DEVELOPMENT OF PHYSICALLY BASED FLOOD FREQUENCY MODELS

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By

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Gratis

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled, **DEVELOPMENT OF PHYSICALLY BASED FLOOD FREQUENCY MODELS** in fulfilment of the requirement for the award of the Degree of **Doctor of Philosophy** submitted in the **Department of Hydrology, University of Roorkee,** is an authentic record of my own work carried out during a period from January, 1993 to December, 1995, under the supervision of **Dr. B.S. Mathur** and **Dr. N.K. Goel**.

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

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ABSTRACT

Flood frequency analysis is one of the most active areas of hydrological research. In the past, efforts have been mainly concentrated on the statistical analysis of available flood data. Statistical flood frequency methods require long term homogeneous series of flood characteristics such as peak discharge, volume, duration etc. Collection of flood data for a long period is tedious and expensive. Keeping this in view, attempts have been made in the past to develop physically based flood frequency models. These models use readily available rainfall data and catchment characteristics.

The physically based flood frequency models or derived flood frequency distributions (DFFD) were first introduced by Eagleson (1972). The DFFD models have three components viz. (i) stochastic rainfall model, (ii) infiltration model and (iii) effective rainfall-runoff model. The stochastic rainfall model used by most of the researchers assumes bivariate exponential distribution of rainfall intensity and duration and these variables are considered to be independent of each other. The ϕ-index, Philip's infiltration equation and SCS curve number method have been tried as infiltration models. Kinematic wave (KW), geomorphologic instantaneous unit hydrograph (GIUH) and geomorphoclimatic instantaneous unit hydrograph (GcIUH) have been used as effective rainfall-runoff models.

The physically based flood frequency models provide a potentially attractive and alternative solution to ungauged watersheds. The impact of watershed changes on flood magnitudes and frequencies can be studied through DFFD models.

In the present study, existing DFFD models have been applied to five watersheds of Sub zone - 3C (India) and their performance evaluated. New DFFD models using bivariate exponential distribution for correlated and independent rainfall intensity and duration have also been developed. The new DFFD models developed in the study were also applied to three watersheds of U.S.A.

Detailed at site/regional and regional flood frequency analysis for Sub zone - 3c has been carried out for comparing the performance of various DFFD models.

It has been found in the present study that the parameters of stochastic rainfall model are most sensitive input to the DFFD models and therefore, should be estimated carefully. Out of the three infiltration models used, the parameter of SCS curve number model can be estimated quite easily with reasonable accuracy. The DFFD models based on this infiltration model perform better than the other models. GcIUH and KW theory based effective rainfall-runoff models perform equally well in DFFD models.

The quantiles estimated by the DFFD models which consider rainfall intensities to be independent of their durations are higher than the flood quantiles estimated by the proposed model which accounts for the negative correlation between these variables. DFFD models for positively correlated case still need to be developed.

Physically based flood frequency models are relatively new in the field of hydrology, and are under development stage. There is a need for application of these models to more watersheds having long term reliable rainfall and runoff data before recommending them for field use.

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CHAPTER 1

INTRODUCTION

1.1 FLOODS AND THEIR IMPORTANCE

Since time immemorial floods have been causing untold misery throughout the world and India is no exception to this. Due to ever increasing population and economic reasons, pressure on flood plains has built up unabated which is causing progressive increase in flood damages. In India, according to Rashtriya Barh Ayog (National Flood Commission) 40 mha of land is prone to floods and annual flood damages are of the order of 350 crores of rupees (around 100 million US dollars). Developed countries like USA and Japan also incur several million dollars as average annual losses due to floods and droughts. Structural and non-structural measures are taken up to control and mitigate floods. For both these measures the estimate of design flood is required.

1.2 CURRENT METHODS OF FLOOD ESTIMATION AND THEIR LIMITATIONS

For design flood estimation a number of methods such as empirical formulae, enveloping curves, rational method, deterministic rainfall-runoff models and flood frequency analysis are in vogue. However, these methods have their own limitations.

Unit hydrograph or other rainfall-runoff models in combination with standard project storm are used for estimation of peak flows. These single event models may give better results only after proper calibration. However, the major assumption of above methods is that the frequency of the flood event will be same as that of rainfall event. Similar assumption is made in rational method. In nature, this may not be true as different combinations of rainfall intensities and loss rates may produce same peak discharge.

Continuous simulation models can generate runoff data provided long term hourly precipitation data are available. However, these models also need calibration. They are costly to run, require more computer time and trained personnel.

Statistical flood frequency methods require long term homogeneous series of flood characteristics such as peak discharge, volume, duration etc. Collection of flood data for a long period is tedious and expensive, as a result available flood series remain short in most cases. Development activities in the watershed do change the watershed response. As a result the flood series at a site does not remain homogeneous. This makes traditional flood frequency methods inapplicable in most cases. Keeping these limitations in view, attempts have been made in the past to develop physically based flood frequency models. These models use readily available rainfall data and catchment characteristics. However, they are under development stage and need refinement before they could be recommended for field applications. The present work is in this direction.

1.3 PHYSICALLY BASED FLOOD FREQUENCY MODELS

The physically based flood frequency model is a derived flood frequency distribution (DFFD) model first introduced by Eagleson (1972). The DFFD model consists of the following three major components:

1. Stochastic rainfall model
2. Infiltration model and
3. Effective rainfall-runoff model.

The stochastic rainfall model used by most of the researchers assumes the bivariate exponential distribution of rainfall intensity and duration and these variables have been taken to be independent of each other.

The ϕ-index, Philip infiltration equation and SCS curve number method have been tried as infiltration models. The probability density function (PDF) of effective rainfall intensity and duration is derived using stochastic rainfall model and infiltration model. Derived distribution technique (Benjamin and Cornell, 1970) is used for this purpose.

Kinematic wave (KW), geomorphologic instantaneous unit hydrograph (GIUH) and geomorphoclimatic instantaneous unit hydrograph (GcIUH) have been used as effective rainfall-runoff models.

The flood frequency model is derived using PDF of effective rainfall intensity and duration and one of the effective rainfall-runoff models.

1.4 OBJECTIVES

The DFFD models are relatively new in the field of hydrology and are still under development stage. In the present study, available DFFD models have been applied to five Indian watersheds and their performance evaluated.

Attempts have been made to develop new DFFD model using SCS method for excess rainfall computation and KW as effective rainfall-runoff model. New DFFD models have also been developed for watersheds where rainfall intensity and rainfall duration tend to be correlated.

The objectives of the present work may thus be summarized as below:

i) to apply and evaluate the performance of existing DFFD models using data of Indian watersheds,

ii) to develop a new DFFD model using bivariate exponential rainfall model of intensity and duration, SCS curve number model of infiltration and KW as effective rainfall-runoff model and

iii) to develop DFFD models using bivariate exponential model of negatively correlated intensity and duration of rainfall, ϕ-index as infiltration model and GcIUH and KW as effective rainfall-runoff models.

1.5 CHAPTERIZATION

The subject matter of this thesis has been arranged in the following chapters:

The current chapter, named **"Introduction"** gives the overall view of physically based flood frequency models and objectives of the present study.

The second chapter which describes the earlier works on the topic of DFFD is entitled **"Review of Literature"**. In this chapter, components of DFFD models and previous works conducted by various researchers have been presented.

The Chapter 3 is named as **"Model Development - Independent Rainfall Intensity and Duration"**. In this chapter, derivation of previous DFFD models have been described in first part. The second part is devoted to development of a new DFFD model. The derivations of PDF of effective rainfall intensity and duration are explained. Transformation of this PDF into CDF of peak discharge has been given using derived distribution technique.

In Chapter 4 derivations of DFFD models for correlated intensity and duration have been presented. This Chapter has been entitled as **"Model Development - Correlated Rainfall Intensity and Duration"**.

The Chapter 5 entitled **"Description of Study Area and Data Availability"** gives details of watersheds selected for the present study and availability of rainfall and runoff data.

The sixth chapter is named as **"Estimation of Parameters for Component Models"**. Estimation of parameters of various component models of DFFD has been described in this chapter.

The Chapter 7 entitled **"Results and Discussion"** presents the results and the analysis of various models.

The eighth chapter is named as **"Conclusions"**. General observations and specific conclusions drawn from the present study are given in this chapter. At the end of this chapter, some areas of future research work on DFFD have been suggested.

CHAPTER 2

REVIEW OF LITERATURE

2.1 INTRODUCTION

Flood frequency analysis has been one of the most active areas of hydrological research for the last forty or more years. Almost every issue of a journal related to Water Resources Research contains papers on this topic. Cunnane (1987) gives a phasewise review of statistical models for flood frequency estimation. This review clearly indicates that in the past, efforts have been mainly concentrated on the statistical analysis of available flood data of the site/region under consideration without taking into account the dynamics of the catchment.

Klemes (1993) gives practical limitations of this standard approach and states "If more light is to be shed on the probabilities of hydrological extremes, then it will have to come from more information on the physics of the phenomena involved, not from more mathematics".

The traditional methods of flood frequency analysis need refinement as extrapolation of small sample to draw remote distribution tails must be supplemented by physically based components. The floods are caused by an unusual combination of hydrometerological factors and a possible range of variation in runoff factors can not be represented by a small sample of flood series alone.

At most of the sites, where quantile estimates are needed, no streamflow data are available. Two techniques are in common use in such situations: predictions from catchment characteristics using linear regression i.e. regional analysis and rainfall-runoff modelling. During the past few years there have been several advancements in these methods (Potter, 1987). However, for regional analysis sound criteria of homogeneity are not available. Keeping this in view Cunnane (1987) concludes that "The largest single obstacle to progress lies in the ungauged catchment case where the relation between mean annual (Index) flood and catchment characteristics remains stubbornly imprecise. It is hoped that physically based flood frequency models may be of some assistance in this in future".

Due to men's influence and water resource development activities, the flood series available at most of the sites are not virgin and homogeneous. In such situations, strictly speaking, the traditional flood frequency methods may not be applicable. This calls for the development of detailed physically based models.

The attempts made for developing physically based flood frequency model are briefly described in the following sections.

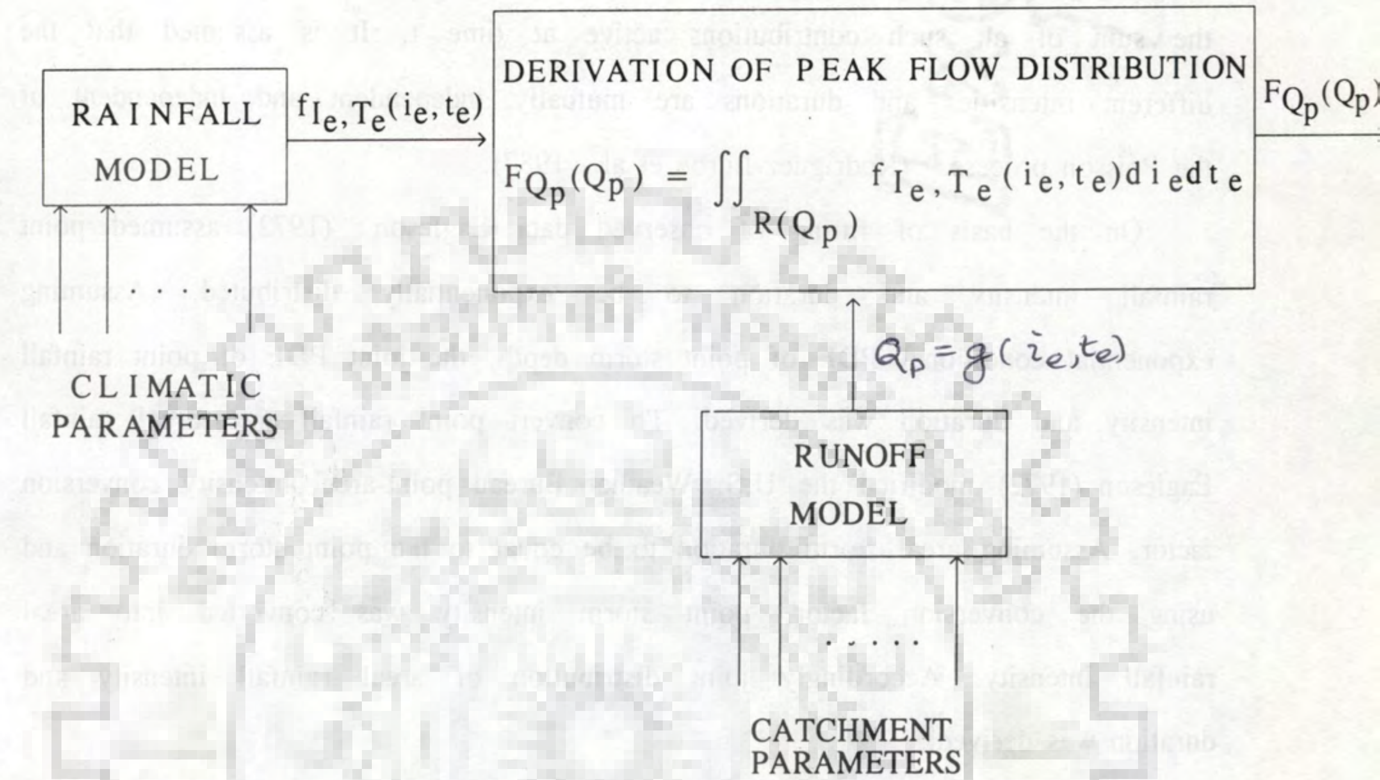
2.2 COMPONENTS OF PHYSICALLY BASED FLOOD FREQUENCY MODELS

The developments in the area of physically based flood frequency model started with the pioneering work of Eagleson (1972). The framework for derived flood frequency distribution (DFFD) model is shown in (Fig.2.1). As shown in this figure, for physically based flood frequency models, now onwards will be called DFFD, climatic parameters are used to make rainfall model and catchment parameters are used to make runoff model. These two models are then linked to transform effective rainfall distribution into peak flow distribution using derived distribution technique (Benjamin and Cornell, 1970).

In the following sections details of various components of DFFD models are discussed.

2.2.1 Stochastic Rainfall Models

Stochastic rainfall model is one of the major components of DFFD. In all the DFFD models developed so for Rectangular Pulses Poisson Model (RPPM) has been used as stochastic rainfall model. "The model is built from rectangular pulses



NOTATIONS

i_e = effective rainfall intensity
t_e = effective rainfall duration
Q_p = peak discharge
$f_{I_e,T_e}(i_e,t_e)$ = PDF of i_e and t_e
$F_{Q_p}(Q_p)$ = CDF of Q_p
$R(Q_p)$ = region of integration where $Q \leq Q_p$

FIG.2.1 - METHOD USED IN DERIVING FLOOD FREQUENCY RELATION. (AFTER EAGLESON, 1972)

associated with a Poisson process. That is, each point of a Poisson process of rate λ per unit time is associated with a rectangular pulse of random duration L and random height X, representing rainfall intensity. The total intensity Y(t) is the sum of all such contributions active at time t. It is assumed that the different intensities and durations are mutually independent and independent of the Poisson process." (Rodriguez-Iturbe et al., 1987).

On the basis of fitting of observed data Eagleson (1972) assumed point rainfall intensity and duration to be exponentially distributed. Assuming exponential conditional PDF of point storm depth, the joint PDF of point rainfall intensity and duration was derived. To convert point rainfall into areal rainfall Eagleson (1972) modified the U.S. Weather Bureau point-areal intensity conversion factor. Assuming areal storm duration to be equal to the point storm duration and using the conversion factor, point storm intensity was converted into areal rainfall intensity. Accordingly joint distribution of areal rainfall intensity and duration was derived.

Hebson and Wood (1982), Diaz-Granados et al. (1983, 1984), Cadavid et al. (1991) and Raines and Valdes (1993) used the stochastic rainfall model proposed by Eagleson (1972).

Rodriguez-Iturbe et al. (1984) derived the second-order properties of the aggregated process for the case of exponential PDF of both rainfall intensity and duration. Using these derivations, model parameters, arrival rate (number of events in a fixed period), inverse of mean intensity and inverse of mean duration were estimated for a particular level of aggregation.

Identification of Independent Storm and Parameter Estimation

The recorded precipitation data has to be separated into statistically independent events to utilize the stochastic rainfall model. Restrepo-Posada and Eagleson (1982) proposed an easily applied approximate criteria for the separation of point precipitation records into statistically independent storms. They used exponentiality of the time between storms t_b as a sufficient condition for the statistical independence of storm arrivals within the restriction that overlapping is negligible. A sample coefficient of variation of unity was accepted as sufficiently unambiguous criteria for independence of arrivals on the basis of observations of exponential like distributions of t_b for numerous series of raw storms. Diaz-Granados et al. (1983, 1984) and Moughamian et al. (1987) used this criteria for identification of independent storms.

Raines and Valdes (1993) adopted the parameter estimation procedure proposed by Rodriguez-Iturbe et al. (1984). This procedure fits mean, variance and lag-one autocorrelation of recorded rainfall depths at a fixed level of aggregation. They estimated parameters using one hour as level of aggregation and used the average annual statistics.

PDF for Correlated Intensity and Duration

The stochastic rainfall models described in previous section assume that the random variables, rainfall intensity and duration are independent of each other. In reality, the independence may not hold and there may be a negative or positive correlation between these two random variables. Cordova and Rodriguez-Iturbe (1985) studied the effect of positive correlation between intensity and rainfall on storm surface runoff and concluded that the correlation has an important impact on the probabilistic structure of storm surface runoff.

Singh and Singh (1991) derived several bivariate PDFs with exponential marginals. They applied this to model the positively correlated random variables i.e. rainfall intensity and depth.

Bacchi et al. (1994) used the joint PDF of intensity and duration described by Gumbel (1960). This PDF considers negative correlation between the random variables having exponential marginals. They applied this distribution to describe the extreme rainfall and suggested a numerical procedure for parameter estimation.

2.2.2 Infiltration Models

Different infiltration models have been used to derive the joint PDF of effective rainfall intensity and duration. Eagleson (1972) and Hebson and Wood (1982) applied a temporally and spatially averaged potential loss rate ϕ to get rainfall excess intensity from total rainfall intensity. They considered runoff producing events in their analyses whereas, Diaz-Granados et al. (1983, 1984) included average annual number of independent rainfall events while deriving joint PDF of effective rainfall intensity and duration using conceptual model (ϕ-index) of infiltration.

Eagleson (1978a) gave expressions for infiltration sorptivity S_i and gravitational infiltration rate A_o of Philip's (1969) infiltration model. This physically based infiltration model was used by Diaz-Granados et al. (1983, 1984) and Shen et al. (1990). Cadavid et al. (1991) also used this model but they replaced A_o by the hydraulic conductivity K_s and S_i was computed using a simple expression given by Koch (1985).

Raines and Valdes (1993) substituted the Philip's equation by SCS curve number model. This model requires only one parameter, curve number CN to be estimated for which the data is more readily available.

Parameter Estimation

Eagleson (1972) used average annual runoff as a fraction of average annual precipitation ϕ_1 and average annual direct runoff as a fraction of average annual runoff ϕ_2 to compute average annual number of rainfall excess events. These parameters could be estimated from the observations taken in nearby or hydrologically similar watersheds (Moughamian et al. 1987).

Parameters of Philip's equation were estimated by Diaz-Granados et al. (1983, 1984) and Raines and Valdes (1993) using ecological climatic climax model given by Eagleson (1982) and Eagleson and Tellers (1982). Cadavid et al. (1991) used simple expression for S_i (Koch, 1985). They replaced A_o by K_s. They found that, the infiltration parameters for sandy loam and loam soils, for Santa Anita Creek and Ralston Creek were close to the values reported by Rawls et al. (1983). McCuen (1989) gives detailed tables of curve numbers for different land use, soil groups and hydrologic conditions of watershed. Raines and Valdes (1993) estimated curve number for their test watershed using procedure described by McCuen (1989).

2.2.3 Effective Rainfall-runoff Models

Simple effective rainfall-runoff models have been used for DFFD models. These models are based on Kinematic Wave theory (KW) or Geomorphologic/Geomorphoclimatic Instantaneous Unit Hydrograph.

Kinematic Wave

Kinematic wave (KW) runoff model was first used by Eagleson (1972) to derive flood frequency distribution of peak discharge. He considered the open book type catchment-stream geometry. Analytical solutions developed by Eagleson (1970) for overland and stream flow routing were used to derive expressions for peak discharges for different forms of hydrograph. Only three important flow regimes were considered after rejecting least probable alternatives.

Shen et al. (1990) used the concept of average overland plane (AOP) for overland flow routing. They developed different flow regimes and their expressions for peak discharge. These expressions were tested using WATRUN a numerical model of KW. They applied their analysis on four contrived basins to get the derived flood frequency distributions under different soil and initial moisture conditions.

Cadavid et al. (1991) used the open book type catchment-stream geometry for runoff routing model. They included the fourth flow regime in their analysis which was omitted by Eagleson (1972). Expressions were derived using method of characteristics for different flow regimes. Out of the four flow regimes it is not possible to develop solutions for two flow regimes, therefore, approximate solutions developed for the two flow regimes were tested using WATRUN. Data generated for these two flow regimes were used for regression analysis. Regression equations to compute peak discharge were given for these cases. The resulting equations were used to derive flood frequency distribution of two real catchments.

Geomorphologic Instantaneous Unit Hydrograph

The geomorphologic instantaneous unit hydrograph (GIUH) [Rodriguez-Iturbe and Valdes, 1979] which incorporates the effects of catchment scale and shape into runoff dynamics was used by Hebson and Wood (1982) as effective rainfall runoff model for deriving flood frequency distribution. The GIUH is parameterized by average first order stream lengths, bifurcation, area and length ratio which are physically meaningful and can be determined easily. Peak velocity, which is an important factor of GIUH is assumed constant through out the catchment.

Geomorphoclimatic Instantaneous Unit Hydrograph

Rodriguez-Iturbe et al. (1982) introduced geomorphoclimatic instantaneous unit hydrograph (GcIUH), which is a stochastic reinterpretation of GIUH. Using Eagleson's (1970) analytical solutions of KW equations and derived distribution technique, peak velocity in GIUH was replaced by effective rainfall intensity to develop the GcIUH. Expressions for IUH peak and time to peak were derived as functions of catchment parameters and effective rainfall intensity. Diaz-Granados et al. (1983, 1984) used GcIUH as effective rainfall-runoff model in their study. Raines and Valdes (1993) also used GcIUH for the study of flood frequency distribution of four real catchments.

Parameters required for KW, GIUH and GcIUH models can be estimated from topographic maps of the catchments. Manning's coefficients for overland and channel roughness are difficult to estimate. They are found by fitting the KW model to one or more sets of observed rainfall-runoff data. Peak velocity of GIUH is also difficult to estimate however, it can be obtained by field measurements.

The following parameters of KW model can be estimated from topographic maps of the catchment. Average overland plane, and channel slopes, catchment area, plane width, channel length. Channel cross-sections can be measured in the field and the coefficient and exponent of hydraulic radius and cross-sectional flow area relationship are determined. If field observations are not feasible, empirical methods suggested by Henderson (1966) as reported by Shen et al. (1990) may be used to estimate these coefficients. Manning's roughness coefficients for plane and channel are difficult to estimate. However, these coefficients may be found by fitting the KW model to one or more sets of observed rainfall runoff data (Eagleson, 1972).

Most of the parameters required for GIUH and GcIUH can be estimated from topographic map of the catchment. If the field measurements are not possible, estimation of peak flow velocity is difficult. Moughamian et al. (1987) used both judgemental and least square technique to estimate peak flow velocity and KW parameter α_Ω using available rainfall-runoff data of few events. Raines and Valdes (1993) used channel characteristics estimated from topographic maps to compute these parameters.

2.3 AVAILABLE DFFD MODELS

All the DFFD models developed so for by researchers assume bivariate exponential distribution for rainfall intensity and duration. The available models may be categorised as

i) Kinematic wave theory based models

ii) GIUH and GcIUH based models

2.3.1 KW Theory Based Models

Eagleson (1972) used bivariate exponential model of rainfall intensity and duration in his analysis. Both these parameters were assumed independent of each other. He used a conceptual (ϕ-index) model of infiltration. KW runoff model was used for transformation of effective rainfall distribution into distribution of peak discharge.

Shen et al. (1990) also used the same stochastic rainfall model. For derivation of PDF of effective rainfall and duration they used Philip's model of infiltration as proposed by Diaz-Granados et al. (1983, 1984). Effective rainfall-runoff model used by Eagleson (1972) was modified to include five different regimes of flow.

Cadavid et al. (1991) applied stochastic rainfall, and infiltration models used by Shen et al. (1990). They considered one of the omitted regimes by Eagleson (1972) in their effective rainfall-runoff model.

2.3.2 GIUH and GcIUH Based Models

Geomorphological instantaneous unit hydrograph developed by Rodriguez-Iturbe and Valdes (1979) was first used by Hebson and Wood (1982) as effective rainfall-runoff model in their derivation of CDF of peak discharge. They applied constant loss rate (ϕ-index) infiltration model to derive PDF of effective rainfall intensity and duration. Stochastic rainfall model used was similar to Eagleson (1972).

Diaz-Granados et al. (1983, 1984) used GcIUH as effective rainfall runoff model. They were first to introduce Philip's infiltration model to derive PDF of effective rainfall intensity and duration using the stochastic rainfall model of Eagleson (1972). They also included the concept of probability of null runoff by using all the independent rainfall events.

Moughamian et al. (1987) compared the GIUH (Hebson and Wood, 1982) and GcIUH (Diaz-Granados et al. (1984)) based models. They concluded that improvements are needed in stochastic rainfall and watershed response models.

Raines and Valdes (1993) compared the models of Hebson and Wood (1982), and Diaz-Granados et al. (1984) with their model which introduced SCS-CN model of infiltration. This model was proposed to avoid uncertainty in estimation of infiltration parameters of other models. However, no method could produce better results for the four catchments tested by them. They suggested a need for improved methods of parameter estimation.

Wood and Hebson (1986) and Sivapalan et al. (1990) have also developed dimensionless flood frequency distributions using GIUH.

2.4 SUMMARY

As reported by several researchers parameter estimation has been the most difficult task in using DFFD models. Therefore, an attempt has been made in the present study to improve the methods of parameter estimation for stochastic rainfall model.

One of the major difficulties in DFFD model is estimation of parameters of infiltration model. In SCS curve number model, infiltration is computed using CN for the catchment, which can be easily estimated from soil and land use data. An attempt has been made to develop a new DFFD model which uses SCS model as infiltration model and KW as effective rainfall-runoff model.

In the past the researchers have used bivariate exponential stochastic rainfall model in which intensity and duration are assumed to be independent of each other. In reality these two random variables may be correlated. This correlation may have significant effect on the flood quantiles. In the present study the methodology for negatively correlated intensity and duration has been developed.

CHAPTER 3

MODEL DEVELOPMENT
INDEPENDENT RAINFALL INTENSITY AND DURATION

3.1 INTRODUCTION

As discussed in Chapter 2 and shown in Fig. 2.1, in DFFD models, climatic parameters are used to develop rainfall model whereas catchment parameters are put to use for developing a suitable runoff model. These two models are then linked to transform effective rainfall distribution into peak flow distribution. Most of the DFFD models assume rainfall intensity and rainfall duration to be independent of each other. But these variables can be correlated as well. The mathematical derivations for various derived flood frequency distributions which assume rainfall intensity and duration as independent are presented in this chapter. All these models assume bivariate exponential distribution for rainfall intensity and duration.

In the present study, a new DFFD model using bivariate exponential distribution for stochastic rainfall, SCS as infiltration model and kinematic wave as effective rainfall-runoff model has been developed. Detailed derivations of this DFFD model are also described in this chapter (section 3.4.5).

The development of the theory for DFFD models which account for correlation between rainfall intensity and duration will be presented in Chapter 4.

3.2 STOCHASTIC RAINFALL MODEL

In this section, stochastic rainfall model used earlier (Eagleson, 1972; Hebson and Wood, 1982; Diaz-granados et al., 1984; Cadavid et al., 1991 and Raines and Valdes, 1993) has been discussed.

The point storm duration t_r and average point storm intensity i are assumed to be exponentially distributed. Therefore, their PDFs can be expressed as:

$$f_I(i) = \beta^* \exp(-\beta^* i) \qquad i \geq 0 \tag{3.1}$$

$$f_{T_r}(t_r) = \delta \exp(-\delta t_r) \qquad t_r \geq 0 \tag{3.2}$$

where β^* and δ are inverses of mean storm intensity m_i and mean storm duration m_{t_r} respectively.

Assuming that the areal storm duration is equal to the point storm duration and using the area reduction factor K, the areal rainfall intensity i_r is given by

$$i_r = Ki \tag{3.3}$$

and its PDF is defined as

$$f_{I_r}(i_r) = \frac{\beta^*}{K} \exp\left(-\frac{\beta^*}{K} i_r\right) \qquad i_r \geq 0$$

defining $\beta = \beta^*/K$

$$f_{I_r}(i_r) = \beta \exp(-\beta i_r) \qquad i_r \geq 0 \tag{3.4}$$

Assuming the areal storm intensity and duration to be independent of each other, the joint PDF will be

$$f_{I_r,T_r}(i_r,t_r) = \beta\delta \exp(-\beta i_r - \delta t_r) \qquad i_r,\ t_r \geq 0 \tag{3.5}$$

The joint PDF of rainfall intensity and duration as expressed by (3.5) will be used to derive the probability of null runoff, PDF of effective rainfall duration and PDF of effective rainfall intensity i_e and duration t_e.

3.2.1 Derivation of $f_{I_e,T_e}(i_e,t_e)$ with ϕ - index as Infiltration Model

PDF of effective rainfall intensity and duration can be derived using ϕ - index as infiltration model as follows.

Effective rainfall intensity and duration for a spatially averaged potential loss rate ϕ are given by

$$i_e = i_r - \phi \qquad \text{and} \qquad t_e = t_r \qquad \text{if } i_r > \phi \tag{3.6a}$$

$$i_e = 0 \qquad \text{and} \qquad t_e = 0 \qquad \text{if } i_r \leq \phi \tag{3.6b}$$

Probability of null runoff (P_{NR})

When i_r is less than or equal to ϕ, no runoff is generated. In terms of distribution of i_e and t_e, this situation is represented by discrete probability at i_e=0 and t_e=0. The probability of null runoff P_{NR} can be obtained by integrating PDF of i_r and t_r under the region (Fig. 3.1) where no runoff is generated. The P_{NR} is given by

$$P(i_e=t_e=0) = \int_0^{\infty}\left[\int_0^{\phi} \beta\delta\exp(-\beta i_r - \delta t_r)di_r\right]dt_r$$

$$= \int_0^{\infty} \beta\delta\exp(-\delta t_r)\left[\frac{\exp(-\beta\phi) - 1}{-\beta}\right]dt_r$$

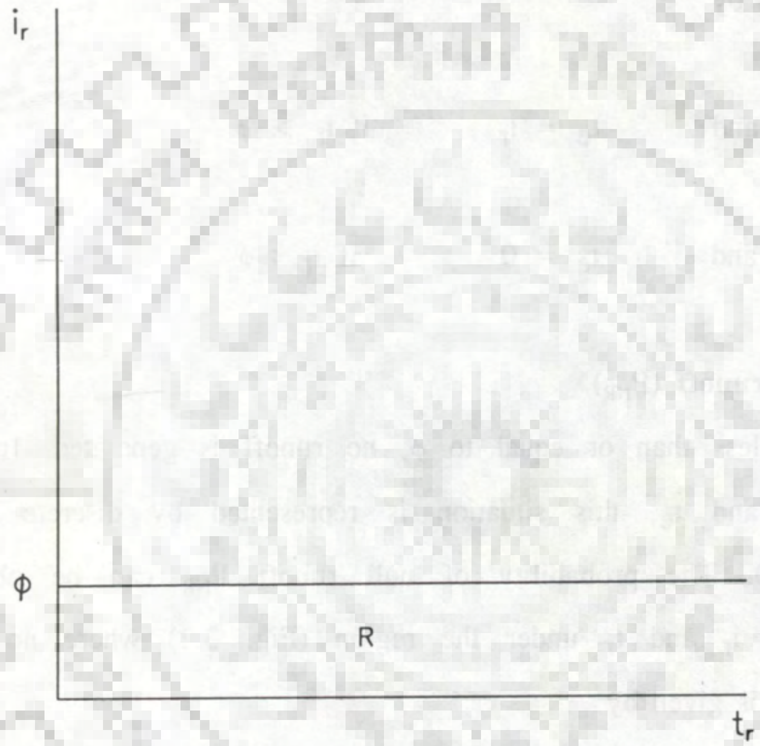


FIG. 3.1 — INTEGRATION REGION FOR NO RUNOFF (PHI—INDEX)

$$= 1 - \exp(-\beta\phi) \tag{3.7}$$

Evaluation of $f_{I_e,T_e}(i_e,t_e)$

PDF of i_e and t_e can be derived using the technique of derived distribution as follows:

$$f_{I_e,T_e}(i_e,t_e) = f_{I_r,T_r}\left[g_1^{-1}(i_e,t_e),\ g_2^{-1}(i_e,t_e)\right]\left|\frac{\partial(i_r,t_r)}{\partial(i_e,t_e)}\right| \tag{3.8}$$

Using (3.5) and (3.6a) we get

$$f_{I_e,T_e}(i_e,t_e) = \beta\delta\exp[-\beta(i_e + \phi) - \delta t_e]$$

$$= \beta\delta\exp(-\beta\phi)\exp(-\beta i_e - \delta t_e) \qquad i_e,\ t_e > 0 \tag{3.9}$$

The joint PDF of effective rainfall intensity and duration given by above equation will be used to derive the CDF of peak discharge.

3.2.2 Derivation of $f_{I_e,T_e}(i_e,t_e)$ with Philip's Equation as Infiltration Model

Using Philip's (1969) infiltration equation, Eagleson (1978a) has represented infiltration rate f_i as

$$f_i = 0.5S_i t^{-0.5} + A_o \tag{3.10}$$

where

S_i = infiltration sorptivity (cm/hr$^{1/2}$)

A_o = gravitational infiltration rate (cm/hr)

t = elapsed time (hr)

Eagleson (1978b) gives an approximation for time of ponding t_o and surface runoff Rs as

$$t_o = \frac{S_i^2}{2(i_r - A_o)^2} \text{ ,and} \tag{3.11}$$

$$Rs = (i_r - A_o)t_r - S_i(t_r/2)^{1/2} \tag{3.12}$$

As shown in Fig. 3.2 effective rainfall duration and intensity can be expressed as under

$$t_e = t_r - t_o \tag{3.13}$$

$$i_e = \frac{Rs}{t_e} \tag{3.14}$$

Probability of Null Runoff (P_{NR})

When rainfall intensity of a storm is less than or equal to A_o, no runoff is generated (Fig. 3.3a). The storms, having intensities greater than A_o but durations less than or equal to t_o (Fig. 3.3b) will not produce any runoff. Therefore, the shaded portion of the i_r-t_r plane (Fig. 3.3c) is the region of integration for evaluating P_{NR}. The P_{NR} is given by the following relationship.

$$P(i_e = t_e = 0) = \int_{R1,2} f_{I_r,T_r}(i_r,t_r)\,di_r\,dt_r$$

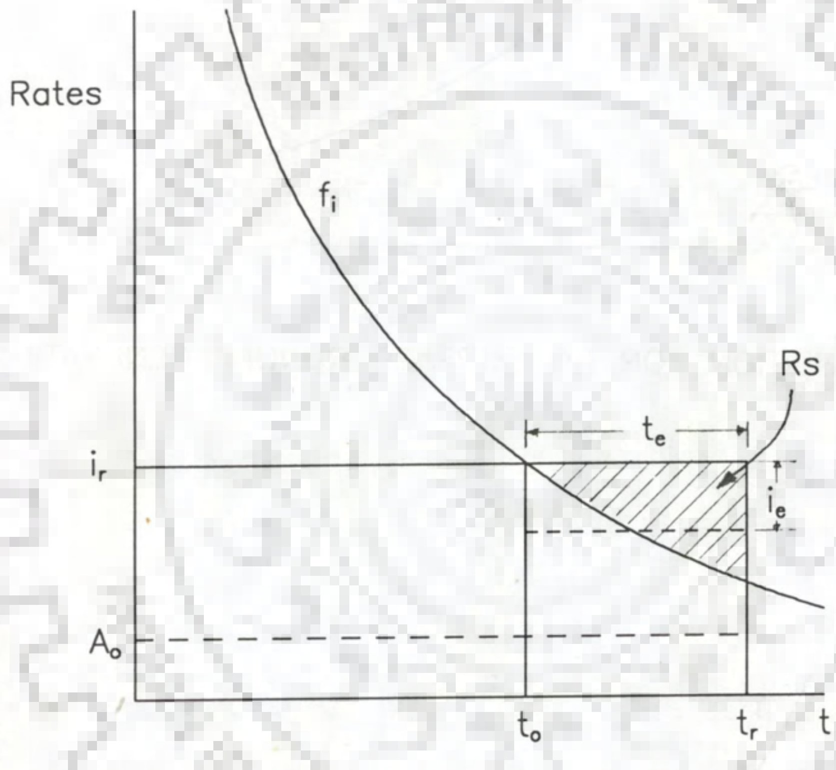


FIG. 3.2 – SURFACE RUNOFF GENERATION DURING TYPICAL STORM.

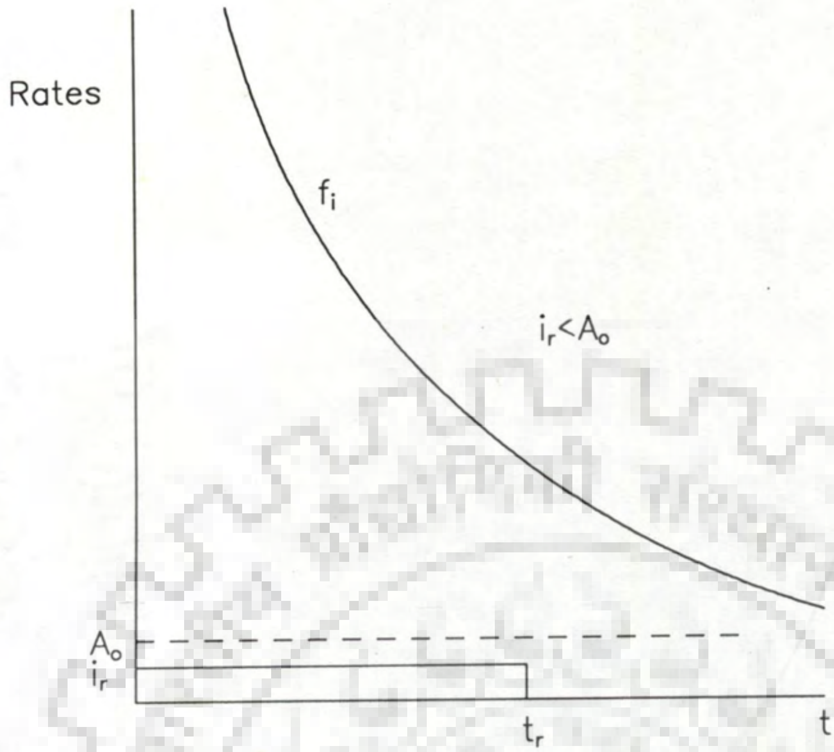


FIG. 3.3 (a) – CONDITION FOR NO SURFACE RUNOFF GENERATION.

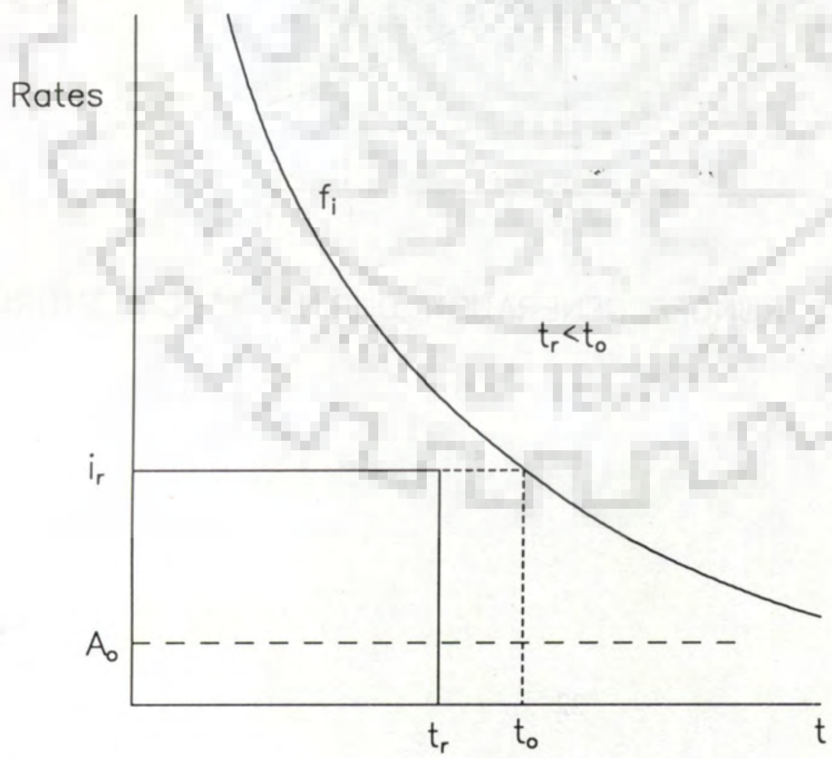


FIG. 3.3 (b) – CONDITION FOR NO SURFACE RUNOFF GENERATION.

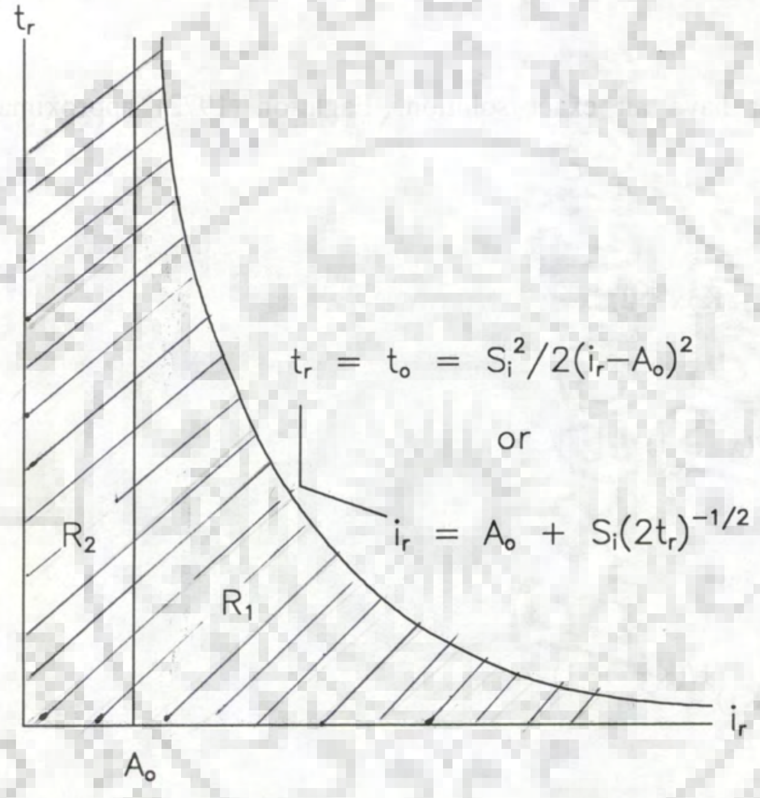


FIG. 3.3 (c) – INTEGRATION REGION FOR NO RUNOFF (PHILIP'S EQN.).

$$= \int_0^{\infty}\left[\int_0^{A_\Theta + S_i(2t_r)^{-1/2}} \beta\delta\exp(- \beta i_r - \delta t_r)di_r\right]dt_r$$

$$= \int_0^{\infty}\delta\exp(- \delta t_r)\{1 - \exp[- \beta(A_o + S_i(2t_r)^{-1/2})]\}dt_r$$

$$= 1 - \delta\exp(-\beta\ A_o)\int_0^{\infty}\exp[- \delta t_r - \beta S_i(2t_r)^{-1/2}]dt_r \qquad (3.15)$$

This integral does not have an exact solution. Eagleson (1972) approximated it in the following manner.

$$I_o = K_1\int_0^{\infty}\exp(- K_1 x - K_2 x^{-K_3})dx$$

$$= \exp(- \sigma/K_3)\sigma^{-\sigma}\Gamma(\sigma + 1)$$

where

$$\sigma = K_1(K_2K_3/K_1)^{1/(K_3 + 1)}$$

Substituting values of K_1, K_2 and K_3 from (3.15) we get

$$\sigma = \delta\left(\frac{\beta S_i}{2\sqrt{2}\ \delta}\right)^{2/3}$$

Using the above solution, the following expression is arrived at.

$$P_{NR} = 1 - \exp(-\beta A_o - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1) \tag{3.16}$$

Probability of null runoff as expressed by (3.16) will be used to derive the CDF of peak discharge.

