3.45		2000.3	4.4		13.0	4.0	42.5
	7.0	95.11	1.46	3.44		2024.8	3.2		13.5	8.0	38.2
3	6.0	68.13	3.79	11.41	1392.1	869.8	37.5	16.5	16.5	.0	523.5
	5.5	68.15	4.16	11.19		866.8	37.7		16.0	3.0	532.1
	5.0	66.53	4.63	11.28		858.5	38.3		16.0	3.0	537.4
	6.5	68.45	3.37	11.34		869.8	37.5		17.0	3.0	521.3
	7.0	67.27	3.09	11.33		854.0	38.7		17.5	6.1	516.6
4	6.0	53.97	1.28	-5.27	919.3	1127.0	22.6	5.5	8.0	45.5	39.3
	5.5	68.07	1.57	-5.27		1084.2	17.9		8.0	45.5	34.8
	5.0	79.81	1.87	-5.28		1048.9	14.1		7.5	36.4	30.8
	4.5	88.97	2.17	-5.28		1005.7	9.4		7.0	27.3	27.4
	4.0	94.84	2.52	-5.28		950.9	3.4		7.0	27.3	25.5
	3.5	84.54	4.09	1.23		825.0	10.3		5.5	.0	41.9

Table 5.2 : Observed hydrograph ordinates and computed ordinates for various trial runs for Event 1
(Time of interval = 30 minutes)

Time in hours	Observed Flow (cumecs)	Time of Concentration (t_c) 6.0	5.5	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5
.5	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3
1.0	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3
1.5	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3	8.3
2.0	8.2	9.3	9.3	9.3	9.3	9.3	9.3	9.3	9.3	9.3	9.3	9.3
2.5	8.1	10.3	10.3	10.3	10.3	10.3	10.3	10.3	10.3	10.3	10.3	10.3
3.0	8.0	14.1	15.0	13.4	13.0	13.1	13.2	13.3	13.5	13.8	14.3	15.0
3.5	7.9	26.4	29.7	23.6	21.6	21.0	20.7	20.5	20.5	20.5	21.4	22.8
4.0	7.9	46.9	50.3	42.2	38.3	36.6	34.7	33.3	32.4	31.8	33.0	34.8
4.5	7.9	63.7	65.4	61.9	59.5	57.3	53.6	52.1	51.4	50.6	51.7	53.7
5.0	7.8	85.8	89.7	85.2	83.4	80.5	79.5	80.0	78.9	78.1	78.9	83.5
5.5	9.1	137.2	151.9	129.3	122.3	118.5	118.7	118.4	117.6	121.4	123.7	128.3
6.0	10.3	234.2	252.3	213.8	196.5	187.2	181.5	178.2	178.9	182.4	186.8	194.5
6.5	18.9	349.1	395.2	329.4	312.9	297.2	280.7	273.9	273.1	273.8	282.1	293.0
7.0	27.5	531.3	613.5	484.9	445.6	430.7	422.8	422.4	416.1	413.7	420.5	438.3
7.5	76.5	786.2	942.0	703.0	636.1	602.4	578.3	581.9	584.5	600.6	611.8	633.4
8.0	125.6	1144.1	1329.5	986.5	875.4	814.5	770.8	758.7	746.6	770.8	797.3	832.9
8.5	226.9	1533.3	1721.0	1363.0	1185.7	1057.8	990.4	952.2	930.0	937.8	942.9	985.1
9.0	328.2	1931.1	2064.9	1767.7	1580.3	1408.3	1264.0	1174.3	1129.5	1102.7	1088.4	1101.0
9.5	496.0	2285.7	2349.4	2170.1	1999.1	1831.6	1679.7	1503.4	1372.9	1302.0	1254.2	1223.5
10.0	663.7	2558.5	2558.7	2507.4	2417.8	2275.1	2130.6	1957.6	1774.9	1619.3	1508.9	1410.9
10.5	1739.0	2756.2	2660.5	2767.6	2749.5	2694.8	2590.9	2450.3	2298.1	2118.8	1923.8	1745.1
11.0	2814.3	2841.2	2770.5	2952.7	2997.2	3020.3	3009.2	2932.4	2836.1	2692.9	2549.5	2332.8
11.5	3441.8	2958.3	2957.1	3032.1	3152.5	3257.1	3319.8	3343.8	3331.1	3246.2	3184.0	3054.5
12.0	4069.3	3149.6	3237.0	3146.2	3221.0	3352.0	3512.8	3631.2	3715.5	3730.8	3717.0	3696.7
12.5	4396.6	3429.2	3463.2	3341.6	3332.5	3403.9	3533.1	3749.7	3935.4	4072.9	4157.9	4191.3
13.0	4723.8	3661.5	3595.7	3633.3	3531.7	3529.6	3578.7	3727.4	3958.9	4182.6	4384.0	4535.8
13.5	4801.5	3785.0	3681.2	3862.0	3835.9	3739.2	3726.3	3774.8	3907.5	4083.3	4354.2	4595.6
14.0	4879.2	3866.2	3750.6	3978.8	4058.9	4053.9	3952.3	3930.9	3967.9	4025.7	4124.4	4390.7
14.5	4839.6	3935.5	3884.6	4052.1	4161.7	4271.9	4278.5	4170.1	4127.9	4120.7	4094.6	4126.0
15.0	4800.0	4070.4	4049.3	4118.3	4218.9	4348.4	4475.2	4505.8	4383.9	4299.8	4245.4	4161.0

Table 5.2 Contd...

Time in hours	Observed Flow (cumecs)	Time of Concentration (t_c)										
		6.0	5.5	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5
15.5	4732.5	4213.7	4172.4	4238.2	4273.6	4388.4	4514.2	4664.3	4714.7	4571.2	4448.8	4355.5
16.0	4665.0	4310.2	4238.0	4358.6	4364.6	4427.0	4533.8	4665.8	4821.2	4880.5	4727.4	4584.3
16.5	4590.0	4347.6	4299.8	4430.0	4455.6	4479.4	4544.9	4651.7	4769.8	4926.3	5000.8	4849.2
17.0	4515.0	4366.9	4322.4	4426.4	4494.5	4545.7	4561.2	4610.0	4712.7	4812.9	4968.8	5082.1
17.5	4422.1	4357.3	4269.4	4413.7	4449.3	4540.2	4594.1	4590.4	4620.5	4706.2	4775.4	4941.3
18.0	4329.1	4257.3	4164.3	4358.3	4401.4	4462.1	4548.5	4578.6	4558.1	4566.4	4634.8	4662.8
18.5	4296.2	4126.2	4023.2	4225.1	4298.6	4372.1	4431.6	4503.5	4520.2	4463.0	4439.3	4467.4
19.0	4263.2	3960.0	3889.0	4064.2	4134.5	4222.7	4293.3	4342.0	4410.5	4410.5	4327.6	4234.6
19.5	4159.9	3817.0	3738.8	3881.8	3946.8	4036.0	4097.7	4161.3	4207.9	4266.4	4263.0	4142.9
20.0	4056.6	3663.1	3564.1	3730.1	3752.9	3825.4	3885.4	3940.9	3979.5	4021.7	4094.3	4083.7
20.5	3993.7	3471.6	3348.9	3562.3	3597.9	3622.2	3656.8	3709.0	3744.5	3755.8	3793.1	3893.9
21.0	3930.7	3237.0	3075.7	3353.2	3416.7	3465.3	3452.0	3471.0	3498.0	3507.9	3493.9	3536.6
21.5	3696.9	2946.2	2804.9	3100.3	3190.1	3268.0	3302.8	3268.8	3256.5	3256.2	3247.1	3235.8
22.0	3463.1	2665.4	2547.2	2793.7	2919.6	3024.0	3090.5	3112.5	3066.8	3026.6	2999.6	2995.8
22.5	3169.2	2409.0	2327.4	2504.8	2601.6	2737.3	2830.4	2888.3	2906.9	2857.0	2795.8	2767.2
23.0	2875.4	2191.8	2139.0	2247.3	2310.5	2405.2	2532.6	2615.3	2669.7	2698.5	2658.9	2593.1
23.5	2438.3	1997.3	1947.8	2028.1	2053.1	2109.7	2192.2	2305.0	2389.3	2454.2	2498.5	2471.3
24.0	2001.2	1797.4	1752.1	1829.8	1832.2	1855.4	1896.4	1957.0	2071.0	2162.4	2248.8	2310.5
24.5	1534.2	1604.6	1581.1	1629.3	1630.8	1635.1	1649.3	1665.2	1713.2	1834.2	1945.3	2044.4
25.0	1067.1	1439.0	1439.7	1439.5	1432.2	1432.6	1430.3	1423.0	1427.0	1471.7	1598.6	1718.8
25.5	888.5	1300.3	1305.4	1280.2	1248.0	1238.9	1227.8	1210.2	1191.5	1199.3	1230.3	1359.3
26.0	709.9	1169.6	1173.1	1148.1	1097.9	1062.6	1045.7	1019.3	986.9	974.9	980.0	988.4
26.5	628.7	1041.7	1055.0	1023.8	975.5	923.8	883.3	851.9	812.1	781.5	767.3	763.6
27.0	547.5	928.7	949.7	902.5	861.0	813.4	757.4	706.0	663.4	628.4	586.8	572.5
27.5	480.6	829.0	855.7	796.7	749.7	710.1	660.8	596.6	537.3	502.2	467.1	424.4
28.0	413.7	740.9	771.8	704.4	654.1	609.4	570.1	517.2	447.6	398.6	371.3	342.7
28.5	398.7	663.2	697.0	624.0	572.0	524.6	480.9	441.4	388.5	331.3	289.2	276.6
29.0	383.8	594.6	630.3	553.9	501.6	453.2	407.7	365.1	329.2	292.8	244.4	213.9
29.5	369.5	534.1	570.8	492.8	441.3	393.2	347.7	304.6	266.3	248.7	225.9	190.4
30.0	355.3	480.7	517.8	439.6	389.5	342.8	298.5	256.7	218.8	196.7	193.9	189.6