Evaluation of $f_{I_e,T_e}(i_e,t_e)$

The joint PDF of i_e and t_e will be derived using conditional PDF of $i_e|t_e$ and marginal PDF of t_e. The joint PDF is given by

$$f_{I_e,T_e}(i_e,t_e) = f_{I_e|T_e}(i_e,t_e).f_{T_e}(t_e) \tag{3.17}$$

Evaluation of $f_{T_e}(t_e)$

The integration of $f_{I_r,T_r}(i_r,t_r)$ over the shaded area (Fig. 3.4) will give the probability of t_e to be between 0 and t_{e1}.

$$P(0 < t_e \leq t_{e1}) = \int_{A_o}^{\infty}\left[\int_{t_o}^{t_o+t_{e1}} \beta\delta\exp(-\beta i_r - \delta t_r)dt_r\right]di_r$$

$$= \int_{A_o}^{\infty} \beta\exp(-\beta i_r)\{\exp(-\delta t_o) - \exp[-\delta(t_o + t_{e1})]\}di_r$$

$$= \int_{A_o}^{\infty} \beta\exp\left[-\beta i_r - \delta\frac{S_i^2}{2(i_r - A_o)^2}\right]\left[1 - \exp(-\delta t_{e1})\right]di_r \tag{3.18}$$

Substituting $y = i_r - A_o$

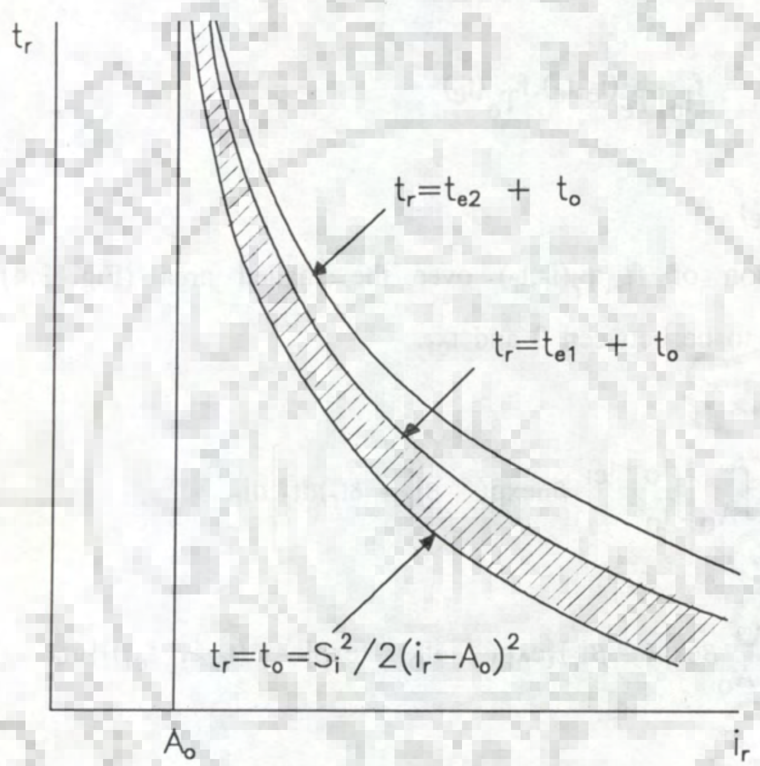


FIG. 3.4 — INTEGRATION REGION FOR EVALUATING THE CDF OF t_e (PHILIP'S EQN.).

$$P(0 < t_e \le t_{e1}) = \beta \exp(-\beta A_o)[1 - \exp(-\delta t_{e1})]\int_0^{\infty} \exp\left[-\beta y - \delta \frac{S_i^2}{2y^2}\right] dy \qquad (3.19)$$

The last integral is a function of β, δ and S_i and can be expressed as $k(\beta,\delta,S_i)$. Therefore, complete CDF of t_e is given by

$$F_{T_e}(t_e) = P(t_e=0) + \beta \exp(-\beta A_o)[1 - \exp(-\delta t_e)]k(\beta,\delta,S_i) \qquad (3.20)$$

where

$$P(t_e=0) = 1 - \exp(-\beta A_o - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)$$

The function $k(\beta,\delta,S_i)$ can be approximated in the same manner as done for P_{NR} but this will be evaluated indirectly using the following property of CDF.

$F_{T_e}(t_e) \to 1$ as $t_e \to \infty$

Therefore, when t_e equals ∞ (3.20) gives

$$k(\beta,\delta,S_i) = \exp(-2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)/\beta$$

Substituting this value of $k(\beta,\delta,S_i)$ in (3.20), we get

$$F_{T_e}(t_e) = 1 - \exp(-\beta A_o - 2\sigma - \delta t_e)\sigma^{-\sigma}\Gamma(\sigma + 1) \qquad (3.21)$$

Differentiation of (3.21) gives

$$f_{T_e}(t_e) = \delta\exp(-\beta A_o - 2\sigma - \delta t_e)\sigma^{-\sigma}\Gamma(\sigma + 1) \qquad t_e > 0 \qquad (3.22)$$

Evaluation of $\mathcal{F}_{I_e|T_e}(i_e,t_e)$

Using (3.12) and (3.14) i_e can be written as

$$i_e = \frac{Rs}{t_e} = \frac{(i_r - A_o)t_r - S_i(t_r/2)^{1/2}}{t_e} \qquad (3.23)$$

Substituting $t_r = t_o + t_e$ in (3.23)

$$i_e = \frac{(i_r - A_o)(t_o + t_e)}{t_e} - \frac{S_i\left(\frac{t_o + t_e}{2}\right)^{1/2}}{t_e}$$

$$= (i_r - A_o)\left(1 + \frac{t_o}{t_e}\right) - \frac{S_i}{\sqrt{2}}\left(\frac{t_o}{t_e^2} + \frac{1}{t_e}\right)^{1/2} \qquad (3.24)$$

Defining $c = t_e/t_o$ and substituting i_r-A_o using (3.11) and after some manipulation we get

$$i_e = \frac{S_i}{(2t_o)^{1/2}}\left[\left(1 + \frac{1}{c}\right) - \left(\frac{1}{c^2} + \frac{1}{c}\right)^{1/2}\right]$$

$$= (i_r - A_o)[1 + c - (1 + c)^{1/2}]/c$$

$$= (i_r - A_o)k(c) \tag{3.25}$$

In order to make (3.25) tractable k(c) is approximated by (Diaz-Granados et al., 1983)

$$k(c) \simeq 0.60729c^{0.09229} \tag{3.26}$$

Substituting (3.26) and value of c in (3.25) we get

$$i_e \simeq (i_r - A_o)0.60729\left(\frac{t_e}{t_o}\right)^{0.09229} \tag{3.27}$$

Replacing t_o in above equation and solving for i_r we get

$$i_r \simeq 1.4434S_i^{0.1558}t_e^{-0.0779}i_e^{0.8442} \tag{3.28}$$

which can be used as $g^{-1}(i_e)$ to obtain conditional distribution of i_e given t_e. Using (3.4) and (3.29) conditional PDF of i_e given t_e can be derived as

$$f_{I_e|T_e}(i_e,t_e) = \left|\frac{di_r}{di_e}\right| f_{I_r}(g^{-1}i_e)$$

$$= 1.2185(S_i/i_e)^{0.1558}t_e^{-0.0779}$$

$$.\beta\exp(-1.4434\beta S_i^{0.1558}t_e^{-0.0779}i_e^{0.8442}) \qquad i_e,t_e > 0 \tag{3.29}$$

Using (3.17), (3.22) and (3.29) we get

$$f_{I_e,T_e}(i_e,t_e) = 1.2185\beta\delta\exp(-\beta A_o - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)(S_i/i_e)^{0.1558}t_e^{-0.0779}$$
$$.\exp(-\delta t_e - 1.4434\beta S_i^{0.1558}t_e^{-0.0779}i_e^{0.8442}) \qquad i_e,t_e > 0 \qquad (3.30)$$

The joint PDF of effective rainfall intensity and duration given by above equation will be used to derive the CDF of peak discharge.

3.2.3 Derivation of $f_{I_e,T_e}(i_e,t_e)$ with SCS Curve Number as Infiltration Model

The excess rainfall depth R is computed as a function of total rainfall depth P and the maximum potential retention S and is given by

$$R = \frac{(P - 0.2S)^2}{(P + 0.8S)} \qquad P > 0.2S \qquad (3.31a)$$

$$R = 0 \qquad P \le 0.2S \qquad (3.31b)$$

where

$$S = \frac{2540}{CN} - 25.4 \qquad (3.32)$$

and units of P, R and S are in cm.

If we express excess rainfall intensity and duration by i_e and t_e respectively, we can write equation (3.31a) as follows:

$$i_e t_e = \frac{(i_r t_r - 0.2S)^2}{(i_r t_r + 0.8S)} \qquad t > t_o \qquad (3.33)$$

Also,

$$t_e = t_r - t_o \tag{3.34}$$

$$i_e = t_e = 0 \qquad\qquad t \le t_o \tag{3.35}$$

where t_o is the time of ponding at which excess rainfall begins. The ponding time t_o can be obtained by using the condition of zero excess rainfall. This is given by

$$i_r t_r - 0.2S = 0 \tag{3.36}$$

$$t_r = t_o = \frac{0.2S}{i_r} \tag{3.37}$$

Eqn. (3.37) will be used to derive the probability of null runoff.

Probability of null runoff (P_{NR})

The probability of no effective rainfall is obtained by integrating the joint PDF of i_r and t_r over the area where no runoff is produced. This area is shown in Fig. 3.5. The P_{NR} is given by

$$\mathrm{Prob}(i_e=0,\ t_e=0) = \int_0^{\infty}\left[\int_0^{0.2S/t_r} \beta\delta\exp(-\beta i_r - \delta t_r)\,di_r\right]dt_r$$

$$= \int_0^{\infty} \delta\exp(-\delta t_r)\left[1 - \exp(-0.2S\beta/t_r)\right]dt_r$$

$$= 1 - \int_0^{\infty}\left[\delta\exp(-\delta t_r - 0.2S\beta/t_r)\right]dt_r \tag{3.38}$$

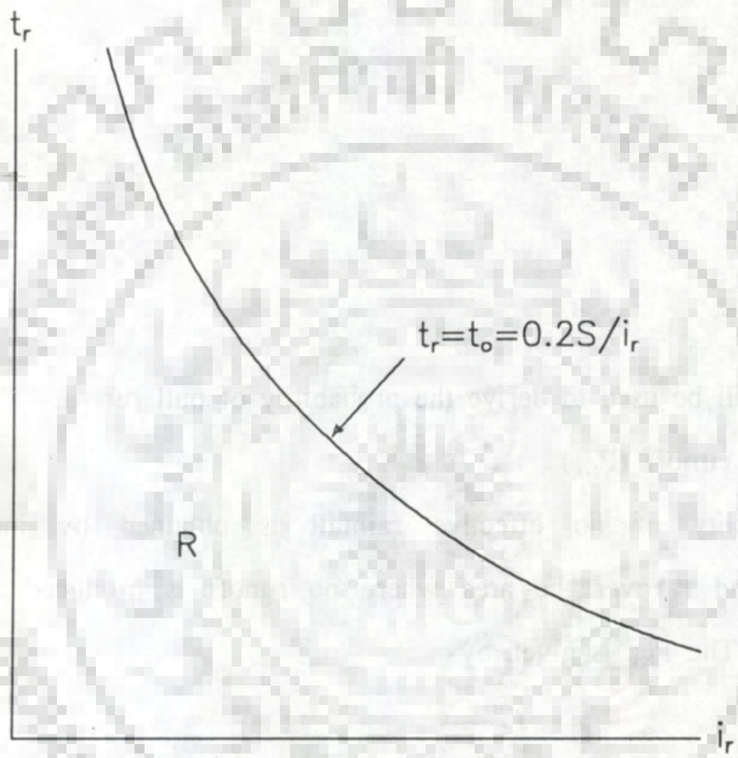


FIG. 3.5 — INTEGRATION REGION FOR NO RUNOFF (SCS METHOD).

This last integral does not have an exact solution. Approximate solution as given by Eagleson(1972) gives

$$\text{Prob}(i_e=0,\ t_e=0) = 1 - \exp(-\sigma)\Gamma(\sigma + 1)\ \sigma^{-\sigma} \tag{3.39}$$

where

$$\sigma = \delta\left(0.2\ \beta\ \frac{S}{\delta}\right)^{1/2} \tag{3.40}$$

Evaluation of $f_{I_e,T_e}(i_e,t_e)$

The continuous part of the joint PDF of i_e and t_e can be computed as the product of the conditional PDF of i_e given t_e and the marginal PDF of t_e.

$$f_{I_e,T_e}(i_e,t_e) = f_{I_e|T_e}(i_e,t_e).f_{T_e}(t_e) \tag{3.41}$$

Eqn. (3.41) will be used to find the joint PDF of i_e and t_e.

Evaluation of $f_{I_e|T_e}(i_e,t_e)$

The excess rainfall depth R is given by

$$R = \frac{(P - 0.2S)^2}{(P + 0.8S)} \tag{3.42}$$

R, P and S can be expressed in terms of i_r, t_r, i_e, t_e and t_o as

$R = i_e t_e$, $P = i_r t_r$, $S = 5 i_r t_o$ (equation 3.37) and $t_r = t_o + t_e$.

Substituting above expressions in (3.42) we get

$$i_e t_e = \frac{(i_r t_r - i_r t_o)^2}{(i_r t_r + 4 i_r t_o)} \tag{3.43}$$

$$i_e = \frac{(i_r t_r)^2 - 2 i_r^2 t_r t_o + (i_r t_o)^2}{t_e i_r (t_r + 4 t_o)}$$

$$i_e = \frac{i_r^2[(t_o + t_e)^2 - 2(t_o + t_e)t_o + t_o^2]}{t_e i_r (t_e + 5 t_o)}$$

$$i_e = \frac{i_r t_e}{(t_e + 5 t_o)} \tag{3.44}$$

substituting $c = \frac{t_e}{t_o}$ in (3.44)

$$i_e = \frac{c}{(c + 5)} i_r \quad \text{or} \quad i_e = K(c)\ i_r \tag{3.45}$$

where

$$K(c) = \frac{c}{(c + 5)} \tag{3.46}$$

In order to make equation (3.44) tractable, K(c) as given by equation (3.46) is to be approximated by some other function. The approximation chosen by Raines and Valdes (1993) is as under.

$$K(c) \simeq 0.15517c^{0.79086} \tag{3.47}$$

Introducing the above function into equation (3.44) and replacing c in terms of t_e and t_o, we get

$$i_r \simeq \frac{i_e}{0.15517(t_e/t_o)^{0.79086}}$$

$$i_r \simeq \frac{i_e}{0.15517 t_e^{0.79086}(5i_r/S)^{0.79086}}$$

$$i_r \simeq 1.39047 i_e^{0.55839} S^{0.44161} t_e^{-0.44161} \tag{3.48}$$

Using derived distribution technique, we get conditional PDF of i_e and t_e as

$$f_{I_e|T_e}(i_e,t_e) = \left|\frac{di_r}{di_e}\right| f_{I_r}(g^{-1}i_e) \tag{3.49}$$

Using (3.48) and marginal distribution of i_r, we get

$$f_{I_e|T_e}(i_e,t_e) = 0.77642\,\beta\, i_e^{-0.44161} t_e^{-0.44161} S^{0.44161} \cdot \exp(-1.39047\,\beta\, S^{0.44161} t_e^{-0.44161} i_e^{0.55839}) \quad i_e,t_e>0 \tag{3.50}$$

Evaluation of $f_{T_e}(t_e)$

Fig. 3.6 shows the i_r-t_r plane where the dashed lines represent different values of t_e, i.e., t_{e1} and t_{e2}. The shaded area corresponds to the values of t_e

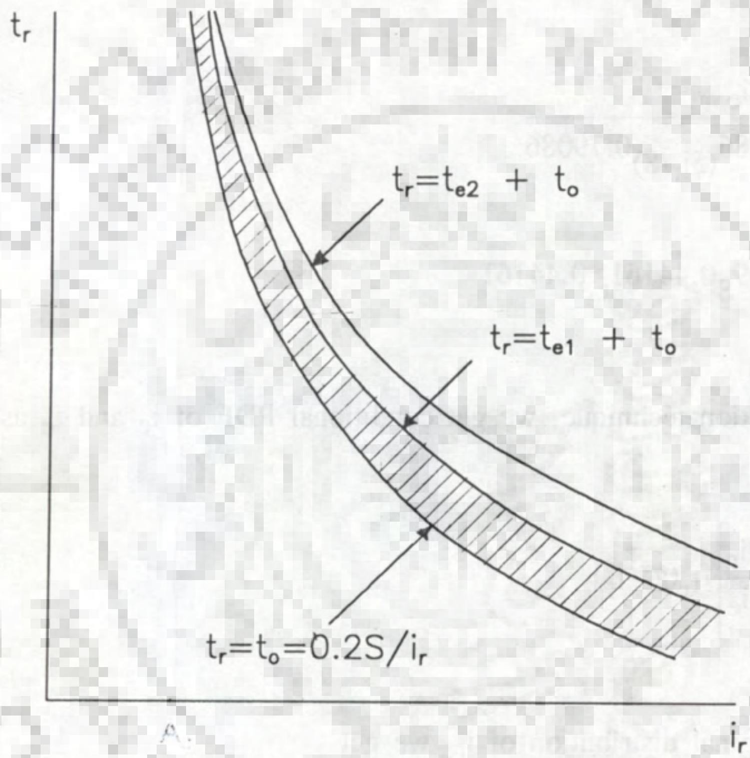


FIG. 3.6 – INTEGRATION REGION FOR EVALUATING THE CDF OF t_e (SCS METHOD).

between 0 and t_{e1}. Therefore, integration of $f_{I_r,T_r}(i_r,t_r)$ over that area will give the probability that t_e is between 0 and t_{e1}

$$\text{Prob}(0 < t_e \le t_{e1}) = \int_{0.2S/t_o}^{\infty} \left[\int_{t_o}^{t_{e1}+t_o} \delta\beta \exp(-\delta t_r - \beta i_r) dt_r\right] di_r$$

$$= \beta[1 - \exp(-\delta t_{e1})] \int_{0.2S/t_o}^{\infty} \exp[-(0.2S\delta/i_r) - \beta i_r] di_r \quad (3.51)$$

Substituting $y = i_r - 0.2S/t_o$ we get

$$\text{Prob}(0 < t_e \le t_{e1}) = \beta(1 - e^{-\delta t_{e1}}) \int_0^{\infty} \exp(-\beta(y + \frac{0.2S}{t_o}) - \frac{0.2\delta S}{y + 0.2S/t_o}) dy \quad (3.52)$$

where the last integral may be approximated in the same manner as before, however, in order to preserve the properties of any CDF, it is evaluated indirectly as follows. The CDF of t_e can be expressed as:

$$F_{T_e}(t_e) = \text{Prob}(t_e = 0) + \beta(1 - e^{-\delta t_e}).K(\beta,\delta,S)$$

Substituting value of $\text{Prob}(t_e=0)$ from (3.39)

$$F_{T_e}(t_e) = 1 - e^{-\sigma}\Gamma(\sigma + 1)\sigma^{-\sigma} + \beta(1 - e^{-\delta t_e}).K(\beta,\delta,S) \quad (3.53)$$

$K(\beta,\delta,S)$ is then calculated such that

$$F_{T_e}(t_e) \to 1 \text{ as } t_e \to \infty$$

Consequently,

$$K(\beta,\delta,S) = e^{-\sigma}\Gamma(\sigma + 1)\sigma^{-\sigma}/\beta \tag{3.54}$$

and

$$F_{T_e}(t_e) = 1 - e^{-\sigma}\Gamma(\sigma + 1)\sigma^{-\sigma}e^{-\delta t_e} \tag{3.55}$$

Differentiation of (3.55) gives

$$f_{T_e}(t_e) = \delta e^{-\sigma}\Gamma(\sigma + 1)\sigma^{-\sigma}e^{-\delta t_e} \qquad t_e > 0 \tag{3.56}$$

Using (3.41), (3.50) and (3.56), continuous part of the joint PDF of i_e and t_e is given by

$$f_{I_e,T_e}(i_e,t_e) = 0.77642\beta\delta\exp(-\delta t_e - \sigma)\Gamma(\sigma + 1)\sigma^{-\sigma}\left(\frac{S}{i_e t_e}\right)^{0.44161}$$

$$.\exp\left[-1.39047\beta\left(\frac{S}{t_e}\right)^{0.44161} i_e^{0.55839}\right] \qquad i_e,\ t_e > 0 \tag{3.57}$$

Above equation will be used to derive the CDF of peak discharge.

3.3 EFFECTIVE RAINFALL-RUNOFF MODELS

In the following sections brief information about geomorphoclimatic instantaneous unit hydrograph (GcIUH) and kinematic wave (KW) theory based effective rainfall-runoff models is presented. These models are one of the main components of the DFFD models. For detailed description of these models Rodriguez-Iturbe et al. (1982); Diaz-Granados et al. (1983); Wooding (1965); Eagleson (1970); and Cadavid et al., (1991) may be referred.

3.3.1 GcIUH

Rodriguez-Iturbe and Valdes (1979) introduced the concept of geomorphologic instantaneous unit hydrograph (GIUH). GIUH of a basin is a function of Horton numbers, length of highest order stream and peak velocity of response. A stochastic reinterpretation of GIUH was then proposed by Rodriguez-Iturbe et al. (1982) as GcIUH. Velocity term of GIUH is expressed as a function of storm intensity and duration in GcIUH. Expressions for IUH peak q_p and time to peak t_p were derived as functions of catchment parameters and effective rainfall intensity i_e. The q_p and t_p of GcIUH are given by

$$q_p = \frac{0.871(i_e A R_L)^{2/5} \alpha_\Omega^{3/5}}{L_\Omega} \tag{3.58}$$

$$t_p = \frac{0.585 L_\Omega}{(i_e A R_L)^{2/5} \alpha_\Omega^{3/5}} \tag{3.59}$$

where

L_Ω = Length of highest order stream (km)

A = Area of the watershed (km^2)

R_L = Length ratio

α_Ω = KW parameter of the highest order stream ($m^{-1/3} s^{-1}$)

i_e = Effective rainfall intensity (cm/hr)

3.3.2 Kinematic Wave

The kinematic wave equations relate the storm parameters and hydraulic and physical parameters of the catchment to compute the discharge hydrograph at the outlet. Wooding (1965 and 1966) applied kinematic wave theory to real catchments. A simple catchment - stream geometry (Fig. 3.7) was considered. The catchment was assumed to be two identical rectangular planes joined to form a V, along the apex of which a line stream can flow. Following Iwagaki (1955); Lighthill and Witham (1955) and Wooding (1965), Eagleson (1970) presented analytical solutions of kinematic equations for different combinations of storm intensity and duration. Cadavid et al. (1991) have used a combination of analytical and regression equations for different cases. This is discussed below.

The watershed is assumed as two rectangular planes discharging into a first order stream located at the middle of the watershed. Solutions for plane and stream are as follows:

Solution for Plane

The time of concentration t_c for a plane of width W due to effective rainfall intensity i_e of duration t_e using method of characteristics is given by

$$t_c = \left[\frac{W i_e^{1-\beta_p}}{\alpha_p} \right]^{1/\beta_p} \tag{3.60}$$

where

$$\alpha_p = \frac{S_p^{1/2}}{n_p} \qquad \text{and} \qquad \beta_p = \frac{5}{3} \tag{3.61}$$

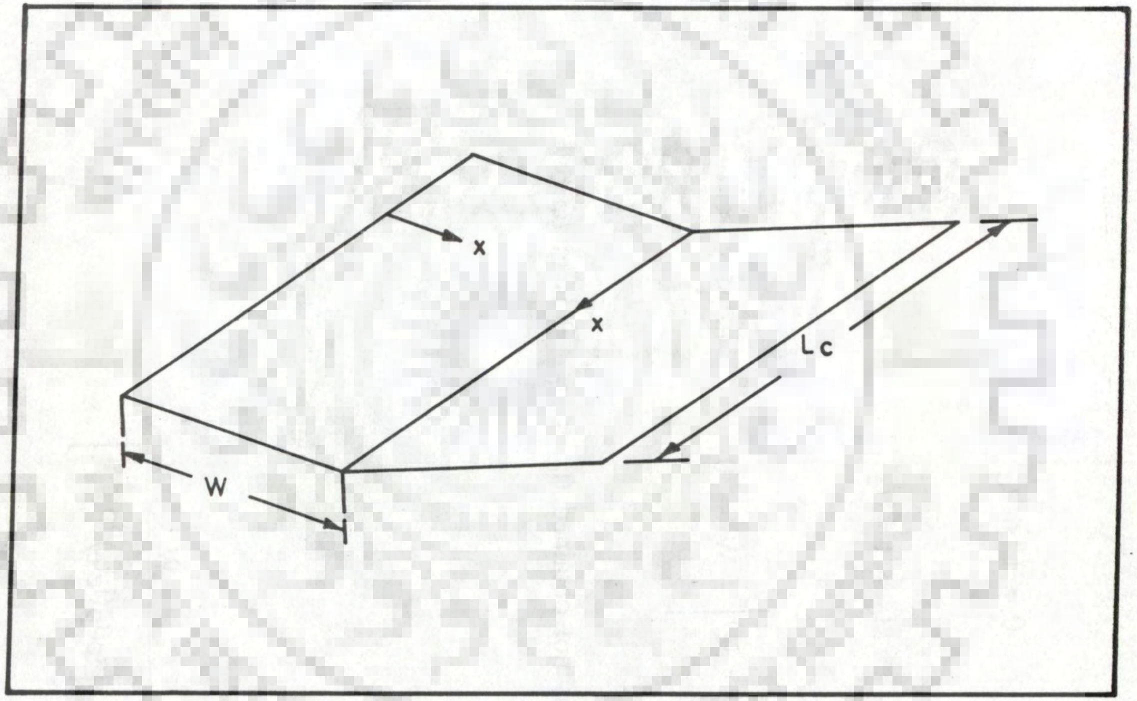


FIG. 3·7. CACTHMENT-STREAM GEOMETRY

and S_p and n_p are plane slope and Manning's roughness respectively. The plane has been assumed as wide open channel.

For a given intensity, when complete plane contributes to runoff (concentration), the flow depth y at the plane outlet, is given by the set of following equations.

$$y = i_e t \qquad 0 \le t < t_c \tag{3.62}$$

$$y = i_e t_c \qquad t_c \le t \le t_e \tag{3.63}$$

$$\alpha_p y^{\beta_p - 1} \left[\frac{y}{i_e} + \beta_p (t - t_e) \right] - W = 0 \qquad t_e < t < \infty \tag{3.64}$$

when concentration is not obtained ($t_e < t_c$), y is given by

$$y = i_e t \qquad 0 \le t \le t_e \tag{3.65}$$

$$y = i_e t_e \qquad t_e < t \le t_p \tag{3.66}$$

$$t_p = \left[\frac{\beta_p - 1}{\beta_p} \right] t_e + \frac{W}{\left[\alpha_p \beta_p (i_e t_e)^{\beta_p - 1} \right]} \tag{3.67}$$

when concentration is not obtained, flow depth remains constant between t_e and t_p.

Flow depth for recession limb is calculated using (3.64). However, (3.67) is applicable in the range of $t > t_p$ for the second case ($t < t_c$).

Discharge q_L entering stream at the plane outlet is given by

$$q_L = \alpha_p y^{\beta_p} \tag{3.68}$$

Solution for Stream

The equations for channel flow are (Eagleson, 1970)

$$\frac{dx}{dt} = \alpha_c \beta_c A^{\beta_c - 1} \tag{3.69}$$

$$\frac{dQ}{dx} = 2q_L \tag{3.70}$$

$$\frac{dA}{dt} = 2q_L \tag{3.71}$$

Based on Manning's equation, α_c and β_c are

$$\alpha_c = \frac{a^{2/3} S_c^{1/2}}{n_c} \qquad \text{and} \qquad \beta_c = 1 + \frac{2b}{3}$$

where S_c and n_c are channel slope and roughness respectively, a and b are the coefficient and exponent of the relationship of hydraulic radius R and flow cross sectional area A. Cadavid et al. (1991) used a = 0.25 and b = 0.35 for FPS system of units.

Four possible cases have been considered to describe the hydrograph in the channel. These four cases are given in Table-3.1.

Table -3.1 Combination of overland and channel flow

Case	Reached concentration	
	Plane	Channel
1	Yes	Yes
2	Yes	No
3	No	Yes
4	No	No

(Source: Cadavid et al., 1991)

Maximum discharge for these four cases is given by

$$Q_{p1} = 2L_c W i_e \qquad t_e \geq t^* \tag{3.72}$$

$$Q_{p3} = 2L_c \alpha_p (i_e t_e)^{\beta_p} \qquad t_e + t_s'' \leq t_p \text{ and } t_e < t_c \tag{3.73}$$

$$Q_{p2} = 0.2\left[-129.697 + 49.878 \ln\left(\frac{100 t_e}{t^*}\right)\right] L_c W i_e \qquad t^* > t_e > t_c \tag{3.74}$$

$$Q_{p4} = 0.2\left[-118.552 + 47.458 \ln\left(\frac{100 t_p}{t_e + t_s''}\right)\right] L_c \alpha_p (i_e t_e)^{\beta_p}$$

$$t_c > t_e, \quad t_p < t_e + t_s'' \tag{3.75}$$

where

L_c = length of main channel

$t^* = t_c + t_s$

$$t_s = \left[\frac{L_c}{\left[\alpha_c (2Wi_e)^{\beta_c} - 1\right]} \right]^{1/\beta_c}$$

$$t_s'' = \left[\frac{L_c}{\left[\alpha_c [2\alpha_p (i_e t_e)^{\beta_p})]^{\beta_c} - 1\right]} \right]^{1/\beta_c}$$

Above equations of peak discharge for different cases will be used to define the integration regions for derivation of the CDF of peak discharge.

3.4 DERIVATION OF FLOOD FREQUENCY DISTRIBUTIONS USING DIFFERENT APPROACHES

This section describes the derivation of flood frequency distribution based on GcIUH or KW effective rainfall-runoff models using different infiltration models.

3.4.1 GcIUH-ϕ-index

This DFFD model was introduced by Diaz-Granados et al. (1983). Considering triangular IUH, Henderson (1963) describes the peak discharge at the outlet of basin as

$$Q_p = \frac{2 i_e t_e A}{t_b} \left(1 - \frac{t_e}{2 t_b} \right) \qquad \text{for } t_e < t_b \qquad (3.76)$$

$$Q_p = i_e A \qquad \text{for } t_e \geq t_b \qquad (3.77)$$

For a triangular IUH:

$$q_p t_b = 2$$

where t_b is the time base of the IUH. Using above relationship, (3.76) and (3.77) become

$$Q_p = i_e t_e A q_p \left(1 - \frac{q_p t_e}{4}\right) \qquad \text{for } t_e < \frac{2}{q_p} \qquad (3.78)$$

$$Q_p = i_e A \qquad \text{for } t_e \geq \frac{2}{q_p} \qquad (3.79)$$

Using expression for q_p (3.58) as given by Rodriguez-Iturbe et al. (1982) (3.78) and (3.79) can be written as

$$Q_p = 0.871 K_1 A i_e^{7/5} t_e \left(1 - \frac{0.871 K_1 i_e^{2/5} t_e}{4}\right) \qquad \text{for } t_e < \frac{2}{0.871 K_1} i_e^{-2/5} \qquad (3.80)$$

$$Q_p = i_e A \qquad \text{for } t_e \geq \frac{2}{0.871 K_1} i_e^{-2/5} \qquad (3.81)$$

where

$$K_1 = \frac{(AR_L)^{2/5} \alpha_\Omega^{3/5}}{L_\Omega}$$

Solving (3.80) for t_e we get

$$t_e = \frac{2}{0.871K_1} i_e^{-2/5}\left[1 - \left(1 - \frac{Q_p}{Ai_e}\right)^{1/2}\right] \quad (3.82)$$

Defining $t_e = t_e^*$ and $Q_p/A = Q_p^*$ Diaz-Granados et al. (1983) evaluated CDF of Q_p as

$$F_{Q_p}(Q_p) = P_{NR} + \int_0^{Q_p^*}\left[\int_0^{\infty} f_{I_e,T_e}(i_e,t_e)dt_e\right]di_e + \int_{Q_p^*}^{\infty}\left[\int_0^{t_e^*} f_{I_e,T_e}(i_e,t_e)dt_e\right]di_e \quad (3.83)$$

Substituting (3.9) in (3.83), first (I_1) and second (I_2) integrals are evaluated as

$$I_1 = \int_0^{Q_p^*}\left[\int_0^{\infty} \beta\delta\exp(-\beta\phi)\exp(-\beta i_e - \delta t_e)dt_e\right]di_e$$

$$= \int_0^{Q_p^*} \beta\exp(-\beta\phi)\exp(-\beta i_e)di_e$$

$$= \left[1 - \exp(-\beta Q_p^*)\right]\exp(-\beta\phi)$$

$$I_2 = \int_{Q_p^*}^{\infty}\left[\int_0^{t_e^*} \beta\delta\exp(-\beta\phi)\exp(-\beta i_e - \delta t_e)dt_e\right]di_e$$

$$= \int_{Q_p^*}^{\infty} \beta\exp(-\beta\phi)\left[1 - \exp(-\delta t_e^*)\right]\exp(-\beta i_e)di_e$$

$$= \exp(-\beta\phi - \beta Q_p^*) - \beta\exp(-\beta\phi)\int_{Q_p^*}^{\infty} \exp(-\beta i_e - \delta t_e^*)di_e$$

substituting values of I1, I2 and P_{NR} from (3.7) in (3.83) we get

$$F_{Q_p}(Q_p) = 1 - \beta\exp(-\beta\phi)\int_{Q_p^*}^{\infty} \exp(-\beta i_e - \delta t_e^*)di_e \tag{3.84}$$

where t_e^* is given by (3.82).

The computer programme for this model (modified from Diaz-Granados et al.,1983) is given in Appendix - 1.

3.4.2 GcIUH-Philip

This DFFD model was given by Diaz-Granados et al. (1983, 1984). For GcIUH based DFFD models CDF of Q_p is given by (3.83). Using (3.30) for PDF of effective rainfall in (3.83) we get first integral as

$$I_1 = 1.2185\beta\delta\exp(-\beta A_0 - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)S_i^{k} \cdot\int_0^{\infty} t_e^{j}\exp(-\delta t_e)\left[\int_0^{Q_p^*} i_e^{-k} \exp(-1.4434\beta S_i^{k} i_e^{l} t_e^{j})di_e\right]dt_e \tag{3.85}$$

where

$l = 0.8442,$

$k = 0.1558$ and

$j = -0.0779$

Defining $A^* = 1.2185\beta\delta\exp(-\beta A_0 - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)S_i^{k}$ and substituting $y = i_e^{l}$ and changing the limits of integral (3.85) can be expressed as

$$I_1 = A^* \int_0^{\infty} t_e^{j} \exp(-\delta t_e) \left[\int_0^{Q_p^{*l}} y^{-k/l} \exp(-1.4434 \beta S_i^{k} y t_e^{j}) \frac{dy}{l y^{(l-1)/l}} \right] dt_e$$

$$= A^* \int_0^{\infty} t_e^{j} \exp(-\delta t_e) \left[\frac{1}{l} \left(\frac{\exp(-1.4434 \beta S_i^{k} t_e^{j} Q_p^{*l}) - 1}{-1.4434 \beta S_i^{k} t_e^{j}} \right) \right] dt_e \tag{3.86}$$

Replacing value of A^* we get

$$I_1 = \exp(-\beta A_0 - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1) \left[1 - \int_0^{\infty} \delta \exp(-1.4434 \beta S_i^{k} t_e^{j} Q_p^{*l} - \delta t_e) dt_e \right] \tag{3.87}$$

The second integral of (3.83) for this case can be given by

$$I_2 = A^* \int_{Q_p^*}^{\infty} i_e^{-k} \left[\int_0^{t_e^*} t_e^{j} \exp(-1.4434 \beta S_i^{k} t_e^{j} i_e^{l} - \delta t_e) dt_e \right] di_e$$

The inner integral can not be evaluated analytically and since t_e^* is a function of i_e, it is not possible to change the integration order as done for I_1. Numerical integration will take more computer time, therefore, following approximations were made by Diaz-Granados et al. (1983).

$$t_e^* = \frac{2}{0.871 K_1} i_e^{-2/5} \left[1 - \left(1 - \frac{Q_p^*}{i_e} \right)^{1/2} \right] \tag{3.88}$$

Replacing

$$1 - \left(1 - \frac{Q_p^*}{i_e}\right)^{1/2} \simeq \left(\frac{Q_p^*}{i_e}\right)^{3.1358} \qquad Q_p^* \le i_e \le 1.2Q_p^*$$

$$\simeq \left(0.80482 \frac{Q_p^*}{i_e}\right)^{1.36396} \qquad 1.2Q_p^* \le i_e \le 2Q_p^*$$

$$\simeq \left(0.6595 \frac{Q_p^*}{i_e}\right)^{1.10812} \qquad 2Q_p^* \le i_e \le 5Q_p^*$$

$$\simeq \left(0.5 \frac{Q_p^*}{i_e}\right) \qquad 5Q_p^* \le i_e \le \infty$$

Using above approximations the integral is split up into four integrals, whose integration limits are determined as follows:

$Q_p^* \le i_e \le 1.2Q_p^*$

$$t_e^* = \frac{2}{0.871K_1} i_e^{-2/5} \left(\frac{Q_p^*}{i_e}\right)^{3.1358} \tag{3.89}$$

substituting $i_e = Q_p^*$ and $i_e = 1.2Q_p^*$ in (3.89) we get

$t_e^* = 2.2962{Q_p^*}^{-0.4}/K_1$ for $i_e = Q_p^*$

$$t_e^* = 1.2051 Q_p^{*-0.4} / K_1 \qquad \text{for } i_e = 1.2Q_p^*$$

$$i_{e1} = [2Q_p^{*3.1358}/0.871K_1t_e]^{1/3.5358}$$

Similarly other limits are computed. These limits are given below.

$$i_{e2} = [2(0.80482Q_p^*)^{1.36396}/0.871K_1t_e]^{1/1.76396}$$

$$i_{e3} = [2(0.65295Q_p^*)^{1.10812}/0.871K_1t_e]^{1/1.50812}$$

$$i_{e4} = [Q_p^*/0.871K_1t_e]^{1/1.4}$$

The limits of t_e will be as follows:

$$t_{e1} = 2.2962Q_p^{*-0.4}/K_1$$

$$t_{e2} = 1.2216Q_p^{*-0.4}/K_1$$

$$t_{e3} = 0.5031Q_p^{*-0.4}/K_1$$

$$t_{e4} = 0.1235Q_p^{*-0.4}/K_1$$

The second integral can now be evaluated as follow:

$$I_2 = \int_0^{t_{e4}}\left[\int_{Q_p^*}^{i_{e4}} f_{I_e,T_e}(i_e,t_e)di_e\right]dt_e + \int_{t_{e4}}^{t_{e3}}\left[\int_{Q_p^*}^{i_{e3}} f_{I_e,T_e}(i_e,t_e)di_e\right]dt_e$$

$$+ \int_{t_{e3}}^{t_{e2}}\left[\int_{Q_p^*}^{i_{e2}} f_{I_e,T_e}(i_e,t_e)di_e\right]dt_e + \int_{t_{e2}}^{t_{e1}}\left[\int_{Q_p^*}^{i_{e1}} f_{I_e,T_e}(i_e,t_e)di_e\right]dt_e \qquad (3.90)$$

The inner integrals of (3.90) can be evaluated using similar method as done for I_1. As a result each right hand side component of (3.90) has the following form:

$$\delta\exp(-\beta A_o - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)\Big\{\int_{t_{ei+1}}^{t_{ei}} \exp(-\delta t_e - 1.4434\beta S_i^k Q_p^{*l} t_e^j)dt_e$$
$$- \int_{t_{ei+1}}^{t_{ei}} \exp(-\delta t_e - 1.4434\beta S_i^k i_{ei}^l t_e^j)dt_e\Big\} \qquad (3.91)$$

The four positive terms of (3.91) when added will yield

$$\delta\exp(-\beta A_o - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)\int_0^{t_{e1}} \exp(-\delta t_e - 1.4434\beta S_i^k Q_p^{*l} t_e^j)dt_e$$

The four negative terms of (3.91) after substituting the values of limits and i_{e1}, i_{e2}, i_{e3} and i_{e4} can be expressed as

$$J_i = \int_{a_i Q_p^{*-0.4}/K_1}^{b_i Q_p^{*-0.4}/K_1} \exp\Big\{-\delta t_e - 1.4434\beta S_i^k t_e^j$$
$$\cdot[2(c_i Q_p^*)^{d_i}/0.871K_1 t_e]^{l/e_i}\Big\}dt_e \qquad (3.92)$$

Using (3.16), (3.87), (3.91) and (3.92) we get

$$F_{Q_p}(Q_p) = 1 - \delta\exp(-\beta A_o - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)\left(I + \sum_{i=1}^{4} J_i\right) \qquad (3.93)$$

where

$$I = \int_{2.2962Q_p^{*-0.4}/K_1}^{\infty} \exp(-\delta t_e - 1.4434\beta S_i^k Q_p^{*l} t_e^j) dt_e$$

and J_i is given by (3.92).

Coefficients a_i, b_i, c_i, d_i and e_i are listed in Table 3.2.

Table 3.2 - Coefficients of J_i

i	a_i	b_i	c_i	d_i	e_i
1	0.0000	0.1235	0.5000	1.0000	1.4000
2	0.1235	0.5033	0.6529	1.1081	1.5081
3	0.5033	1.2216	0.8048	1.3640	1.7640
4	1.2216	2.2962	1.0000	3.1358	3.5358

(Source:Diaz-Granados et al. (1983,1984)

The computer programme for this model (modified from Diaz-Granados et al.,1983) is given in Appendix - 2.

3.4.3 GcIUH-SCS

This DFFD model was given by Raines and Valdes (1993). Derivation of PDF of i_e and t_e has already been given in section 3.2.3. Using this PDF and the method used by Diaz-Granados et al. (1983) as described in section 3.4.2 the CDF of peak discharge is given by

$$F_{Q_p}(Q_p) = 1 - \delta \exp(-\sigma)\sigma^{-\sigma}\Gamma(\sigma + 1)\left[I + \sum_{i=1}^{4} J_i \right] \qquad (3.94)$$

where

$$I = \int_{2.2962 Q_p^{*-0.4}/K_1}^{\infty} \exp(-\delta t_e - 1.39047 \beta S^k Q_p^{*l} t_e^j) dt_e$$

and

$$J_i = \int_{a_i Q_p^{*-0.4}/K_1}^{b_i Q_p^{*-0.4}/K_1} \exp\left\{-\delta t_e - 1.39047 \beta S^k t_e^j [2(c_i Q_p^*)^{d_i}/0.871 K_1 t_e]^{1/e_i}\right\} dt_e$$

Coefficients a_i, b_i, c_i, d_i and e_i are listed in Table 3.2 and

$j = -0.44161$,

$k = 0.44161$ and

$l = 0.55839$.

The computer programme for this model (developed in FORTRAN language) is given in Appendix - 3.

3.4.4 KW-Philip

This DFFD model is based on models given by Eagleson (1972) and Diaz-Granados et al. (1983, and 1984). Cadavid et al. (1991) included the omitted case (case 3 in Table 3.1) by Eagleson (1972) and gave regression equations as given in section 3.3.2. Using the PDF of i_e and t_e given by Diaz-Granados et al. (1983, and 1984), Cadavid et al. (1991) derived the CDF of peak discharge as:

$$F_{Q_p}(Q_p) = P_{NR} + \int_{R_i} f_{I_e,T_e}(i_e,t_e) di_e dt_e \qquad (3.95)$$

The regions of integration were defined according to the four cases considered

(Fig. 3.8). Integration of (3.95) with respect to i_e between limits i_{e1} and i_{e2} gives

$$\int_{i_{e1}}^{i_{e2}} f_{I_e,T_e}(i_e,t_e)\,di_e = g(i_{e1},\ i_{e2},t_e) \tag{3.96}$$

where

$$g(i_{e1},\ i_{e2},\ t_e) = \delta\exp(-\beta A_o - 2\sigma)\sigma^{-\sigma}\Gamma(\sigma+1)\exp(-\delta t_e)$$
$$.[\exp(-1.4434\beta S_i^{k} i_{e1}^{l} t_e^{j}) - \exp(-1.4434\beta S_i^{k} i_{e2}^{l} t_e^{j})]$$

Using these results and referring to Fig. 3.8 the CDF of Q_p can be expressed as

$$\begin{aligned} F_{Q_p}(Q_p) = P_{NR} &+ \int_{t_{e12}}^{t_{emax}} g(i_{e11\text{-}2}(t_e),\ i_{e21}(t_e),t_e)dt_e \\ &+ \int_{t_{e24}}^{t_{e12}} g(i_{e12\text{-}4}(t_e),i_{e22}(t_e),t_e)dt_e \\ &+ \int_{t_{e12}}^{t_{emax}} g(i_{e12\text{-}4}(t_e),i_{e21\text{-}2}(t_e),t_e)dt_e \\ &+ \int_{t_{e43}}^{t_{e24}} g(i_{e14\text{-}3}(t_e),i_{e24}(t_e),t_e)dt_e \\ &+ \int_{t_{e24}}^{t_{emax}} g(i_{e14\text{-}3}(t_e),i_{e22\text{-}4}(t_e),t_e)dt_e \\ &+ \int_{t_{emin}}^{t_{e43}} g(i_{e1}(i_e=0),i_{e23}(t_e),t_e)dt_e \\ &+ \int_{t_{e43}}^{t_{emax}} g(i_{e1}(i_e=0),i_{e24\text{-}3}(t_e),t_e)dt_e \end{aligned} \tag{3.97}$$

where

i_{e2i}, i = 1-4 is the upper limit of integration defined by the equations of peak discharge for case 1 to 4

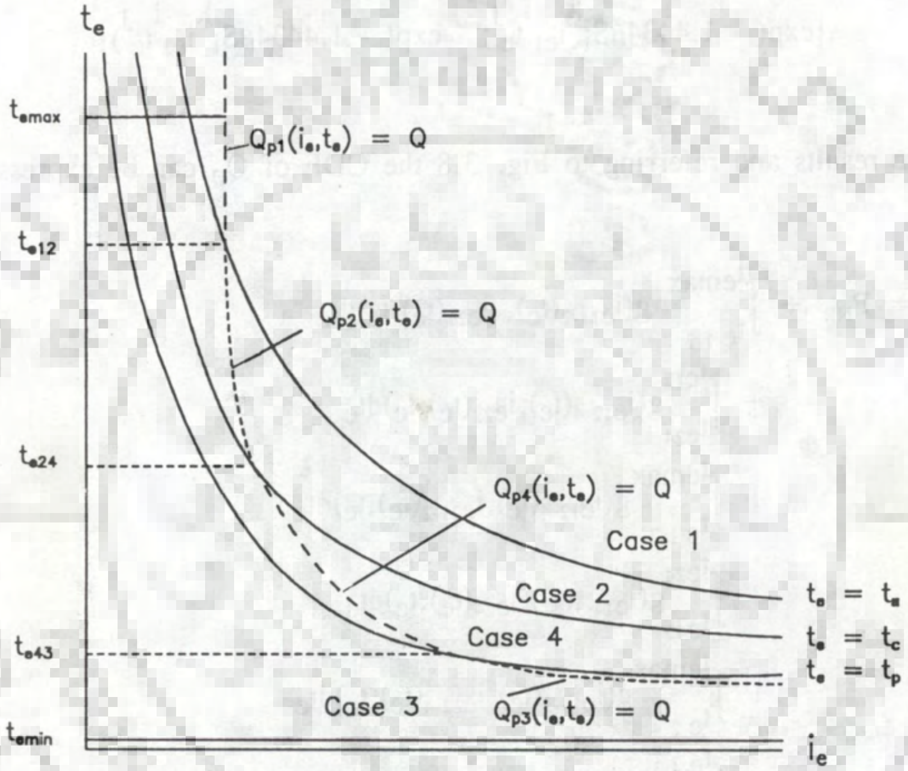


FIG. 3.8–INTEGRATION REGIONS FOR COMPUTATION OF CDF OF PEAK DISCHARGE (Cadavid et al.,1991).

i_{eij-k}, $i = 1\text{-}2$ is the upper ($i = 1$) or lower ($i = 2$) limit of integration defined by the equation of the boundary between cases j and k

$i_{e1}(i_e=0)$ is the axis t_e at which $i_e = 0$.

t_{emin} is defined by the user to get a specific tolerance t_l in integration (0.05 second as used by Cadavid et al., 1991).

$$t_{emax} = -\frac{f_s}{\delta}\ln\left[\frac{t_l}{\delta(1 - P_{NR})}\right] \qquad t_l < \delta(1 - P_{NR})$$

where f_s is factor of safety (Cadavid et al.(1991) used $f_s = 1.2$)

The computer programme for this model (developed in FORTRAN language) is given in Appendix - 4.

3.4.5 KW-SCS

In DFFD models which use KW as effective rainfall-runoff model, researchers have used only ϕ - index and Philip's infiltration equation to derive PDF of effective rainfall intensity and duration. Since estimation of parameters of these infiltration models is quite difficult, the following DFFD model has been developed. The components of this new model are

1) Bivariate exponential distribution of rainfall intensity and duration

2) SCS curve number method for infiltration and

3) KW theory as effective rainfall-runoff model.

The CDF of peak discharge Q_p is obtained by integration of joint PDF of i_e and t_e (derived using SCS model of infiltration) over regions where Q_p is less than or equal to a given value. These regions are shown in Fig. 3.9. Boundaries of the regions and the relationships of Q_p as a function of i_e, t_e and other catchment characteristics are given by (3.72)-(3.75). The CDF of Q_p is then

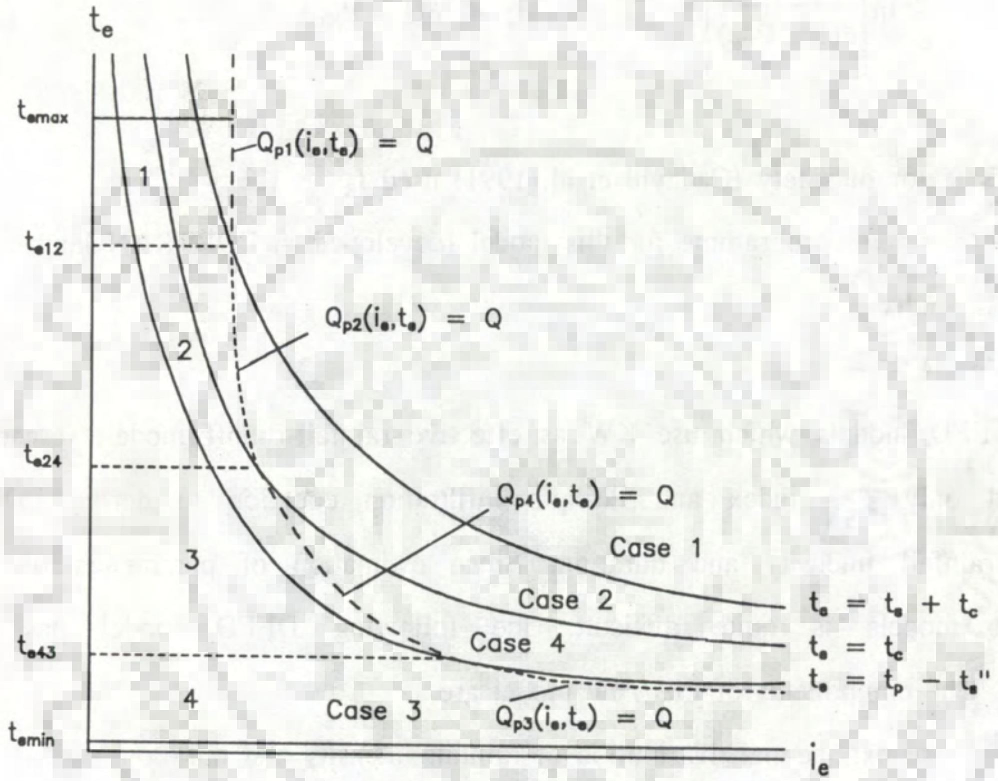


FIG. 3.9 – INTEGRATION REGIONS FOR COMPUTATION OF CDF OF PEAK DISCHARGE (FOR KW–SCS & KW–PHI MODELS).

computed as

$$F_{Q_p}(Q_p) = P_{NR} + \sum_{i=1}^{4} \int_{Ri} f_{I_e,T_e}(i_e,t_e)di_e dt_e \tag{3.98}$$

Integration of (3.57) in the direction of i_e, between a and b, a < b, yields

$$\int_a^b f_{I_e,T_e}(i_e,t_e)di_e = \int_a^b 0.77642\beta\delta\exp(-\delta t_e - \sigma)\Gamma(\sigma+1)\sigma^{-\sigma}\left(\frac{S}{t_e}\right)^{0.44161} .i_e^{-0.44161}\exp\left[-1.39047\beta\left(\frac{S}{t_e}\right)^{0.44161} i_e^{0.55839}\right]di_e \tag{3.99}$$

Substituting $A^* = 0.77642\beta\delta\exp(-\delta t_e - \sigma)\Gamma(\sigma+1)\sigma^{-\sigma}\left(\frac{S}{t_e}\right)^{0.44161}$

$$\int_a^b f_{I_e,T_e}(i_e,t_e)di_e = A^* \int_a^b i_e^{-0.44161} .\exp\left[-1.39047\beta\left(\frac{S}{t_e}\right)^{0.44161} i_e^{0.55839}\right]di_e \tag{3.100}$$

Substituting $k_1 = 0.44161$, $y = i_e^{1-k_1}$, and $B^* = 1.39047\beta\left(\frac{S}{t_e}\right)^{0.44161}$

and changing the limits

$$\int_a^b f_{I_e,T_e}(i_e,t_e)di_e = A^*\int_{a^{1-k_1}}^{b^{1-k_1}} \frac{1}{1-k_1} \exp(-B^*y)dy$$

$$= A^* \frac{1}{1-k_1}\left[\frac{\exp(-B^*a^{1-k_1}) - \exp(-B^*b^{1-k_1})}{B^*}\right] \qquad (3.101)$$

Substituting A^* and B^* in above equation we get

$$\int_a^b f_{I_e,T_e}(i_e,t_e)di_e = \delta\exp(-\sigma)\Gamma(\sigma+1)\sigma^{-\sigma}\exp(-\delta t_e)$$

$$\left\{\exp\left[-1.39047\beta\left(\frac{S}{t_e}\right)^{k_1} a^{1-k_1}\right] - \exp\left[-1.39047\beta\left(\frac{S}{t_e}\right)^{k_1} b^{1-k_1}\right]\right\}$$

$$= g(a,b,t_e) \qquad (3.102)$$

Using these results and as shown in Fig. 3.9, we get

$$F_{Q_p}(Q_p) = P_{NR} + \int_{t_{e12}}^{\infty} g(i_e=0,\ i_e \text{ given by } Q_{p1},\ t_e)dt_e$$
$$+ \int_{t_{e24}}^{t_{e12}} g(i_e=0,\ i_e \text{ given by } Q_{p2},\ t_e)dt_e$$
$$+ \int_{t_{e43}}^{t_{e24}} g(i_e=0,\ i_e \text{ given by } Q_{p4},\ t_e)dt_e$$
$$+ \int_{0}^{t_{e43}} g(i_e=0,\ i_e \text{ given by } Q_{p3},\ t_e)dt_e \qquad (3.103)$$

It may be pointed out that integration region has been covered by only four integrals as against seven used by Cadavid et al. (1991). This also avoids use of iterative methods to compute the conditions at the boundaries of different cases.

Iterative method is used only for solution of equations of peak discharge for Q_{p2} and Q_{p4}, as a result total computer time required for the programme has been reduced.

The return period for a given value of discharge Q_p is given by (Eagleson, 1972; Diaz-Granados et al., 1983)

$$T = \frac{1}{\left\{ m_\nu \left[1 - F_{Qp}(Q_p) \right] \right\}} \tag{3.104}$$

where m_ν is the average number of independent rainfall events per year.

The computer programme for this model (developed in FORTRAN language) is given in Appendix - 5.

CHAPTER 4

MODEL DEVELOPMENT
CORRELATED RAINFALL INTENSITY AND DURATION

4.1 INTRODUCTION

The DFFD models discussed in Chapter 3 consider the rainfall intensity and duration as independent of each other. These variables may be correlated also in some cases. The present chapter describes the development of new DFFD models which can take care of the correlation between these variables. These models use bivariate exponential rainfall model for the correlated intensity and duration, constant loss rate (ϕ - index) as infiltration model and GcIUH and KW theory as effective rainfall-runoff models.

4.2 STOCHASTIC RAINFALL MODEL

In this section a bivariate exponential distribution for rainfall intensity and duration has been described which considers the negative correlation between these random variables. This is followed by the derivation of P_{NR} and $f_{I_e,T_e}(i_e,t_e)$.

Gumbel (1960) studied the bivariate PDF of random variables which were negatively correlated. As reported by Bacchi et al. (1994) the joint PDF of intensity and duration can be written in the form of

$$f_{I_r,T_r}(i_r,t_r) = \beta\delta[(1 + \beta\gamma i_r)(1 + \delta\gamma t_r) - \gamma]\exp(-\beta i_r - \delta t_r - \beta\delta\gamma i_r t_r) \qquad (4.1)$$

where marginal PDFs of intensity and duration are exponential with parameters β and δ, representing inverses of mean intensity and mean duration of storm, respectively. Parameter γ in (4.1) describes correlation coefficient $\rho(i_r,t_r)$ between intensity and duration as defined by

$$\rho(i_r,t_r) = -1 + \int_0^\infty \frac{1}{1+\gamma x}\exp(-x)dx \tag{4.2}$$

Probability of null runoff (P_{NR})

Effective rainfall intensity and duration for a spatially averaged potential loss rate ϕ are given by

$$i_e = i_r - \phi \quad \text{and} \quad t_e = t_r \quad \text{if } i_r > \phi \tag{4.3a}$$

$$i_e = 0 \quad \text{and} \quad t_e = 0 \quad \text{if } i_r \le \phi \tag{4.3b}$$

When $i_r \le \phi$, no runoff is generated. In terms of distribution of i_e and t_e this situation is represented by a spike at $i_e=0$ and $t_e=0$. This value is the probability of null runoff (P_{NR}) and is given by

$$P(i_e=0,t_e=0) = \int_0^\infty\left[\int_0^\phi f_{I_r,T_r}(i_r,t_r)di_r\right]dt_r \tag{4.4}$$

Substituting $f_{I_r,T_r}(i_r,t_r)$ from (4.1) in (4.4) the inner integral I will be

$$I = \beta\delta(1-\gamma+\delta\gamma t_r)\exp(-\delta t_r)\int_0^\phi \exp[-(\beta+\beta\delta\gamma t_r)i_r]di_r$$
$$+\ \beta\delta(\beta\gamma+\beta\delta\gamma^2 t_r)\exp(-\delta t_r)\int_0^\phi i_r\exp[-(\beta+\beta\delta\gamma t_r)i_r]di_r$$

$$= \delta\exp(-\delta t_r) - \delta(1+\beta\gamma\phi)\exp[-\beta\phi-(\delta+\beta\delta\gamma\phi)t_r] \tag{4.5}$$

Integrating (4.5) from 0 to ∞ with respect to t_r gives

$$P(i_e=0,t_e=0) = 1-\exp(-\beta\phi) \tag{4.6}$$

Evaluation of $f_{I_e,T_e}(i_e,t_e)$

PDF of i_e and t_e can be derived using the technique of derived distribution as follows:

$$f_{I_e,T_e}(i_e,t_e) = f_{I_r,T_r}\left[g_1^{-1}(i_e,t_e),\ g_2^{-1}(i_e,t_e)\right]\left|\frac{\partial(i_r,t_r)}{\partial(i_e,t_e)}\right| \tag{4.7}$$

Using (4.1) and (4.3) in (4.7) we get

$$f_{I_e,T_e}(i_e,t_e) = \beta\delta\{[1 + \beta\gamma(i_e + \phi)](1 + \delta\gamma t_e) - \gamma\}$$
$$.\exp[-\beta(i_e + \phi) - \delta t_e - \beta\delta\gamma(i_e + \phi)t_e]$$

$$=\beta\delta[(1 + \beta\gamma\phi - \gamma) + (\beta\gamma i_e + \delta\gamma t_e + \beta\delta\gamma^2\phi t_e + \beta\delta\gamma^2 i_e t_e)]\exp(-\beta\phi)$$
$$.\exp(-\beta i_e - \delta t_e - \beta\delta\gamma\phi t_e - \beta\delta\gamma i_e t_e) \qquad i_e,\ t_e > 0 \tag{4.8}$$

Therefore, (4.6) and (4.8) completely define the distribution of i_e and t_e.

4.3 DERIVATION OF CUMULATIVE DISTRIBUTION FUNCTION OF PEAK DISCHARGE

For effective rainfall-runoff modelling two approaches based on geomorphoclimatic instantaneous unit hydrograph and kinematic wave theory have been used. The stochastic rainfall model discussed above has been used to derive the CDF of peak discharge. The details are given below.

4.3.1 GcIUH Based Model

Defining $t_e=t_e^*$ and $Q_p/A=Q_p^*$ Diaz-Granados et al. (1983) evaluated CDF of Q_p as

$$F_{Q_p}(Q_p) = P_{NR} + \int_0^{Q_p^*}\left[\int_0^{\infty} f_{I_e,T_e}(i_e,t_e)dt_e\right]di_e$$

$$+ \int_{Q_p^*}^{\infty}\left[\int_0^{t_e^*} f_{I_e,T_e}(i_e,t_e)dt_e\right]di_e \qquad (4.9)$$

Substituting (4.8) in (4.9) first (I_1) and second (I_2) integrals are evaluated to yield:

$$I_1 = \exp(-\beta\phi)[1 - \exp(-\beta Q_p^*)] \qquad (4.10)$$

$$I_2 = \exp(-\beta\phi)\exp(-\beta Q_p^*) - \beta\exp(-\beta\phi)\int_{Q_p^*}^{\infty}(1 + \delta\gamma t_e^*)$$

$$.\exp[-\beta i_e - (\delta + \beta\delta\gamma\phi + \beta\delta\gamma i_e)t_e^*]di_e \qquad (4.11)$$

Complete CDF of Q_p is given by adding P_{NR} to these integrals. Using (4.6), (4.10) and (4.11) we get

$$F_{Q_p}(Q_p) = 1 - \beta\exp(-\beta\phi)\int_{Q_p^*}^{\infty}(1 + \delta\gamma t_e^*)$$

$$.\exp[-\beta i_e - (\delta + \beta\delta\gamma\phi + \beta\delta\gamma i_e)t_e^*]di_e \qquad (4.12)$$

where t_e^* is given by (3.88).

It may be pointed out that when rainfall intensity and duration are independent of each other, γ equals zero in (4.12) as a result we get

$$F_{Q_p}(Q_p) = 1 - \beta\exp(-\beta\phi)\int_{Q_p^*}^{\infty} \exp(-\beta i_e - \delta t_e^*)di_e \qquad (4.13)$$

Equation (4.13) is the same as derived by Diaz-Granados et al. (1983) for the case of independent rainfall intensity and duration.

The computer programme for this model (developed in FORTRAN language) is given in Appendix - 1.

4.3.2 KW Based Model

Using the procedure described in section 3.4.5 the CDF of peak discharge can be written as

$$F_{Q_p}(Q_p) = P_{NR} + \sum_{i=1}^{4} \int_{R_i} f_{I_e,T_e}(i_e,t_e)di_e dt_e \tag{4.14}$$

substituting $f_{I_e,T_e}(i_e,t_e)$ from (4.8) and integrating in the direction of i_e, between i_{e1} and i_{e2}, $i_{e1} < i_{e2}$ yields

$$\int_{i_{e1}}^{i_{e2}} f_{I_e,T_e}(i_e,t_e)di_e = \int_{i_{e1}}^{i_{e2}} \{\beta\delta[(1 + \beta\gamma\phi - \gamma) + \delta\gamma t_e + \beta\delta\gamma^2\phi t_e]\exp(-\beta\phi)$$
$$+ \beta\delta\gamma(\beta + \beta\delta\gamma t_e)i_e\exp(-\beta\phi)\}\exp[-(\delta + \beta\delta\gamma\phi)t_e]$$
$$.\exp[-(\beta + \beta\delta\gamma t_e)i_e]di_e$$

$$= \delta\exp(-\beta\phi)\exp[-(\delta + \beta\delta\gamma\phi)t_e]\{(1 + \beta\gamma\phi + \beta\gamma i_{e1})$$
$$.\exp[-(\beta + \beta\delta\gamma t_e)i_{e1}] - (1 + \beta\gamma\phi + \beta\gamma i_{e2})$$
$$.\exp[-(\beta + \beta\delta\gamma t_e)i_{e2}]\}$$

$$= g(i_{e1}, i_{e2}, t_e) \tag{4.15}$$

Using these results and as shown in Fig. 3.9 we get

$$F_{Q_p}(Q_p) = P_{NR} + \int_{t_{e12}}^{t_{emax}} g(i_e=0,\ i_e \text{ given by } Q_{p1},\ t_e)dt_e$$
$$+ \int_{t_{e24}}^{t_{e12}} g(i_e=0,\ i_e \text{ given by } Q_{p2},\ t_e)dt_e$$

$$+ \int_{t_{e43}}^{t_{e24}} g(i_e=0,\ i_e \text{ given by } Q_{p4},\ t_e)dt_e$$

$$+ \int_{t_{emin}}^{t_{e\,4\,3}} g(i_e=0,\ i_e \text{ given by } Q_{p3},\ t_e)dt_e \qquad (4.16)$$

The return periods can be computed using (3.104).

The computer programme for this model (developed in FORTRAN language) is given in Appendix - 6.

CHAPTER 5

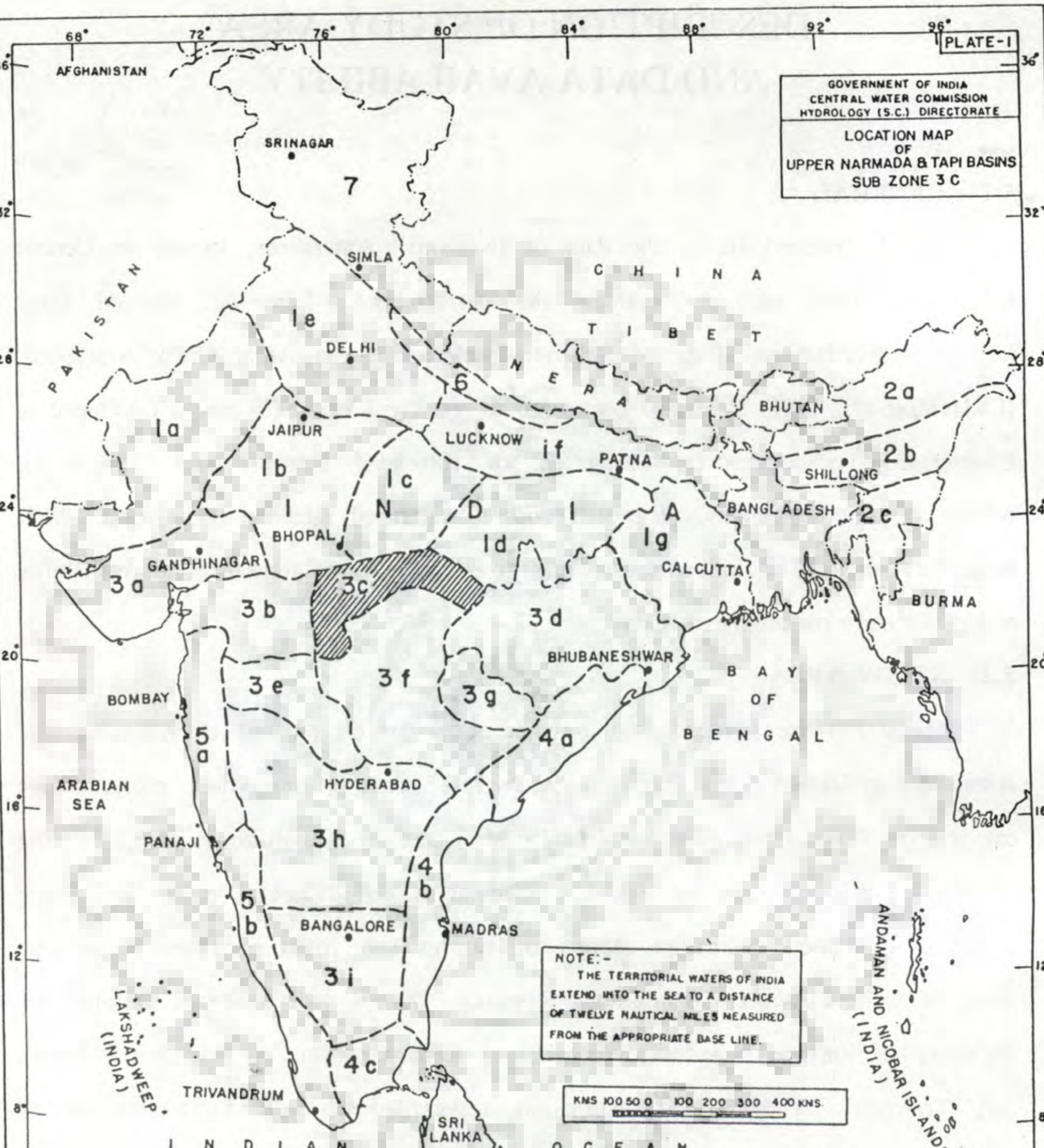
DESCRIPTION OF STUDY AREA AND DATA AVAILABILITY

5.1 GENERAL

In the present study, the data of five small watersheds, located in Central India have been used to evaluate the performance of various derived flood frequency distributions. Data of Ralston Creek and Santa Anita Creek watersheds (Cadavid et al., 1991) have also been used to test the KW - SCS model developed in Chapter 3. To demonstrate the effect of correlated intensity and duration of rainfall on flood quantiles, data of Davidson watershed (Hebson and Wood, 1982) have been used. The following sections give details of watersheds and availability and preliminary processing of data.

5.2 STUDY AREA

India has been divided into 7 major zones for the purpose of systematic and sustained collection of hydro-meteorological data from the representative catchments. These zones have been further divided into 20 sub zones (Fig.5.1). The hydrometeorological data of selected catchments in different zones are being observed and flood estimation reports of various sub zones published as a joint work of Central Water Commission, Research, Design and Standards Organization (Ministry of Railways), India Meteorolcgical Department and Ministry of Shipping and Transport. These flood estimation reports give detailed information on location and hydro-meteorological parameters of sub zones and selected watersheds. Data of five watersheds of sub zone - 3c which cover part of Upper Narmada and Tapi river basins have been used for this study. The details are presented in section 5.3. Data of Ralston Creek, Santa Anita Creek and Davidson watersheds are



THE EXTERNAL BOUNDARY AND COAST LINE OF INDIA ON THE MAPS AGREE WITH THE RECORD COPY CERTIFIED BY THE SURVEY OF INDIA.

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FIG. 5.1 MAP SHOWING DIFFERENT SUB ZONES OF INDIA (REPRODUCED FROM FLOOD ESTIMATION REPORT, 1983) given in section 5.4 under the head other watersheds.

5.3 SUB ZONE-3C WATERSHEDS

The sub zone - 3c is located in Central India and lies between 76°12' to 81°45'E longitude and 20°10' to 23°45'N latitude (Fig.5.1). It occupies about 86353 km^2 area in the States of Madhya Pradesh and Maharashtra. The main soil group of the sub zone is vertisols (Black Soils). The area is mostly cultivated (55 per cent) or under forest (40 per cent). Remaining area covers grassland and wastelands. Annual rainfall of the sub zone - 3c varies from 800 to 1600 mm.

In sub zone - 3c, hydrometeorological data of 18 watersheds have been observed. Out of these 18 watersheds, 5 watersheds have been selected for the present study. These watersheds have varied characteristics. One watershed is under forest while another has only cultivated area. Remaining three have about 50 per cent cultivated area. Annual rainfall of the five watersheds ranges from 800 to 1400 mm and the catchment area varies from 42.7 to 178.07 km^2. All these watersheds mainly have black cotton soil. The locations of these watersheds are shown in Fig.5.2 (Flood Estimation Report, 1983).

The rainfall data were available at two or more stations in each watershed. The rainfall and runoff data, obtained in raw form, were processed before analysis.

Details of the watersheds of sub zone - 3c selected for the present study are given in following sections.

5.3.1 Tairhia Watershed

Watershed Details

Tairhia watershed (Fig. 5.3) is the part of area drained by river Tairhia - a tributary of river Narmada. The discharge gauging station at bridge No. 253 of Gondia - Jabalpur section of South Eastern railway is located at 79°50'08" E longitude and 22°52'36" N latitude. The watershed area as measured from toposheets

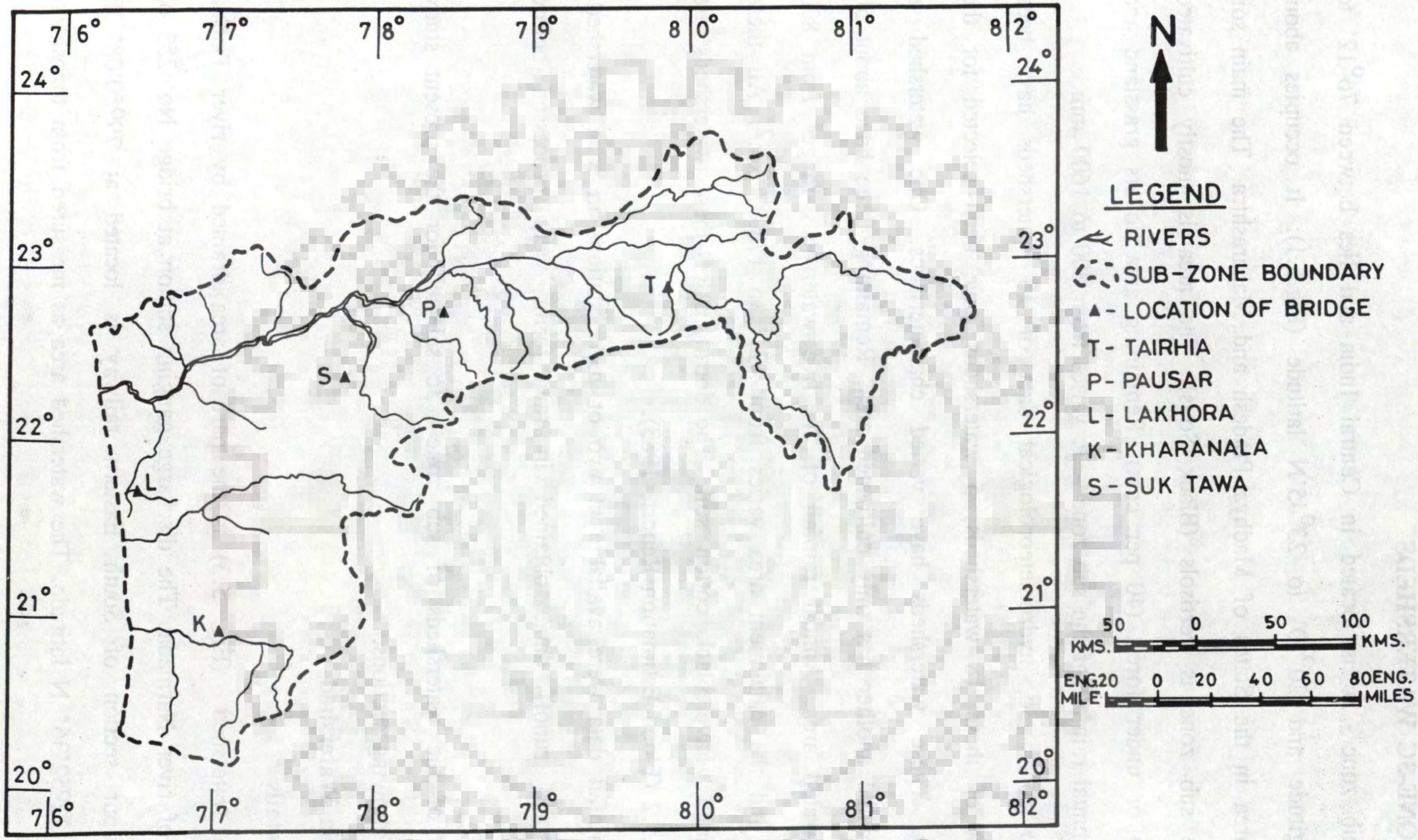

FIG. 5·2. LOCATIONS OF WATERSHEDS UNDER STUDY IN SUB ZONE-3C

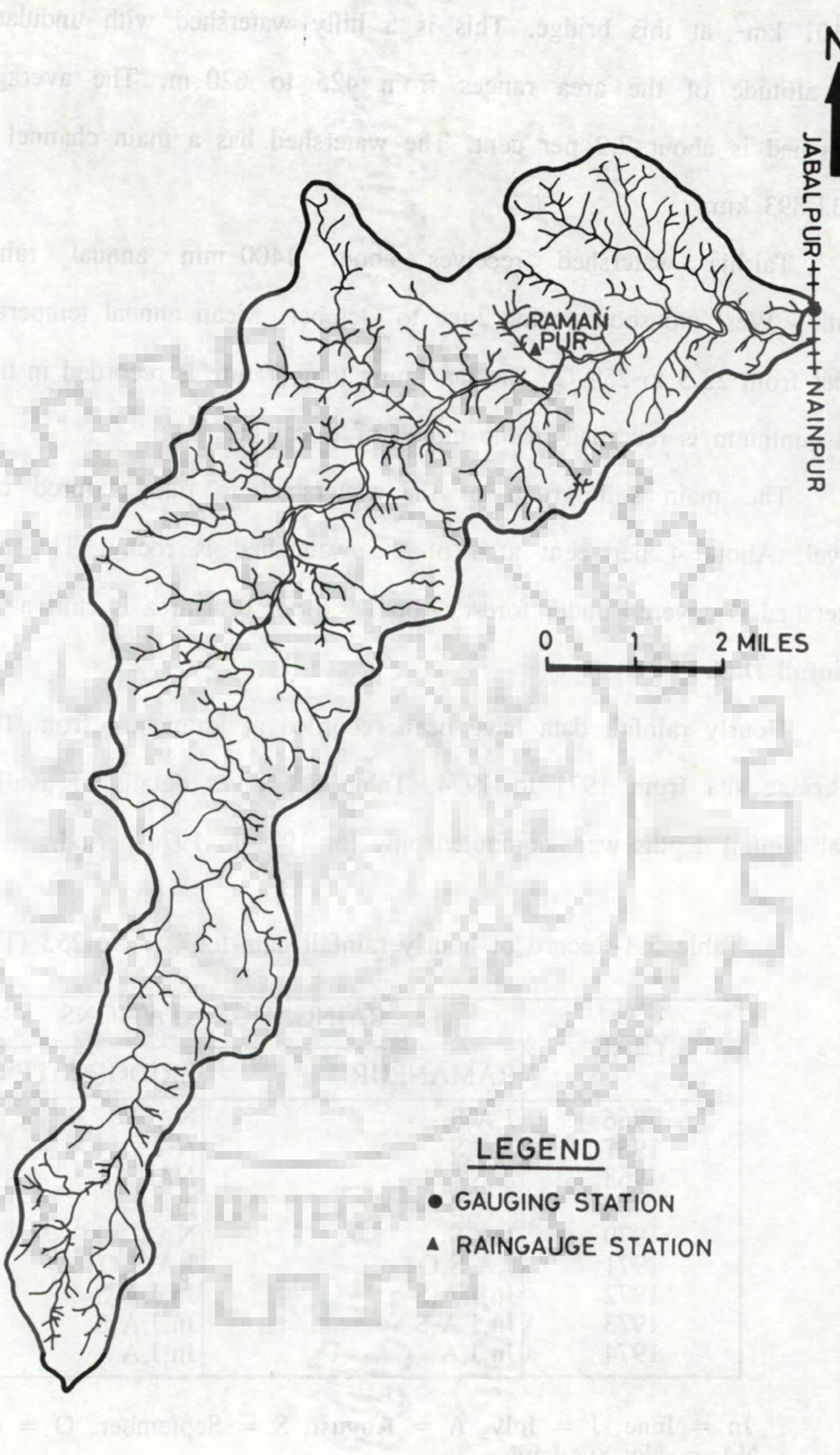

FIG. 5·3. DRAINAGE MAP OF TAIRHIA WATERSHED is 101 km^2 at this bridge. This is a hilly watershed with undulating topography. The altitude of the area ranges from 425 to 620 m. The average slope of the watershed is about 7.2 per cent. The watershed has a main channel having a length of 32.893 km.

Tairhia watershed receives about 1400 mm annual rainfall from the South - West monsoon during June to October. Mean annual temperature of the area varies from 22.5 to 25^{o} C. The maximum temperature is recorded in the month of May and minimum is recorded in the month of December.

The main soil group of the watershed is fine textured clay mixed with gravel. About 4 per cent area of the watershed is rocky. The most part of the watershed is covered under forest. About 9.6 per cent area is cultivated.

Rainfall Data

Hourly rainfall data have been recorded at Ramanpur from 1966 to 1974 and at bridge site from 1971 to 1974. Table 5.1 gives details of available data. Mean areal rainfall depths were computed only for 1971 to 1974 period.

Table 5.1-Record of hourly rainfall data for Br. No.253 (Tairhia)

YEAR	RAINGAUGE STATIONS	
	RAMANPUR	BRIDGE SITE
1966	J,A,S	NA
1967	J,A,S	NA
1968	A,S	NA
1969	J,A,S	NA
1970	J,A,S	NA
1971	J,A,S,O	J,A,S,O
1972	Jn,J,A,S	Jn,J,A,S
1973	Jn,J,A,S	Jn,J,A,S
1974	Jn,J,A	Jn,J,A

Jn = June, J = July, A = August, S = September, O = October
NA = Not available

Discharge Data

Hourly stage data were recorded at the outlet of the watershed round the clock. Velocity measurements were done frequently during day time using current meter or float. The data for the period 1966 to 1974 were processed and used in the analysis. Annual flood series at the bridge site from 1966 to 1989 (RDSO, 1991) is given in Table 5.2. Statistical parameters of original and log transformed series are given in Table 5.3.

Table 5.2-Annual flood peaks of Tairhia watershed (Area=101 km^2)

Year	Discharge(cumec)	Year	Discharge(cumec)
1966	64	1978	285
1967	189	1979	150
1968	37	1980	NG
1969	200	1981	54
1970	197	1982	139
1971	151	1983	400
1972	266	1984	331
1973	606	1985	118
1974	212	1986	NG
1975	433	1987	NG
1976	70	1988	NG
1977	253	1989	315

NG-Not gauged

Table 5.3 - Statistical parameters of original and log transformed series of Tairhia watershed

Statistical Parameter	Original Series	$\log_e$ transformed series
Mean (μ)	223.5	5.186
Standard deviation (σ)	143.6	0.737
Coefficient of skewness (C_s)	1.014	-0.578
Coefficient of kurtosis (C_k)	4.531	3.200
Lag-1 correlation coeff.(r_1)	-0.009	

5.3.2 Pausar Watershed

Watershed Details

Pausar watershed (Fig.5.4) is the part of upper catchment of Pausar river - a tributary of river Narmada. The discharge gauging station at bridge No. 505 of Itarasi - Allahabad section of Central railway is located at 78°21'56" E longitude and 22°45'25" N latitude. The drainage area of the watershed is 67.37 km^2. The watershed is leaf shaped. The altitude of the area ranges from 300 to 600 m. The average slope of the watershed is about 3.04 per cent. The main drainage channel of the watershed is 24.046 km long.

The average annual rainfall of the watershed is 1300 mm. This rainfall is received from June to October during South-West monsoon. The mean annual temperature of the watershed area varies from 22.5 to 25° C. The maximum temperature is recorded in the month of May and minimum is recorded in the month of December.

The main soil group of the watershed is fine textured deep clay. The upper portion of the watershed (40 per cent) is covered under forest and remaining 60 per cent area is under cultivation.

Rainfall Data

Hourly rainfall data have been recorded at four stations. Details of raingauge stations and years of record are given in Table 5.4. Areal rainfall depths were computed for the years 1965 and 1967 using data of raingauge stations at Raikheri, Panari and bridge site. For the years 1968 and 1969, data of Raikheri and Bori raingauge stations were used.

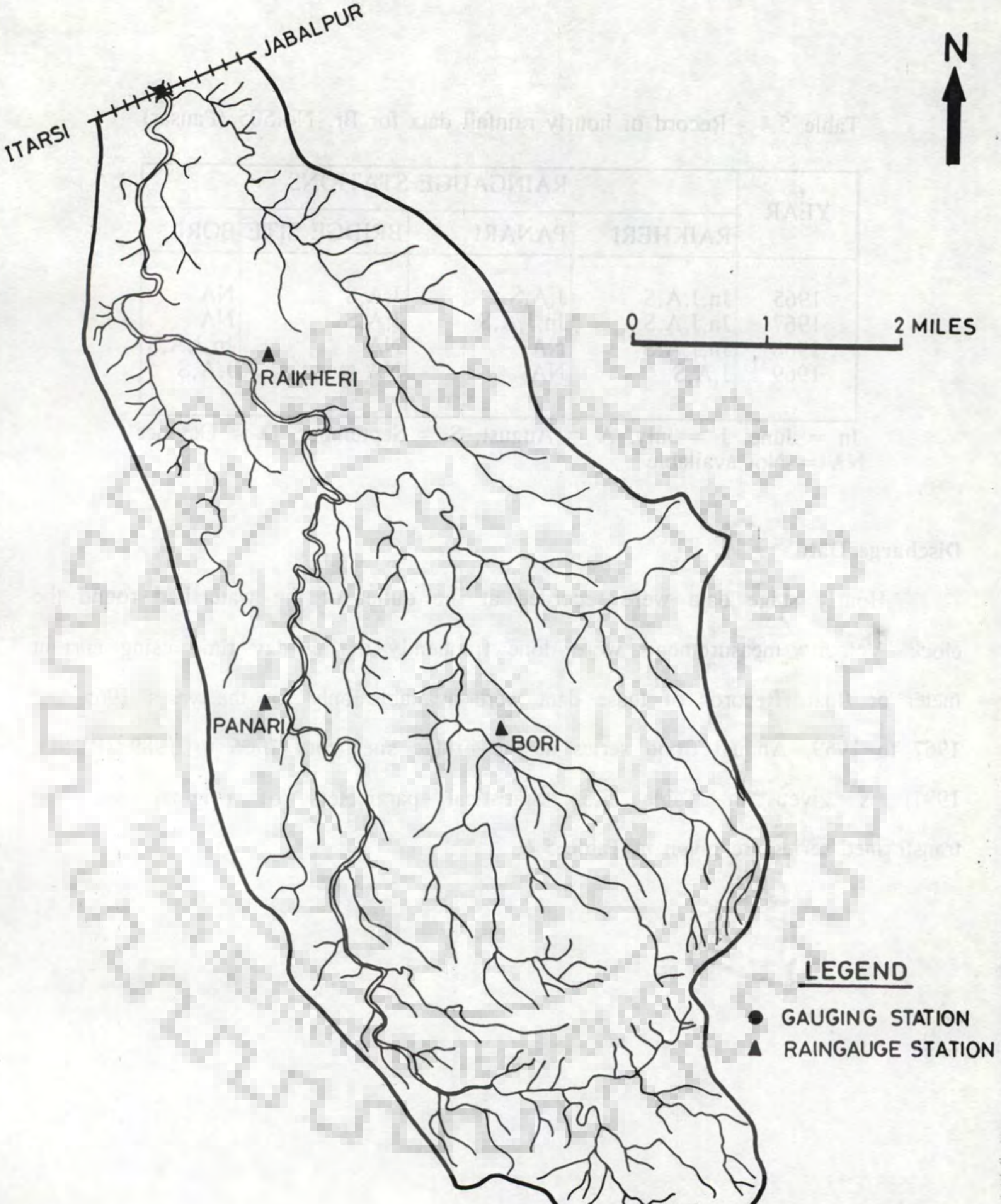

FIG. 5·4. DRAINAGE MAP OF PAUSAR WATERSHED

Table 5.4 - Record of hourly rainfall data for Br. No.505 (Pausar)

YEAR	RAINGAUGE STATIONS			
	RAIKHERI	PANARI	BRIDGE SITE	BORI
1965	Jn,J,A,S	J,A,S	J,A,S	NA
1967	Jn,J,A,S	Jn,J,A,S	J,A,S	NA
1968	Jn,J,A,S	NA	NA	Jn,J,A,S
1969	J,A,S	NA	NA	J,A,S

Jn = June, J = July, A = August, S = September, O = October
NA = Not available

Discharge Data

Hourly stage data were recorded at the outlet of the watershed round the clock. Velocity measurements were done frequently during day time using current meter or float. Records of these data were available only for the years 1965 and 1967 to 1969. Annual flood series at the bridge site from 1966 to 1989 (RDSO, 1991) is given in Table 5.5. Statistical parameters of original and log transformed series are given in Table 5.6.