Table 5.2 Contd...

Time in hours	Observed Flow (cumecs)	Time of Concentration (t_c)										
		6.0	5.5	6.5	7.0	7.5	8.0	8.5	9.0	9.5	10.0	10.5
30.5	328.9	433.6	470.6	393.3	345.1	300.4	258.2	218.7	183.0	160.1	149.0	163.8
31.0	302.4	392.1	428.5	352.9	307.2	264.9	225.3	188.8	156.1	134.3	120.5	119.5
31.5	278.5	355.6	391.0	317.8	274.6	235.0	198.4	165.1	135.9	116.2	102.7	96.4
32.0	254.7	323.4	357.7	287.3	246.8	210.0	176.4	146.5	120.8	103.6	91.6	84.6
32.5	233.3	295.0	328.0	260.8	223.0	189.1	158.6	131.9	109.6	95.0	84.9	78.8
33.0	211.9	270.1	301.5	237.8	202.7	171.6	144.0	120.5	101.3	89.1	81.0	76.2
33.5	193.0	248.1	278.1	217.8	185.4	157.0	132.3	111.6	95.3	85.2	78.8	75.3
34.0	174.0	228.8	257.2	200.5	170.7	144.9	122.8	104.8	91.0	82.8	77.8	75.3
34.5	165.4	211.9	238.7	185.5	158.2	134.8	115.2	99.5	87.9	81.3	77.5	75.8
35.0	156.8	197.1	222.2	172.5	147.5	126.5	109.1	95.6	85.9	80.5	77.7	76.5
35.5	151.9	184.1	207.6	161.3	138.5	119.6	104.3	92.6	84.5	80.3	78.2	77.4
26.0	147.0	172.7	194.7	151.7	130.9	114.0	100.5	90.5	83.8	80.4	78.8	78.3
36.5	134.1	162.7	183.3	143.4	124.6	109.4	97.5	89.0	83.5	80.8	79.6	79.3
37.0	121.3	154.1	173.2	136.2	119.2	105.7	95.3	88.0	83.4	81.4	80.5	80.3
37.5	109.9	146.5	164.3	130.2	114.8	102.7	93.6	87.4	83.7	82.0	81.4	81.3
38.0	98.6	140.0	156.4	125.0	111.1	100.3	92.4	87.2	84.1	82.8	82.4	82.3
38.5	84.3	134.3	149.5	120.6	108.0	98.5	91.6	87.2	84.7	83.7	83.3	83.3
39.0	84.3	129.4	143.4	116.9	105.6	97.1	91.2	87.4	85.4	84.6	84.3	84.3

Table 5.3 : Observed hydrograph ordinates and computed ordinates for various trial runs for Event 2 (Time of interval = 30 minutes)

Time in hours	Observed Flow (cumecs)	Time of Concentration (t_c)			
		6.0	5.5	6.5	7.0
.5	59.8	59.8	59.8	59.8	59.8
1.0	59.8	59.8	59.8	59.8	59.8
1.5	57.5	57.5	57.5	57.5	57.5
2.0	55.3	58.1	58.1	58.1	58.1
2.5	56.6	58.6	58.6	58.6	58.6
3.0	58.0	61.8	61.9	62.0	62.3
3.5	58.8	70.1	70.3	70.5	71.1
4.0	59.8	84.1	84.5	86.1	88.3
4.5	65.1	104.0	104.9	108.8	113.1
5.0	70.3	132.1	134.5	139.8	146.5
5.5	94.4	172.2	180.9	182.7	190.7
6.0	118.4	230.1	241.3	238.4	241.9
6.5	139.3	291.3	311.7	298.6	302.3
7.0	160.1	365.5	401.0	362.3	357.6
7.5	196.4	468.2	519.6	445.6	423.0
8.0	232.7	602.0	646.2	560.7	520.3
8.5	469.1	727.7	746.1	692.5	647.6
9.0	705.6	831.5	845.5	819.2	780.2
9.5	848.6	939.0	968.6	922.1	909.0
10.0	991.5	1078.1	1106.4	1038.1	1020.6
10.5	1301.1	1227.8	1232.6	1197.4	1162.5
11.0	1610.7	1362.3	1347.4	1364.5	1341.5
11.5	1843.1	1471.7	1481.9	1494.7	1512.2
12.0	2075.6	1607.6	1644.3	1599.1	1626.1
12.5	2084.0	1780.3	1801.1	1738.2	1720.7
13.0	2092.4	1923.6	1911.1	1902.5	1860.7
13.5	2026.4	1972.2	1916.1	2000.3	1999.3
14.0	1960.5	1894.9	1818.5	1974.0	2024.8
14.5	1851.5	1767.0	1707.5	1858.0	1919.9
15.0	1742.4	1665.5	1648.8	1723.5	1772.3
15.5	1712.8	1627.9	1651.2	1631.2	1650.6
16.0	1683.1	1658.1	1698.2	1603.7	1587.7
16.5	1720.3	1739.0	1771.3	1658.9	1600.0
17.0	1757.5	1816.9	1781.8	1760.0	1699.3
17.5	1614.6	1771.5	1623.1	1803.7	1798.2
18.0	1471.7	1530.7	1359.0	1689.3	1781.4
18.5	1291.8	1236.3	1123.7	1407.7	1583.6
19.0	1111.9	998.6	937.5	1098.2	1260.1
19.5	1025.8	818.6	793.2	863.8	942.4
20.0	939.6	679.1	676.1	690.5	713.4
20.5	781.5	563.2	573.4	554.4	547.9
21.0	623.5	464.1	483.6	442.1	421.7
21.5	503.6	382.5	408.4	352.8	324.4
22.0	383.8	318.4	347.3	285.6	253.2
22.5	80.0	268.0	297.8	235.1	203.0
23.0	75.5	223.5	258.0	192.3	162.6
23.5	77.9	204.8	242.0	165.8	139.5
24.0	80.3	202.2	245.6	159.6	123.9
24.5	80.0	210.6	236.0	166.9	127.0
25.0	80.0	205.8	206.8	184.4	146.0

Table 5.4 : Observed hydrograph ordinates and computed ordinates for various trial runs for Event 3 (Time of interval = 30 minutes)