Table 5.5 - Annual flood peaks of Pausar watershed (Area=67.37 km^2)

Year	Discharge(cumec)	Year	Discharge(cumec)
1965	145	1978	330
1966	360	1979	120
1967	240	1980	100
1968	265	1981	98
1969	227	1982	105
1970	370	1983	182
1971	172	1984	310
1972	172	1985	410
1973	172	1986	200
1974	342	1987	49
1975	390	1988	NG
1976	235	1989	78
1977	38		

NG-Not gauged

Table 5.6 - Statistical parameters of original and log transformed series of Pausar watershed

Statistical Parameter	Original Series	$\log_e$ transformed series
Mean (μ)	212.9	5.189
Standard deviation (σ)	112.9	0.651
Coefficient of skewness (C_s)	0.242	-0.791
Coefficient of kurtosis (C_k)	2.236	3.393
Lag-1 correlation coeff.(r_1)	0.207	

5.3.3 Lakhora Watershed

Watershed Details

Lakhora watershed (Fig. 5.5) is located in the western portion of sub zone - 3c. This watershed is a part of area drained by river Lakhora - a tributary of river Narmada. The gauging of runoff is done at bridge No. 584 of Khandwa - Akola section of South Central railway. Location of this bridge is at $76^{o}27'15''$ E longitude and $21^{o}44'10''$ N latitude. The area of Lahkora watershed is 151.35 km^2. The watershed has an average slope of about 2.05 per cent, main channel length of 27.6 km and altitude of the watershed area varies from 300 to 400 m.

The average annual rainfall of Lakhora watershed is 900 mm. This amount is received from June to October during South - West monsoon. The mean annual temperature varies from 25 to 27.5^{o} C. May is the hottest month of the year and December is the coldest month of the year.

The main soil group of the watershed is fine textured clay having medium depth. About 67 per cent area of the watershed is under cultivation and 33 per cent under reserve forest.

Rainfall Data

Hourly rainfall data recorded at Gandhawa, Kumta and bridge site are available from 1966 to 1973 (Table 5.7). Areal rainfall depths were computed for the years 1966, 1967 1972 and 1973 using data from above three stations. For the years 1968 - 70 areal rainfall was computed using data of Gandhwa and Kumta stations only.

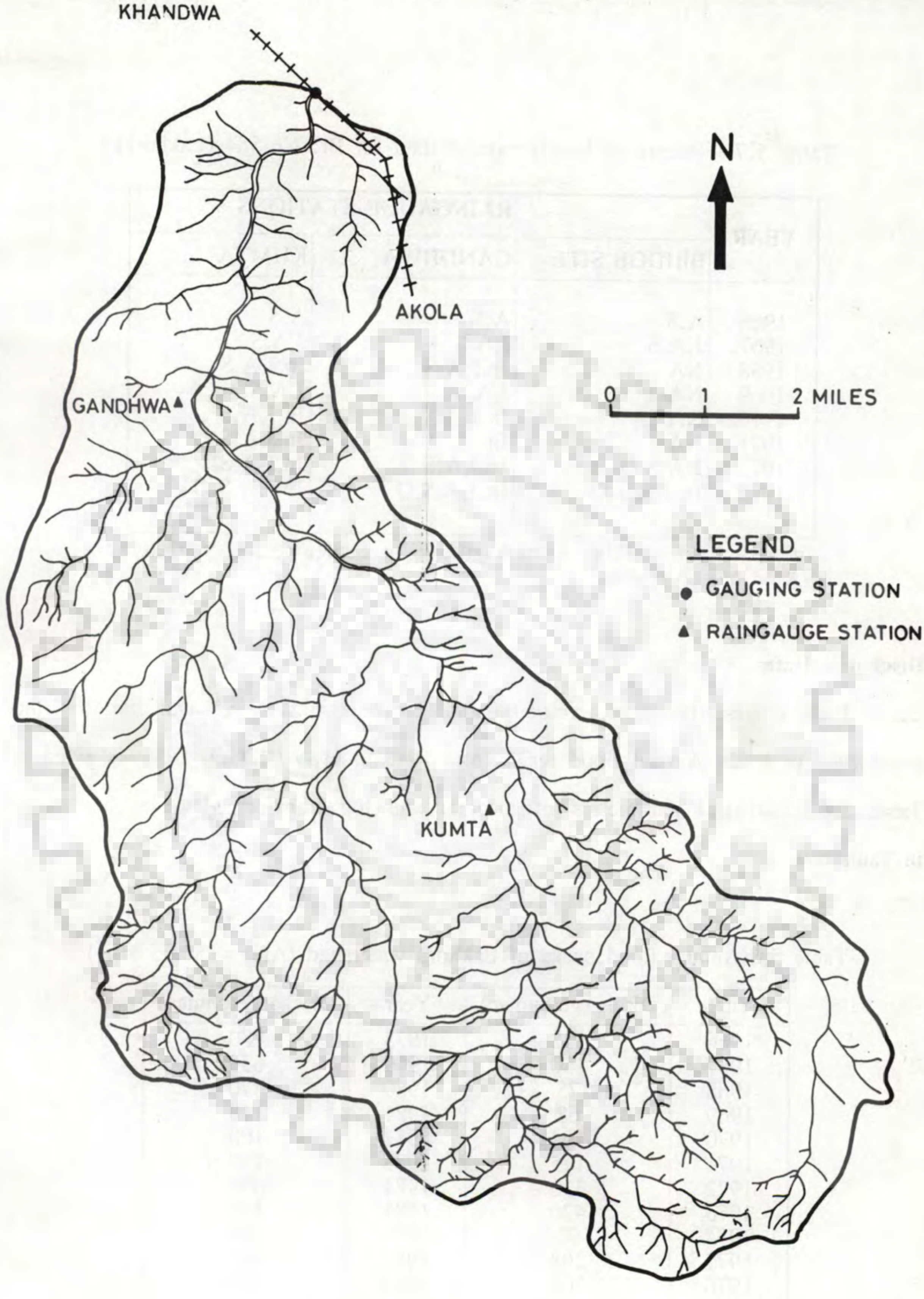

FIG. 5·5. DRAINAGE MAP OF LAKHORA WATERSHED

Table 5.7 - Record of hourly rainfall data for Br. No.584 (Lakhora)

YEAR	RAINGAUGE STATIONS		
	BRIDGE SITE	GANDHWA	KUMTA
1966	A,S	A,S	A,S
1967	J,A,S	J,A,S	J,A,S
1968	NA	Jn,J,A,S	Jn,J,A,S
1969	NA	J,A,S	J,A,S
1970	NA	Jn,J,A,S	Jn,J,A,S
1971	NA	Jn	Jn,J,A,S,O
1972	J,A,S,O	Jn,J,A,S	Jn,J,A,S
1973	Jn,J,A,S,O	Jn,J,A,S,O	Jn,J,A,S,O

Jn = June, J = July, A = August, S = September, O = October
NA = Not available

Discharge Data

Data of hourly stage, velocity and cross-sections were available for the years 1966 to 1973. Annual flood series from 1966 to 1989 (RDSO, 1991) is given in Table 5.8. Statistical parameters of original and log transformed series are given in Table 5.9.

Table 5.8-Annual flood peaks of Lakhora watershed (Area=151.35 km^2)

Year	Discharge(cumec)	Year	Discharge(cumec)
1966	122	1978	700
1967	150	1979	630
1968	75	1980	470
1969	68	1981	110
1970	194	1982	100
1971	175	1983	130
1972	330	1984	698
1973	420	1985	170
1974	92	1986	168
1975	298	1987	140
1976	108	1988	190
1977	21	1989	165

Table 5.9 - Statistical parameters of original and log transformed series of Lakhora watershed

Statistical Parameter	Original Series	$\log_e$ transformed series
Mean (μ)	238.5	5.166
Standard deviation (σ)	199.9	0.821
Coefficient of skewness (C_s)	1.418	-0.201
Coefficient of kurtosis (C_k)	4.180	4.028
Lag-1 correlation coeff.(r_1)	0.189	

5.3.4 Kharanala Watershed

Watershed Details

Kharanala watershed (Fig.5.6) is a part of area drained by Kharanala - a tributary of river Tapi. The discharge gauging is done at bridge No. 710 of Khandwa - Akola section of South Central railway located at 77°02'20" E longitude and 20°59'25" N latitude. This is the smallest watershed out of five test watersheds. The watershed has an area of 42.7 km^2. It is an elongated watershed with 23.657 km long main drainage channel. The altitude of the area ranges from 275 to 370 m. The average slope of the watershed is 0.56 per cent.

The average annual rainfall of the watershed is 800 mm. Rainy season is limited to the South - West monsoon period (June - October). The mean annual temperature of the watershed area varies from 25 to 27.5° C. The maximum temperature is recorded in the month of May and minimum is recorded in the month of December.

The main soil group of the watershed is fine textured deep clay. The area is suitable for cultivation, therefore, complete watershed is under agriculture land use.

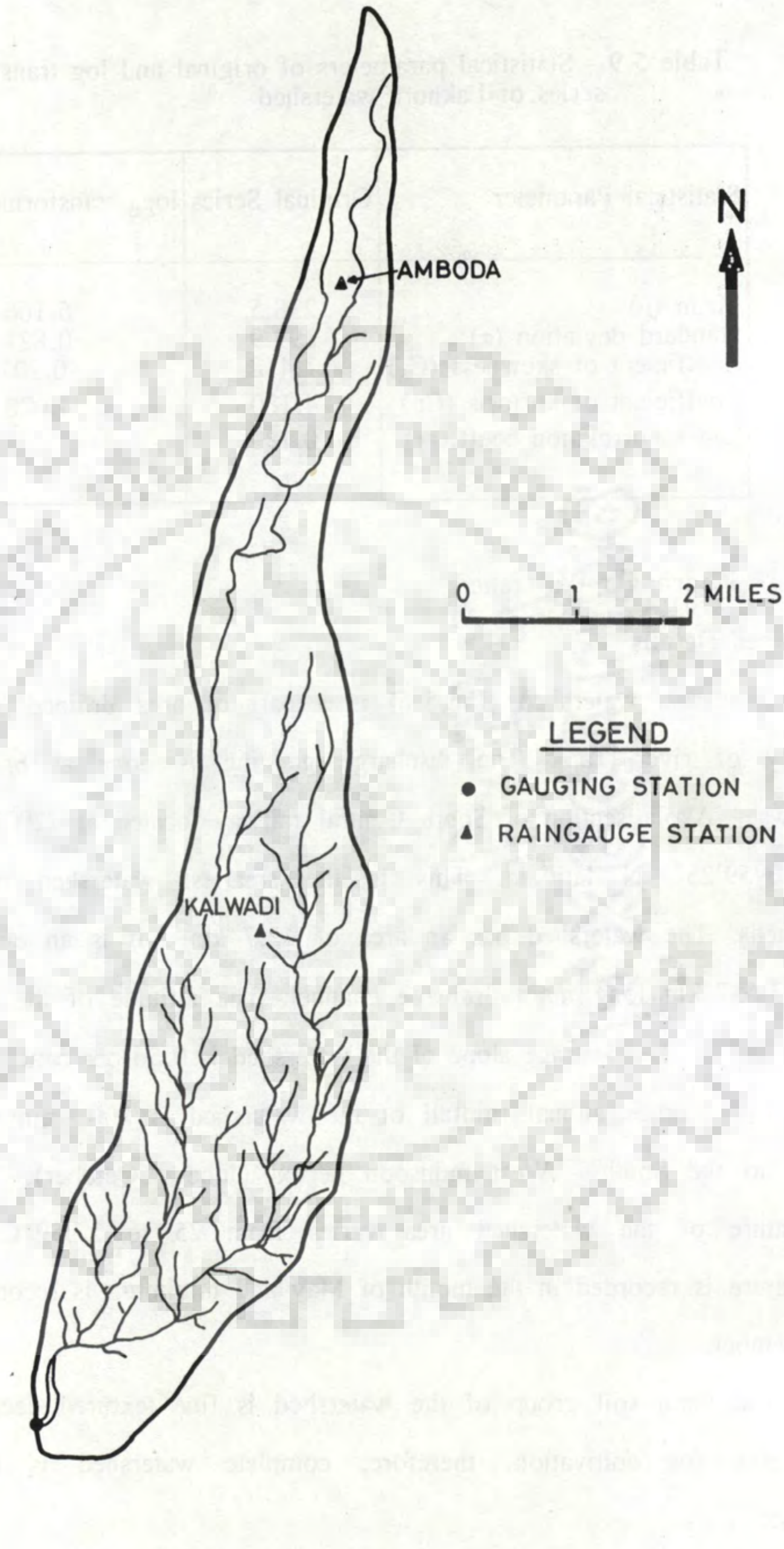

FIG. 5·6. DRAINAGE MAP OF KHARANALA WATERSHED

Rainfall Data

Hourly rainfall data recorded at Kalwadi, Amboda and bridge site are available from 1968 to 1973 (Table 5.10). Areal rainfall depths were computed for the year 1971 using data of Kalwadi and Amboda stations. For the years 1972 and 1973 data of all the three stations were used.

Table 5.10 - Record of hourly rainfall data for Br. No.710 (Kharanala)

YEAR	RAINGAUGE STATIONS		
	KALWADI	AMBODA	BRIDGE SITE
1968	J,A,S	NA	NA
1969	J,A,S	NA	NA
1970	Jn,J,A,S,O	S	NA
1971	Jn,J,A,S,O	Jn,J,A,S,O	NA
1972	J,A,S	J,A,S,O	J,A,S
1973	Jn,J,A,S	Jn,J,A,S	Jn,J,A,S

Jn = June, J = July, A = August, S = September, O = October
NA = Not available

Discharge Data

Data of hourly stage, velocity and cross-sections were available for the years 1968 to 1973. Annual flood series from 1968 - 1989 was available (RDSO, 1991) and is given in Table 5.11. Gauging was not done for the year 1981. Statistical parameters of original and log transformed series are given in Table 5.12.

Table 5.11 - Annual flood peaks of Kharanala watershed (Area=42.7 km^2)

Year	Discharge(cumec)	Year	Discharge(cumec)
1968	160	1979	200
1969	11	1980	100
1970	170	1981	NG
1971	10	1982	107
1972	283	1983	380
1973	125	1984	110
1974	5	1985	32
1975	17	1986	390
1976	29	1987	11
1977	165	1988	270
1978	120	1989	250

NG-Not gauged

Table 5.12 - Statistical parameters of original and log transformed series of Kharanala watershed

Statistical Parameter	Original Series	$\log_e$ transformed series
Mean (μ)	140.2	4.345
Standard deviation (σ)	119.8	1.350
Coefficient of skewness (C_s)	0.720	-0.725
Coefficient of kurtosis (C_k)	3.060	2.554
Lag-1 correlation coeff.(r_1)	-0.278	

5.3.5 Suk Tawa Watershed

Watershed Details

Suk Tawa watershed (Fig.5.7) is the part of area drained by Suk Tawa river - a tributary of river Narmada. The runoff gauging station at bridge No. 776 of Itarasi - Amla section of Central railway is located at 77°49'10" E longitude and 22°24'22" N latitude. Suk Tawa watershed has an area of 179.07 km^2. The altitude of the area ranges from 360 to 600 m. Average slope of the watershed is about 6.4 per cent. The length of the main channel is 23.848 km.

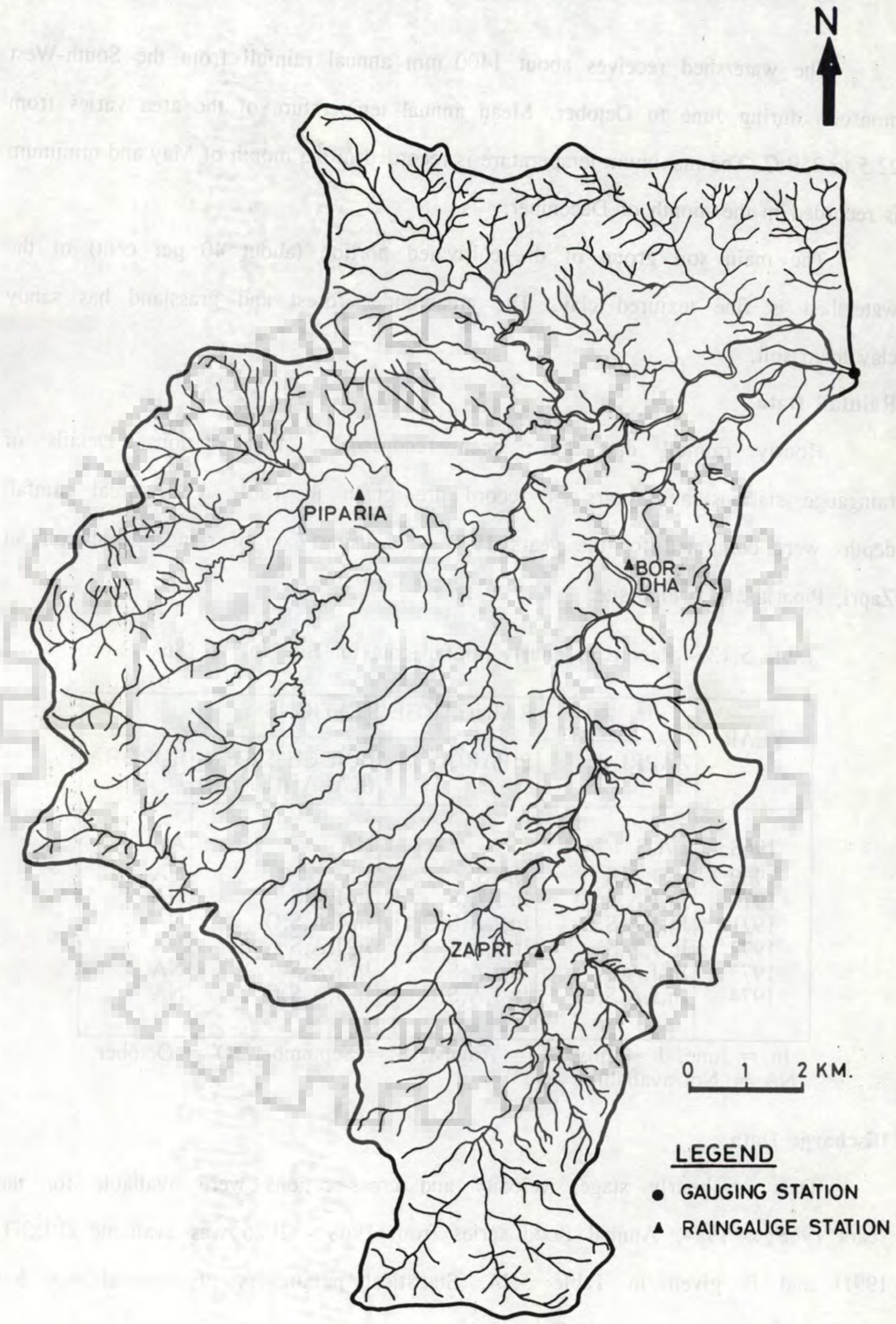

FIG. 5·7. DRAINAGE MAP OF SUK TAWA WATERSHED

The watershed receives about 1400 mm annual rainfall from the South-West monsoon during June to October. Mean annual temperature of the area varies from 22.5 to 25^{o} C. The maximum temperature is recorded in the month of May and minimum is recorded in the month of December.

The main soil group of the cultivated portion (about 40 per cent) of the watershed is fine textured clay. The area under forest and grassland has sandy clay loam soil.

Rainfall Data

Hourly rainfall data have been recorded at four stations. Details of raingauge stations and years of record are given in Table 5.13. areal rainfall depths were computed for the years 1970 - 74 using data of raingauge stations at Zapri, Piparia and bridge site.

Table 5.13 - Record of hourly rainfall data for Br. No.776 (Suk Tawa)

YEAR	RAINGAUGE STATIONS			
	ZAPRI	PIPARIA	BRIDGE SITE (KALA AKHAR)	BORDHA
1968	J,A,S	J	NA	A
1969	Jn,J,A,S	J,A,S	J,S	NA
1970	Jn,J,A,S	Jn,J,A,S	Jn,J,A,S	NA
1971	Jn,J,A,S.O	Jn,J,A,S.O	Jn,J,A,S,O	NA
1972	Jn,J,A,S	Jn,J,A,S	Jn,J,A,S	NA
1973	Jn,J,A,S	Jn,J,A,S	Jn,J,A,S	NA
1974	Jn,J,A,S,O	Jn,J,A,S,O	Jn,J,A,S,O	NA

Jn = June, J = July, A = August, S = September, O = October
NA = Not available

Discharge Data

Data of hourly stage, velocity and cross-sections were available for the years 1968 to 1974. Annual flood series from 1968 - 1986 was available (RDSO, 1991) and is given in Table 5.14. Statistical parameters of original and log transformed series are given in Table 5.15.

Table 5.14 - Annual flood peaks of Suk Tawa watershed (Area = 178.07 km^2)

Year	Discharge(cumec)	Year	Discharge(cumec)
1968	557	1978	380
1969	535	1979	240
1970	399	1980	180
1971	724	1981	400
1972	475	1982	750
1973	740	1983	1250
1974	725	1984	600
1975	860	1985	280
1976	1000	1986	320
1977	450		

Table 5.15 - Statistical parameters of original and log transformed series of Suk Tawa watershed

Statistical Parameter	Original Series	$\log_e$ transformed series
Mean (μ)	571.8	6.235
Standard deviation (σ)	275.7	0.503
Coefficient of skewness (C_s)	0.813	-0.279
Coefficient of kurtosis (C_k)	3.898	3.105
Lag-1 correlation coeff.(r_1)	0.412	

A summary of five test watersheds of sub zone - 3c is given in Table 5.16.

Table 5.16 - Details of watersheds and their soils and land use.

Description	Br.No.253	Br.No.505.	Br.No.584	Br.No.710	Br.No.776
Stream	Tairhia	Pausar	Lakhora	Kharanala	Suk Tawa
Longitude (E)	79°50’08"	78°21’56"	76°27’15"	77°02’20"	77°49’10"
Latitude (N)	22°52’36"	22°45’25"	21°44’10"	20°59’25"	22°24’22"
Area (km^2)	101.0	67.37	151.35	42.70	178.07
Stream order	5	5	6	4	5
Soils	Clay with gravel-96.0% Rocky-4.0%	Clay-100%	Clay-100%	Clay-100%	Clay-40.7% scl-59.3%
Land use	C - 9.6% F - 86.5% B - 3.9%	C - 60% F - 40%	C - 67% F - 33%	C - 100%	C - 40.7% F - 27.2% G - 32.1%

C - Cultivated, B - Barren land, F - Reserve Forest, G - Grassland

5.4 OTHER WATERSHEDS

For the purpose of comparison KW-SCS model has been applied on Ralston Creek and Santa Anita watersheds. Data of Davidson catchment have been used to apply DFFD model which accounts for correlation between intensity and duration. Details of these watersheds are presented below.

5.4.1 Ralston Creek Watershed

The gauging station of Ralston Creek watershed (Fig.5.8) is located within the Iowa City urban perimeter. The gauging station (U.S.Geological Survey station number 5-4550) measures the discharge in Ralston Creek. The watershed area is 3.01 sq. mi. The historical annual flood series is available for the period 1938 - 1965 ("Surface Water" 1971 - U.S. Geological Survey).

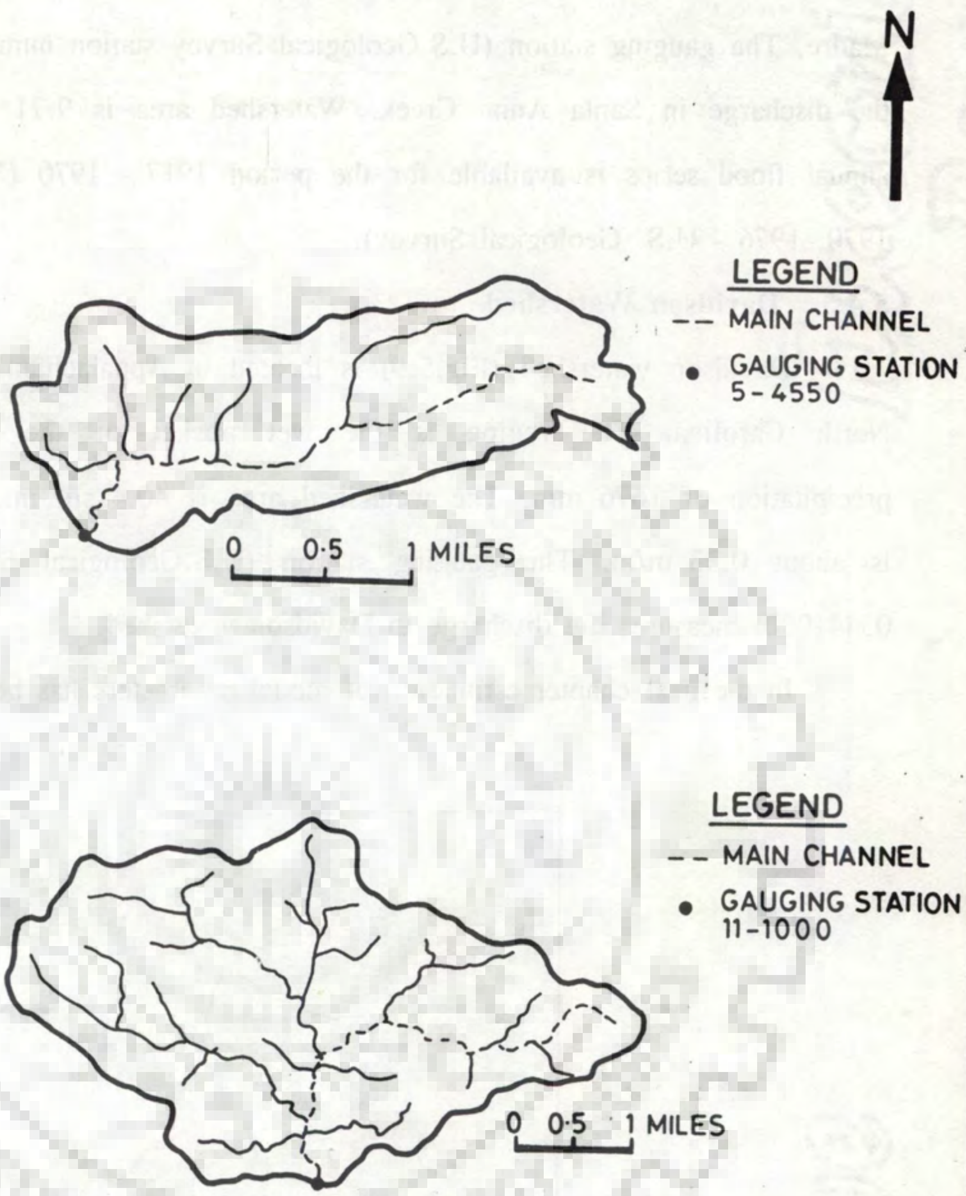

FIG. 5·8. MAPS OF RALSTON CREEK WATERSHED IN IOWA AND SANTA ANITA CREEK WATERSHED IN CALIFORNIA

5.4.2 Santa Anita Creek Watershed

Santa Anita Creek watershed (Fig.5.8) is in California, near the Sierra Madre. The gauging station (U.S.Geological Survey station number 11-1000) measures the discharge in Santa Anita Creek. Watershed area is 9.71 sq. mi. The historical annual flood series is available for the period 1917 - 1970 ("Surface Water" 1965, 1970, 1976 - U.S. Geological Survey).

5.4.3 Davidson Watershed

Davidson watershed (Fig.5.9) is located in Appalachian Mountains of western North Carolina. The region is the wet pocket of the State with a yearly precipitation of 1676 mm. The watershed area is 40.4 sq. mi. The catchment slope is about 0.33 m/m. The gauging station (U.S.Geological Survey station number 03441000) measures the discharge in Davidson watershed.

In the next chapter estimation of model parameters has been discussed.

FIG.5.9. DRAINAGE MAP OF DAVIDSON RIVER CATCHMENT

CHAPTER 6

ESTIMATION OF PARAMETERS FOR COMPONENT MODELS

6.1 INTRODUCTION

The methodology developed in Chapter 3 and 4 was applied to five small watersheds located in Central India. For comparison, data of watersheds reported by Cadavid et al. (1991) and Hebson and Wood (1982) were also used. The parameters of stochastic rainfall model, infiltration models and effective rainfall-runoff models play an important role in derived flood frequency distributions. The present chapter gives the details of procedures used for the estimation of various parameters. Estimated parameters of different component models for five test watersheds and parameters of Ralston Creek, Santa Anita Creek and Davidson watersheds are also given in this chapter.

6.2 STOCHASTIC RAINFALL MODEL

Stochastic rainfall model is one of the major components of DFFD models. The joint distribution of areal rainfall intensity and duration has been used as stochastic rainfall model for different DFFD models. The details of areal rainfall computation, estimation of model parameters, characteristics of the rainfall to be modelled, criteria for separation of independent storms and reasonableness of assumed distributions are presented in this section.

6.2.1 Mean Areal Rainfall Computation

Most of the DFFD models have been applied on small watersheds where point rainfall data of one station (within the watershed or nearest raingauge station) were used for estimation of mean point rainfall intensity. Area reduction factors were then used to convert this point rainfall intensity into mean areal rainfall intensity. The areal rainfall durations were assumed to be equal to point rainfall durations.

As reported in Chapter 5 hourly rainfall data were available for two or more stations of the five watersheds selected for the study. For these watersheds areal rainfall depths were computed using Thiessen polygon method. These rainfall depths were then used for further analysis. The Thiessen weights for different test watersheds are presented in Table 6.1.

6.2.2 Stochastic Rainfall Model Parameters

There are three parameters to be estimated using rainfall data i.e. number of independent events per year m_ν, inverse of mean areal rainfall intensity β and inverse of mean storm duration δ. The parameters β and δ are used to represent joint distribution of areal rainfall intensity and duration. The parameter m_ν is used for computing the return periods of various peak discharges.

As discussed in Chapter 2, for Poissonian occurrences of rainfall events, the interstorm periods are exponentially distributed. The rate of arrival i.e. number of storms per unit time (m_ν if the unit time is equal to one year) depends upon how the storms are separated from each other. As per the requirement of Poisson process, the storms should be identified in such a manner that the resulting series of interstorm period become exponentially distributed. The storms so separated are then used to compute mean intensity and mean duration of different storms for the watershed. The stochastic rainfall model parameters β and δ are simply the inverses of mean rainfall intensity and mean rainfall duration respectively.

Table 6.1 - Thiessen weights for different years for test watersheds

YEARS	STATIONS			
TAIRHIA WATERSHED				
	RAMANPUR	BRIDGE SITE		
1971 TO 1974	0.94078	0.05922		
PAUSAR WATERSHED				
	RAIKHERI	PANARI	BRIDGE SITE	BORI
1965 TO 1967	0.2848	0.6527	0.0625	-
1968 TO 1969	0.34806	-	-	0.65194
LAKHORA WATERSHED				
	BRIDGE SITE	GANDHWA	KUMTA	
1966, 67, 72, 73	0.0734	0.2736	0.6530	
1968 TO 1971	-	0.34704	0.65296	
KHARANALA WATERSHED				
	KALWADI	AMBODA	BRIDGE SITE	
1971	0.76478	0.23522	-	
1972 TO 1973	0.5395	0.2350	0.2255	
SUK TAWA WATERSHED				
	ZAPRI	PIPARIA	BRIDGE SITE	
1970 TO 1974	0.3736	0.4624	0.1640	

6.2.3 Characteristics of Rainfall

The watersheds selected for the present study receive rainfall during South-West monsoon only. Monsoon rains start from June, 15. The rainy season is limited to a period of four months (June, 15 to October, 15) only. The monsoon retreats at the end of September. However, a few storms are received till October, 15. Remaining period of the year receives no rainfall in most of the years.

Since upper layer of soil is recharged due to rains in the month on June, floods occur during the months of July and August. July is the wettest month of the year followed by August. Generally, intense storms of longer durations are observed in these months. Most of the hilly areas of sub zone -3c forming the upper portions of the watersheds receive high intensity rains.

6.2.4 Identification of Independent Storms

Restrepo-Posada and Eagleson (1982) suggested a simple procedure for selecting minimum interstorm period t_{b_0}. They used exponentiality of interstorm period as a sufficient criterion for the statistically independent storm arrivals. A sample coefficient of variation of unity was accepted as a sufficient criterion. Diaz-Granados et al. (1983,1984) and Moughamian et al. (1987) applied this criterion for identification of independent storms. The above criterion when applied on the rainfall data of five test watersheds gave very long mean durations of storms which were physically unrealistic. This also resulted in very low mean rainfall intensity and less number of independent events per year.

In the present study, a simple conceptual criterion has been used which partially fulfills the theoretical assumption of exponentially distributed interstorm periods as well as matches with the actual rainfall pattern of the test watersheds. The criterion uses time of concentration of the watershed as the basis of separating two storms. Conceptually if two storms are separated by a period equal to or more than time of concentration of the watershed, they will produce two separate peaks and can be considered as independent storms.

Time of Concentration

There are various methods for estimation of time of concentration of a watershed. In the present study, the method given by Soil Conservation Service (1986) was adopted. In this method time of concentration is obtained by adding sheet flow travel time and channel flow travel time. Sheet flow is limited to a distance of 90 m only. Sheet flow travel time t_{sf} (hr) was computed by the following equation.

$$t_{sf} = \frac{0.0288\ (n_p L)^{0.8}}{(P_2)^{0.5}\ S^{0.4}} \qquad (6.1)$$

where

n_p = Manning's roughness coefficient

P_2 = 2 year 24 hour rainfall (cm)

S = land slope (m/m)

L = flow length (m)

The velocity of flow in the channel can be estimated using Manning's equation. However, the velocity of flow in the channel was estimated using available runoff and cross-section data. Travel time in the channel was computed using channel length and velocity in the channel.

6.2.5 Reasonableness of Assumed Model

Before using the exponential rainfall model for DFFD models it is essential to test the reasonableness of Poissonian assumption. As mentioned in section 6.2.4 a practical approach based on time of concentration has been adopted to identify the independent storms. The time series obtained using this approach were tested for goodness of fit. The results are as under.

Interstorm Period

The observed and computed relative frequencies of interstorm periods (events separated by at least time of concentration) are plotted in Fig. 6.1 to 6.5 for five watersheds. These histograms show that the distributions are exponential-like. Kolmogorov-Smirnov test (Haan, 1977) was also applied to test

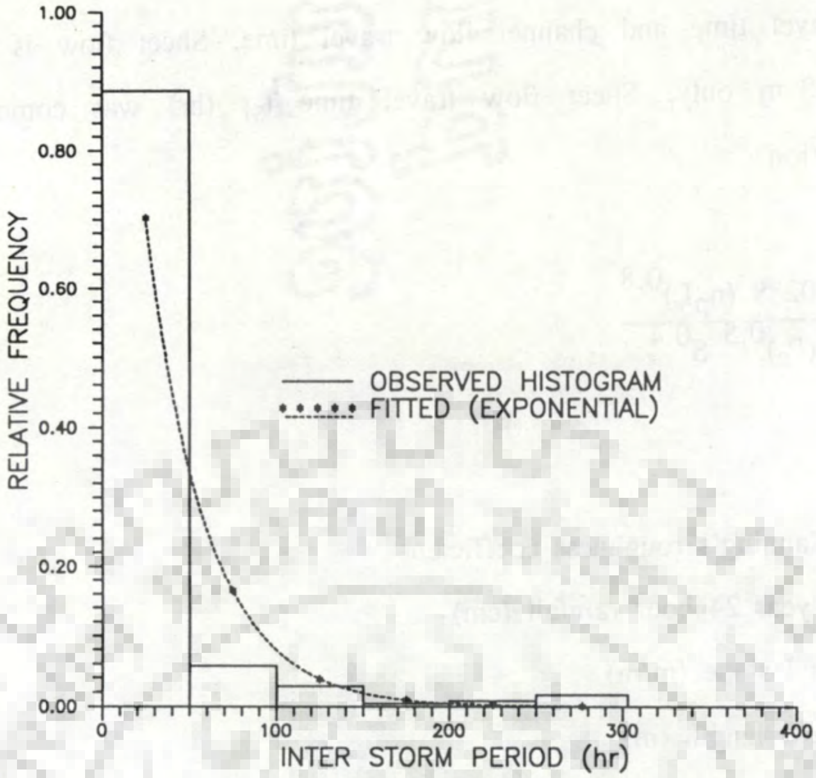

FIG. 6.1—DISTRIBUTION OF INTER STORM PERIOD AT TAIRHIA WATERSHED

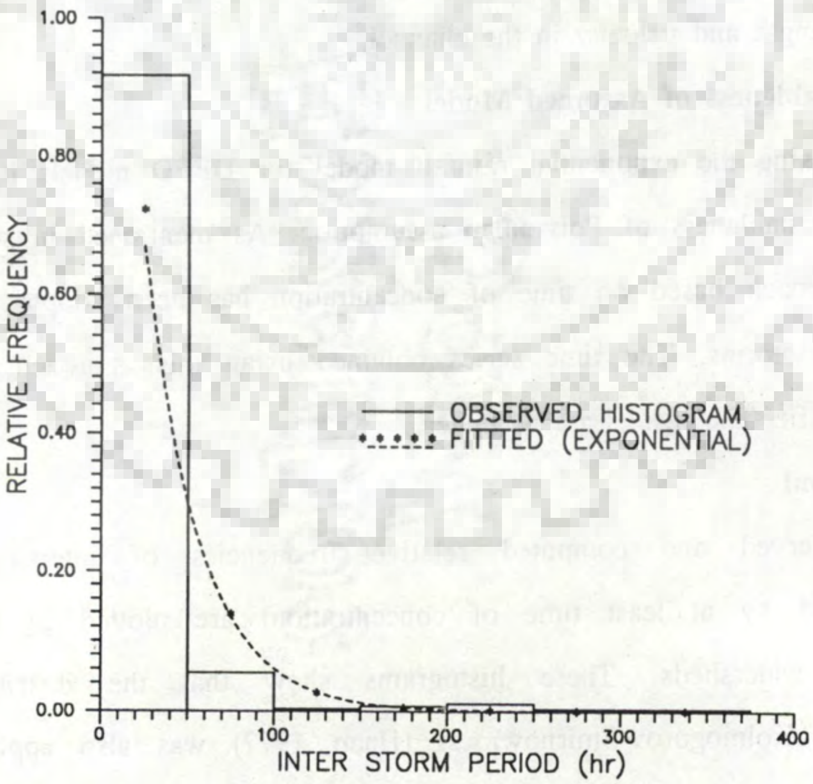

FIG. 6.2—DISTRIBUTION OF INTER STORM PERIOD AT PAUSAR WATERSHED.

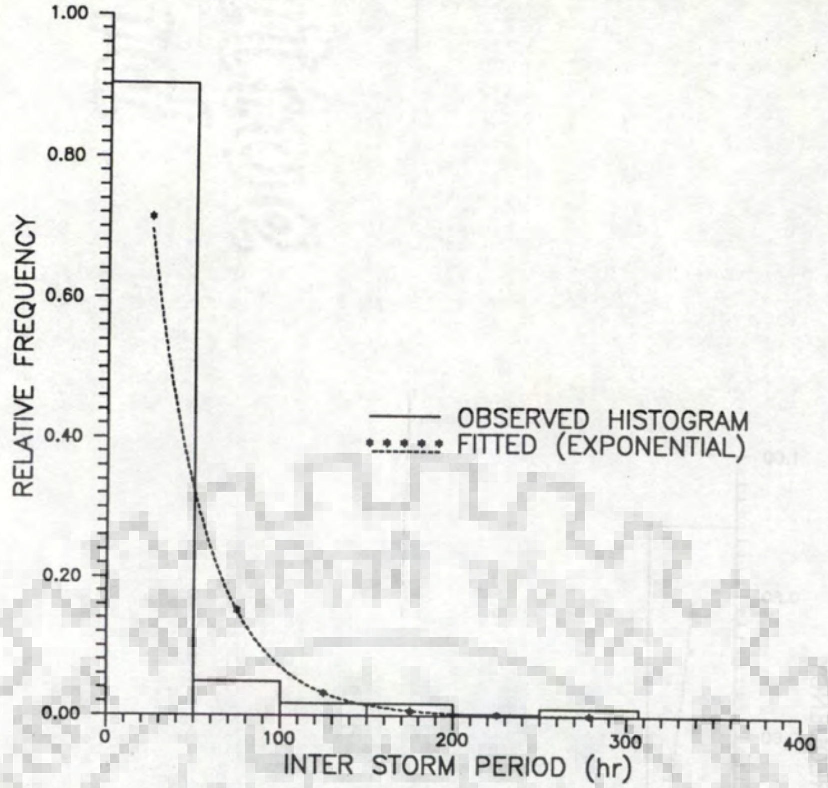

FIG. 6.3—DISTRIBUTION OF INTER STORM PERIOD AT LAKHORA WATERSHED.

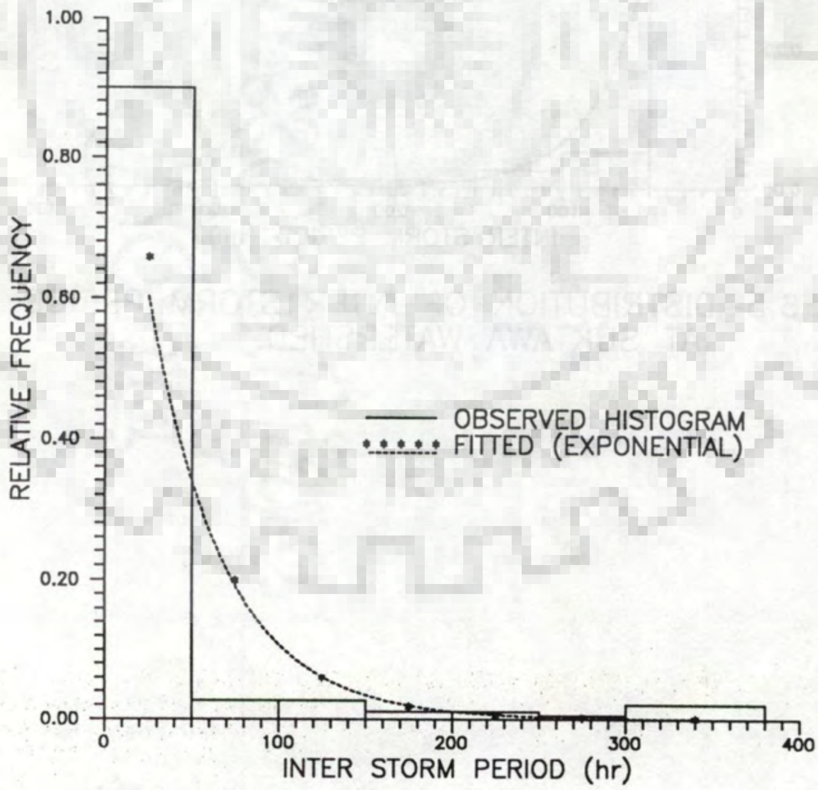

FIG. 6.4—DISTRIBUTION OF INTER STORM PERIOD AT KHARANALA WATERSHED.

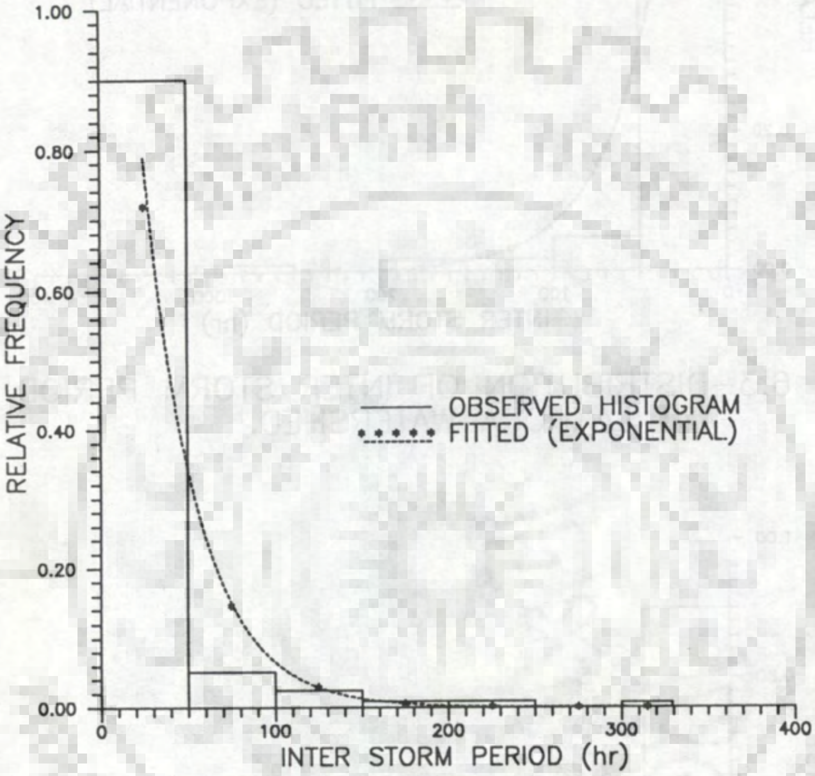

FIG. 6.5–DISTRIBUTION OF INTER STORM PERIOD AT SUKTAWA WATERSHED.

the validity of exponential distribution for interstorm duration. The test uses maximum difference between observed probability and computed CDF as test statistics. The computed K - S statistic is compared with the critical value for a given number of observations. For the data of interstorm period the computed K - S statistics and its critical value at 10 per cent significance level are given in Table 6.2 along with the number of observations. It may be seen from the table that the data of Pausar, Kharanala and Suk Tawa watersheds pass Kolmogorov-Smirnov test at 10 per cent significance level. Data of Kharanala watershed does pass this test even at 1 per cent significance level.

Table 6.2 - K - S statistics for interstorm period

WATERSHED	NO. OF OBSERVATIONS	K - S (COMP.)	K - S (CRIT.) (10%)
Tairhia	250	0.124	0.103
Pausar	279	0.089	0.098
Lakhora	458	0.081	0.076
Kharanala	192	0.050	0.118
Suk Tawa	455	0.074	0.076

Rainfall Intensity

The observed and computed relative frequencies of intensity are depicted in Fig. 6.6 to 6.10. These figures indicate that observed samples of intensities belong to an exponential distribution. The Kolmogorov-Smirnov tests were also conducted for the series of areal rainfall intensity (Table 6.3). Data of all the five test watersheds pass this test even at 1 per cent significance level.

Table 6.3 - K - S statistics for storm intensity

WATERSHED	NO. OF OBS.	K - S (COMP.)	K - S (CRIT.) (1%)
Tairhia	250	0.022	0.077
Pausar	279	0.019	0.073
Lakhora	458	0.013	0.057
Kharanala	192	0.019	0.088
Suk Tawa	455	0.017	0.057

Rainfall Duration

Observed and computed relative frequencies of duration are depicted in Fig. 6.11 to 6.15. These figures indicate that observed samples of duration could be considered exponential-like as depicted in histograms. Kolmogorov-Smirnov test was also conducted for storm durations. The data of Tairhia watersheds were exponentially distributed at 1 per cent significance level as indicated by Kolmogorov-Smirnov test (Table 6.4). In Kharanala watershed Kolmogorov-Smirnov test was passed at 10 per cent significance level. Duration in other three watersheds could not pass this test.

Table 6.4 - K - S statistics for storm duration

WATERSHED	NO. OF OBS.	K - S (COMP.)	K - S (CRIT.) (10%)
Tairhia	250	0.068	0.103
Pausar	279	0.112	0.098
Lakhora	458	0.115	0.076
Kharanala	192	0.102	0.118
Suk Tawa	455	0.119	0.076

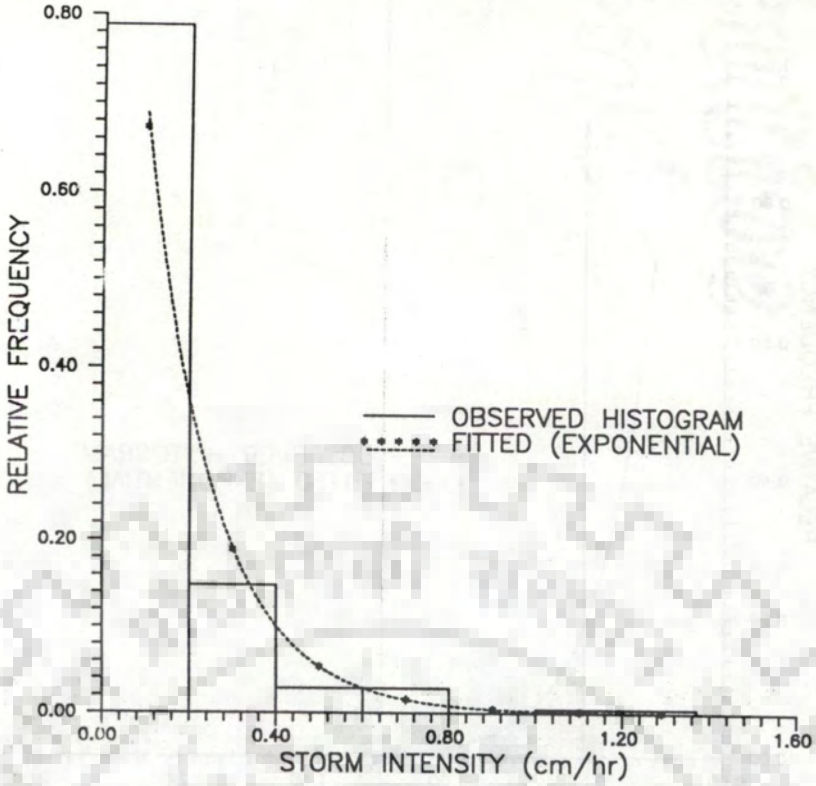

FIG. 6.6–DISTRIBUTION OF STORM INTENSITY AT TAIRHIA WATERSHED.

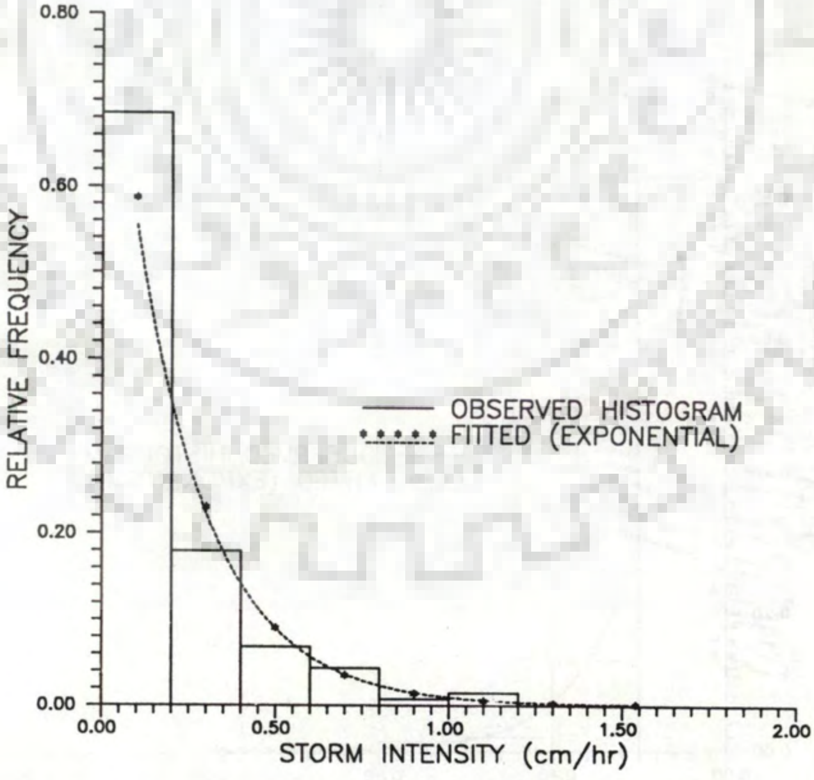

FIG. 6.7–DISTRIBUTION OF STORM INTENSITY AT PAUSAR WATERSHED.

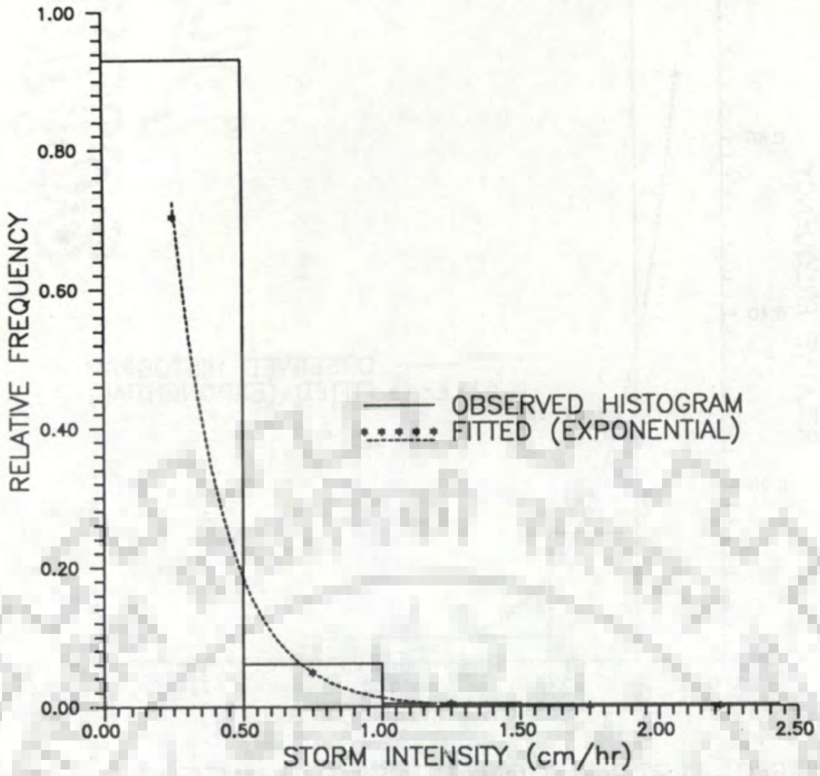

FIG. 6.8–DISTRIBUTION OF STORM INTENSITY AT LAKHORA WATERSHED.

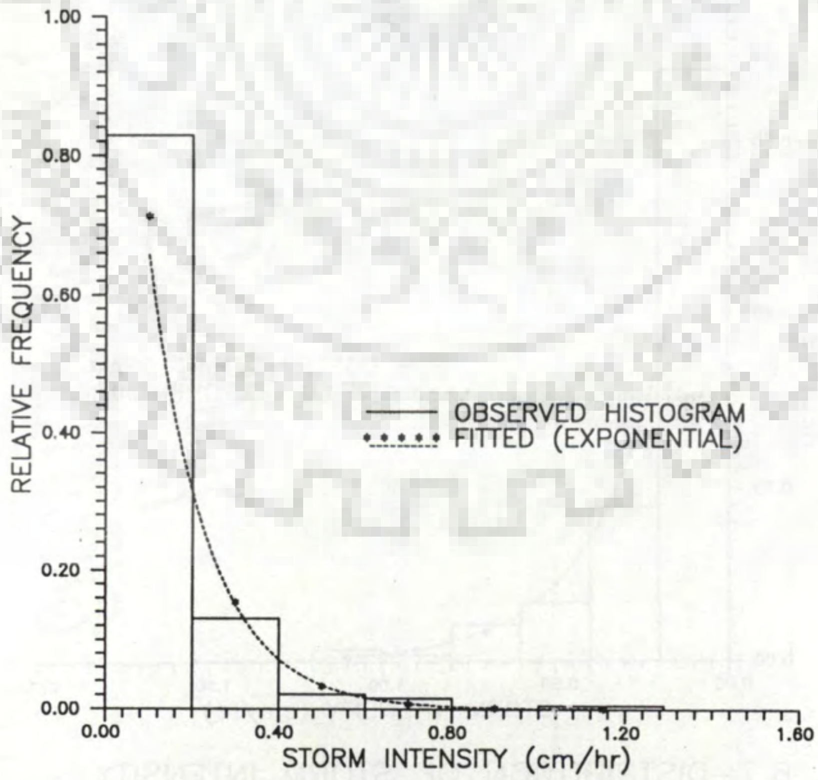

FIG. 6.9–DISTRIBUTION OF STORM INTENSITY AT KHARANALA WATERSHED.

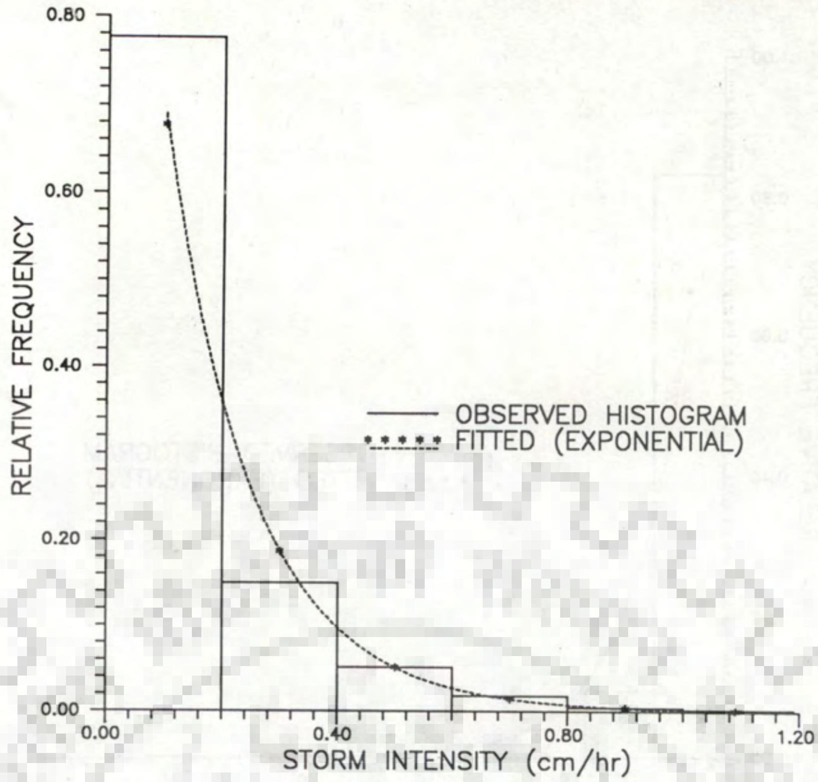

FIG. 6.10—DISTRIBUTION OF STORM INTENSITY AT SUKTAWA WATERSHED.

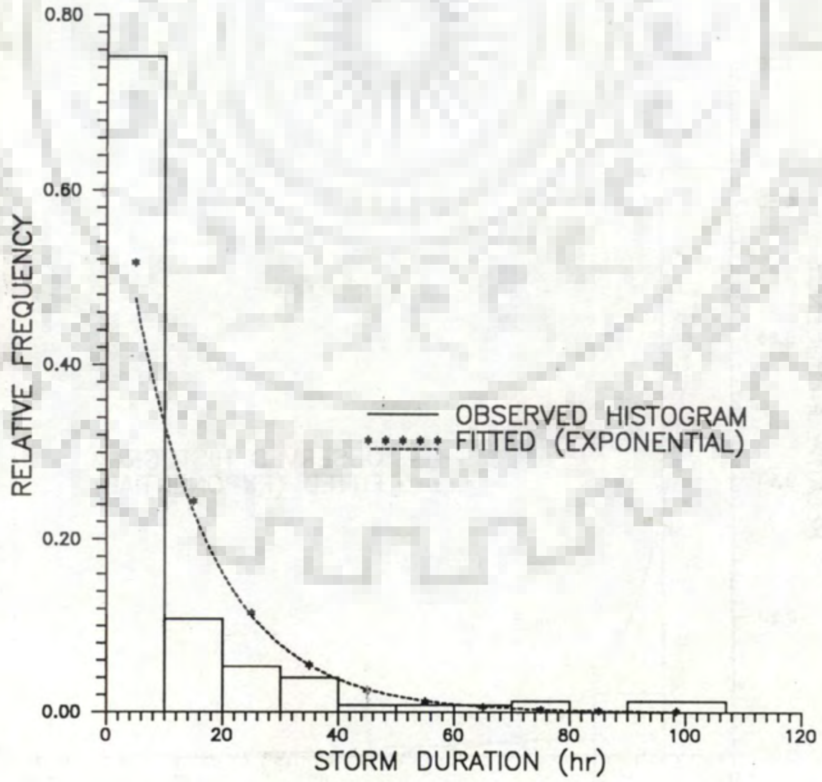

FIG. 6.11—DISTRIBUTION OF STORM DURATION AT TAIRHIA WATERSHED.

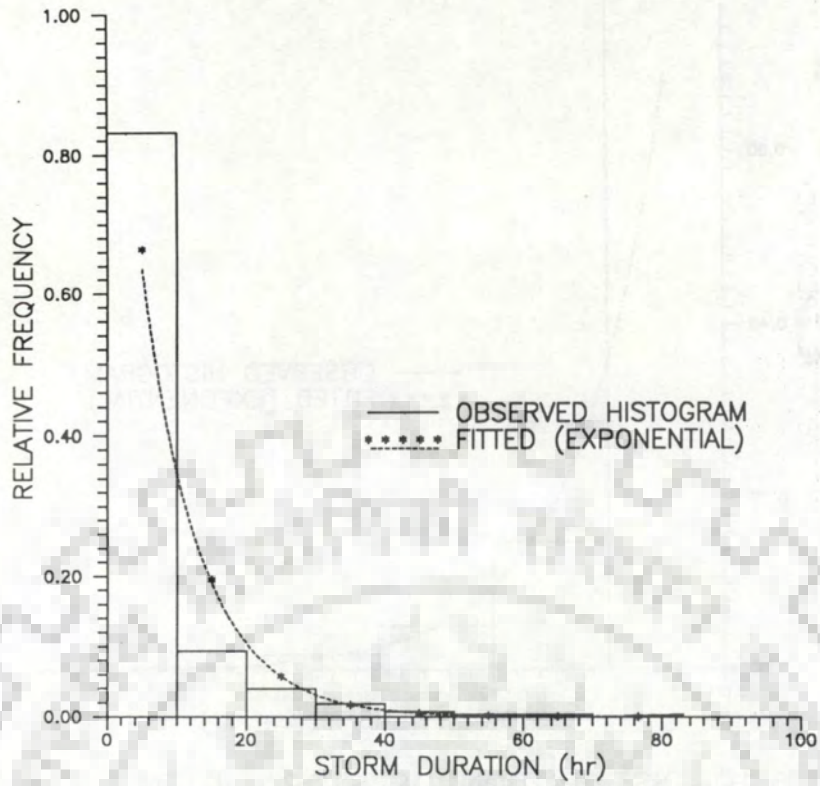

FIG. 6.12—DISTRIBUTION OF STORM DURATION AT PAUSAR WATERSHED.

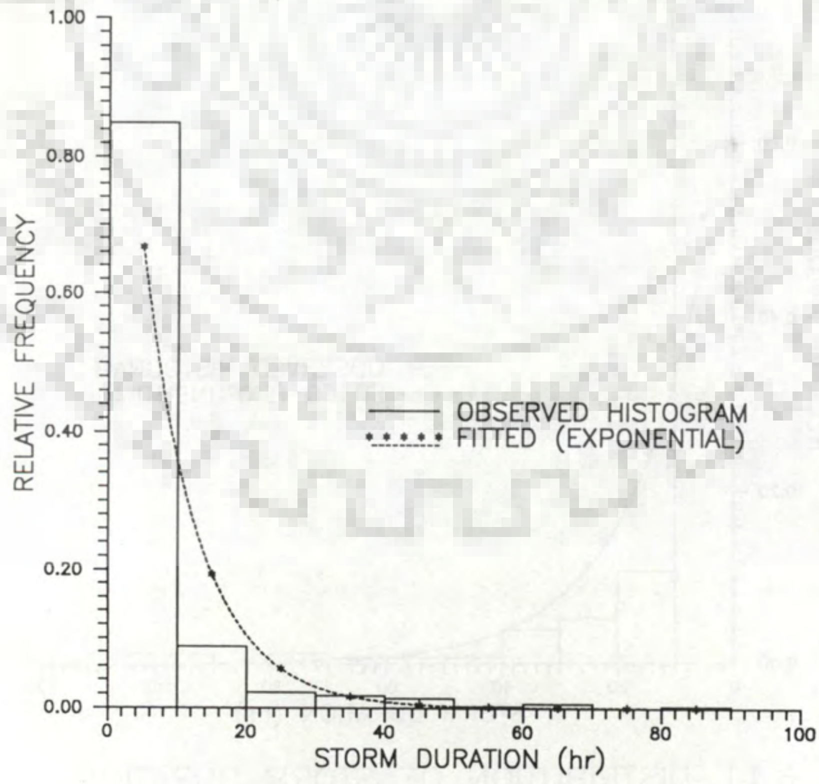

FIG. 6.13—DISTRIBUTION OF STORM DURATION AT LAKHORA WATERSHED.

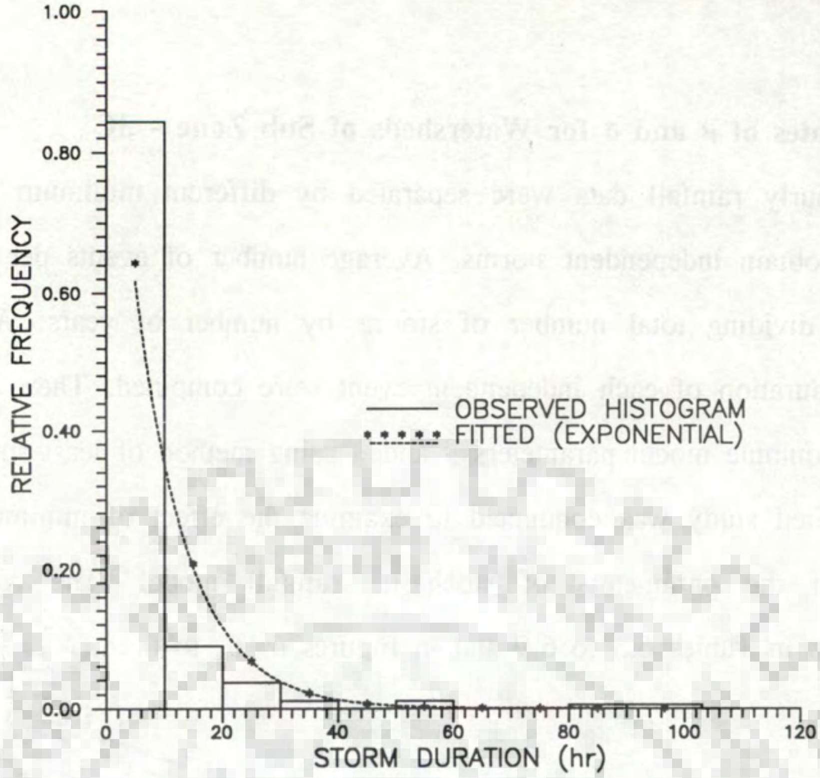

FIG. 6.14—DISTRIBUTION OF STORM DURATION AT KHARANALA WATERSHED.

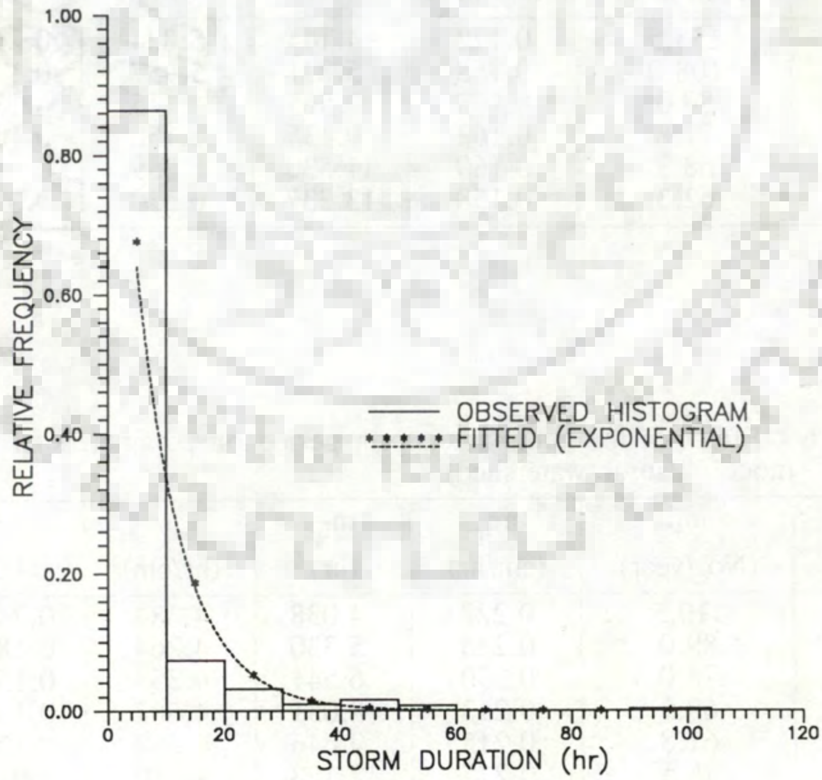

FIG. 6.15—DISTRIBUTION OF STORM DURATION AT SUKTAWA WATERSHED.

6.2.6 Estimates of β and δ for Watersheds of Sub Zone - 3C

The hourly rainfall data were separated by different minimum time between storms t_{b_0} to obtain independent storms. Average number of events per year m_ν was calculated by dividing total number of storms by number of years. Average storm intensity and duration of each independent event were computed. These samples were then used to compute model parameters β and δ using method of least squares.

A detailed study was conducted to examine the effect of minimum interstorm period t_{b_0} on the parameters of stochastic rainfall model. The results of this study are given in Tables 6.5 to 6.9 and in Figures 6.16 to 6.18.

Table 6.5 - Effect of minimum interstorm period on parameters of rainfall model (Tairhia watershed)

t_{b_0} (hr)	m_ν (No./year)	m_{i_r} (cm/hr)	m_{t_r} (hr)	β (hr/cm)	δ (1/hr)
1	135.5	0.172	4.144	5.824	0.24132
2	108.0	0.168	5.831	5.957	0.17149
3	89.0	0.163	7.565	6.115	0.13219
4	77.8	0.162	9.435	6.156	0.10599
5	68.3	0.160	11.530	6.259	0.08673
6	62.3	0.158	13.337	6.330	0.07498

Table 6.6 - Effect of minimum interstorm period on parameters of rainfall model (Pausar watershed)

t_{b_0} (hr)	m_ν (No./year)	m_{i_r} (cm/hr)	m_{t_r} (hr)	β (hr/cm)	δ (1/hr)
1	110.5	0.228	4.038	4.384	0.24767
2	89.0	0.235	5.330	4.264	0.18763
3	78.0	0.230	6.541	4.353	0.15289
4	69.5	0.213	8.051	4.687	0.12421
5	61.8	0.215	9.646	4.658	0.10368
6	55.5	0.217	12.123	4.610	0.08249

Table 6.7 - Effect of minimum interstorm period on parameters of rainfall model (Lakhora watershed)

t_{b_0} (hr)	m_ν (No./year)	m_{i_r} (cm/hr)	m_{t_r} (hr)	β (hr/cm)	δ (1/hr)
1	115.5	0.197	3.615	5.071	0.27661
2	89.1	0.187	5.121	5.342	0.19526
3	74.9	0.185	6.621	5.399	0.15103
4	65.3	0.187	8.059	5.348	0.12408
5	58.6	0.183	9.333	5.466	0.10714
6	53.6	0.184	10.824	5.422	0.09239

Table 6.8 - Effect of minimum interstorm period on parameters of rainfall model (Kharanala watershed)

t_{b_0} (hr)	m_ν (No./year)	m_{i_r} (cm/hr)	m_{t_r} (hr)	β (hr/cm)	δ (1/hr)
1	95.3	0.134	5.376	7.449	0.18602
2	78.3	0.133	6.890	7.505	0.14514
3	63.7	0.127	8.862	7.852	0.11284
4	56.3	0.120	10.461	8.303	0.09559
5	51.7	0.108	11.691	9.269	0.08554
6	46.3	0.101	13.842	9.898	0.07224

Table 6.9 - Effect of minimum interstorm period on parameters of rainfall model (Suk Tawa watershed)

t_{b_0} (hr)	m_ν (No./year)	m_{i_r} (cm/hr)	m_{t_r} (hr)	β (hr/cm)	δ (1/hr)
1	133.8	0.154	4.634	6.493	0.21578
2	105.6	0.154	6.313	6.492	0.15840
3	90.8	0.155	7.776	6.467	0.12861
4	80.0	0.157	9.396	6.351	0.10643
5	73.4	0.154	10.686	6.495	0.09358
6	70.2	0.153	11.568	6.541	0.08649

The following conclusions may be drawn from this analysis.

(1) The mean storm duration increases as t_{b_0} increases.

(2) As the minimum time between storms increases the number of storms per year m_ν decreases.

(3) The mean areal rainfall intensities decrease slightly as the t_{b_0} increases from 1 to 6 hours..

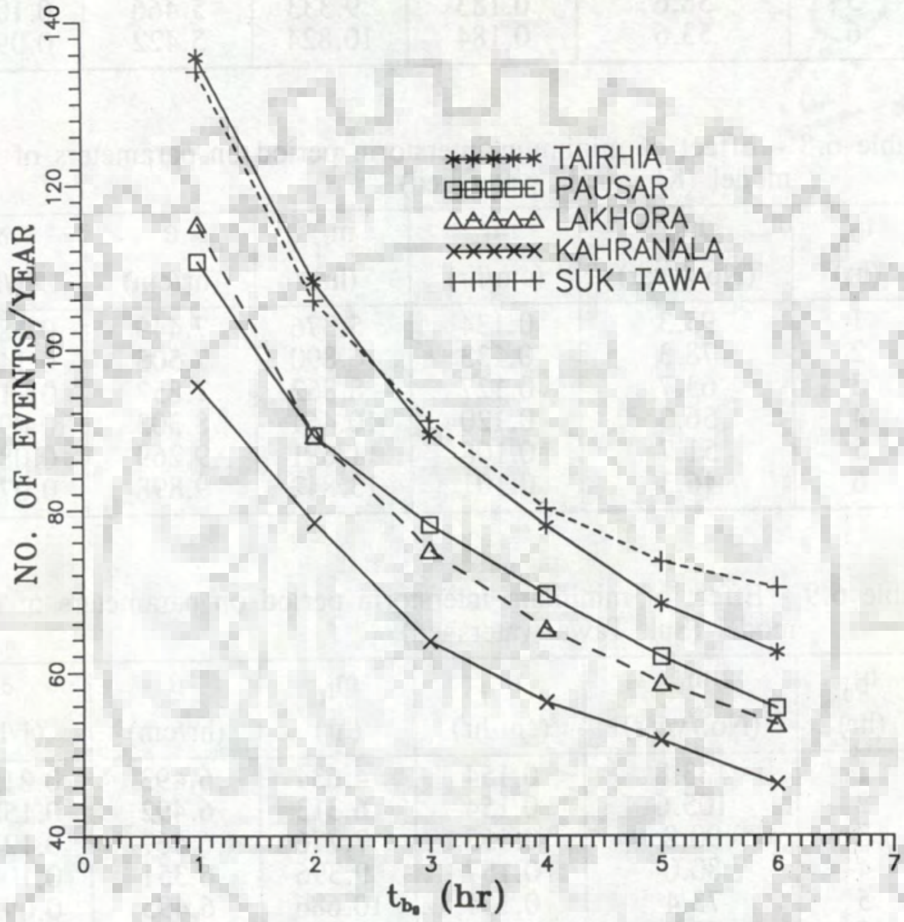

FIG. 6.16—EFFECT OF tb_0 ON NUMBER OF EVENTS/YEAR.

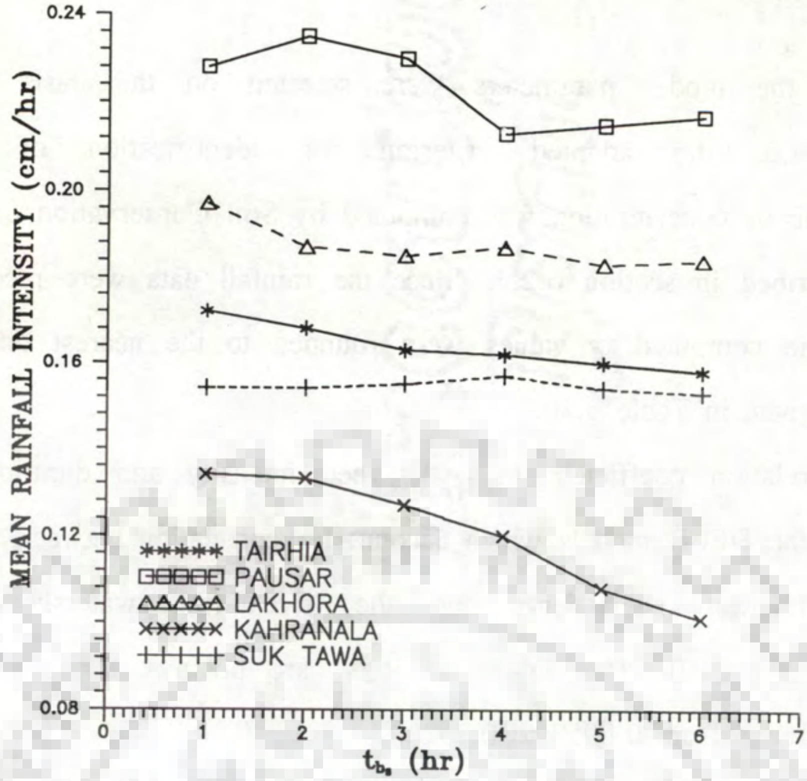

FIG. 6.17—EFFECT OF tb_O ON MEAN RAINFALL INTENSITY.

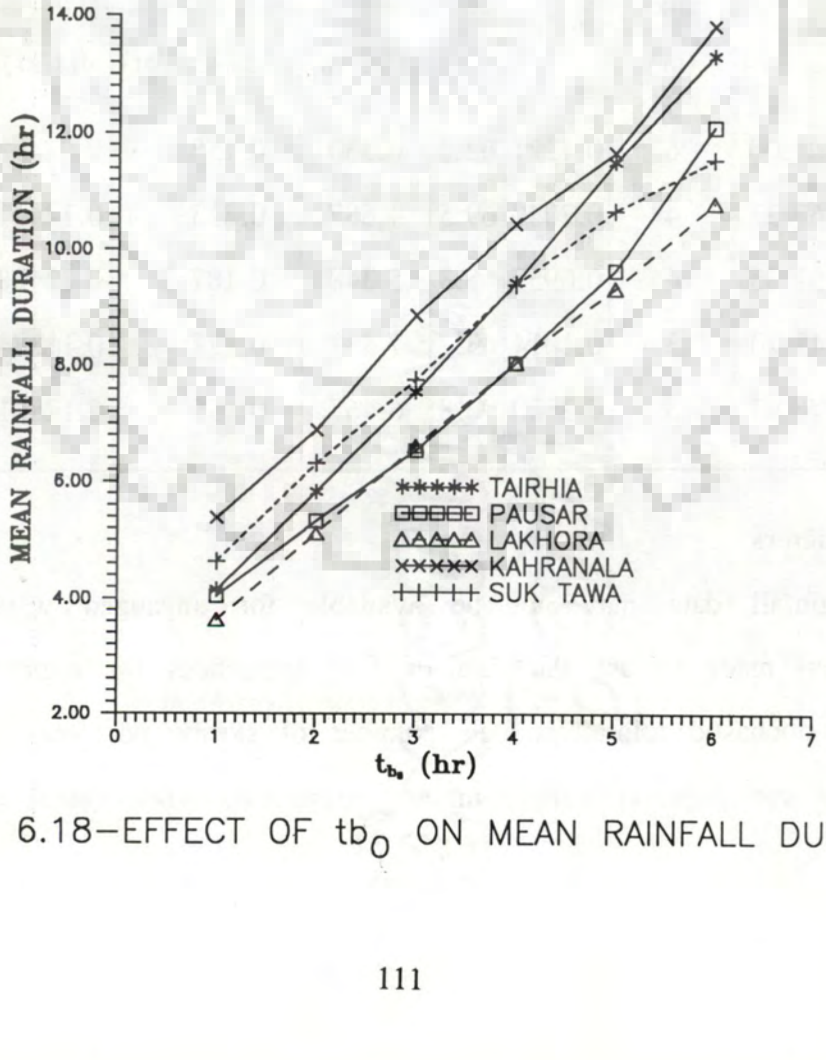

FIG. 6.18—EFFECT OF tb_O ON MEAN RAINFALL DURATION.

Finally, the model parameters were selected on the basis of time of concentration, i.e. the adopted criterion for identification of independent storms. The time of concentration was computed by Soil Conservation Service (1986) method as described in section 6.2.4. Since the rainfall data were recorded at one hour interval the computed t_c values were rounded to the nearest one hour. The parameters are given in Table 6.10.

The correlation coefficient ρ_{i_r,t_r} between intensity and duration of storms is required for the DFFD models which account for correlation between these random variables. Correlation coefficients for the five test watersheds are also presented in Table 6.10. The values of ρ_{i_r,t_r} are positive for all the five test watersheds and range from 0.0957 to 0.2520.

Table 6.10 - Stochastic rainfall model parameters for five test watersheds

Watershed	Area (sq km)	Time of conc. (hr)	ρ_{i_r,t_r}	m_ν	MODEL PARAMETERS			
					β(hr/cm)	mi_r(cm/hr)	δ(1/hr)	mt_r(hr)
Tairhia	101.00	6	0.1221	62.3	6.330	0.158	0.07498	13.337
Pausar	67.37	4	0.2040	69.5	4.687	0.213	0.12421	8.051
Lakhora	151.35	4	0.0957	65.3	5.348	0.187	0.12408	8.059
Kharanala	42.70	3	0.1014	63.7	7.852	0.127	0.11284	8.862
Suk Tawa	178.07	3	0.2520	90.8	6.467	0.155	0.12861	7.776

Regional Parameters

Since rainfall data may not be available for ungauged watersheds, an attempt has been made to use the data of five watersheds for regionalization of parameters for stochastic rainfall model. Number of storms per year, mean areal rainfall intensity and duration were computed using data from Tables 6.5 to 6.9.

Average values were computed for each t_{b_o}. Table 6.11 gives these values for different t_{b_0}.

Table 6.11 - Mean areal rainfall intensity and duration at different t_{b_0} for the sub zone - 3c

t_{b_0} (hr)	No. of storms per year	Mean intensity (cm/hr)	Mean duration (hr)
1	118.04	0.1770	4.3614
2	94.00	0.1754	5.8970
3	79.28	0.1720	7.4730
4	69.78	0.1678	9.0804
5	62.76	0.1640	10.5772
6	57.58	0.1626	12.3388

6.3 INFILTRATION MODELS

In the present study, ϕ - index, Philip's equation and SCS curve number method were used as infiltration models. The following sections describe the procedures adopted for estimating the model parameters.

6.3.1 ϕ - index

The main soil group in the five test watersheds is black cotton soil (clay). This soil has the property of swelling, as a result, more runoff is generated. On an average 10 per cent of gross rainfall is lost as abstractions. As seen from table 6.10 the average rainfall intensity and duration for the five test watersheds are about 0.15 cm/hr and 10 hours respectively. Therefore, out of 1.5 cm average storm depth, 0.15 cm (10 per cent) will be lost in 10 hours duration. This gives a ϕ-index of 0.015 cm/hr. This average value has been used for all the test watersheds.

6.3.2 Philip's Equation

Infiltration sorptivity S_i and gravitational infiltration rate A_0 are the two parameters of Philip's infiltration model (section 3.2.3). Sorptivity is the capacity of soil to absorb water and may be computed as Koch (1985)

$$S_i = [2K_s(\theta_s - \theta_i)H_c]^{1/2} \tag{6.2}$$

where

K_s = the hydraulic conductivity of the soil (cm/hr)

θ_s = soil water content at natural saturation

θ_i = soil water content at the beginning of rainfall

H_c = the suction head (cm)

Cadavid et al. (1991) used above equation to estimate S_i. In the present work, the hydraulic conductivity K_s, soil water content at natural saturation θ_s and suction head H_c were taken from Rawls et al. (1983). Gravitational infiltration rate A_o has been replaced by K_s. One third of the values given in the table (Rawls et al., 1983) were used as K_s. The table gives a wide range of soil water content at natural saturation and suction head. Therefore, typical values should be considered for a particular application. In the study area, the moisture content of the clay soil remains higher during the rainy season after the first few rains. The judgmental values for θ_s, θ_i and H_c have been selected on this basis. Two types of soils were observed in Suk Tawa watershed as a result weighted values of the parameters were computed for this watershed depending upon the areal extent of each soil type. Computation of sorptivity for the five test watersheds is given in Table 6.12.

Table 6.12 - Computation of infiltration sorptivity

Watershed	Soil Type	K_s (cm/hr)	θ_s	θ_i	H_c (cm)	S_i (cm/hr$^{1/2}$)	Weighted S_i (cm/hr$^{1/2}$)
Tairhia	Clay	0.01	0.4	0.3	10.0	0.1	0.1
Pausar	Clay	0.01	0.4	0.3	10.0	0.1	0.1
Lakhora	Clay	0.01	0.4	0.3	10.0	0.1	0.1
Kharanala	Clay	0.01	0.4	0.3	10.0	0.1	0.1
Suk Tawa	Clay (40.7%) Sandy clay loam (59.3%)	0.01 0.05	0.4 0.35	0.3 0.3	10.0 8.0	0.1 0.2	0.16

6.3.3 SCS Curve Number Method

As described in Chapter 5, the soil and land use data were used to estimate curve numbers for each watershed. Curve numbers were assigned from the tables given by SCS (1972) for each combination of soil and land uses. Weighted curve numbers were then calculated for each watershed. The computation of curve numbers is given in Table 6.13.

Table 6.13 - Computation of curve number for test watersheds

Watershed	Land Use	Soil Group	Extent (per cent)	Curve No.	Weighted CN
Tairhia	C	D	9.6	91	
	F	D	86.5	82	83.33
	B	D	3.9	94	
Pausar	C	D	60.0	89	85.00
	RF	D	40.0	79	
Lakhora	C	D	67.0	89	85.70
	RF	D	33.0	79	
Kharanala	C	D	100.0	91	91.00
Suk Tawa	C	D	40.7	91	
	F	C	27.2	82	85.7
	G	C	32.1	82	

C - Cultivated, F - Forest, B - Barren land, RF - Reserve Forest, G - Grassland

A summary of parameters of different infiltration models is given in Table 6.14.

Table 6.14 - Infiltration parameters for five test watersheds

INFILTRATION PARAMETERS	TAIRHIA	PAUSAR	LAKHORA	KHARANALA	SUK TAWA
ϕ - index (cm/hr)	0.015	0.015	0.015	0.015	0.015
Gravitational infiltration rate K_S (cm/hr)	0.01	0.01	0.01	0.01	0.034
Sorptivity S_i (cm/hr$^{1/2}$)	0.1	0.1	0.1	0.1	0.16
Curve number CN	83.33	85.00	85.70	91.00	85.70

6.4 EFFECTIVE RAINFALL-RUNOFF MODEL

In the present study GcIUH and KW have been used as watershed response models. The estimation of parameters of these models is described in this section.

6.4.1 GcIUH Parameters

As explained in Section 3.3.1, equation 3.58, for application of GcIUH, the following parameters are required.

i) Area of the watershed, A

ii) Length ratio, R_L

iii) Length of highest order stream, L_Ω and

iv) Kinematic wave parameter of highest order stream, α_Ω.

To find out the above parameters, toposheets of 1:63360/1:50000 scale of Survey of India were used. These toposheets give details of the watersheds such as location of the bridges (outlets of the watersheds), drainage lines, contours at 50 ft/20 m interval and different land uses. The test watersheds were delineated on the toposheets by careful observation of contours and drainage lines. Areas and lengths of streams were measured using digital planimeter. Lengths of all the streams of different orders were measured to obtain average stream lengths. Length ratio for each watershed was then determined by fitting these average stream lengths with stream orders (Figures 6.19-6.23). Length of highest order stream of each watershed was measured by planimeter. Kinematic wave parameter α_Ω was computed using channel characteristics estimated from the toposheets. The channel was assumed to be a wide open channel for this purpose. GcIUH parameters of five watersheds are summarised in Table 6.15.

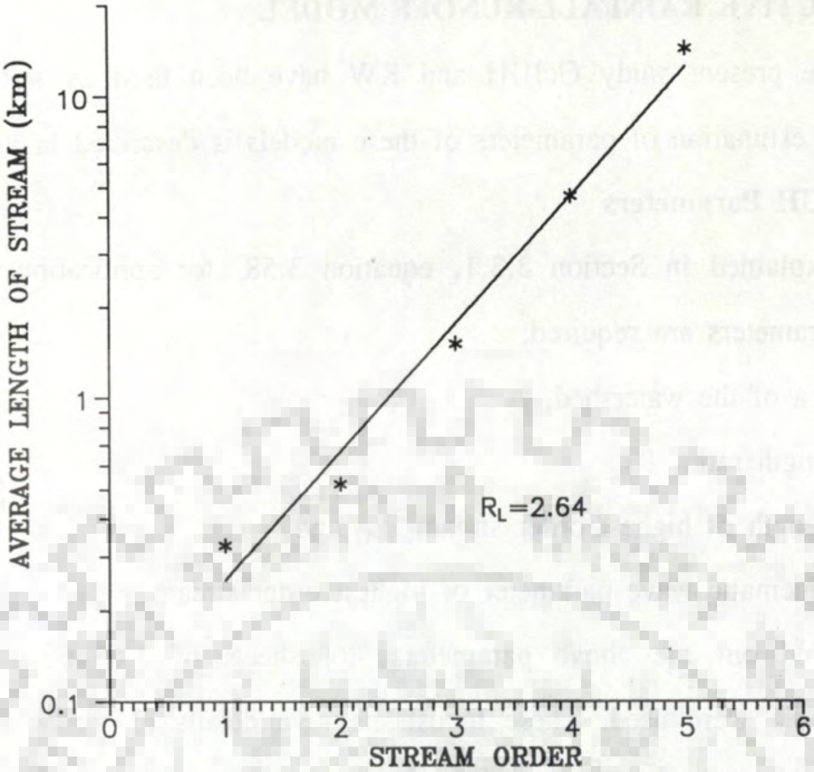

FIG. 6.19–LENGTH RATIO FOR TAIRHIA WATERSHED.