Time in hours	Observed Flow (cumecs)	Time of Concentration (tc)				
		6.0	5.5	5.0	6.5	7.0
.5	7.6	7.6	7.6	7.6	7.6	7.6
1.0	7.6	7.6	7.6	7.6	7.6	7.6
1.5	7.7	8.0	8.0	8.0	8.0	8.0
2.0	7.8	10.5	10.8	11.0	10.6	10.6
2.5	7.9	17.3	20.0	23.5	17.4	17.2
3.0	8.1	44.3	56.5	70.0	39.5	35.0
3.5	8.1	114.3	142.4	170.3	99.1	84.0
4.0	8.1	231.7	248.6	265.5	205.2	176.4
4.5	8.3	316.7	311.1	301.3	305.7	293.2
5.0	8.5	331.8	324.7	316.4	349.6	362.0
5.5	8.7	330.6	339.0	343.5	352.4	365.8
6.0	8.9	354.3	372.5	380.5	356.8	359.3
6.5	9.6	403.8	425.3	416.0	394.8	381.8
7.0	10.3	462.6	453.5	424.1	452.0	438.2
7.5	17.0	477.3	430.9	395.8	490.1	493.1
8.0	23.8	430.8	388.8	357.1	473.3	503.7
8.5	27.4	379.6	349.8	331.4	415.2	457.0
9.0	31.0	348.1	333.1	332.5	368.4	396.5
9.5	45.4	346.4	349.4	345.9	350.3	364.0
10.0	59.8	365.9	355.6	341.3	355.8	356.9
10.5	60.2	362.2	335.5	326.8	366.5	363.8
11.0	60.7	355.3	345.8	355.2	370.2	372.5
11.5	61.6	419.6	444.1	489.6	407.7	399.6
12.0	62.5	579.3	607.1	653.1	532.7	491.5
12.5	610.2	720.4	732.1	737.7	684.9	651.2
13.0	1157.8	780.0	777.1	757.3	775.3	769.7
13.5	1163.6	785.8	777.3	753.6	810.5	805.8
14.0	1169.5	761.9	762.5	736.8	801.0	809.1
14.5	1187.2	738.4	734.5	720.7	769.5	784.8
15.0	1204.9	701.0	728.0	754.7	731.9	750.1
15.5	1219.9	705.7	771.4	826.2	693.3	704.7
16.0	1234.9	757.9	840.9	854.1	704.1	672.3
16.5	1313.5	836.2	866.8	858.5	755.3	691.8
17.0	1392.1	869.8	865.2	846.9	835.3	740.3
17.5	1322.6	867.5	847.5	816.7	869.8	819.7
18.0	1253.2	848.6	811.7	771.5	866.4	854.0
18.5	1149.2	803.3	759.8	695.3	845.6	849.4
19.0	1045.2	744.1	676.6	626.9	794.4	827.3
19.5	931.3	655.3	603.0	565.5	727.2	772.4
20.0	817.4	577.6	537.7	510.5	630.4	700.4
20.5	710.7	509.6	479.9	461.2	547.1	599.6
21.0	603.9	450.0	428.7	416.9	475.3	513.9
21.5	508.8	397.9	383.3	377.3	413.5	441.1
22.0	413.7	352.2	343.2	341.7	360.4	379.2
22.5	320.0	312.3	307.7	309.8	314.6	326.8
23.0	226.3	277.4	276.2	281.3	275.2	282.2
23.5	187.5	246.8	248.4	255.7	241.4	244.4
24.0	148.6	220.1	223.7	232.7	212.2	212.3
24.5	130.1	196.8	202.0	212.2	187.2	185.1
25.0	111.6	176.3	182.7	193.8	165.7	162.1

Table 5.4 Contd..

Time in hours	Observed Flow (cumecs)	Time of Concentration (t_c)				
		6.0	5.5	5.0	6.5	7.0
25.5	105.7	158.5	165.7	177.4	147.2	142.5
26.0	99.8	143.0	150.6	162.6	131.3	126.0
26.5	92.1	129.4	137.3	149.5	117.7	112.0
27.0	84.3	117.5	125.6	137.7	106.0	100.2
27.5	81.0	107.2	115.3	127.1	96.1	90.2
28.0	77.7	98.2	106.1	117.7	87.5	81.7
28.5	75.0	90.4	98.1	109.3	80.2	74.6
29.0	72.4	83.6	91.0	101.9	74.0	68.7
29.5	68.9	77.7	84.8	95.2	68.6	63.7
30.0	65.4	72.6	79.3	89.2	64.1	59.5
30.5	63.5	68.1	74.5	83.9	60.3	56.0
31.0	61.6	64.3	70.3	79.2	57.0	53.1
31.5	60.2	61.0	66.7	75.0	54.3	50.7
32.0	58.9	58.2	63.5	71.3	52.0	48.7
32.5	57.5	55.7	60.7	68.0	50.1	47.1
33.0	56.2	53.7	58.2	65.1	48.5	45.8
33.5	54.0	51.9	56.1	62.6	47.2	44.7
34.0	51.9	50.4	54.3	60.3	46.1	43.9
34.5	50.3	49.1	52.8	58.4	45.3	43.3
35.0	48.6	48.1	51.4	56.6	44.6	42.8
35.5	47.0	47.3	50.3	55.1	44.1	42.5
36.0	45.5	46.6	49.4	53.8	43.7	42.2
36.5	44.0	46.0	48.6	52.7	43.4	42.1
37.0	42.4	45.6	47.9	51.7	43.2	42.1
37.5	40.3	45.2	47.4	50.9	43.1	42.1
38.0	40.3	45.0	47.0	50.2	43.1	42.2

Table 5.5 : Observed hydrograph ordinates and computed ordinates for various trial runs for Event 4 (Time of interval = 30 minutes)

Time in hours	Observed Flow (cumecs)	Time of Concentration (t_c)					
		6.0	5.5	5.0	4.5	4.0	3.5
.5	38.0	38.0	38.0	38.0	38.0	38.0	38.0
1.0	38.0	38.0	38.0	38.0	38.0	38.0	38.0
1.5	38.0	38.0	38.0	38.0	38.0	38.0	38.0
2.0	38.0	38.0	38.0	38.0	38.0	38.0	38.0
2.5	38.0	38.0	38.0	38.0	38.0	38.0	43.6
3.0	38.0	42.2	42.2	42.3	42.5	42.7	48.2
3.5	38.1	52.5	52.9	53.6	54.7	55.9	48.0
4.0	38.2	72.7	76.0	79.8	84.3	94.0	57.2
4.5	44.6	113.6	123.0	133.3	139.7	147.6	102.9
5.0	51.0	181.6	184.0	189.5	207.9	236.5	398.5
5.5	485.2	236.8	250.0	271.1	315.3	402.6	822.5
6.0	919.3	310.7	345.3	403.8	497.8	631.4	825.0
6.5	897.0	417.5	486.4	588.4	723.6	849.9	778.2
7.0	874.7	563.4	681.1	810.4	928.6	946.4	708.8
7.5	839.1	774.2	894.9	999.5	1005.7	950.9	645.0
8.0	803.4	976.3	1065.9	1048.9	977.8	868.9	576.8
8.5	739.8	1127.0	1084.2	987.0	867.4	737.6	516.7
9.0	676.1	1107.9	982.8	848.1	716.5	613.8	463.5
9.5	543.4	959.8	812.1	678.4	579.5	512.5	416.6
10.0	410.6	754.3	622.8	530.8	471.0	429.7	375.1
10.5	362.9	546.2	466.1	418.2	385.0	362.0	338.6
11.0	315.1	385.1	352.8	332.4	317.0	306.7	306.3
11.5	270.7	276.9	270.9	267.1	263.2	261.5	277.9
12.0	226.3	204.5	211.8	217.4	220.6	224.7	252.8
12.5	204.7	156.1	169.2	179.6	187.1	194.6	230.8
13.0	183.1	123.7	138.5	151.0	160.6	170.2	211.3
13.5	161.9	102.3	116.5	129.3	139.8	150.3	194.2
14.0	140.6	88.1	100.8	112.9	123.4	134.2	179.2
14.5	131.7	78.9	89.7	100.6	110.6	121.1	166.0
15.0	122.7	72.9	81.8	91.4	100.6	110.6	154.4
15.5	115.1	69.2	76.4	84.6	92.8	102.1	144.2
16.0	107.6	67.0	72.7	79.6	86.9	95.3	135.3
16.5	98.8	65.7	70.2	76.0	82.3	89.9	127.6
17.0	90.1	65.2	68.6	73.4	78.8	85.6	120.8
17.5	86.1	65.1	67.7	71.6	76.3	82.2	114.9
18.0	82.0	65.3	67.3	70.5	74.4	79.6	109.7
18.5	78.8	65.7	67.2	69.8	73.1	77.6	105.3
19.0	75.5	66.3	67.4	69.4	72.2	76.1	101.4
19.5	67.4	66.9	67.8	69.4	71.7	75.1	98.1
20.0	67.4	67.6	68.3	69.6	71.5	74.4	95.3