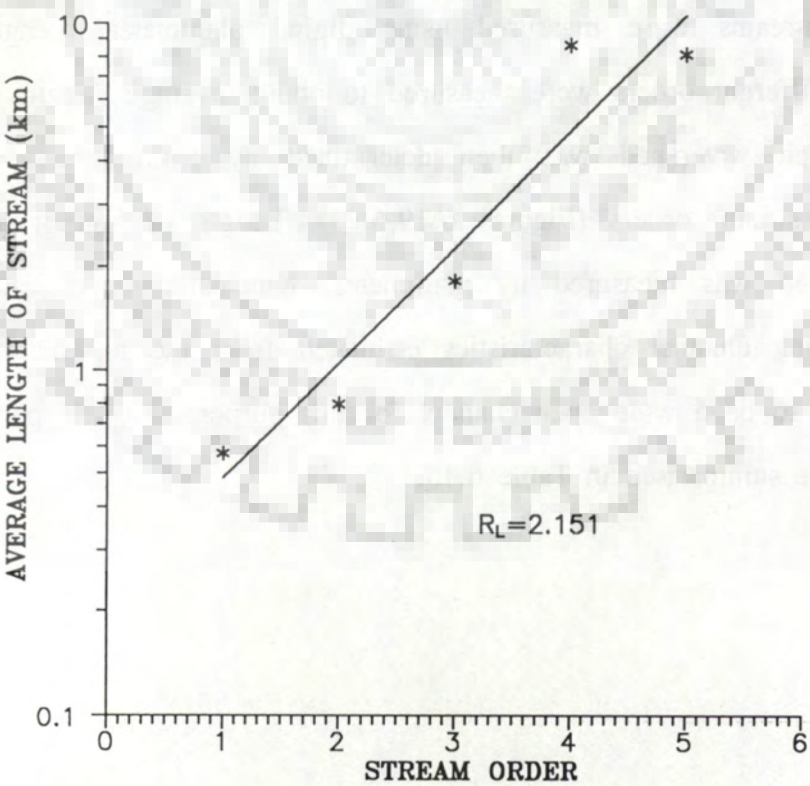

FIG. 6.20–LENGTH RATIO FOR PAUSAR WATERSHED.

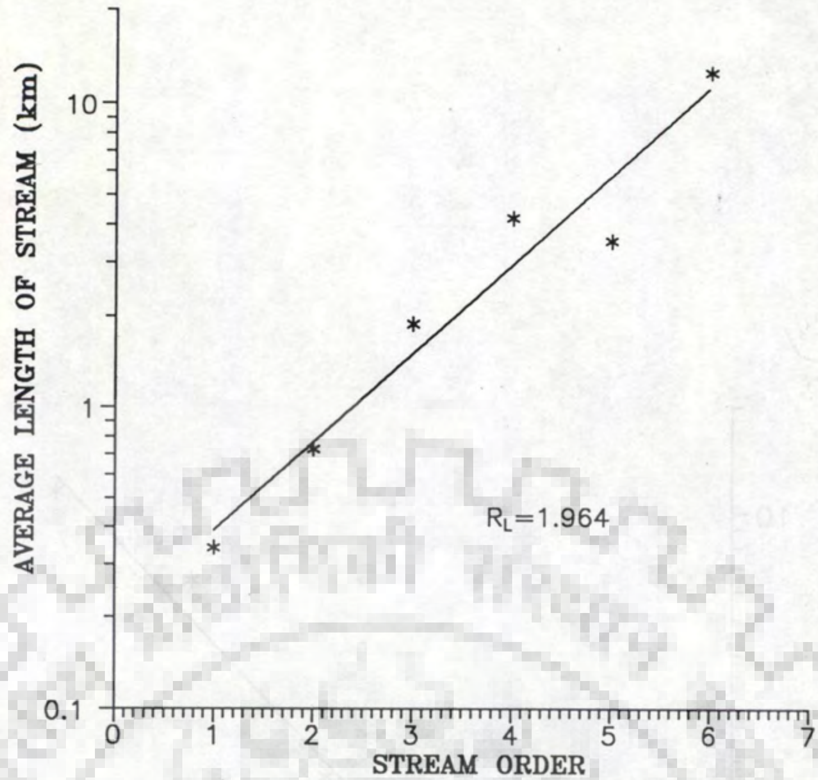

FIG. 6.21–LENGTH RATIO FOR LAKHORA WATERSHED.

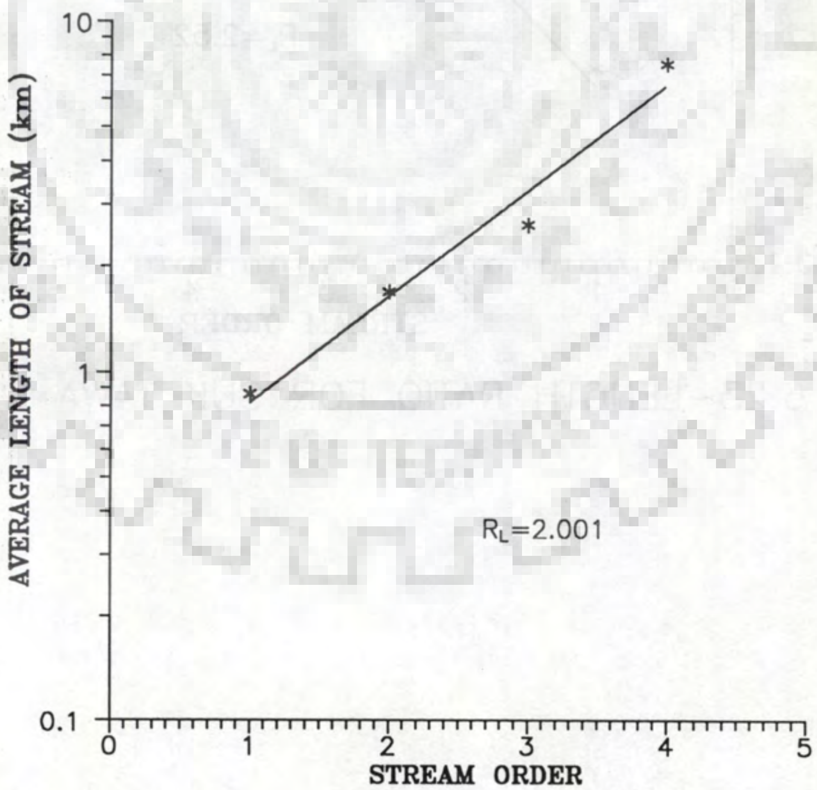

FIG. 6.22–LENGTH RATIO FOR KHARANALA WATERSHED.

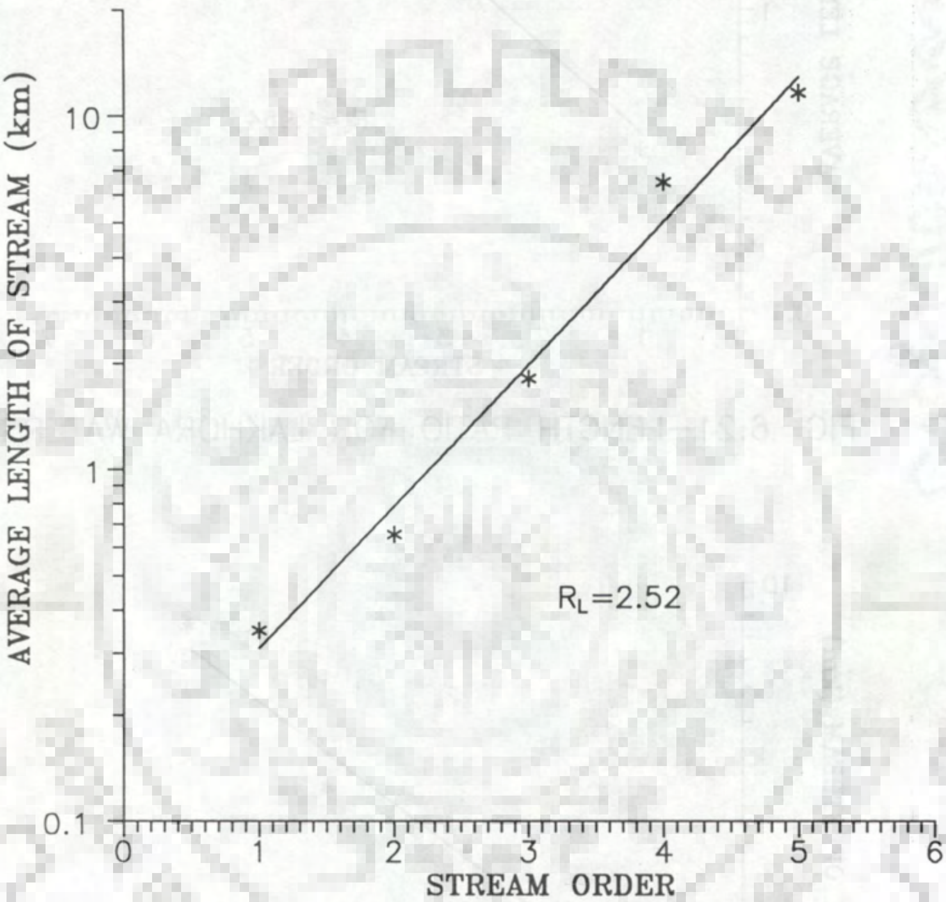

FIG. 6.23—LENGTH RATIO FOR SUK TAWA WATERSHED.

Table 6.15 - GcIUH parameters of the five test watersheds

GcIUH PARAMETERS	TAIRHIA	PAUSAR	LAKHORA	KHARANALA	SUK TAWA
Area of the watershed A (km^2)	101.00	67.37	151.35	42.70	178.07
Length ratio R_L	2.64	2.151	1.964	2.001	2.522
Length of highest order stream L_Ω (km)	14.064	8.053	12.656	7.570	11.350
KW parameter of highest order stream α_Ω ($s^{-1}m^{-1/3}$)	0.144	0.127	0.125	0.259	0.076

6.4.2 KW Parameters

The KW effective rainfall-runoff model requires following parameters.

i) Length of main channel, L_c

ii) Width of overland plane, W

iii) Channel slope, S_c

iv) Overland plane slope, S_p

v) Roughness coefficient for channel, n_c

vi) Roughness coefficient for plane, n_p and

vii) Coefficient 'a' and exponent 'b' of hydraulic radius-area relationship.

Main channel length of each watershed was measured using planimeter. The watershed area was divided by twice the main channel length to obtain average plane width. The plane slope was computed using grid method. Equivalent channel slope was computed by Gray's method (Singh, 1993) for each watershed.

The coefficient and exponent of hydraulic radius-area relationship were taken as 0.175 and 0.35 respectively (Cadavid et al., 1991). All the parameters estimated above are listed in Table 6.16.

Table 6.16 - Kinematic wave parameters of the five test watersheds

KW PARAMETERS	TAIRHIA	PAUSAR	LAKHORA	KHARANALA	SUK TAWA
Length of main channel L_c (m)	32893.5	24046.0	27600.0	23567.0	23848.5
Width of overland plane W (m)	1335.26	1400.86	2741.85	907.74	3733.36
Channel slope S_c	0.004304	0.002433	0.001975	0.002937	0.002858
Plane slope S_p	0.072267	0.030432	0.020517	0.005614	0.064034
Channel Roughness n_c	0.040	0.023	0.018	0.021	0.020
Plane roughness n_p	0.3	0.3	0.3	0.3	0.3
a (metric system)	0.175	0.175	0.175	0.175	0.175
b (metric system)	0.35	0.35	0.35	0.35	0.35

6.5 MODEL PARAMETERS OF RALSTON CREEK AND SANTA ANITA CREEK WATERSHEDS

Cadavid et al. (1991) applied their model on Santa Anita and Ralston Creek watersheds. To compare their results with the model developed in this study curve numbers for the two watersheds were optimised by visual fitting of flood frequency curves. Final rainfall parameters and KW parameters as adopted by Cadavid et al. (1991) were used for this application. Parameters of the two watersheds used are given in Table 6.17.

Table 6.17 - Parameters of watersheds tested by Cadavid et al. (1991)

PARAMETERS	RALSTON CREEK	SANTA ANITA CREEK
Mean rainfall intensity $1/\beta$ (in/hr)	0.60	0.94
Mean rainfall duration $1/\delta$ (hr)	0.90	0.30
Number of storms/year m_ν	20	20
Hydrauliuc conductivity K_s (in/hr)	0.25	0.80
Sorptivity S_i (in/hr$^{1/2}$)	1.1	1.12
Length of main stream L_c (ft)	16266	23795
Width of overland plane (ft)	2579	5688
Slope of the channel S_c	0.005	0.172
Slope of overland plane S_p	0.106	0.582
Roughness of channel n_c	0.04	0.04
Roughness of overland plane n_p	0.3	0.3
a (fps system)	0.25	0.25
b (fps system)	0.35	0.35

6.6 MODEL PARAMETERS OF DAVIDSON WATERSHED

Hebson and Wood (1982) applied their model on Davidson catchment. The details of this catchment are listed below (Table 6.18).

Table 6.18 - Parameters of Davidson catchment.

PARAMETERS	VALUE
Inverse of mean rainfall intensity (hr/cm)	2.46
Inverse of mean rainfall duration (1/hr)	0.19
Number of events/year	24
Φ - index (cm/hr)	1.125
Area A (km^2)	104.6
Length ratio R_L	2.41
Length of highest order stream L_Ω (km)	8.8
Kinematic wave parameter of highest order stream α_Ω ($m^{-1/3}s^{-1}$)	1.0

Diaz-Granados et al. (1983) applied their model on Davidson watershed and found that good results could be obtained if 50 per cent contributing area (52.3 km^2) is considered with a ϕ - index value of 0.72 cm/hr. The correlation coefficient between intensity and duration is not known for the data of Davidson catchment. However, just to demonstrate the effect of correlation the model developed in Chapter 4 was applied assuming different values of negative correlation coefficients between intensity and duration.

CHAPTER 7

RESULTS AND DISCUSSION

7.1 INTRODUCTION

The results of application of various derived flood frequency distributions to five small watersheds located in Central India are presented in this chapter. For these models, the parameters estimated in previous chapter have been used. The results of at site/regional and regional only flood frequency analysis for the Sub zone - 3c are also presented. At site/regional and regional flood frequency analyses have been carried out using General Extreme Value (GEV), Extreme Value Type 1 (EV1) and Wakeby distributions. The parameters were estimated using standardised probability weighted moments (PWM). The results of GcIUH based models are then presented for various infiltration models. Similar presentation has been made for KW theory based models.

The effect of correlation between rainfall intensity and duration on flood quantiles has been demonstrated using data of Davidson catchment. The model for negatively correlated intensity and duration developed in Chapter 4 has been used for this purpose.

7.2 REGIONAL FLOOD FREQUENCY ANALYSIS OF SUB ZONE - 3C

The regional flood frequency analysis for Sub zone - 3c was carried out using the annual flood series of 15 bridge sites (RDSO, 1991). The watershed areas of these sites range from 42.7 to 2110.85 km^2. The details are presented in the following sections.

7.2.1 Regional Homogeneity

The sites of Sub zone - 3c were tested for homogeneity using USGS homogeneity test (Dalrymple, 1960) and C_V of C_V based test (Cunnane, 1989). As illustrated in Fig 7.1, all the fifteen sites fall within 95 per cent confidence limits. The data of all the fifteen sites were considered for further analysis. The C_V of C_V for these 15 sites is 0.2736. Hence the Sub zone -3c may be considered as homogeneous.

7.2.2 Regional Analysis

The mean annual flood of 10 calibration bridge sites (excluding five test watersheds) and their catchment areas are plotted in Fig. 7.2. The relationship between mean annual flood and catchment area is obtained as under.

$$Q = 17.1209\, A^{0.6056} \qquad (7.1)$$

$r = 0.81183$

Area range = 53.68 to 2110.85 km^2

This relationship may be used for estimating mean annual flood for ungauged catchments of Sub zone - 3c. The dependence of mean annual flood on other physiographic and climatic characteristics could not be studied in detail because of lack of data.

Regional Flood Frequency Relationship

Regional flood frequency relationships have been developed using standardised probability weighted moments (Cunnane, 1988). This method proposed by Wallis (1980), is simple to apply and avails of the excellent properties (Hosking et al., 1985) of the PWM method of parameter estimation (Greenwood et al., 1989). Using annual flood series of 10 bridge sites and standardised probability weighted moment method (Wallis, 1980) the regional parameters of EV1, GEV and Wakeby-4 and 5 parameters distributions were obtained, as given below:

EV1 Distrbution

Location parameter $u = 0.7013$

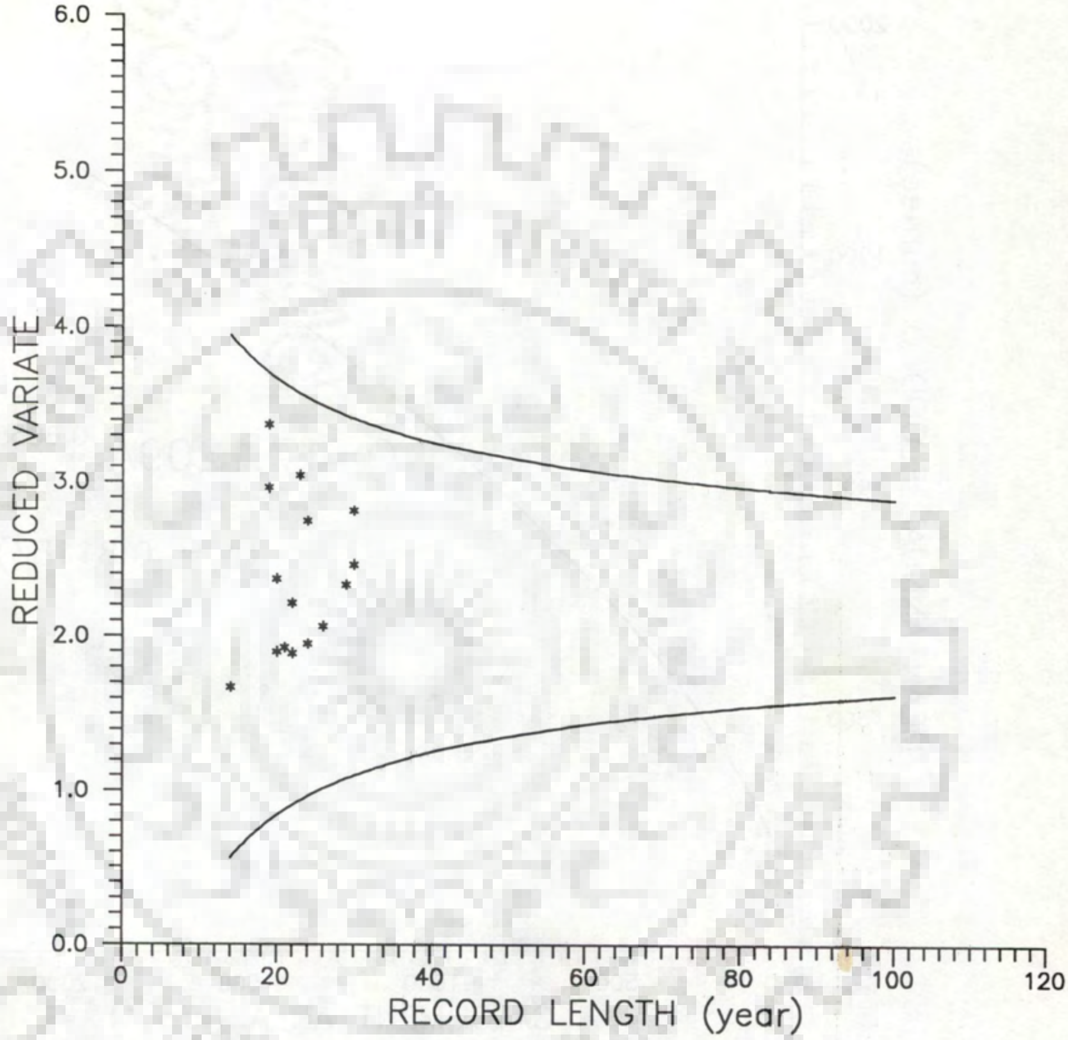

FIG. 7.1 HOMOGENEITY TEST (Dalrymple,1960) FOR SUB ZONE–3C.

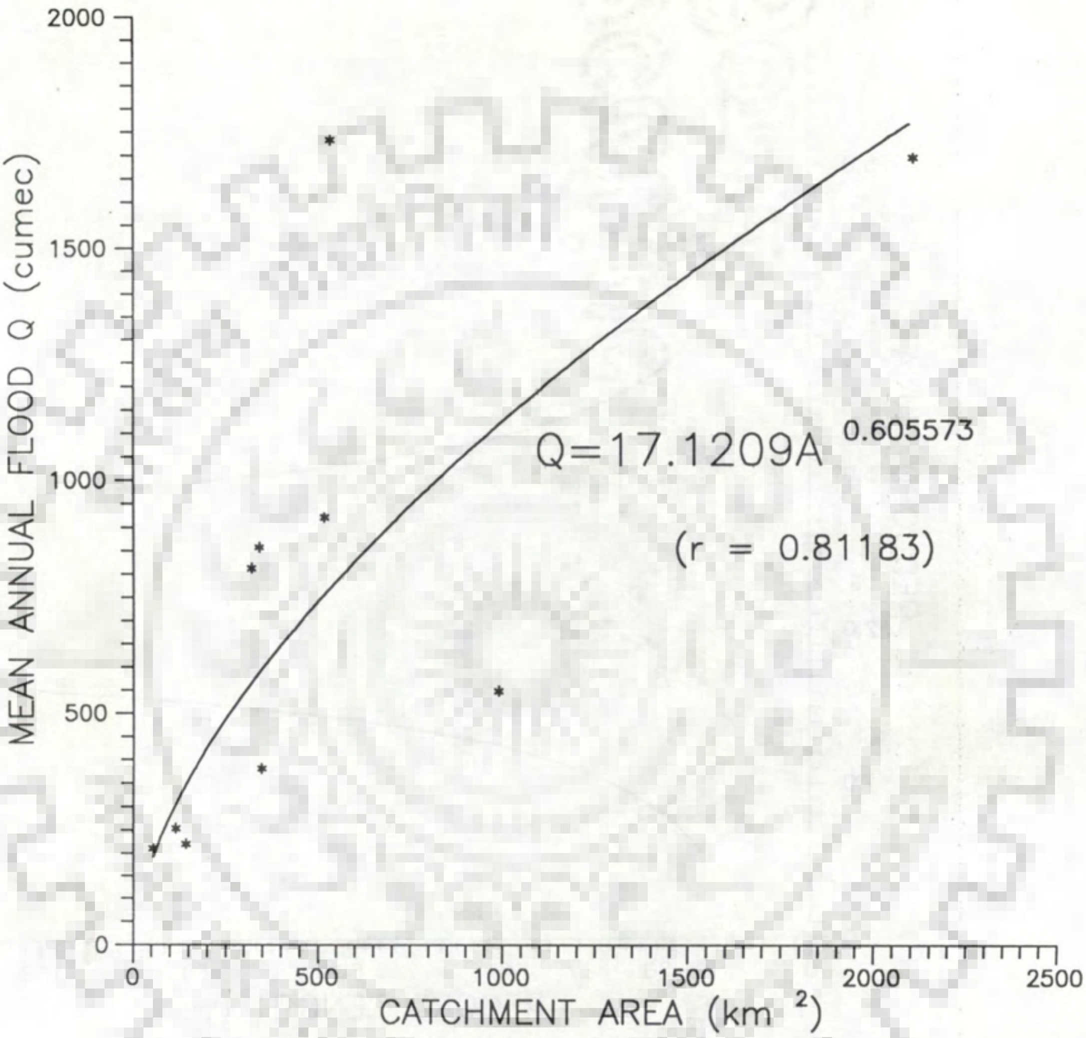

FIG. 7.2 RELATIONSHIP BETWEEN AREA AND MEAN ANNUAL FLOOD.

Scale parameter $\alpha = 0.5175$

GEV Distribution

Location parameter $u = 0.6752$

Scale parameter $\alpha = 0.4579$

Shape parameter $k = -0.11883$

Wakeby-4 parameter (A,B,C and D in Landwehr et al., 1979 notation)

A = 0.292, B = 17.807, C = -8.077 and D = -0.098

Wakeby-5 parameter (A,B,C D and M in Landwehr et al., 1979 notation)

A = 0.237, B = 10.092, C = -9.279 D = -0.083 and M = 0.075

Using the above regional parameters for EV1, GEV, Wakeby-4 as well as Wakeby-5 parameters distributions, the following two types of analyses were performed.

i) At site/regional analysis

ii) Regional only analysis.

In case of at site/regional analysis, the mean annual floods were computed using the observed data while for regional only analysis the mean annual floods were computed using equation 7.1.

For the two analyses, observed and computed quantiles by various distributions were compared on the basis of (i) average of relative deviations between computed and observed quantiles (ADA), (ii) average of squares of relative deviations between computed and observed quantiles (ADR) and (iii) efficiency as follows:

$$ADA = \sum_{i=1}^{N} \left| \frac{Q_i - \hat{Q}_i}{Q_i} \right| \frac{100}{N} \qquad (7.2)$$

$$ADR = \left[\sum_{i=1}^{N} \left(\frac{Q_i - \hat{Q}_i}{Q_i} \right)^2 \right] \frac{100}{N} \tag{7.3}$$

$$Efficiency = \left[1 - \frac{\sum_{i=1}^{N} (Q_i - \hat{Q}_i)^2}{\sum_{i=1}^{N} (Q_i - Q)^2} \right] 100 \tag{7.4}$$

where Q_i = ith observed quantile

$\hat{Q}_i$ = ith computed quantile

Q = observed mean quantile

N = number of observations

On the basis of above criteria, EV1 distribution performed well for test watersheds as well as for calibration watersheds. Therefore, quantiles were computed for five test watersheds using regional parameters of EV1 distribution for both at site/regional and regional only cases. These quantiles are then compared with the output of DFFD models. The detailed results for at site and regional analyses for other distributions like GEV and Wakeby are not being presented, as the aim is to compare the performance of various DFFD models and not to inter compare the performance of regional methods itself.

7.3 PRESENTATION OF RESULTS

The results of application of DFFD models to five test watersheds are presented in the following sections. All these models use joint PDF of intensity and duration as stochastic rainfall model. Infiltration process has been represented by ϕ-index, Philip's equation and SCS curve number method for GcIUH as well as for KW theory based models. For computation of return periods corresponding to observed flood data, Gringorton plotting position formula has been used. The presentation of results has been done in graphical and tabular forms. In both these forms the computed discharges by at site/regional and regional only methods are also presented in order to compare the results of physically based models with that of current methods of flood frequency analysis for ungauged catchments.

7.3.1 GcIUH Based Models

For the five test watersheds, the parameters of stochastic rainfall model and GcIUH model were estimated in Chapter 6 and are presented in tables 6.10 and 6.15 respectively. Using parameters of various infiltration models GcIUH based models were applied to five Indian watersheds.

GcIUH - ϕ - index Model

Estimation of ϕ - indices for five watersheds has been presented in section 6.3.1. Using methodology explained in section 3.4.1, and programme listed in Appendix-I, GcIUH - ϕ - index model was applied. The flood quantiles corresponding to different return periods, are plotted in Figs. 7.3 to 7.7. Observed and computed discharges are also summarised in Tables 7.1 to 7.5. along with the results of other models. These tables are presented latter in section 7.4.

GcIUH - Philip Model

Table 6.12 lists the parameters of Philip's infiltration equation for the five watersheds. GcIUH - Philip model was applied using these parameters and procedure given in section 3.4.2 (Programme listed in Appendix-II). The results are shown in Figs. 7.8 to 7.12. Tables 7.1 to 7.5 also present the comparative performance of GcIUH-Philip model with the other models.

GcIUH - SCS Model

For this model SCS curve numbers have been presented in Table 6.13. These curve numbers along with parameters of GcIUH and stochastic rainfall model were

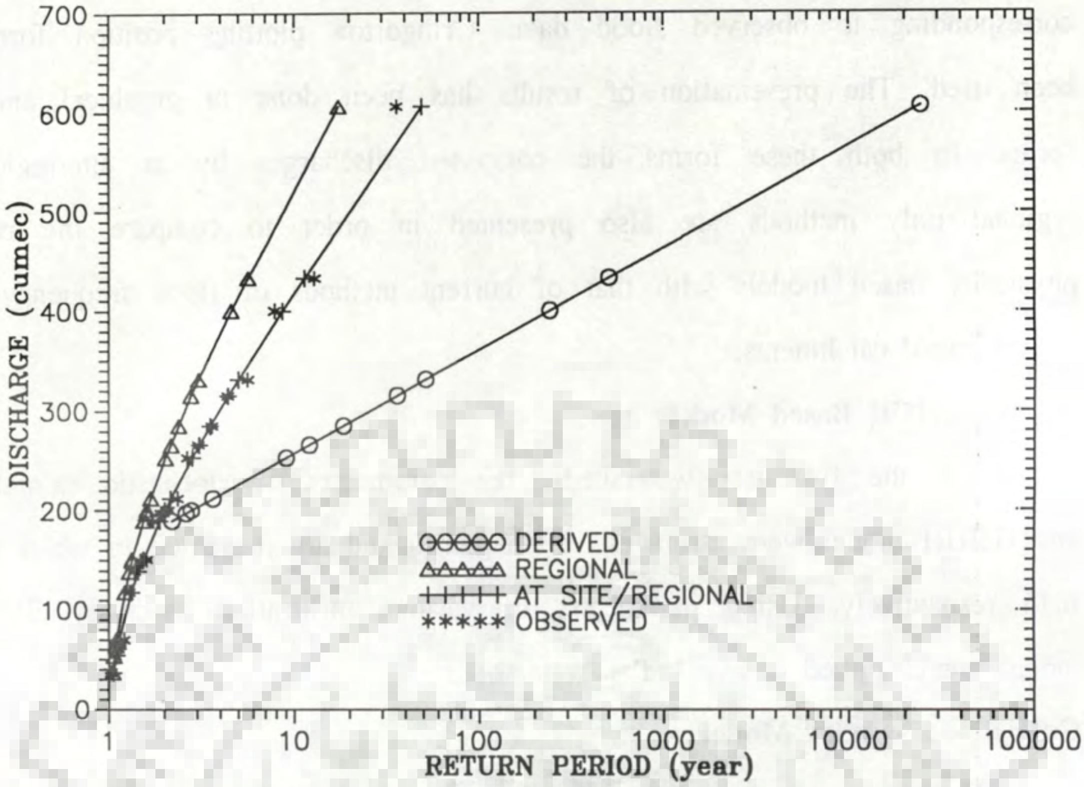

FIG. 7.3–COMPARISON OF FLOOD FREQUENCIES AT TAIRHIA WATERSHED (GcIUH–PHI).

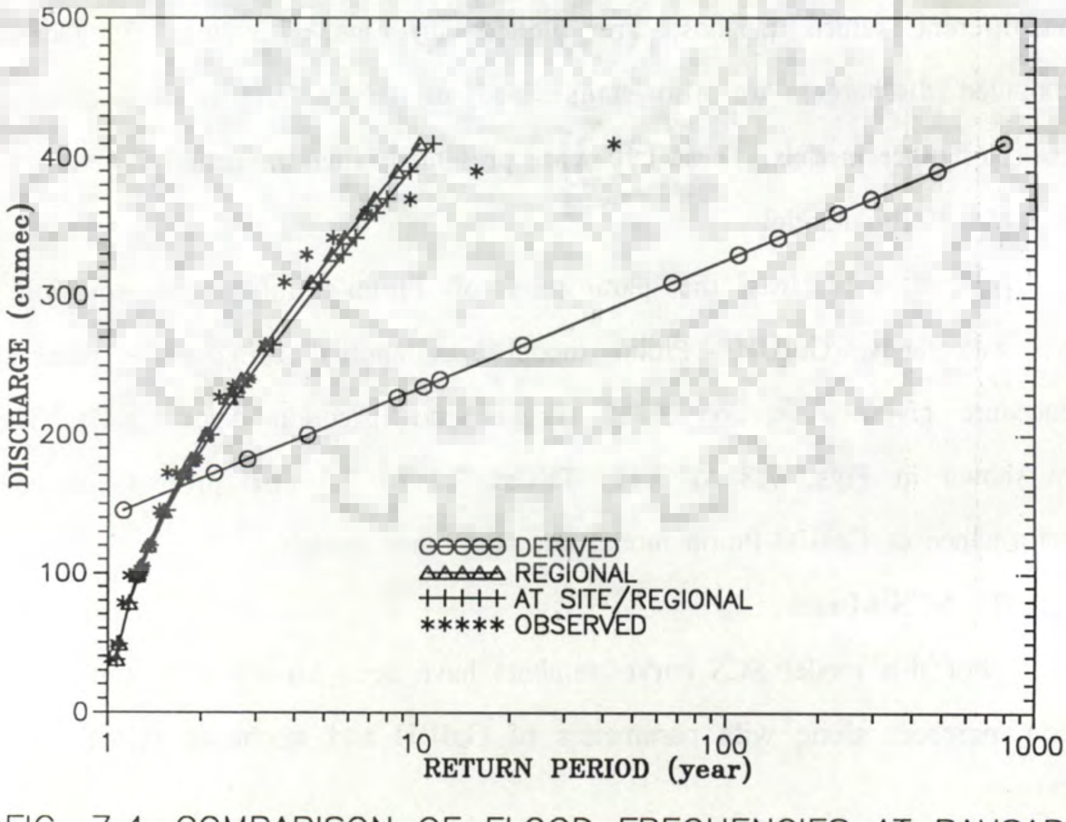

FIG. 7.4–COMPARISON OF FLOOD FREQUENCIES AT PAUSAR WATERSHED (GcIUH–PHI).

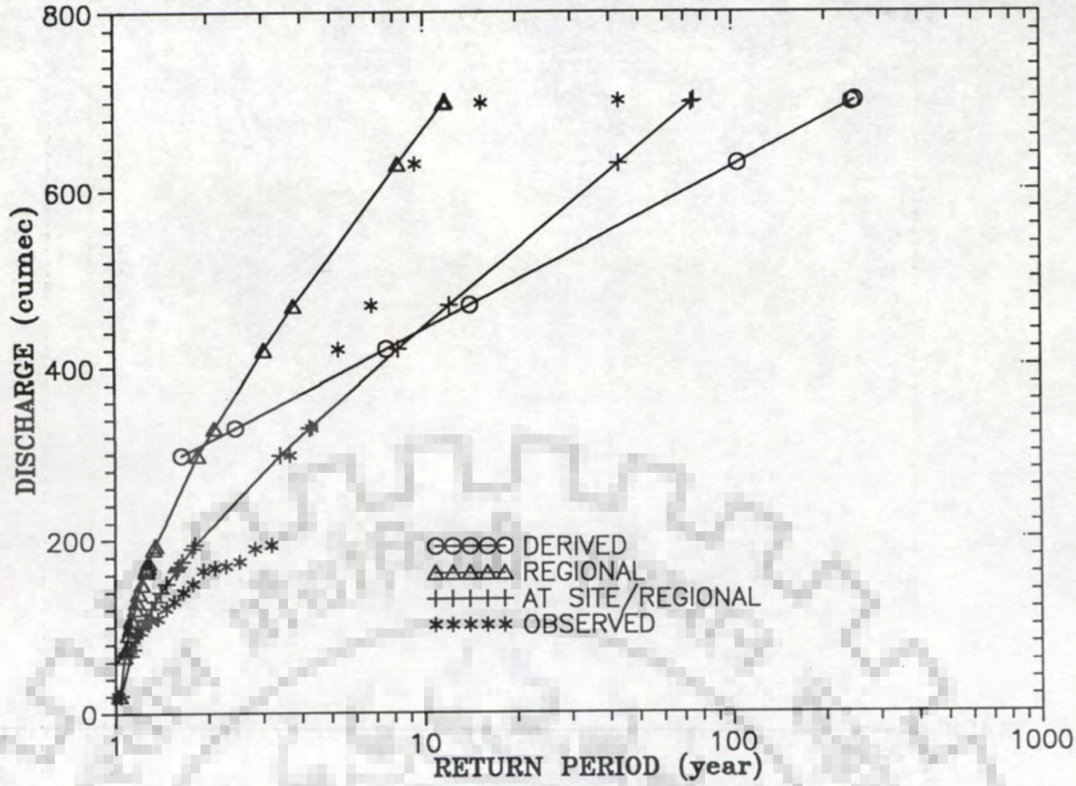

FIG. 7.5–COMPARISON OF FLOOD FREQUENCIES AT LAKHORA WATERSHED (GcIUH–PHI).

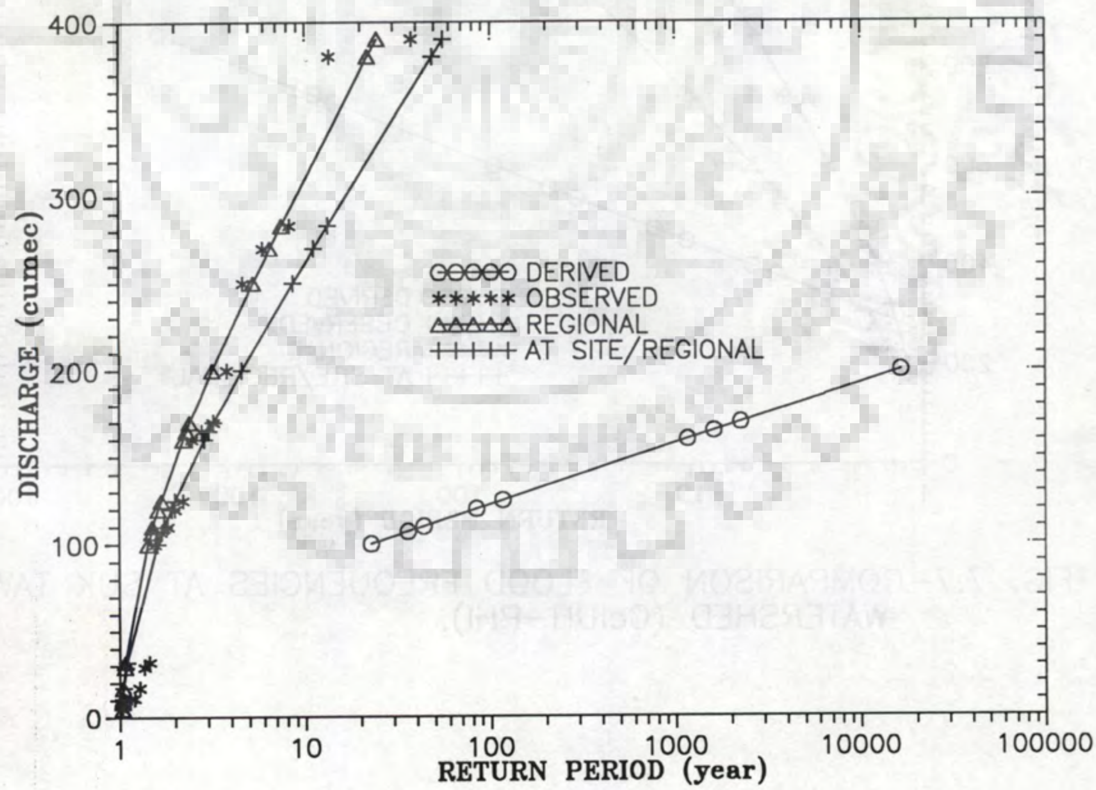

FIG. 7.6–COMPARISON OF FLOOD FREQUENCIES AT KHARANALA WATERSHED (GcIUH–PHI).

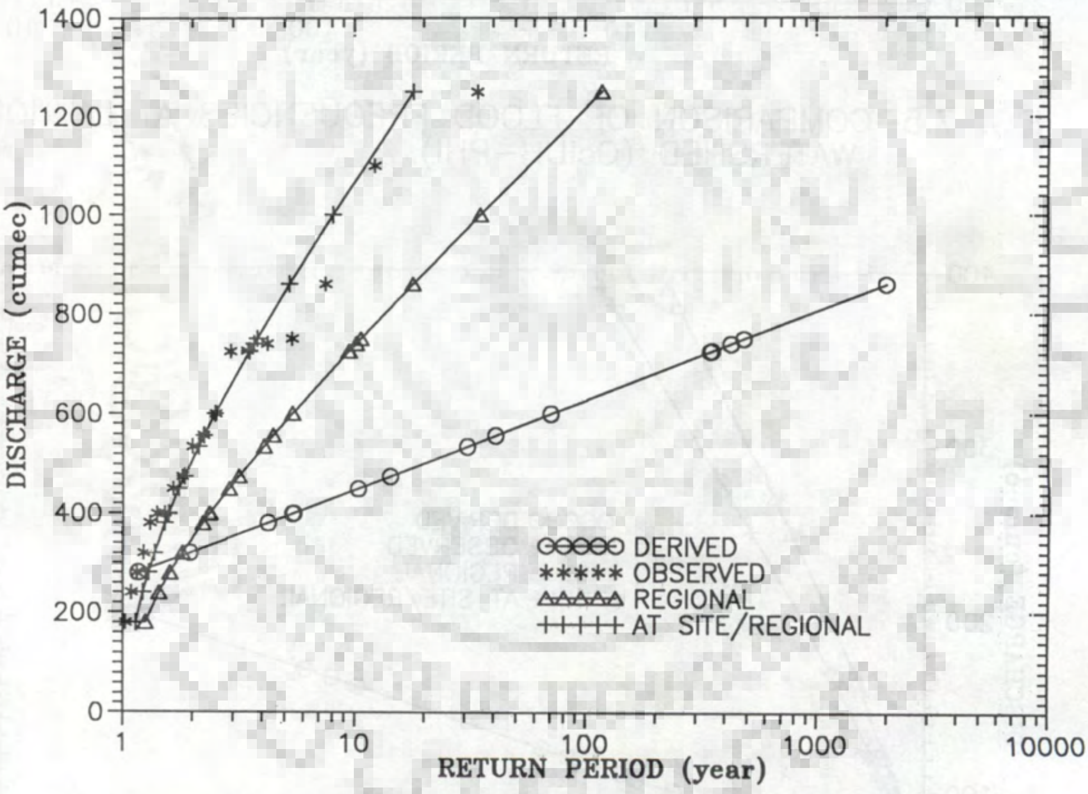

FIG. 7.7–COMPARISON OF FLOOD FREQUENCIES AT SUK TAWA WATERSHED (GcIUH–PHI).

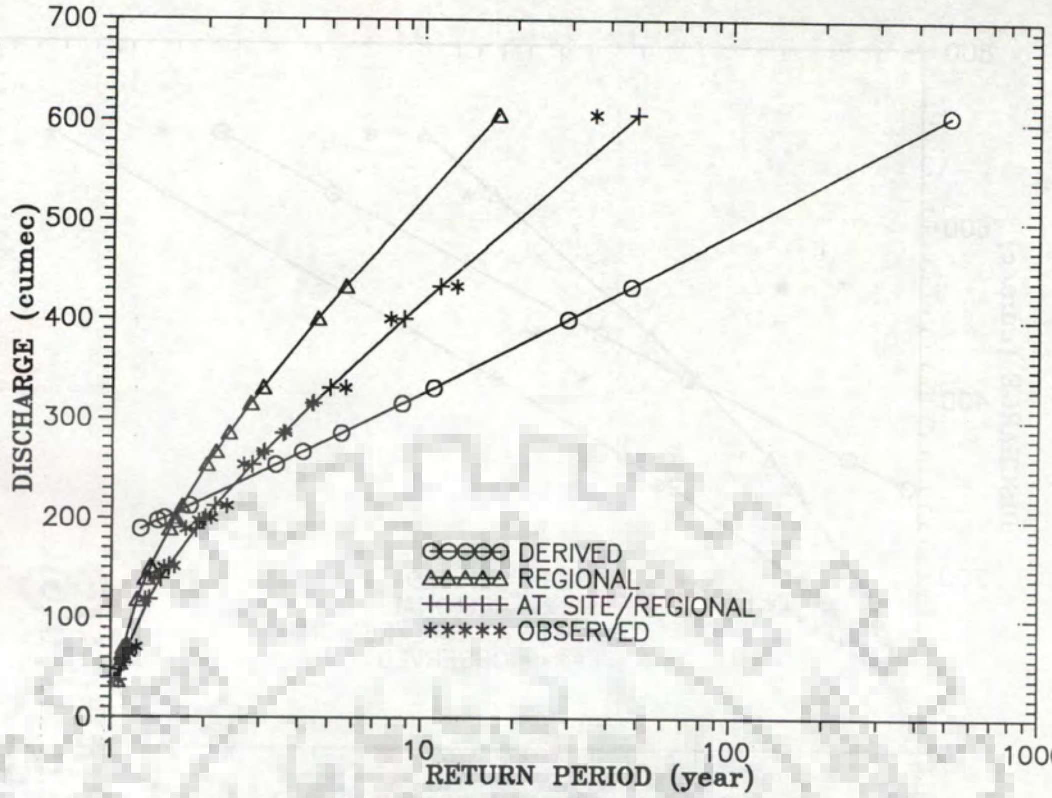

FIG. 7.8–COMPARISON OF FLOOD FREQUENCIES AT TAIRHIA WATERSHED (GcIUH–PHILIP).