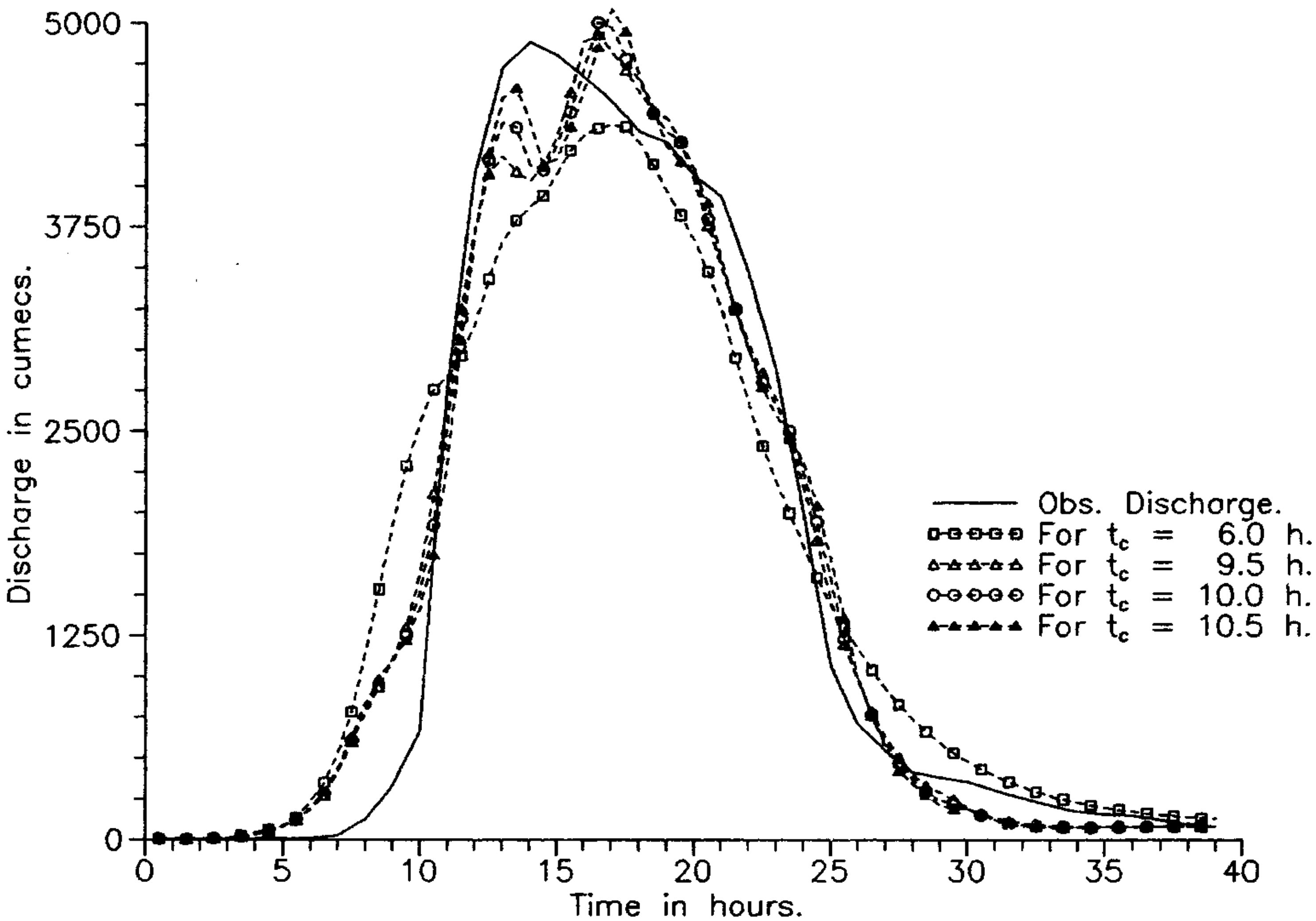

Fig. 5.1 : Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 1 (Time interval = 0.5h).

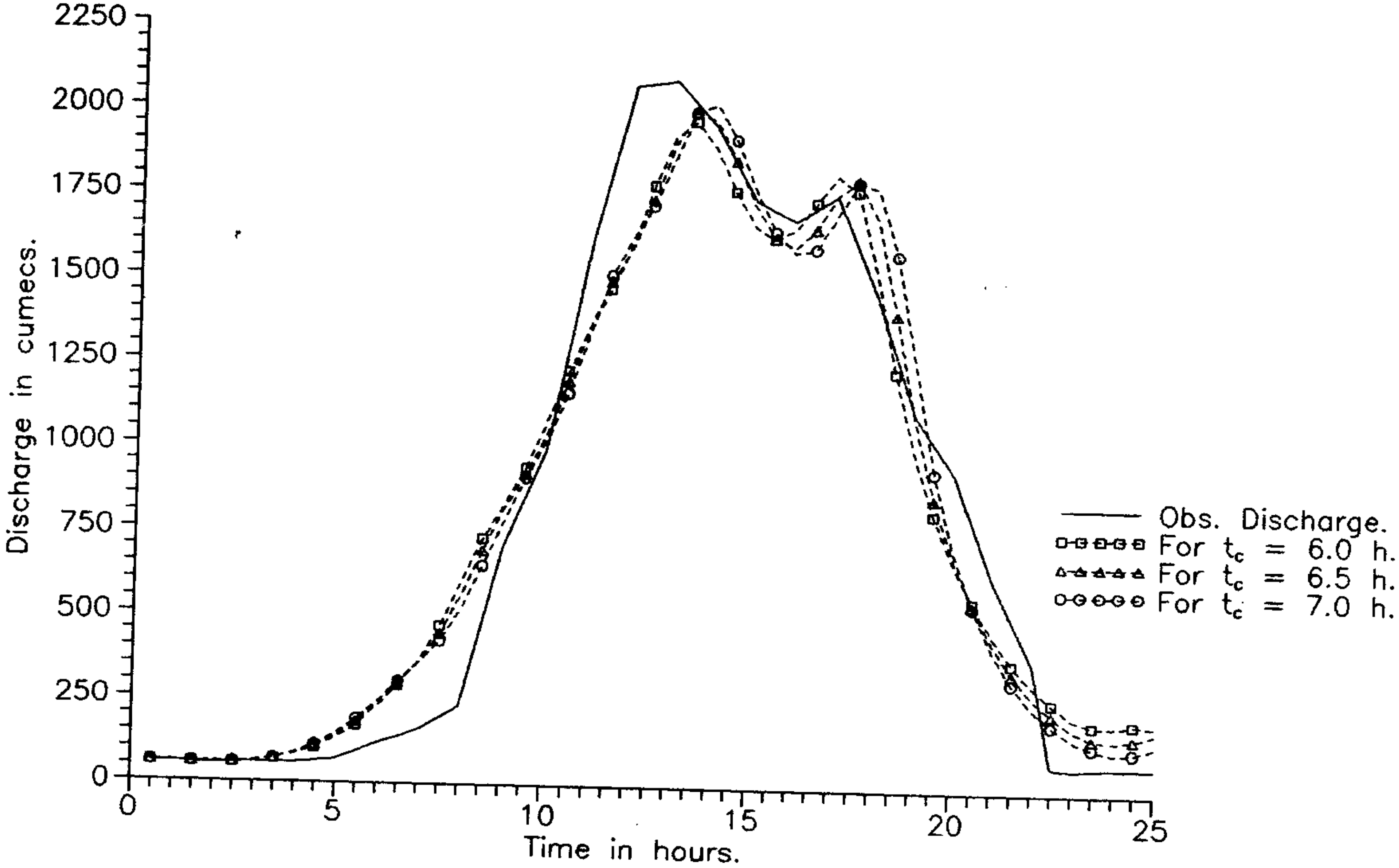

Fig. 5.2 : Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 2 (Time interval = 0.5h).