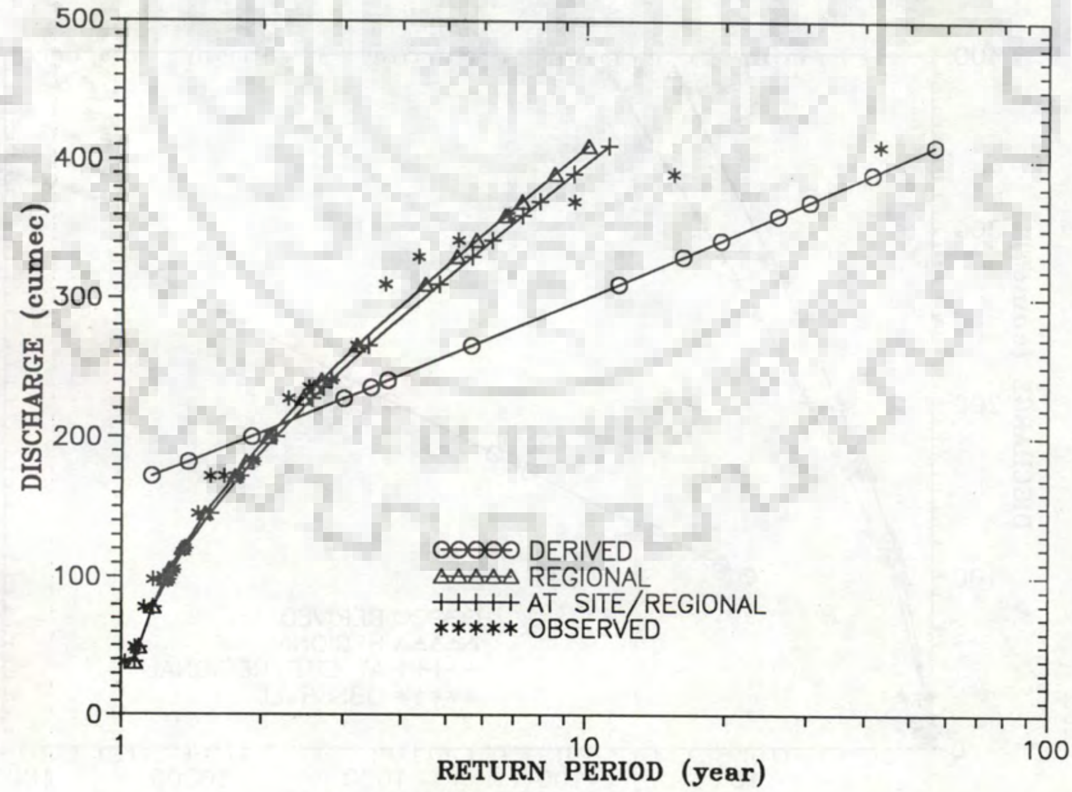

FIG. 7.9–COMPARISON OF FLOOD FREQUENCIES AT PAUSAR WATERSHED (GcIUH–PHILIP).

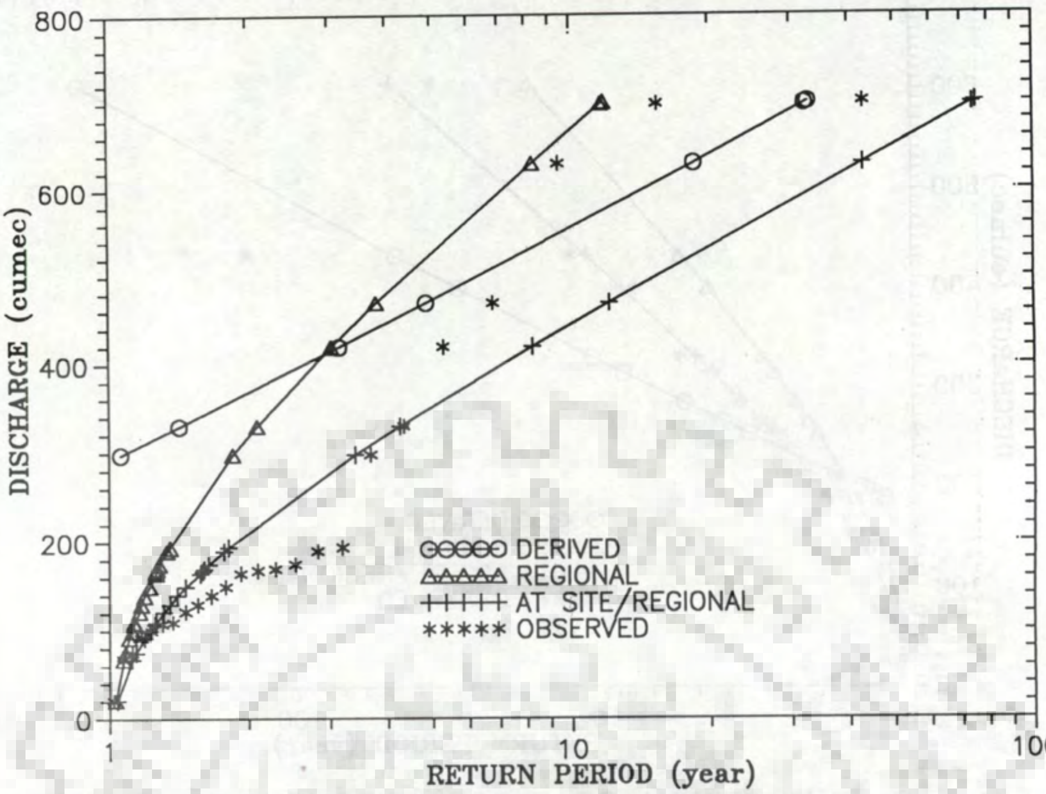

FIG. 7.10–COMPARISON OF FLOOD FREQUENCIES AT LAKHORA WATERSHED (GcIUH–PHILIP).

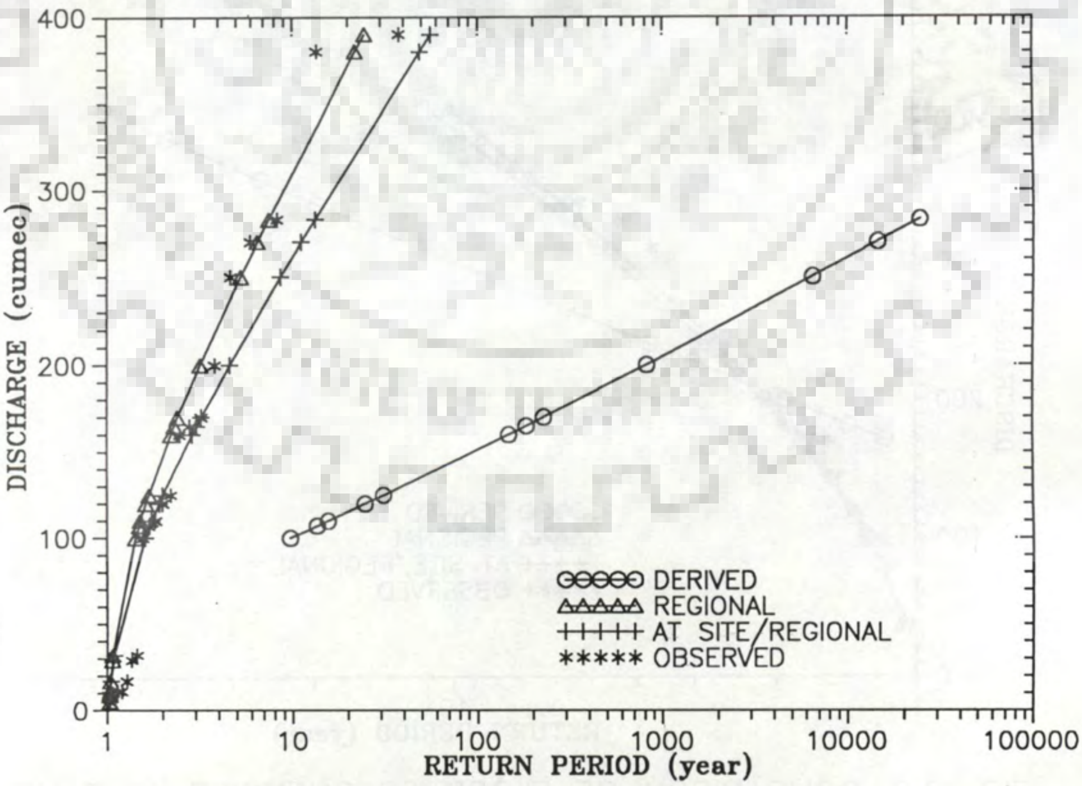

FIG. 7.11–COMPARISON OF FLOOD FREQUENCIES AT KHARANALA WATERSHED (GcIUH–PHILIP).

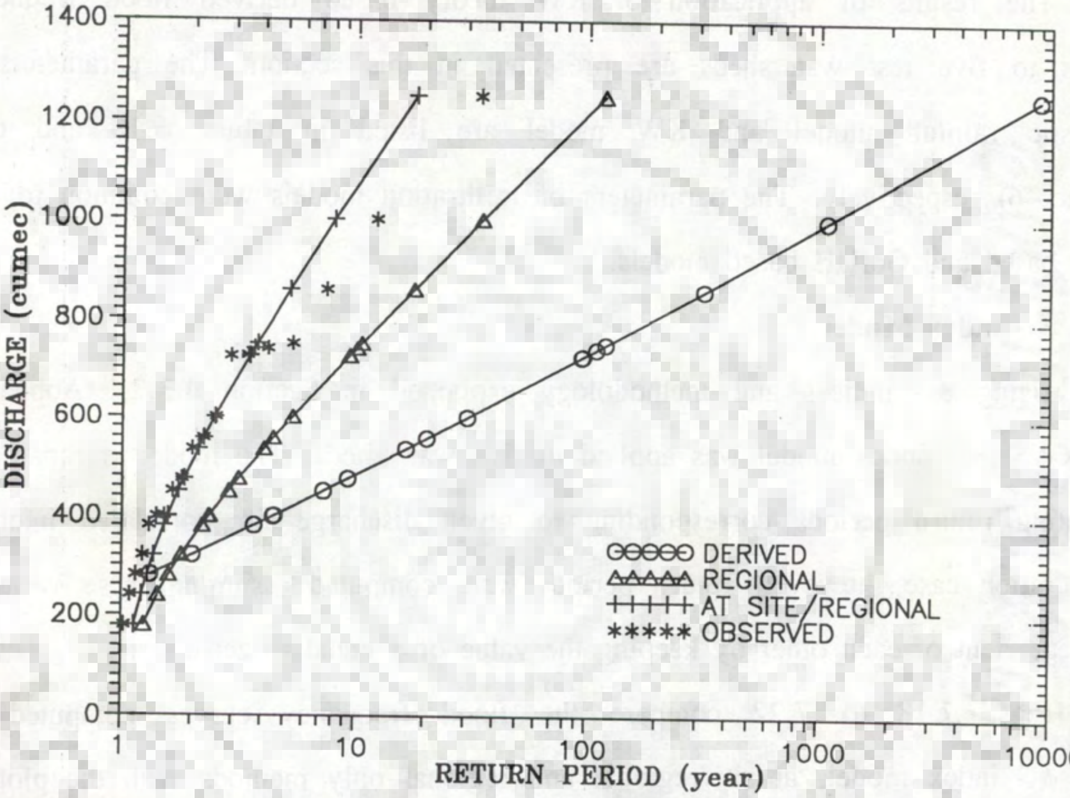

FIG. 7.12–COMPARISON OF FLOOD FREQUENCIES AT SUK TAWA WATERSHED (GcIUH–PHILIP).

used to apply this model. The methodology explained in section 3.4.3 (Appendix-III lists the programme) was used to apply GcIUH - SCS model. Figs. 7.13 to 7.17 illustrate the computed return periods corresponding to observed discharges. The results are also presented in tables 7.1 to 7.5 along with the results of other models.

7.3.2 KW Theory Based Models

The results of application of KW theory based derived flood frequency models to five test watersheds are presented in this section. The parameters of stochastic rainfall model and KW model are listed in tables 6.10 and 6.16 (Chapter 6) respectively. The parameters of infiltration models were common to KW theory based and GcIUH based models.

KW - ϕ - index Model

Using ϕ - indices and methodology explained in Section 4.3.2, (Appendix-VI), KW - ϕ - index model was applied to five watersheds. The model is capable of computing return periods corresponding to given discharge for correlated intensity and duration case, however, return periods were computed assuming these variables as independent of each other by keeping the value of γ equal to zero.

Figs. 7.18 to 7.22 compare the flood frequency curves computed by KW - ϕ - index model, at site/regional and regional only methods with the plotting positions obtained from historical flood series. The results are also compared in Tables 7.1 to 7.5 with the other models.

KW - Philip Model

Table 6.12 (Chapter 6) lists the parameters of Philip's infiltration equation. Using these parameters and methodology explained in section 3.4.4 (programme listed in Appendix-IV), KW - Philip model was applied to five watersheds.

The flood quantiles corresponding to different return periods, computed by

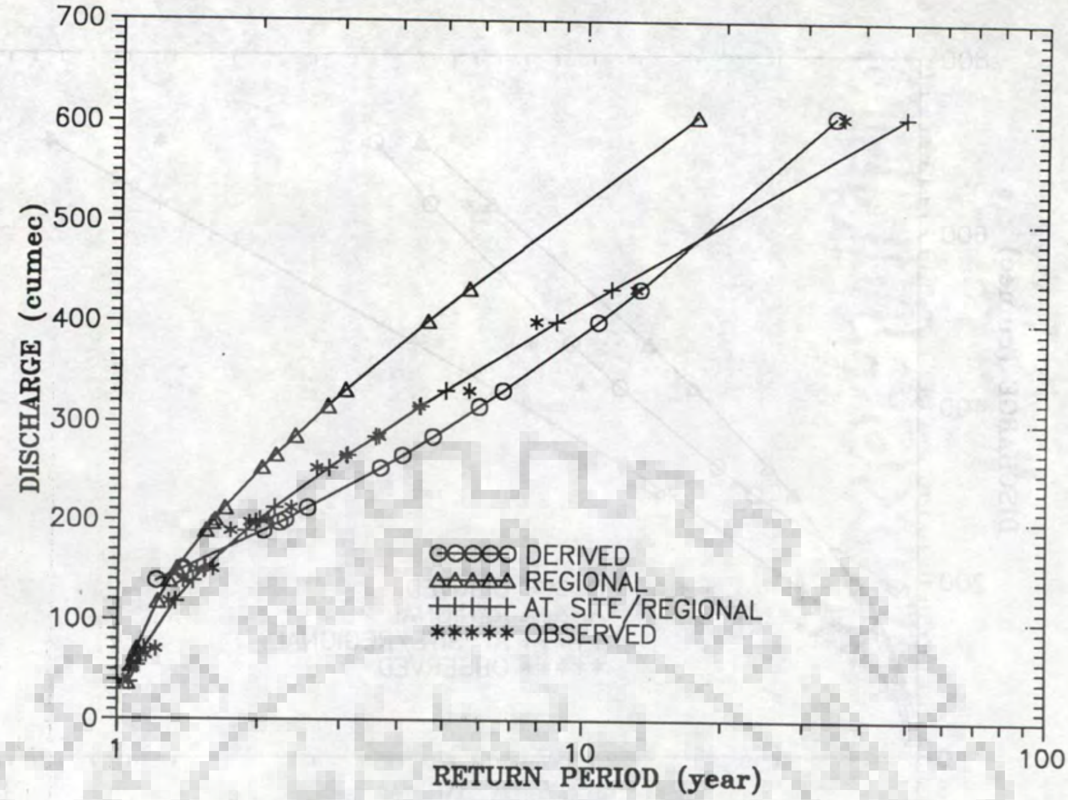

FIG. 7.13—COMPARISON OF FLOOD FREQUENCIES AT TAIRHIA WATERSHED (GcIUH—SCS).

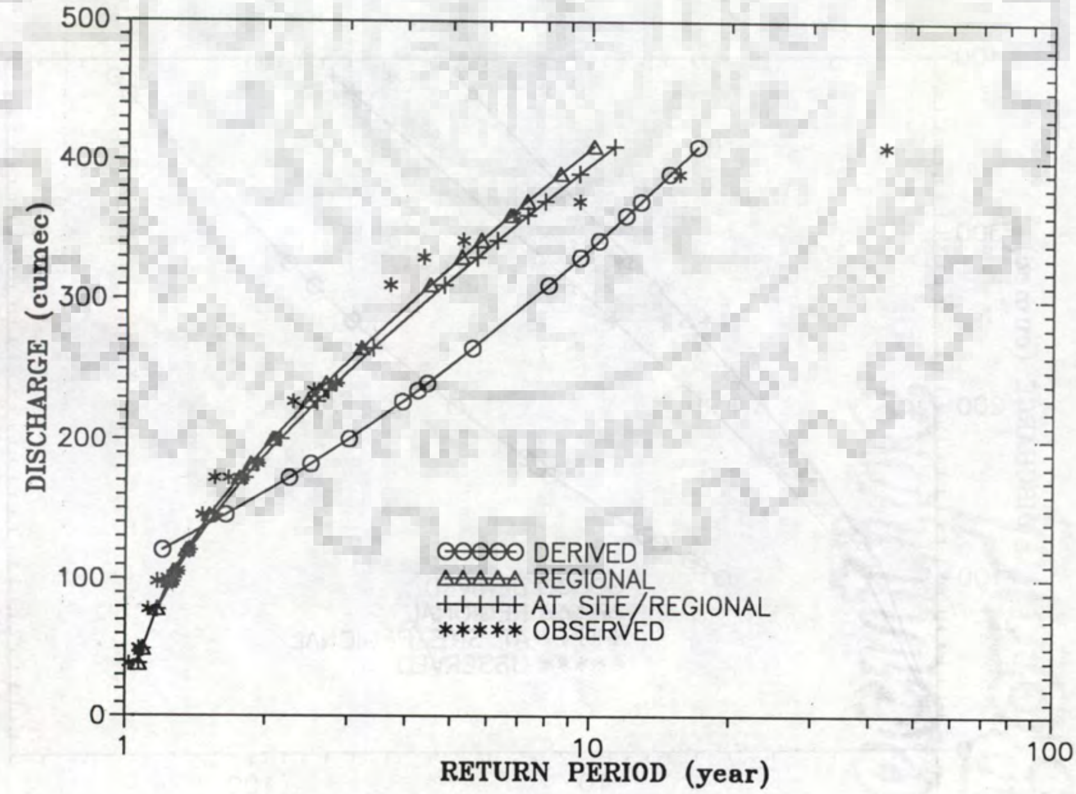

FIG. 7.14—COMPARISON OF FLOOD FREQUENCIES AT PAUSAR WATERSHED (GcIUH—SCS).

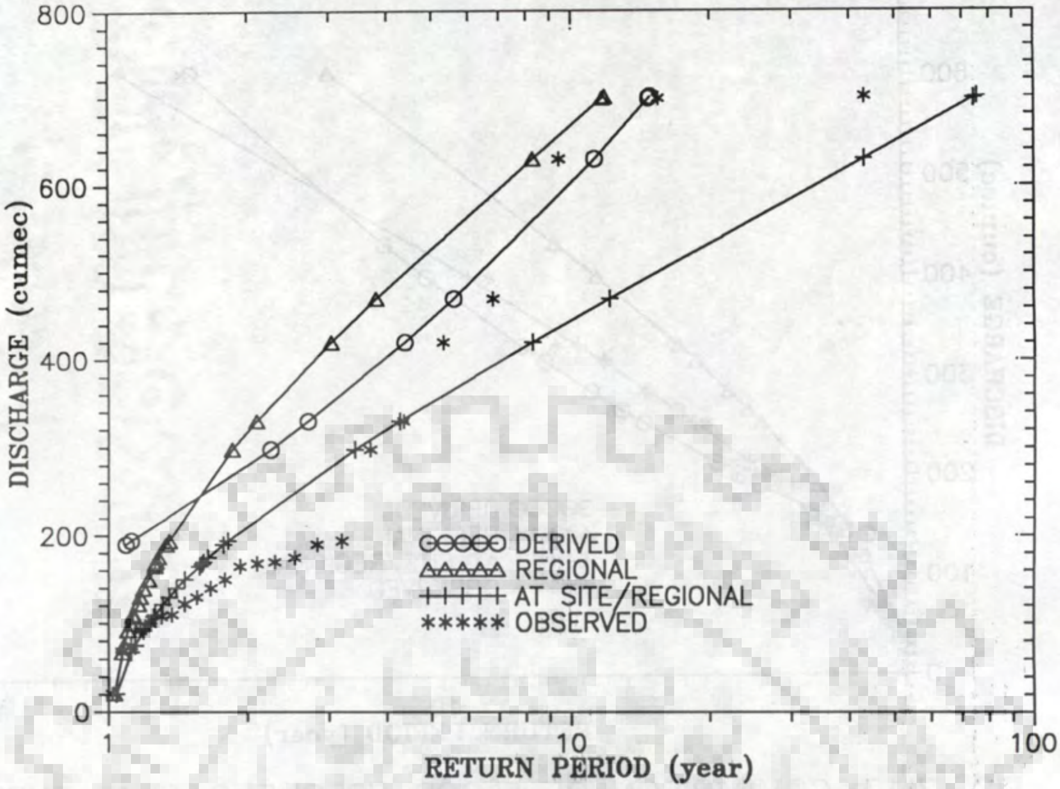

FIG. 7.15–COMPARISON OF FLOOD FREQUENCIES AT LAKHORA WATERSHED (GcIUH–SCS).

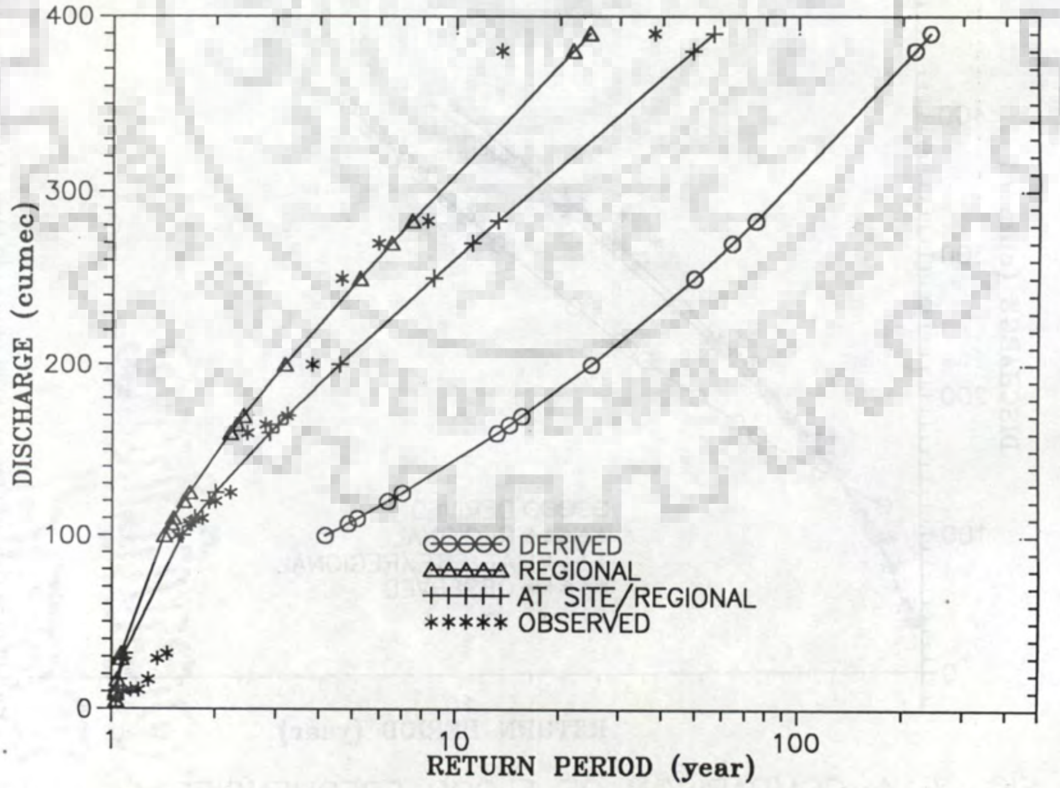

FIG. 7.16–COMPARISON OF FLOOD FREQUENCIES AT KHARANALA WATERSHED (GcIUH–SCS).

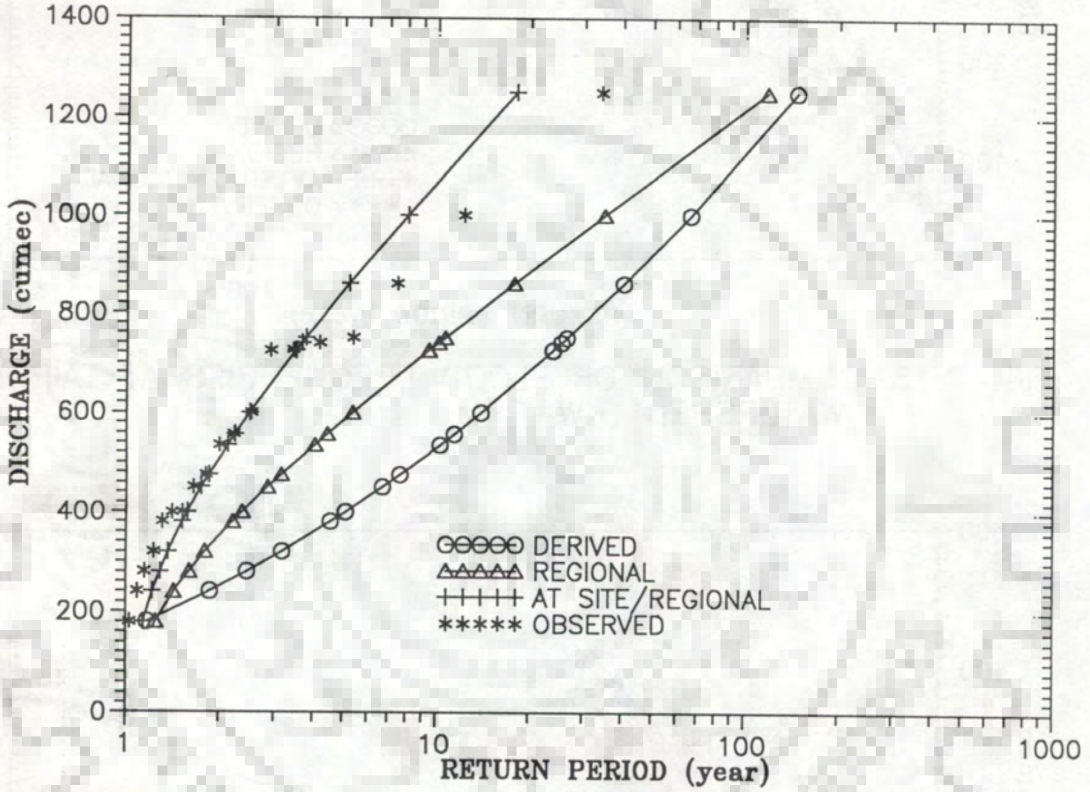

FIG. 7.17—COMPARISON OF FLOOD FREQUENCIES AT SUKTAWA WATERSHED (GcIUH—SCS).

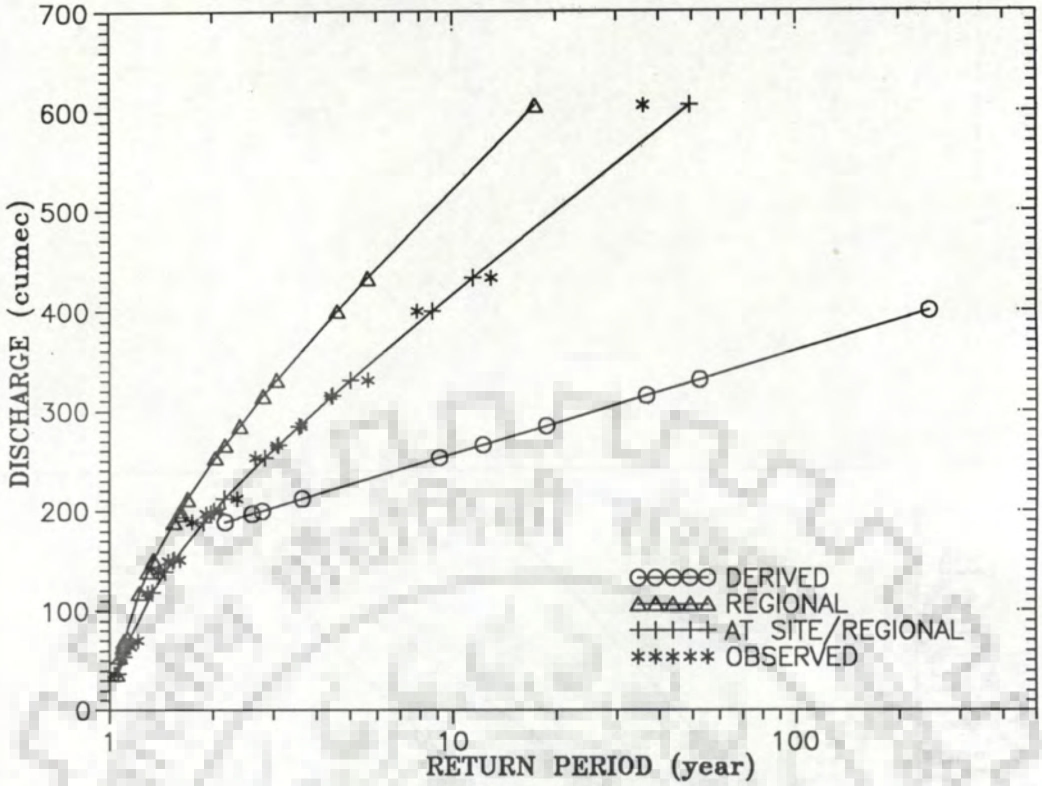

FIG. 7.18—COMPARISON OF FLOOD FREQUENCIES AT TAIRHIA WATERSHED (KW—PHI).

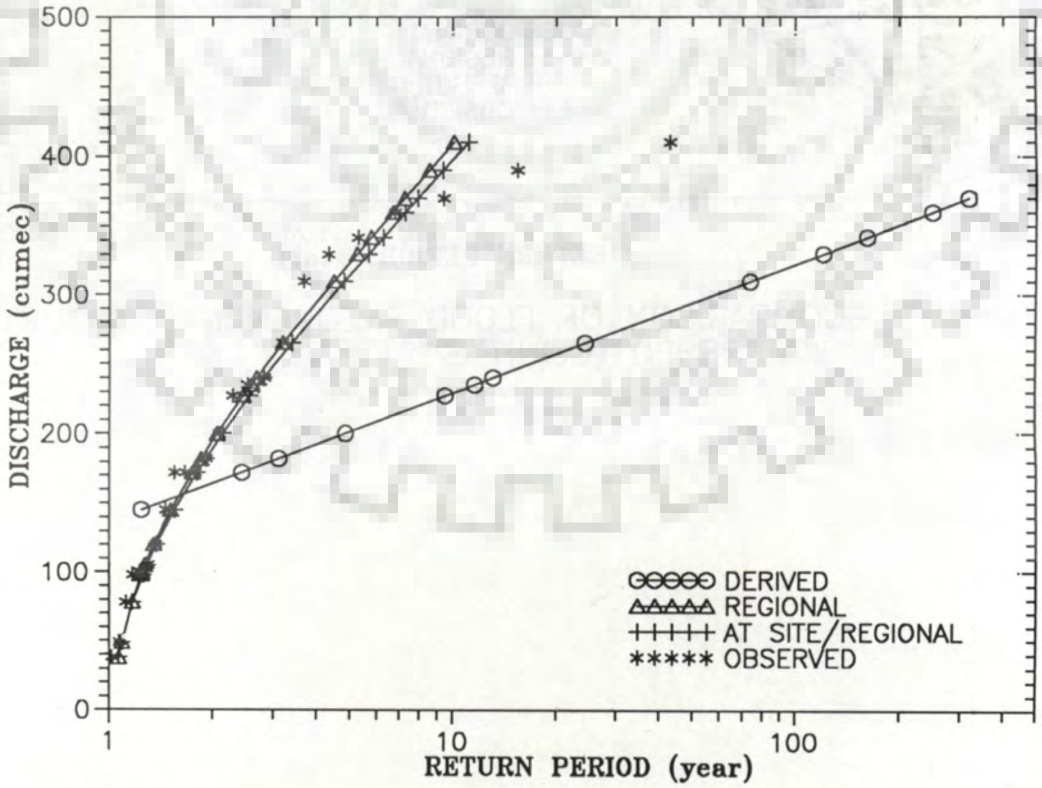

FIG. 7.19—COMPARISON OF FLOOD FREQUENCIES AT PAUSAR WATERSHED (KW—PHI).

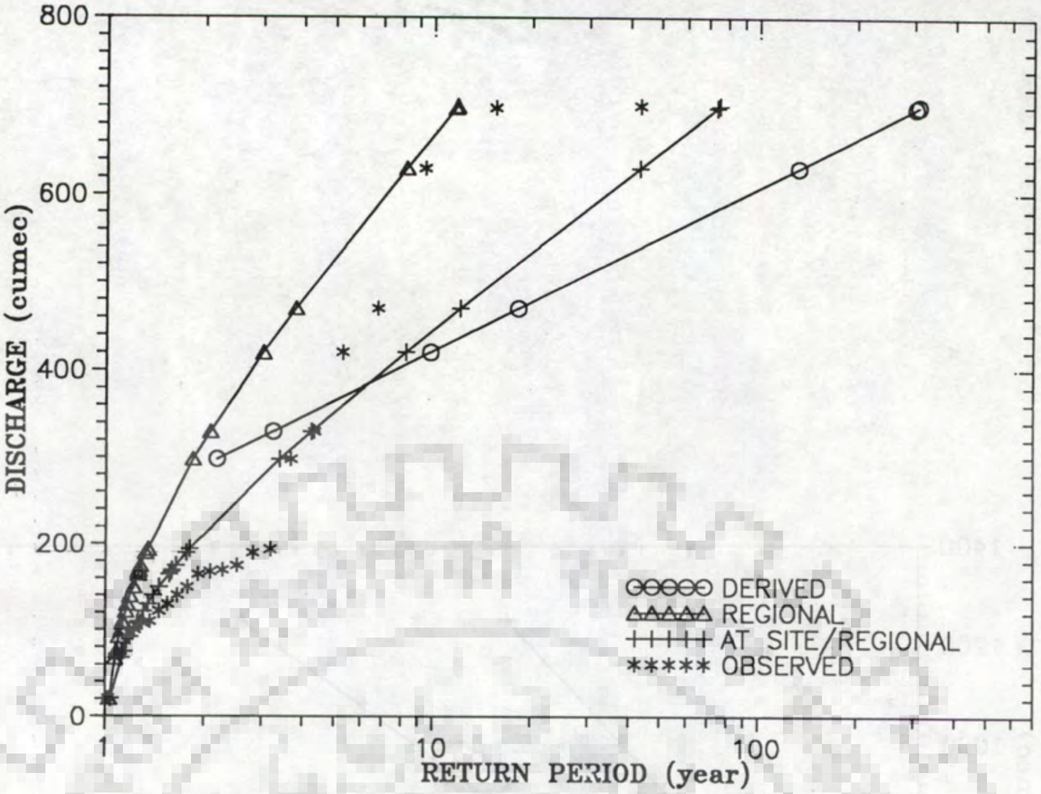

FIG. 7.20–COMPARISON OF FLOOD FREQUENCIES AT LAKHORA WATERSHED (KW–PHI).

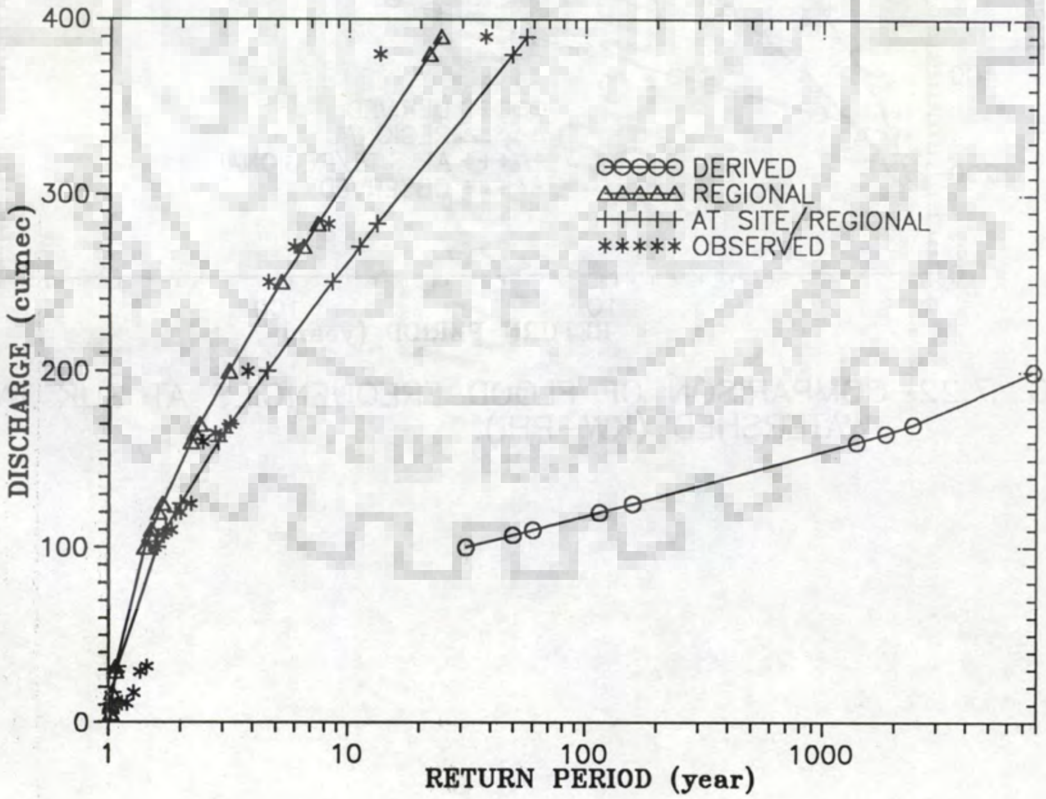

FIG. 7.21–COMPARISON OF FLOOD FREQUENCIES AT KHARANALA WATERSHED (KW–PHI).

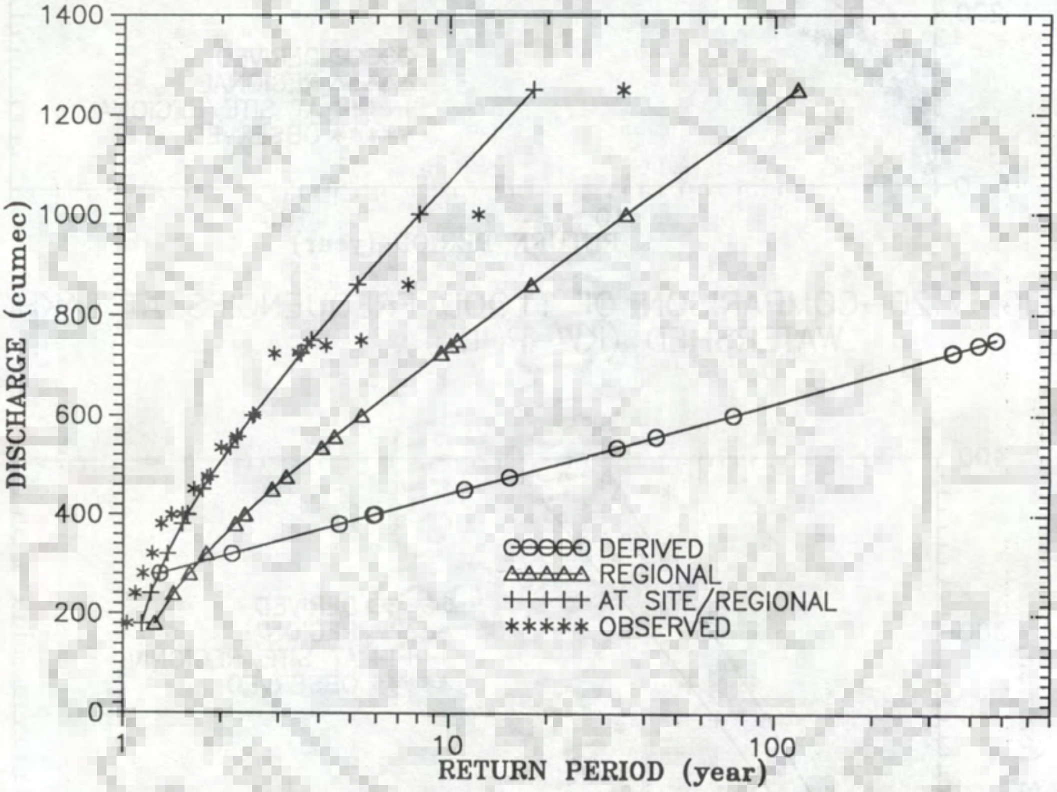

FIG. 7.22—COMPARISON OF FLOOD FREQUENCIES AT SUK TAWA WATERSHED (KW—PHI).

KW - Philip model are plotted in Figs. 7.23 to 7.27. Observed flood series, for the five test watersheds is also illustrated in these figures. The results are also given in tables 7.1 to 7.5.

KW - SCS Model

The SCS curve numbers estimated in Chapter 6 have been presented in Table 6.13. These curve numbers and the methodology explained in section 3.4.5 (Appendix-V) was used to apply KW-SCS model.

The flood frequency curves, computed by KW - SCS model are illustrated in Figs. 7.28 to 7.32. The observed flood series is also plotted in these figures. The results are also presented in tables 7.1 to 7.5 along with the results of other models.

7.4 DISCUSSION OF RESULTS

The results of application of DFFD models and at site/regional and regional only analyses are summarised in tables 7.1 to 7.5, for the five watersheds.

Gringorton plotting position formula has been used to compute the return periods corresponding to the observed annual maximum discharges for the five watersheds. Other formula could also be used. Though this approach is not perfect and plotting position estimates at the upper end of the flood frequency curve are highly uncertain (as can be seen for the first and second quantiles of Lakhora (700 and 698), Pausar (410 and 390) and Kharanala (390 and 380) watersheds). This was done in order to evaluate the performance of various models. It should be quite clear that the standard Q Vs T relationship is not known and the performance can be evaluated only qualitatively.

7.4.1 Comparison Criteria

There are number of ways to evaluate the performance of a DFFD model. In the present study, observed and computed discharges by various models have been compared on the basis of following criteria:

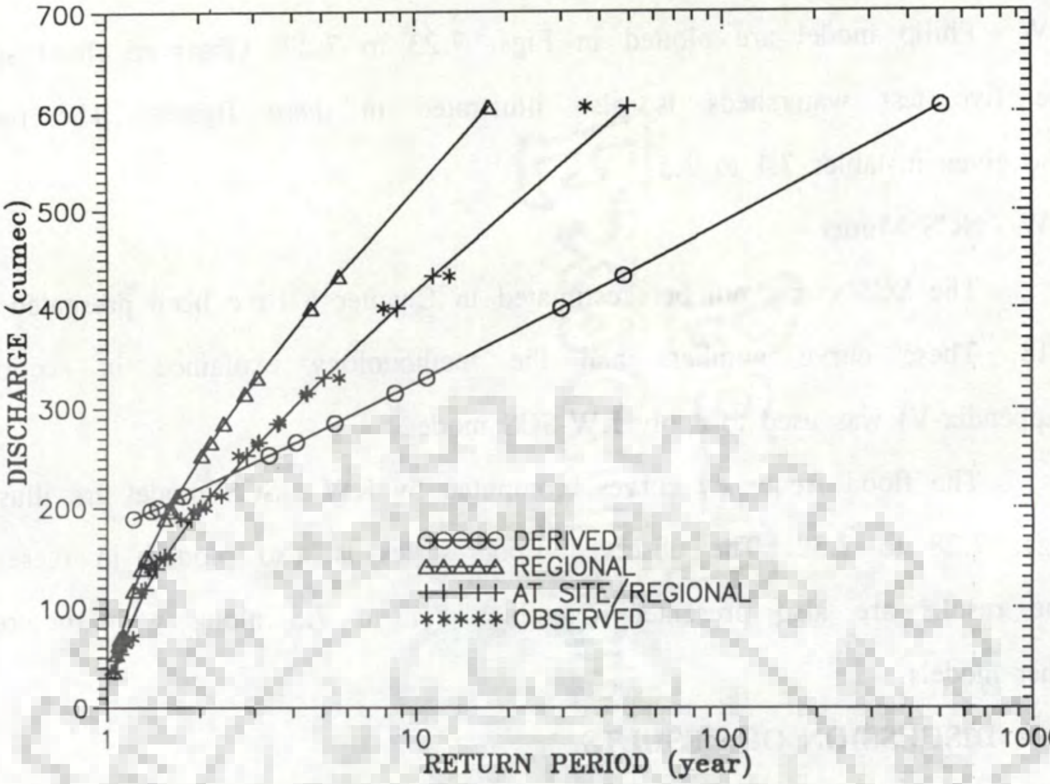

FIG. 7.23–COMPARISON OF FLOOD FREQUENCIES AT TAIRHIA WATERSHED (KW–PHILIP).