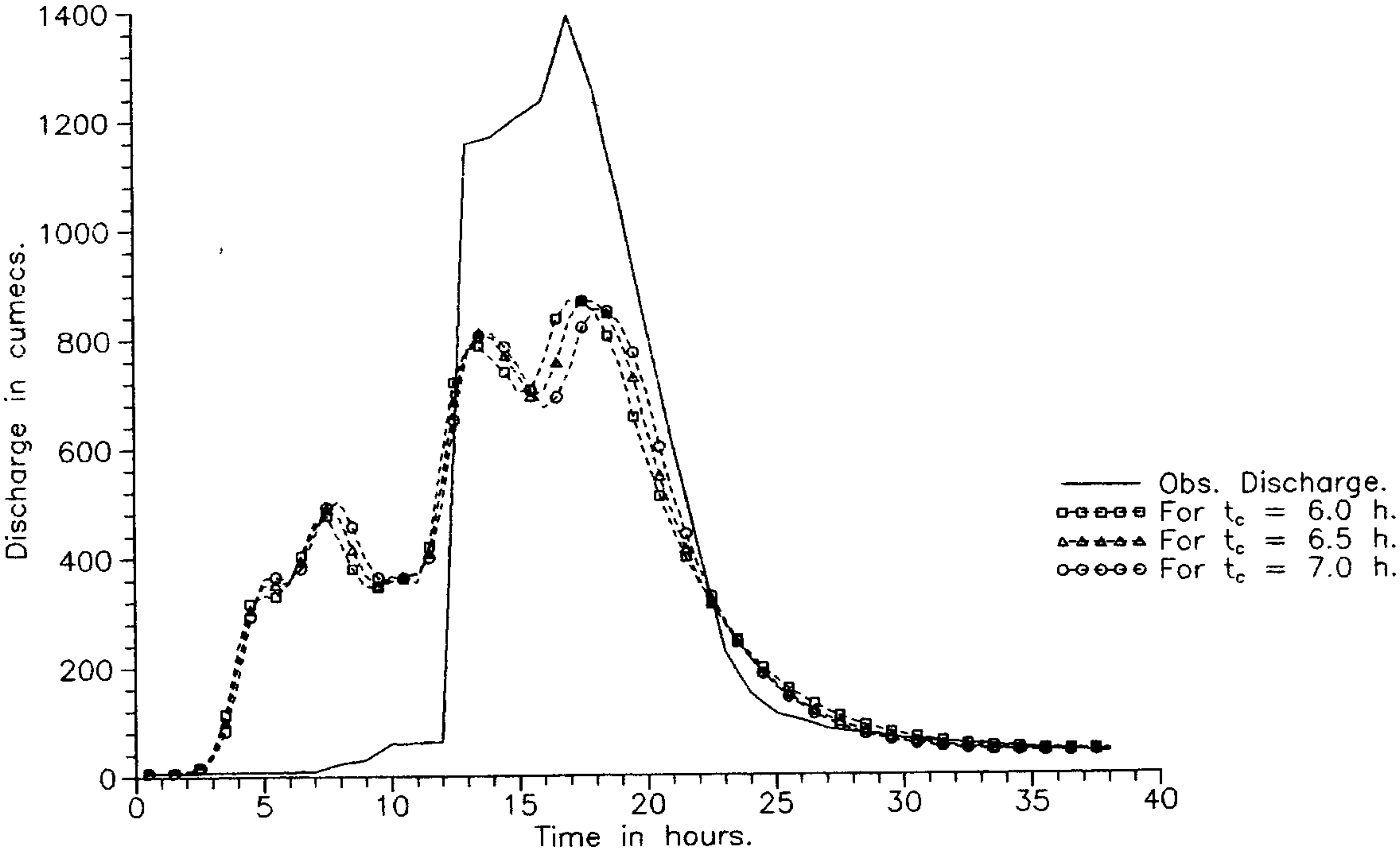

Fig. 5.3 : Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 3 (Time interval = 0.5h).

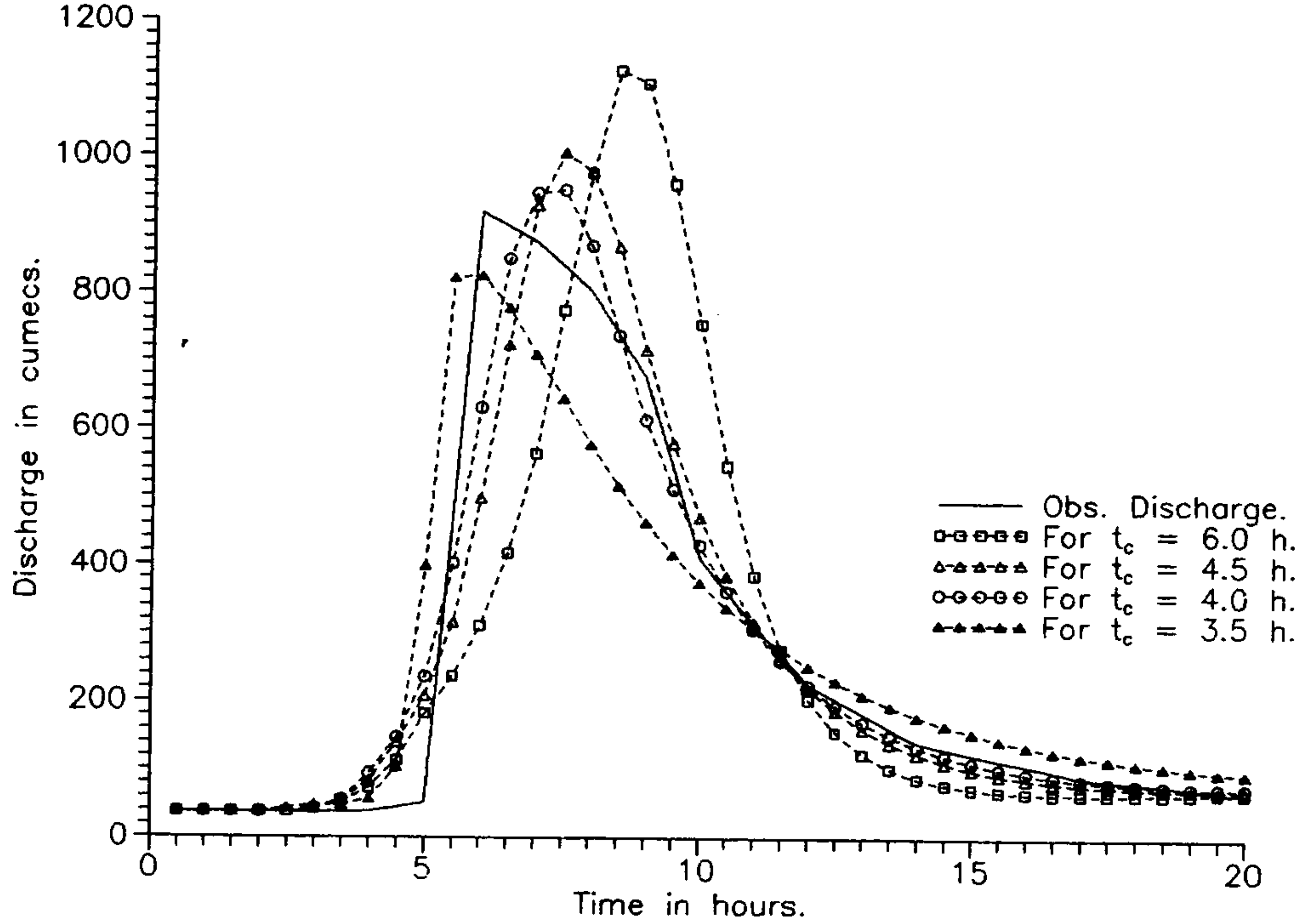

Fig. 5.4 : Comparison of calibrated results for different values of time of concentrations (t_c) with the observed data for Event 4 (Time interval = 0.5h).

Table 5.6 : Set of values of t_c and R for maximum efficiency for events 1, 2, 3 and 4

(Time interval = 30 minutes)

Event No.	t_c	R	ϕ	Obs. Peak Flow (cumecs)	Comp. Peak Flow (cumecs)	% Error in Peak Flow	Obs. Time to Peak(h)	Comp. to Peak (h)	% Error in Time to Peak	Ave. % Absolute Error	Efficie-ncy
1	10.0	1.14	3.23	4879.2	5000.8	2.5	13.5	16.0	18.5	144.8	96.98
2	6.5	1.77	3.45	2092.4	2000.3	4.4	12.5	13.0	4.0	42.5	95.39
3	6.5	3.37	11.34	1392.1	869.8	37.5	16.5	17.0	3.0	521.3	68.45
4	4.0	2.52	-5.28	919.3	950.9	3.4	5.5	7.0	27.3	25.5	94.84

Substituting the value of (t_c+R) in the later we get :

$$R = 0.2610 * 8.95$$

or $$R = 2.3365h \qquad (16)$$

putting this value of R in eq.(11) we get :

$$t_c = 6.61 \text{ h}$$

For making t_c a whole number we assume :

$$t_c = 6.5 \text{ h}$$

re-substituting this value of t_c in eq.(11) we get :

$$R = 2.45 \text{ h}$$

Thus the calibrated values of R and tc are 2.45 h and 6.5 h respectively.

On the basis of these values of R and t_c and the rate of infiltration loss (ϕ-index) as given in the previous chapter the model is run for the validation of the Events 5 and 6. The summary of the results is given in Table 5.7. The observed and computed hydrographs for these Events are given in Table 5.8 and 5.9. The plots showing the comparison of the simulated flows computed on the basis of 30 minutes as time interval with the observed flow for the Events 5 and 6 are given in Fig. 5.5 and 5.6.

The sensitivity of the process of taking finer time interval is shown in Fig. 5.7 and 5.8 in which the observed flows for the Events 5 and 6 are compared with the simulated flows computed on

Table 5.7 : Summary of the results of validation runs for Events 5 and 6.
(Time interval = 30 minutes)

Event No.	t_c	R	ϕ	Obs. Peak Flow (cumecs)	Comp. Peak Flow (cumecs)	% Error in Peak Flow	Obs. Time to Peak (h)	Comp. to Peak (h)	% Error in Time to Peak	Ave.% Absolute Error	Efficiency
5	6.5	2.45	9.80	1333.0	1797.8	14.5	17.5	34.9	20.7	109.2	82.36
6	6.5	2.45	2.91	2029.0	1792.0	13.5	16.5	11.7	22.2	47.7	73.56

Table 5.8 : Observed hydrograph ordinates and computed ordinates for validation runs for Event 5 (Time of interval = 30 minutes)

Time in hours	Observed Flow (cumecs)	Computed Flow (cumecs)
.5	42.3	65.0
1.0	42.3	65.0
1.5	42.1	65.0
2.0	41.8	65.0
2.5	41.7	65.0
3.0	41.5	68.1
3.5	41.4	76.9
4.0	41.2	97.4
4.5	41.0	140.7
5.0	40.8	208.6
5.5	40.4	271.9
6.0	40.0	301.7
6.5	40.0	311.6
7.0	40.0	334.4
7.5	40.0	386.8
8.0	40.0	423.4
8.5	68.8	443.3
9.0	97.5	464.4
9.5	153.8	465.4
10.0	210.0	485.7
10.5	275.0	551.0
11.0	340.1	674.4
11.5	527.9	772.7
12.0	715.6	802.1
12.5	792.4	837.1
13.0	869.2	899.2
13.5	999.6	974.9
14.0	1130.0	1021.0
14.5	1231.5	1023.1
15.0	1333.0	999.8
15.5	1323.0	982.1
16.0	1313.0	1072.0
16.5	1303.5	1264.8
17.0	1294.1	1549.0
17.5	1274.8	1747.1
18.0	1255.7	1797.8
18.5	1236.7	1700.5
19.0	1217.8	1465.7
19.5	1201.0	1244.4
20.0	1184.2	1026.0
20.5	1110.4	848.0
21.0	1036.6	703.0
21.5	768.0	584.9
22.0	499.3	488.6
22.5	363.0	410.1
23.0	226.6	346.2
23.5	217.3	294.1
24.0	208.0	251.7
24.5	200.8	217.1
25.0	193.6	188.9
25.5	187.8	166.0
26.0	182.0	147.3
26.5	180.0	132.0
27.0	178.0	119.6
27.5	174.3	109.5
28.0	170.5	101.3
28.5	166.8	94.5
29.0	163.1	89.1
29.5	159.4	84.6
30.0	155.8	81.0
30.5	152.3	78.0
31.0	148.7	75.6
31.5	141.8	73.6
32.0	135.0	72.0
32.5	131.6	70.7
33.0	128.3	69.7
33.5	125.7	68.8
34.0	123.1	68.1
34.5	120.0	67.5
35.0	116.8	67.0
35.5	114.3	66.7
36.0	111.8	66.4
36.5	110.6	66.1
37.0	109.4	65.9
37.5	108.2	65.7
38.0	106.9	65.6
38.5	105.2	65.5
39.0	103.4	65.4
39.5	101.7	65.3
40.0	100.0	65.2
40.5	97.5	65.2
41.0	97.5	65.2

Table 5.9 : Observed hydrograph ordinates and computed ordinates for validation runs for Event 6 (Time of interval = 30 minutes)

Time in hours	Observed Flow (cumecs)	Computed Flow (cumecs)
.5	60.6	64.8
1.0	60.6	64.8
1.5	60.8	64.8
2.0	60.9	64.8
2.5	61.0	64.8
3.0	61.0	66.7
3.5	61.1	78.5
4.0	61.1	106.4
4.5	110.6	138.6
5.0	160.0	157.9
5.5	230.0	168.9
6.0	300.0	184.7
6.5	370.0	209.3
7.0	440.0	245.8
7.5	510.0	293.8
8.0	580.0	369.8
8.5	660.0	474.0
9.0	740.0	643.7
9.5	820.0	868.1
10.0	900.0	1080.4
10.5	1012.8	1211.7
11.0	1125.5	1232.2
11.5	1394.7	1194.8
12.0	1663.9	1145.5
12.5	1799.2	1128.5
13.0	1934.5	1208.3
13.5	1981.7	1354.9
14.0	2029.0	1510.1
14.5	2005.2	1570.3
15.0	1981.5	1544.9
15.5	1822.7	1593.1
16.0	1663.9	1685.2
16.5	1350.3	1781.6
17.0	1036.7	1792.0
17.5	891.0	1696.8
18.0	745.2	1531.4
18.5	553.2	1317.8
19.0	361.1	1109.3
19.5	330.5	915.9
20.0	299.9	758.3

Time in hours	Observed Flow (cumecs)	Computed Flow (cumecs)
20.5	272.0	629.8
21.0	244.1	525.2
21.5	232.0	439.9
22.0	220.0	370.5
22.5	215.0	313.9
23.0	210.0	267.7
23.5	205.0	230.1
24.0	200.0	199.5
24.5	200.0	174.6
25.0	200.0	154.2
25.5	199.0	137.7
26.0	198.0	124.2
26.5	196.5	113.2
27.0	195.0	104.2
27.5	194.3	96.9
28.0	193.6	91.0
28.5	191.8	86.1
29.0	190.0	82.2
29.5	185.0	78.9
30.0	180.0	76.3
30.5	179.0	74.2
31.0	178.0	72.4
31.5	171.5	71.0
32.0	165.0	69.9
32.5	162.5	68.9
33.0	160.0	68.1
33.5	159.0	67.5
34.0	158.0	67.0
34.5	154.8	66.6
35.0	151.5	66.3
35.5	150.8	66.0
36.0	150.1	65.8
36.5	148.0	65.6
37.0	145.9	65.4
37.5	140.4	65.3
38.0	135.0	65.2
38.5	121.9	65.1
39.0	121.9	65.1

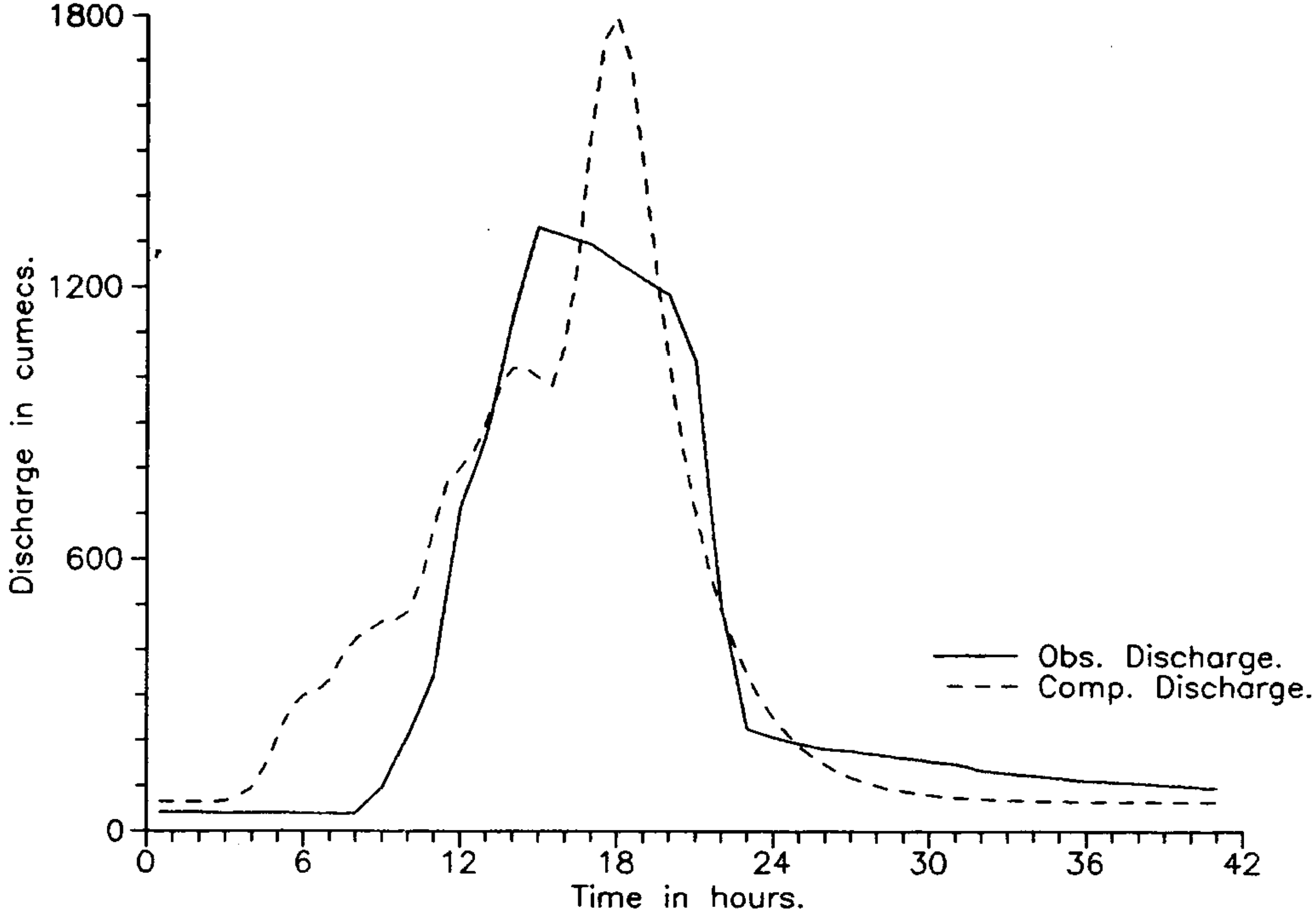

Fig. 5.5 : Validation of the model by comparing observed and computed hydrograph for the Event 5 (Time interval = 0.5h).

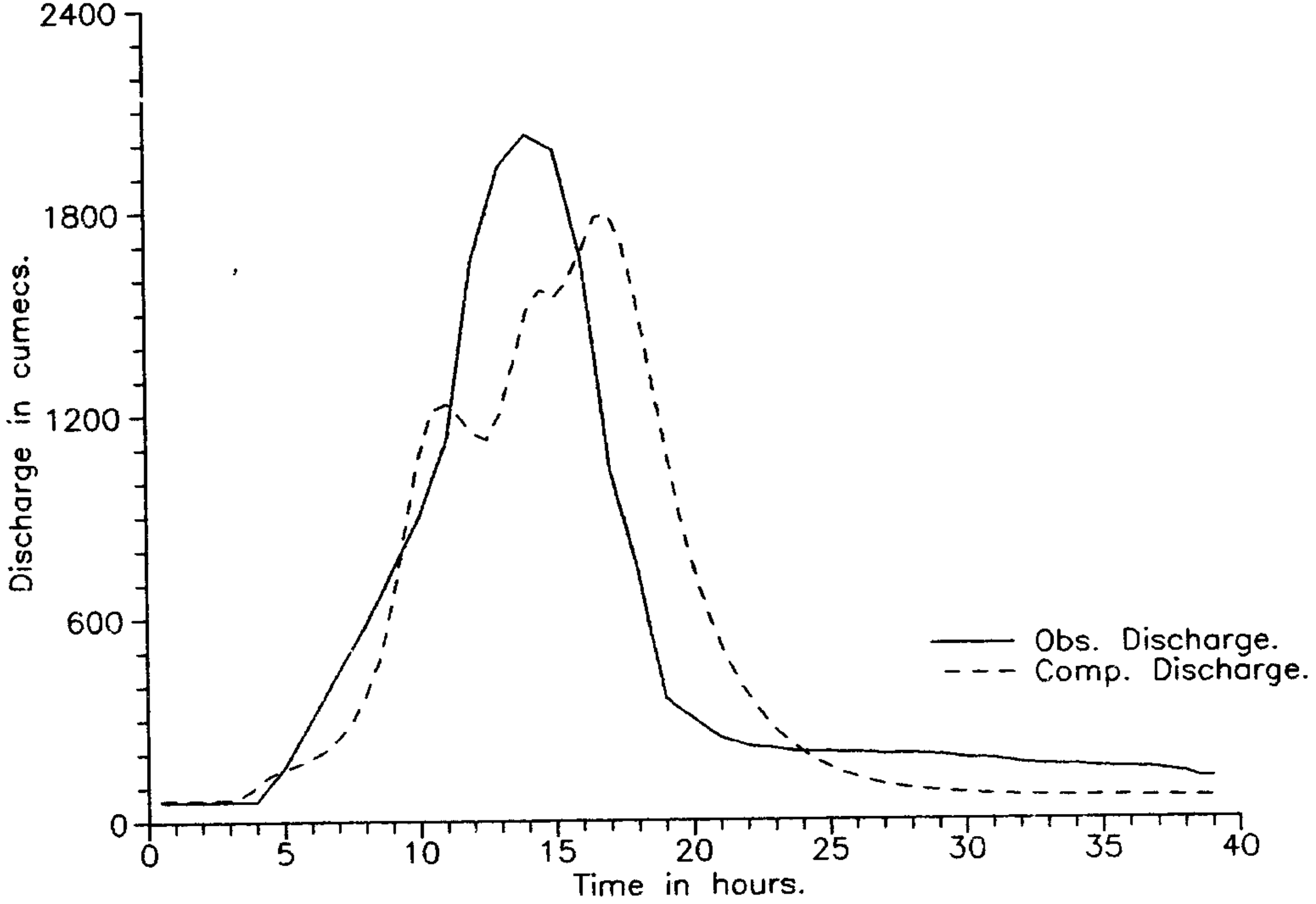

Fig. 5.6 : Validation of the model by comparing observed and computed hydrograph for the Event 6 (Time interval = 0.5h).

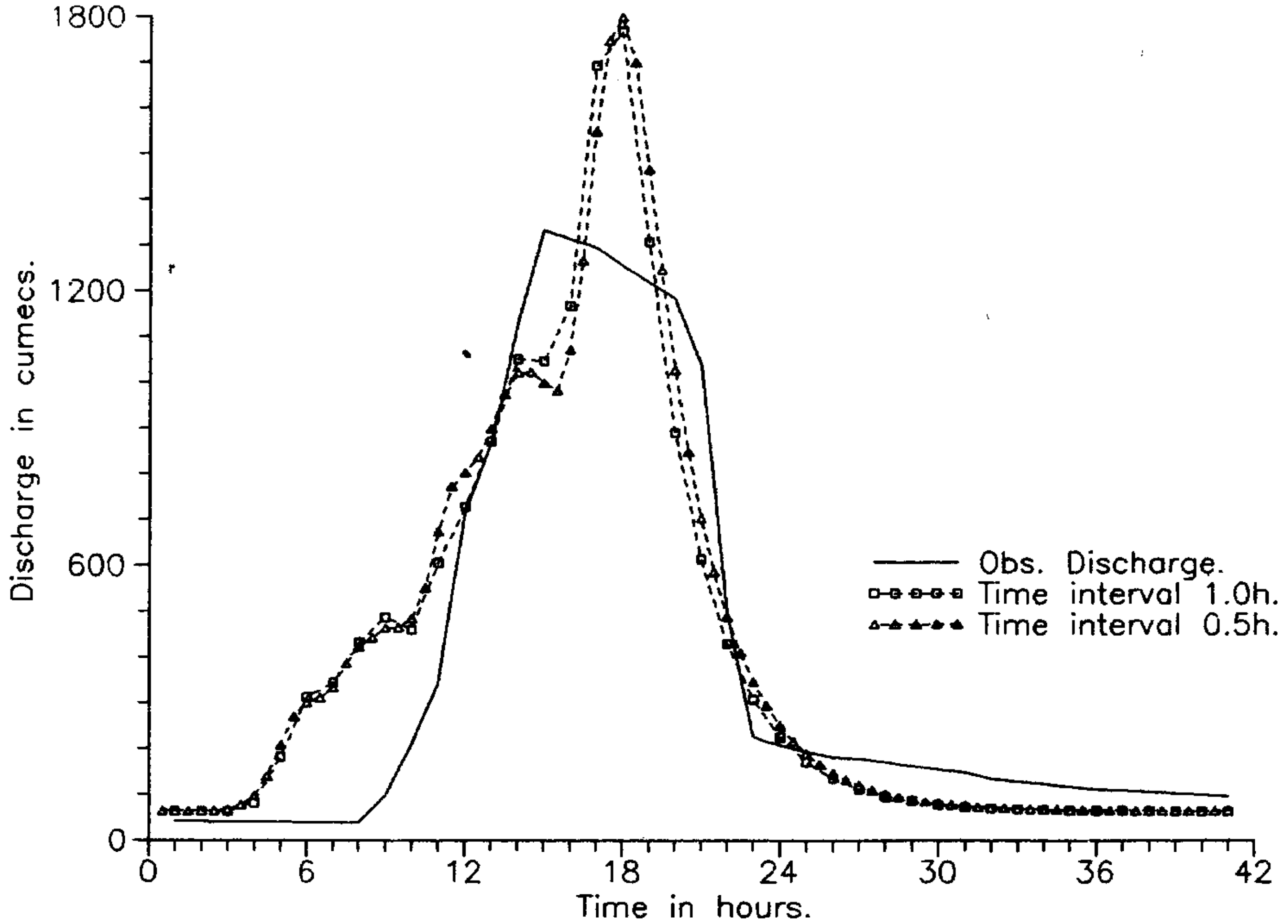

Fig. 5.7 : Sensitivity analysis for Time intervals equal to 0.5 hrs. and 1.0 hrs.

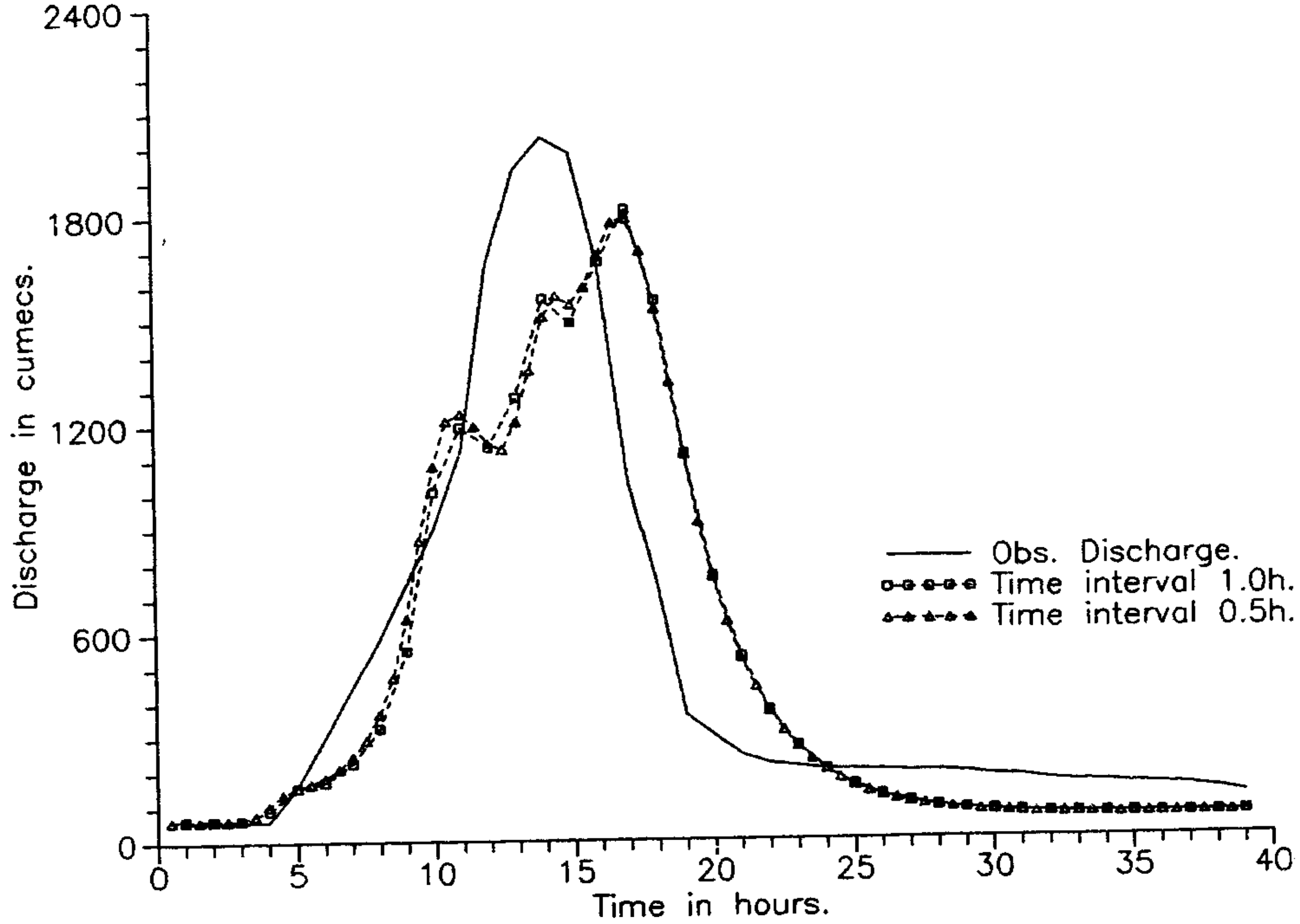

Fig. 5.8 : Sensitivity analysis for Time intervals equal to 0.5 hrs. and 1.0 hrs (Event 6).

the basis of 1.0 h and 30 minutes time interval. It is observed that though qualitatively there is no appreciable difference between the two cases, there seems to be some finer difference. This may be so because of the fact that 1.0 h is already such a small interval of time that further reduction is not showing any appreciable refinement in the computed hydrograph. It may also be seen from the summary of the results that efficiency increases in two cases i.e; Events 2 and 4 while it reduces in the other two.

It is inferred that since 1.0 h interval is sufficiently small for achieving a fairly satisfactory degree of efficiency and further reduction in the time interval has not resulted in a commensurate increase in the efficiency. The reduction in the efficiency in the two cases may be attributed to the numerical purbertation and not as that due to the lowering of time interval. Also, since we are generating the observed data for the smaller interval of time and are not really using the observed data for the smaller time interval, it is not expected to give very accurate results.

6.0 CONCLUSIONS

Application of this Event Based Distributed Rainfall-Runoff Model comes as very handy tool for the transformation of rainfall to runoff for discrete events. The data required for this model is not very difficult to prepare. Much emphasis is put on the preparation of time-area diagram called the Isochronal map in which contours of equal time of travel are plotted over the catchment area. Though an emperical approach is used for calculation of time of travel, it is only made use of for getting a preliminary idea about the scale of the time of travel. An optimisation technique is later used to further refine it to achieve better efficiency of the model. However, the importance of using the emperical formulae is in having a distribution of the catchment area into isochronal areas of varying magnitudes depending on the topography of the region. This is the most important aspect of this model that it takes the topography of the catchment into consideration while simulating the rainfall-runoff process.

The structure of the model is based on the simple concept of linear channels draining simultaneously from different isochronal areas and lagging differently according to the placement of these areas. This is again routed through a linear reservoir to get the final response from the catchment. Thus the model is extremely simple to understand and operate. The computer software of the mathematical model is made in ForTran language for use on personal computers.

The simulation runs by the model shows a fairly satisfactory degree of acceptance. In the present stage of the model development the infiltration loss rate is computed using uniform loss rate (ϕ-index) procedure. It is envisaged to compute the excess rainfall with the help of SCS method of abstraction. A module to calculate the loss rate by SCS method of abstraction may suitably be incorporated in the model in the future stages of model development.

Sensitivity analysis,in this case, did not show any marked improvement in the efficiency of the model by reducing the time interval of the analysis. This may be attributed to three things : (i) to the difference between the two time intervals taken i.e; 1.0 h and 30 minutes is so small that the numerical purbertation overwhelms the change in efficiency and (ii) to the fact that 1.0 h itself is such a small interval that further reduction does not bring appreciable improvement in the efficiency of the model and (iii) to the fact that synthetic procedure was applied for obtaining the observed rainfall and runoff data which may not represent the real values. However, this aspect may be further investigated to ascertain the sensitivity of the model to the lowering of the time interval.

Performance of this model for the present case study is found to be good. It is proposed that more such applications for catchments of different hydro-meteorological regions would give a complete picture of the applicability of the model under such conditions.However, it may be said that since there was no bias in the selection of the catchment and during subsequent application of the model, the model should be able to be applicable in similar type of situations faced in rainfall-runoff transformation process.

This model may very conveniently be put to use while carrying out : (i) design flood studies, (ii) filling of short term runoff records, (iii) reservoir inflow studies, (iv) extension of runoff records and (v) real time flow forecasting etc..

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