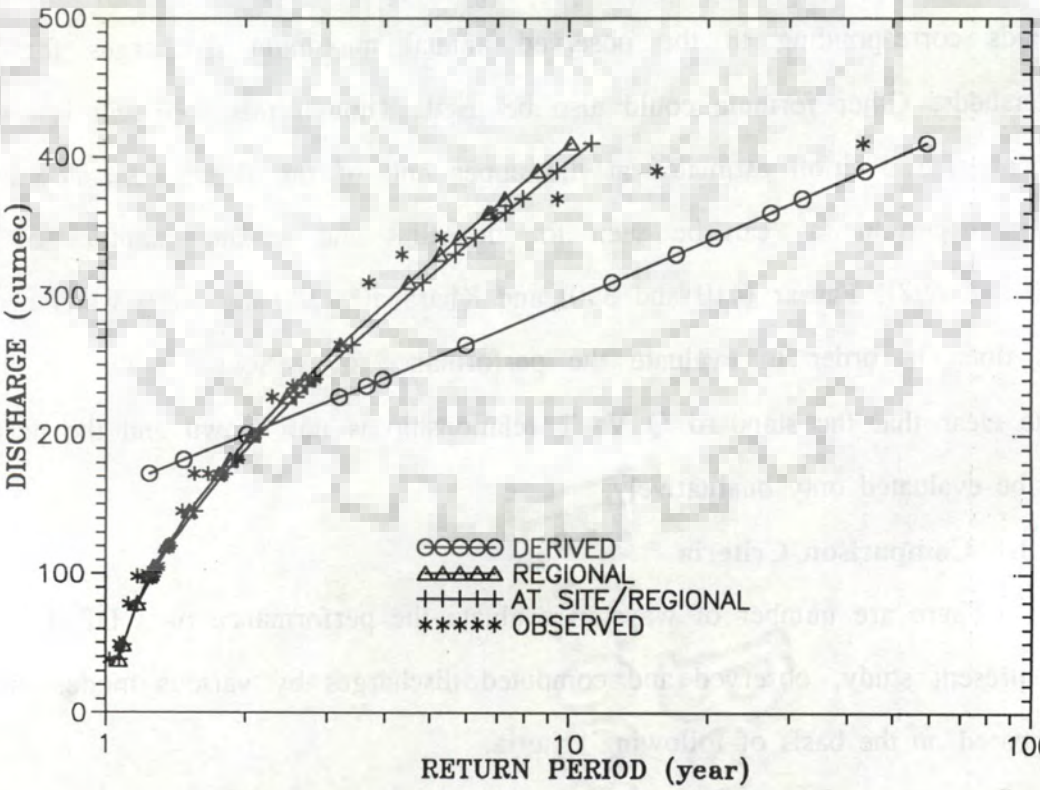

FIG. 7.24–COMPARISON OF FLOOD FREQUENCIES AT PAUSAR WATERSHED (KW–PHILIP).

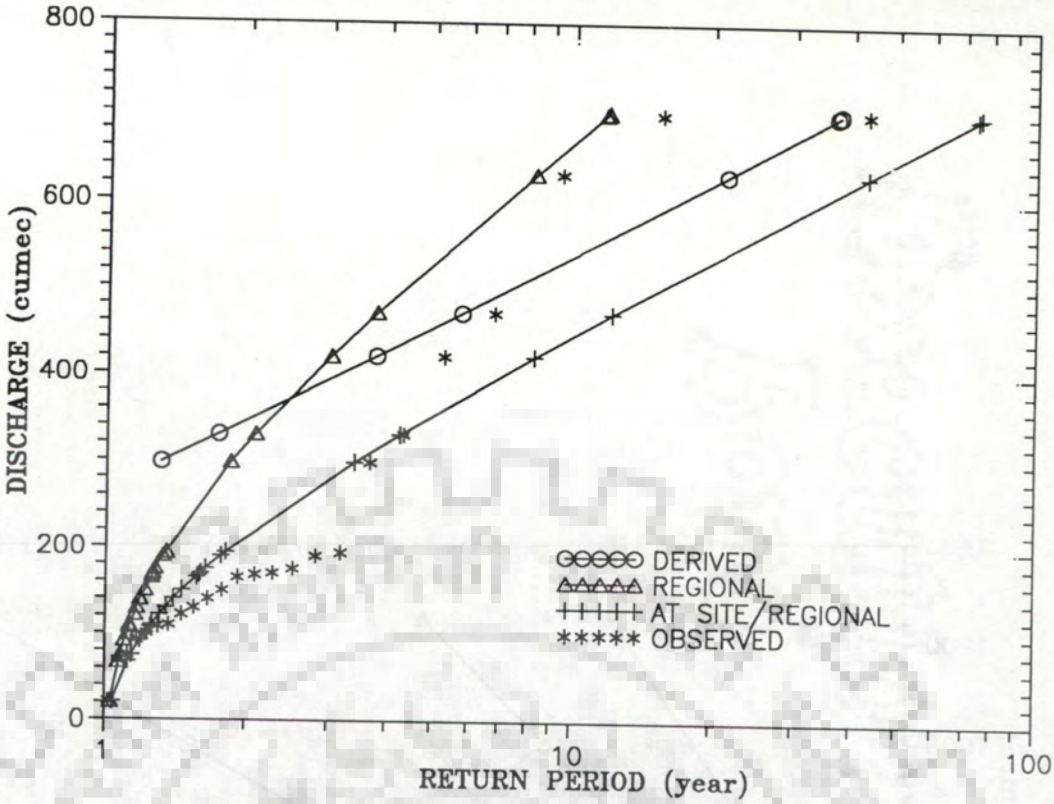

FIG. 7.25–COMPARISON OF FLOOD FREQUENCIES AT LAKHORA WATERSHED (KW–PHILIP).

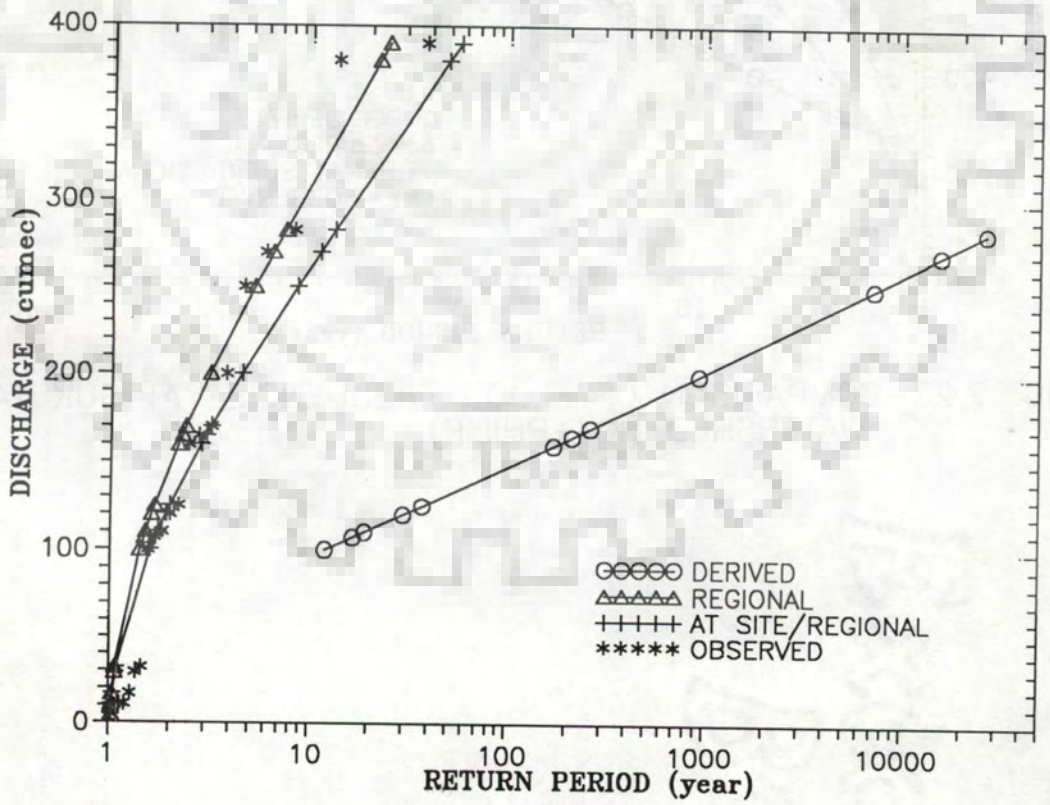

FIG. 7.26–COMPARISON OF FLOOD FREQUENCIES AT KHARANALA WATERSHED (KW–PHILIP).

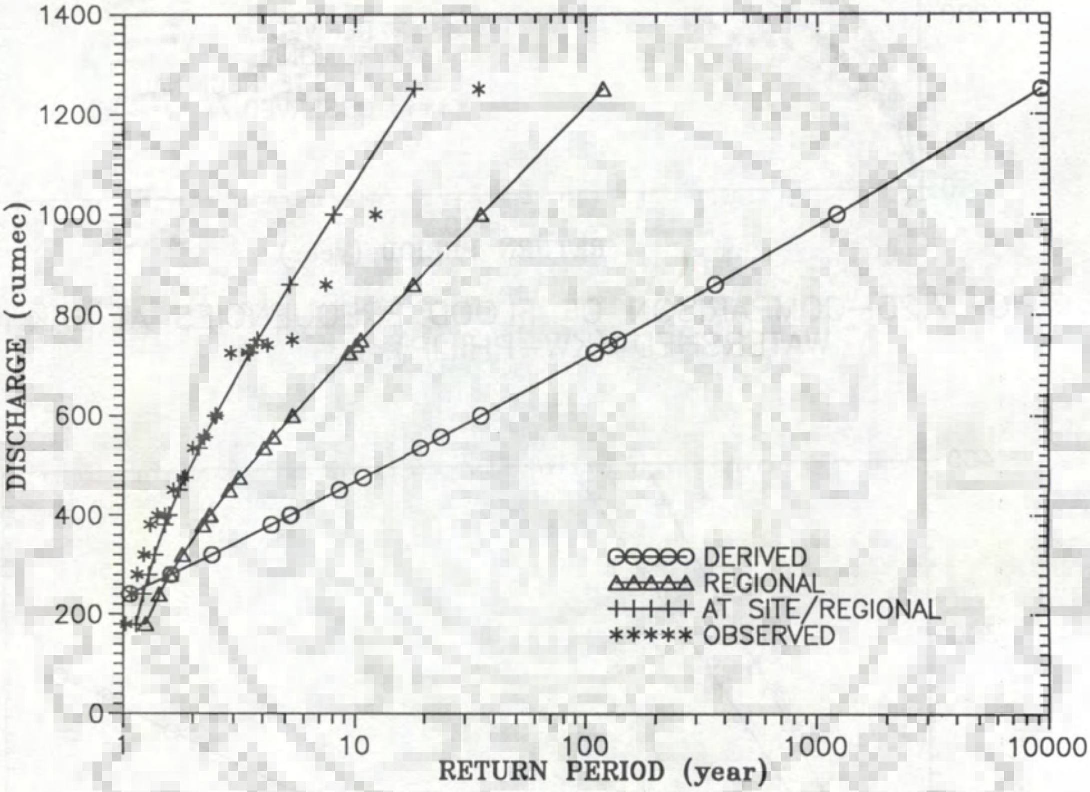

FIG. 7.27—COMPARISON OF FLOOD FREQUENCIES AT SUK TAWA WATERSHED (KW—PHILIP).

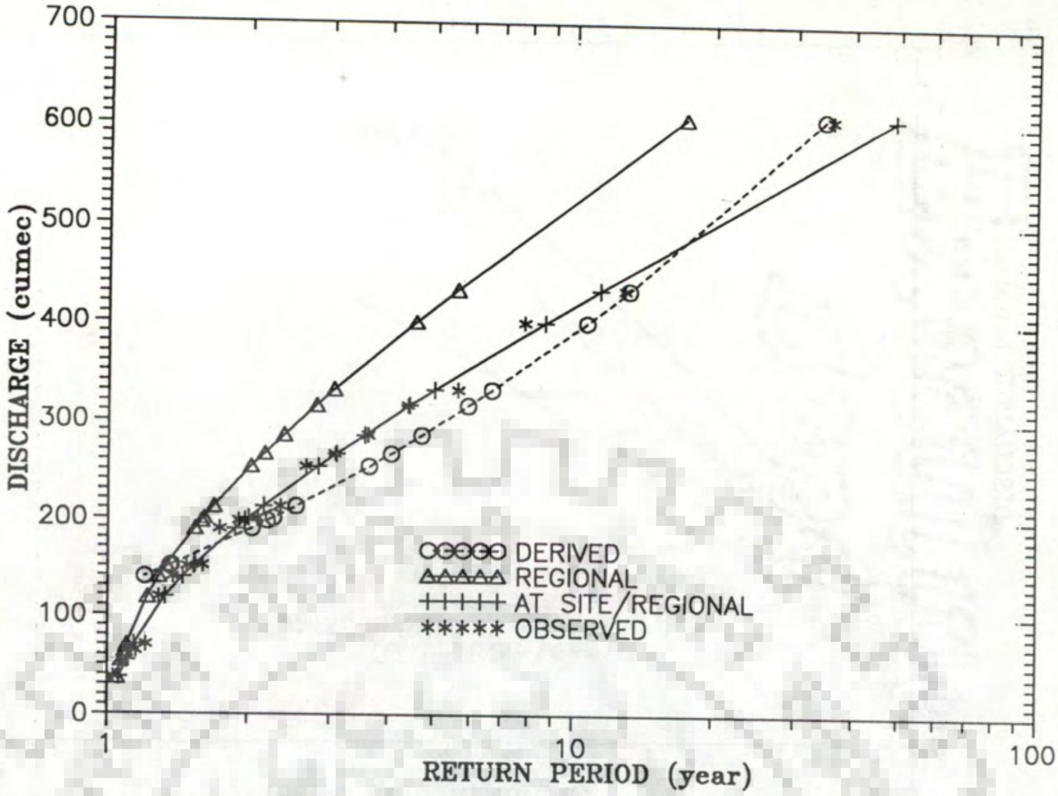

FIG. 7.28–COMPARISON OF FLOOD FREQUENCIES AT TAIRHIA WATERSHED (KW–SCS).

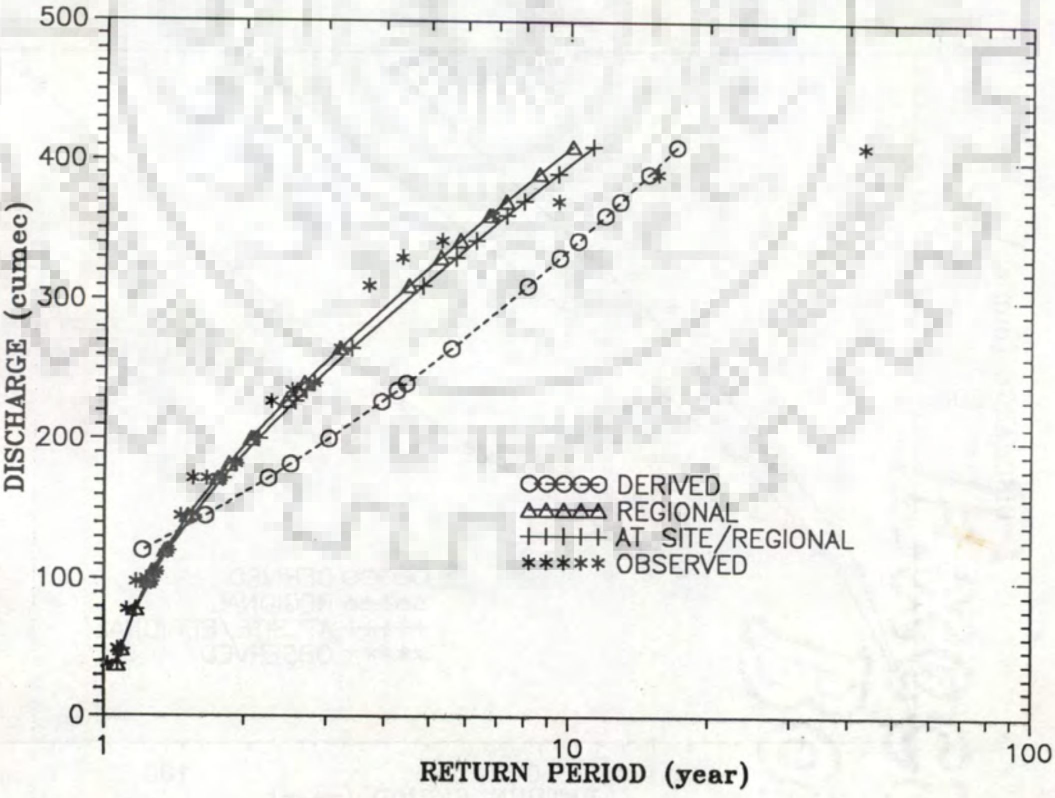

FIG. 7.29–COMPARISON OF FLOOD FREQUENCIES AT PAUSAR WATERSHED (KW–SCS).

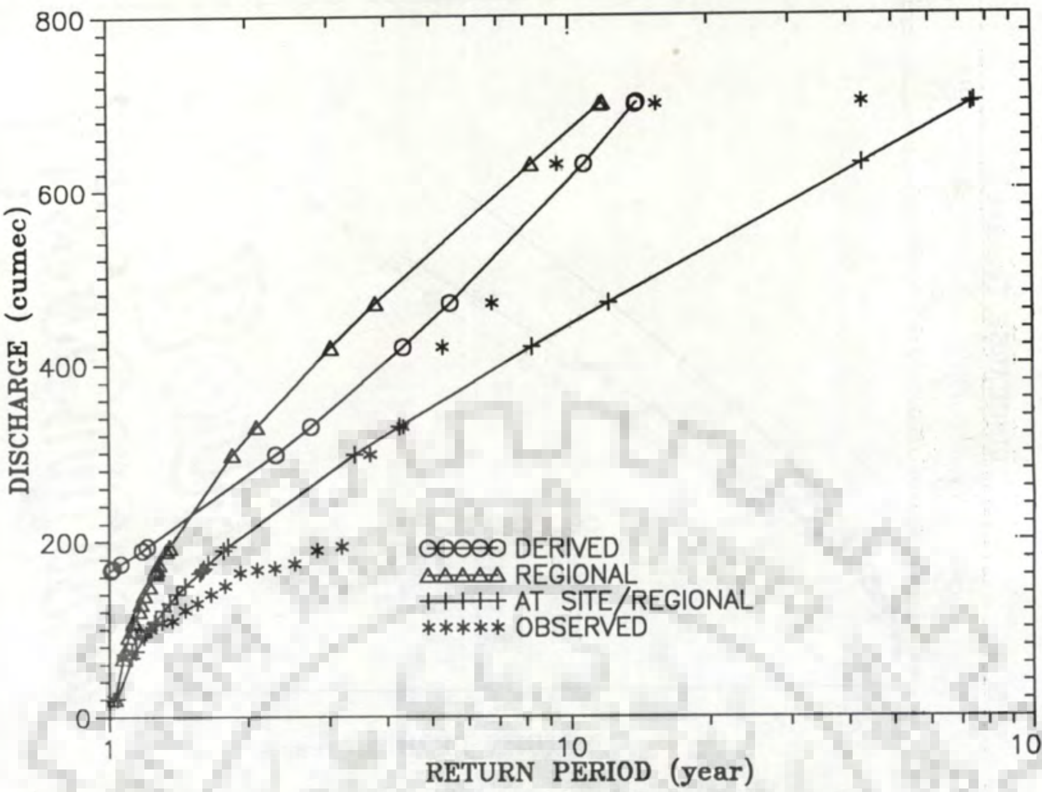

FIG. 7.30–COMPARISON OF FLOOD FREQUENCIES AT LAKHORA WATERSHED (KW–SCS).

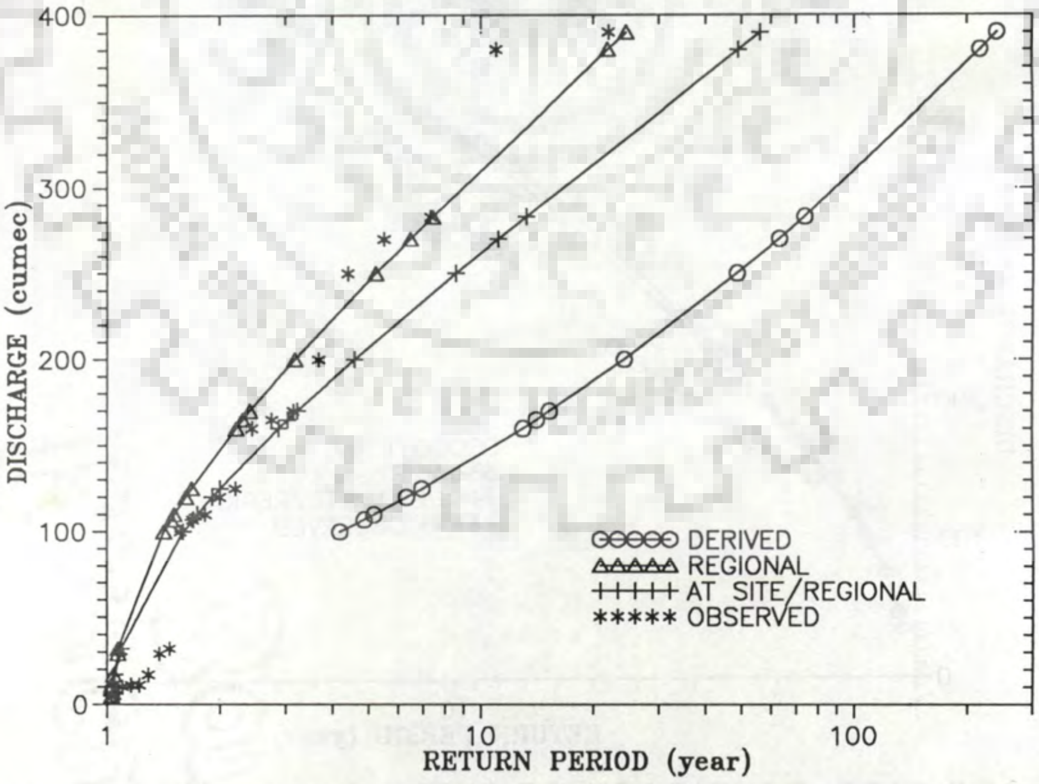

FIG. 7.31–COMPARISON OF FLOOD FREQUENCIES AT KHARANALA WATERSHED (KW–SCS).

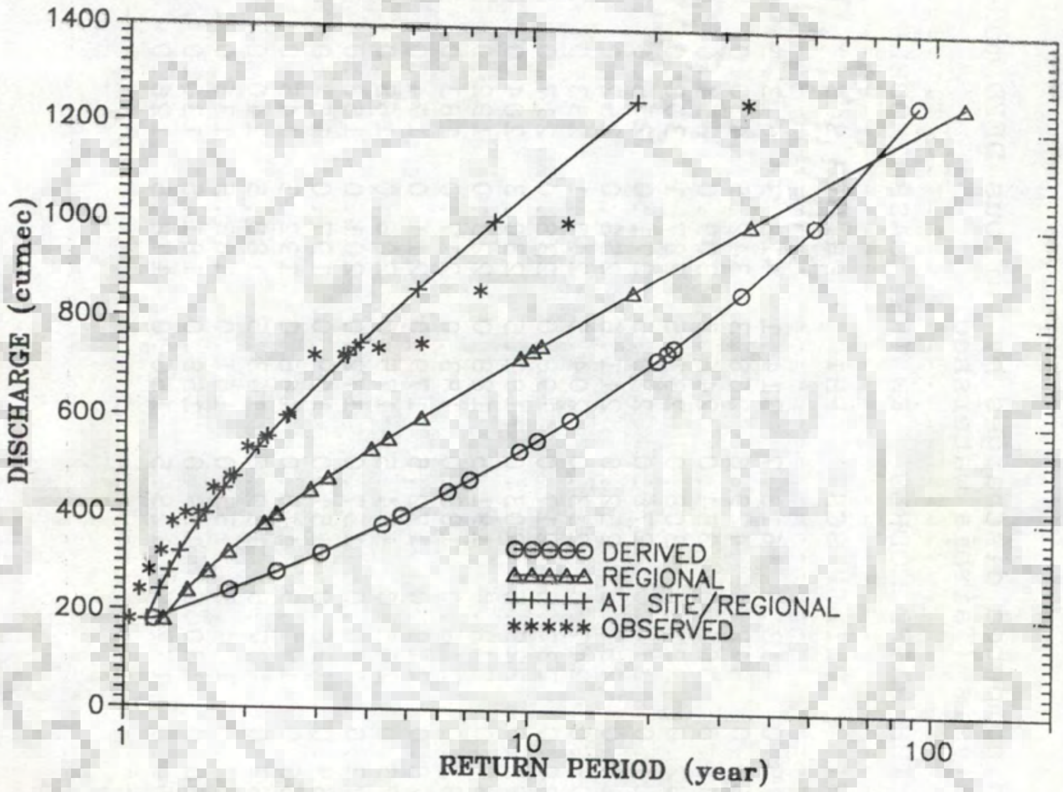

FIG. 7.32—COMPARISON OF FLOOD FREQUENCIES AT SUKTAWA WATERSHED (KW—SCS).

Table 7.1 Observed and computed discharges by various DFFD models and regional analysis (Tairhia watershed)

S.NO.	T	DISCH.	GcIUH BASED MODELS			KW THE. BASED MODELS			AT SITE REG.	REG. ONLY
			PHI	PHILIP	SCS	PHI	PHILIP	SCS		
1	35.9	606.0	315.0	413.4	613.7	314.1	413.3	613.5	569.1	713.2
2	12.9	433.0	269.0	342.4	429.6	268.3	342.5	430.0	447.7	561.0
3	7.9	400.0	247.0	308.8	354.0	246.3	309.0	354.0	387.3	485.3
4	5.7	331.0	232.0	287.0	308.0	231.5	287.1	308.0	345.9	433.4
5	4.4	315.0	221.0	270.0	276.0	220.5	271.0	275.6	313.8	393.2
6	3.6	285.0	212.0	257.0	252.0	211.6	258.0	252.0	287.2	359.9
7	3.1	266.0	205.0	247.0	233.0	204.5	247.1	233.0	264.2	331.1
8	2.7	253.0	198.0	237.5	217.0	198.0	238.0	217.0	243.7	305.4
9	2.4	212.0	193.0	229.5	203.0	192.5	230.3	204.0	225.0	281.9
10	2.1	200.0	187.0	222.0	191.5	188.0	223.0	192.0	207.5	260.0
11	1.9	197.0	183.0	216.0	182.0	183.0	217.0	183.0	190.9	239.2
12	1.7	189.0	178.0	210.0	172.5	179.0	211.0	174.0	174.9	219.2
13	1.6	151.0	175.0	205.0	164.5	175.0	206.0	165.0	159.2	199.5
14	1.5	150.0	172.0	200.0	157.0	172.0	201.0	158.0	143.6	179.9
15	1.4	139.0	169.0	196.0	151.0	169.0	197.0	152.0	127.7	160.0
16	1.3	118.0	166.0	191.5	145.0	166.0	192.5	145.0	111.1	139.2
17	1.2	70.0	162.0	187.5	139.0	163.5	188.5	140.0	93.3	116.9
18	1.1	64.0	162.0	184.5	135.0	161.0	185.5	136.0	73.1	91.6
19	1.1	54.0	158.0	180.5	129.0	158.0	181.5	130.0	48.2	60.4
20	1.0	37.0	154.0	177.5	125.5	156.0	178.5	126.0	9.3	11.6

Table 7.2 Observed and computed discharges by various DFFD models and regional analysis (Pausar watershed)

S.NO.	T	DISCH.	GcIUH BASED MODELS			KW THE. BASED MODELS			AT SITE REG.	REG. ONLY
			PHI	PHILIP	SCS	PHI	PHILIP	SCS		
1	43.1	410.0	292.5	392.5	560.5	288.1	389.3	560.4	562.4	578.9
2	15.5	390.0	251.0	327.2	397.0	246.8	323.9	397.0	447.2	460.4
3	9.4	370.0	231.0	296.3	329.5	226.7	293.0	329.0	390.2	401.7
4	6.8	360.0	217.5	276.0	288.5	213.5	273.0	288.0	351.3	361.7
5	5.3	342.0	207.5	261.0	260.0	203.5	257.6	259.0	321.4	330.9
6	4.3	330.0	199.5	249.0	238.0	195.5	245.6	237.0	296.8	305.5
7	3.7	310.0	193.0	239.0	220.0	188.7	235.6	220.0	275.7	283.8
8	3.2	265.0	187.0	231.0	206.0	183.0	227.1	205.0	257.0	264.6
9	2.8	240.0	182.0	223.0	194.0	178.0	220.0	193.0	240.1	247.2
10	2.5	235.0	177.5	217.0	183.0	173.5	213.1	182.5	224.6	231.2
11	2.3	227.0	173.5	211.0	174.0	169.0	207.2	173.0	210.1	216.3
12	2.1	200.0	170.0	206.0	166.0	166.0	202.2	165.0	196.3	202.1
13	1.9	182.0	166.5	201.0	158.5	162.5	197.2	158.0	183.1	188.5
14	1.8	172.0	163.5	196.0	152.0	159.5	192.5	151.0	170.4	175.4
15	1.7	172.0	160.5	192.0	146.0	157.0	188.5	145.0	157.8	162.5
16	1.6	172.0	158.0	188.0	140.5	154.0	185.5	139.5	145.4	149.7
17	1.5	145.0	155.5	185.0	135.5	151.5	181.8	134.0	133.0	136.9
18	1.4	120.0	153.0	181.0	130.5	147.0	178.0	129.0	120.3	123.8
19	1.3	105.0	151.0	178.0	126.5	146.0	175.0	125.5	107.1	110.2
20	1.2	100.0	148.5	175.0	122.0	144.3	171.8	121.0	93.1	95.9
21	1.2	98.0	146.5	172.0	118.0	143.0	169.0	117.0	77.9	80.2
22	1.1	78.0	145.0	170.0	115.0	141.0	167.8	114.0	60.4	62.1
23	1.1	49.0	143.0	167.0	112.0	139.0	164.5	111.0	38.4	39.5
24	1.0	38.0	141.0	164.5	109.0	137.0	161.0	107.0	3.4	3.5

Table 7.3 Observed and computed discharges by various DFFD models and regional analysis (Lakhora watershed)

S.NO.	T	DISCH.	GcIUH BASED MODELS			KW THE. BASED MODELS			AT SITE REG.	REG. ONLY
			PHI	PHILIP	SCS	PHI	PHILIP	SCS		
1	43.1	700.0	559.0	734.0	1010.0	539.3	717.8	1088.6	630.0	945.1
2	15.5	698.0	477.0	608.0	711.0	456.5	590.0	726.5	500.9	751.6
3	9.4	630.0	437.5	548.0	587.0	416.5	529.2	596.0	437.1	655.8
4	6.8	470.0	411.0	509.0	512.0	389.7	489.5	517.0	393.5	590.4
5	5.3	420.0	391.5	480.0	459.0	370.0	460.0	461.0	360.0	540.2
6	4.3	330.0	375.5	457.0	419.0	353.5	436.3	419.3	332.5	498.8
7	3.7	298.0	362.5	438.0	386.5	340.0	416.6	386.0	308.8	463.3
8	3.2	194.0	351.0	421.0	360.0	328.5	400.2	358.0	287.9	431.9
9	2.8	190.0	341.0	407.0	338.0	318.5	387.0	335.0	269.0	403.6
10	2.5	175.0	332.0	394.5	318.0	309.0	373.0	314.0	251.6	377.5
11	2.3	170.0	324.0	383.0	301.0	301.0	361.0	297.0	235.3	353.1
12	2.1	168.0	317.0	373.5	287.0	294.0	351.0	282.0	219.9	329.9
13	1.9	165.0	310.5	364.0	273.0	288.0	342.5	267.0	205.2	307.8
14	1.8	150.0	304.5	355.0	261.0	281.0	333.0	255.0	190.8	286.3
15	1.7	140.0	299.0	347.5	250.0	275.5	325.0	244.0	176.8	265.3
16	1.6	130.0	293.5	340.0	240.0	270.0	317.0	233.0	162.9	244.4
17	1.5	122.0	289.0	333.0	231.0	265.0	310.0	223.0	148.9	223.4
18	1.4	110.0	283.5	326.0	222.0	260.0	304.0	213.0	134.7	202.1
19	1.3	108.0	279.5	320.5	214.0	256.0	297.0	204.0	120.0	180.0
20	1.2	100.0	275.0	314.5	206.0	252.0	291.0	196.0	104.3	156.5
21	1.2	92.0	271.0	309.0	200.0	247.5	285.0	189.0	87.2	130.9
22	1.1	75.0	268.0	304.0	193.0	243.3	281.0	183.0	67.6	101.4
23	1.1	68.0	264.0	299.0	187.0	240.0	275.5	176.0	43.0	64.5
24	1.0	21.0	260.0	294.0	183.0	237.0	270.0	170.0	3.8	5.7

Table 7.4 Observed and computed discharges by various DFFD models and regional analysis (Kharanala watershed)

S.NO.	T	DISCH.	GcIUH BASED MODELS			KW THE. BASED MODELS			AT SITE	REG.
			PHI	PHILIP	SCS	PHI	PHILIP	SCS	REG.	ONLY
1	37.7	390.0	107.8	129.3	229.5	102.7	125.2	231.1	360.7	427.7
2	13.5	380.0	92.0	107.0	161.5	86.9	102.5	163.4	284.6	337.4
3	8.3	283.0	84.5	96.5	134.0	79.3	91.8	134.1	246.8	292.6
4	5.9	270.0	79.5	89.5	117.0	74.2	84.7	116.8	220.9	261.9
5	4.6	250.0	76.0	84.5	105.0	70.4	79.5	105.0	200.9	238.2
6	3.8	200.0	73.0	80.3	96.0	67.3	75.3	95.5	184.4	218.6
7	3.2	170.0	70.5	77.0	89.0	64.8	72.0	88.0	170.1	201.7
8	2.8	165.0	68.0	74.0	83.0	62.5	69.0	82.0	157.4	186.6
9	2.5	160.0	66.0	71.5	78.0	60.7	66.4	76.5	145.8	172.9
10	2.2	125.0	64.5	69.5	73.5	59.0	64.0	72.0	135.1	160.2
11	2.0	120.0	63.0	67.5	70.0	57.5	62.1	68.0	124.9	148.1
12	1.8	110.0	61.5	65.5	66.0	56.0	60.3	65.0	115.2	136.6
13	1.7	107.0	60.5	64.0	63.0	54.8	58.6	61.5	105.7	125.4
14	1.6	100.0	59.0	62.5	61.0	53.3	57.1	59.0	96.4	114.3
15	1.5	32.0	58.0	61.0	58.0	52.4	55.6	56.0	87.0	103.2
16	1.4	29.0	57.0	59.5	56.0	51.4	54.3	54.0	77.4	91.8
17	1.3	17.0	56.5	58.5	54.0	50.5	53.1	52.0	67.4	79.9
18	1.2	11.0	55.5	57.5	52.0	49.5	51.8	49.7	56.5	67.0
19	1.1	11.0	54.5	56.5	50.0	48.7	51.0	48.2	44.2	52.4
20	1.1	10.0	54.0	55.0	49.0	47.9	50.0	46.5	28.9	34.3
21	1.0	5.0	52.5	54.5	47.0	47.2	48.8	45.0	4.8	5.7

Table 7.5 Observed and computed discharges by various DFFD models and regional analysis (Suk Tawa watershed)

S.NO.	T	DISCH.	GcIUH BASED MODELS			KW THE. BASED MODELS			AT SITE REG.	REG. ONLY
			PHI	PHILIP	SCS	PHI	PHILIP	SCS		
1	34.1	1250.0	543.5	612.3	815.4	538.0	596.8	868.0	1440.8	994.9
2	12.3	1100.0	463.5	506.0	571.5	457.2	487.0	593.7	1129.7	780.0
3	7.5	860.0	424.5	452.0	470.0	418.2	435.3	485.0	974.7	673.0
4	5.4	750.0	399.0	418.0	408.0	392.2	401.0	420.0	868.2	599.5
5	4.2	740.0	379.5	393.0	365.0	372.5	376.0	374.0	785.6	542.4
6	3.4	725.0	364.0	373.0	332.0	357.0	356.0	340.0	717.0	495.1
7	2.9	724.0	351.0	356.0	305.5	344.0	339.0	312.0	657.4	454.0
8	2.5	600.0	340.0	343.0	284.0	333.0	325.0	289.0	604.2	417.2
9	2.2	557.0	330.5	330.0	265.5	323.0	312.0	270.0	555.3	383.4
10	2.0	535.0	322.0	320.0	250.0	314.0	301.0	254.0	509.4	351.8
11	1.8	475.0	314.0	310.0	236.0	306.0	291.5	239.0	465.7	321.5
12	1.7	450.0	307.0	301.0	223.0	298.0	282.0	226.0	423.2	292.2
13	1.5	400.0	300.5	293.0	212.5	292.0	274.0	213.0	381.1	263.1
14	1.4	399.0	295.0	286.0	203.0	286.0	267.0	202.5	338.6	233.8
15	1.3	380.0	289.0	279.0	193.0	280.5	259.5	193.0	294.5	203.4
16	1.2	320.0	284.0	273.0	186.0	275.5	253.5	185.0	247.4	170.8
17	1.2	280.0	279.0	266.5	177.0	270.0	247.0	176.0	194.4	134.3
18	1.1	240.0	275.0	261.5	171.0	265.5	242.0	170.0	129.3	89.3
19	1.0	180.0	270.5	256.0	165.0	261.0	236.0	163.0	27.9	19.3

$$ERRT = 100 \frac{Q_1 - \hat{Q}_1}{Q_1} \tag{7.5}$$

$$ERRT6 = \frac{100}{6} \sum_{i=1}^{6} \left(\frac{Q_i - \hat{Q}_i}{Q_i} \right) \tag{7.6}$$

$$Efficiency = \left[1 - \frac{\sum_{i=1}^{N} (Q_i - \hat{Q}_i)^2}{\sum_{i=1}^{N} (Q_i - Q_i)^2} \right] 100 \tag{7.7}$$

where

ERRT = Relative per cent error in topmost quantile

ERRT6 = Average relative per cent error in top 6 quantiles

The first and second criteria give the comparison in the upper tail region of the frequency curve while third criterion judges the overall fit. The values of the above indices are presented in Table 7.6 for the five watersheds.

7.4.2 Performance of GcIUH Based Models

The results of GcIUH based models are discussed in the following sections. In these models the infiltration losses are represented by ϕ-index, Philip's equation and SCS curve number methods.

GcIUH-ϕ-index Model

As depicted in Fig 7.3, this model under estimates the flood quantiles for the return periods above 1.6 years for Tairhia watershed. Quantiles are over predicted below 1.6 years return periods. Over prediction errors are in the range

Table 7.6 Performance criteria for various DFFD models and regional analysis for five test watersheds

S.NO. T	GcIUH BASED MODELS PHI	GcIUH BASED MODELS PHILIP	GcIUH BASED MODELS SCS	KW THE. BASED MODELS PHI	KW THE. BASED MODELS PHILIP	KW THE. BASED MODELS SCS	AT SITE REG.	REG. ONLY
	Tairhia Watershed							
ERRT	48.0	31.8	-1.3	48.2	31.8	-1.2	6.1	-17.7
ERRT6	34.9	18.8	7.0	35.1	18.7	7.0	0.2	-25.1
EFFICIENCY	-362.5	-84.3	88.4	-373.6	-87.0	88.1	98.9	87.2
	Pausar Watershed							
ERRT	28.7	4.3	-36.7	29.7	5.0	-36.7	-37.2	-41.2
ERRT6	36.7	18.6	7.4	37.9	19.5	7.5	-6.5	-9.6
EFFICIENCY	-187.7	-31.7	70.1	-167.7	-33.2	70.2	92.1	91.3
	Lakhora Watershed							
ERRT	20.1	-4.9	-44.3	23.0	-2.5	-55.5	10.0	-35.0
ERRT6	14.7	-6.7	-14.1	18.9	-2.8	-16.8	16.4	-25.4
EFFICIENCY	-71.2	5.3	67.5	-91.5	8.6	69.3	76.0	70.3
	Kharanala Watershed							
ERRT	72.4	66.8	41.2	73.7	67.9	40.7	7.5	-9.7
ERRT6	70.3	66.2	53.0	72.2	67.9	52.9	15.2	-0.6
EFFICIENCY	-191.4	-196.0	-84.6	-165.1	-168.9	-75.8	82.1	88.3
	Suk Tawa Watershed							
ERRT	56.5	51.0	34.8	57.0	52.3	30.6	-15.3	20.4
ERRT6	51.7	48.7	46.4	52.5	50.7	44.5	-8.7	25.0
EFFICIENCY	-65.5	-45.8	8.1	-60.6	-39.1	14.2	94.4	60.3

of 15.9 to 316.2 per cent and under prediction errors range from 5.8 to 48.0 per cent. Average error in top 6 values is 34.9 per cent. As shown in Table 7.6 the pattern of performance is similar in Pausar, Lakhora, Kharanala and Suk Tawa watersheds. In all the five watersheds the efficiency of overall fit is negative.

Similar trends were observed by Moughamian et al. (1987) and Raines and Valdes (1993) for the watersheds studied by them. This indicates the inability of ϕ-index model to represent the infiltration process.

GcIUH-Philip Model

This model performs better than GcIUH-ϕ-index model for Lakhora, Pausar and Tairhia watersheds in that order. Though, the overall fit criterion as expressed by efficiency is low, relative error in topmost quantiles (4.9, 4.3 and 31.8 per cent) and average error in top six values (6.7, 18.6 and 18.8 per cent) show promising results for these watersheds as we are interested in these criteria for practical purposes. The model under predicts the quantiles for Kharanala and Suk Tawa watersheds as observed by Raines and Valdes (1993) for Turtle Creek watershed. Moughamian et al. (1987) also obtained similar results for Santa Paula Creek watershed.

GcIUH-SCS Model

This model, developed by Raines and Valdes (1993) represent infiltration process by SCS curve number method for which data is more readily available. The quantiles predicted by GcIUH-SCS model were better among GcIUH based models for Tairhia, Pausar and Lakhora watersheds. Better overall fit is explained by 88.4, 70.1 and 67.5 per cent efficiencies for these watersheds.

Best performance is also represented by average relative error in top 6 quantiles (Tairhia-7.0, Pausar-7.4 and Lakhora-14.1). These quantiles correspond to the return periods above 3 years where our interest is centred for practical applications.

The predictions are at par with the regional only analysis which is one of the current practice for ungauged catchments. For Kharanala and Suk Tawa watersheds the results are better among GcIUH based models.

Above results indicate that SCS curve number method represents infiltration losses in a better way as far as DFFD models are concerned. The simple estimation with reasonable accuracy also makes this model suitable for DFFD models. Results of this model on Briar Creek, Turtle Creek and Halls Bayou watersheds were better than the other models tested by Raines and Valdes(1993). However, they estimated the parameters of rainfall model using different method than the one used in the present study.

7.4.3 Performance of KW Theory Based Models

The following sections present the results of KW theory based models. The ϕ-index, Philip's equation and SCS curve number method were used as infiltration models for KW theory based DFFD models.

KW-ϕ-index Model

This model under estimates the quantiles above 1.6 years return periods for Tairhia watershed. As given in Table 7.1, the quantiles are slightly less than the GcIUH ϕ-index model. As a result the performance criteria values are also close to this model. The predictions made for Pausar, Lakhora, Kharanala and Suk Tawa watersheds have similar trends with negative efficiencies.

KW-Philip Model

Out of five watersheds, KW-Philip model has given better performance for Lakhora watershed. Though the efficiency is 8.6 per cent only, the estimation for topmost (-2.5 per cent error) and top six quantiles (average error -2.8 per cent), above 4 years return periods, seems to be acceptable from practical point of view. Results for Pausar and Tairhia watersheds are slightly inferior to Lakhora watershed. The performance for Kharanala and Suk Tawa watersheds is similar to

GcIUH-ϕ-index model.

KW-SCS Model

This model has been developed in the present study, keeping in view the simple estimation procedure of SCS curve number. The model performs almost similar to GcIUH-SCS model for Tairhia, Pausar and Lakhora watersheds. The results are also comparable with regional analyses. For Kharanala and Suk Tawa watersheds the model could not perform well, however, the results are slightly better than GcIUH-SCS model.

7.4.4 Predictive Ability of Different Models

In order to judge the ability of different models in extrapolation, the quantiles were computed for 50 and 100 years return period using above models (Table 7.7). Assuming that the quantiles predicted by at site/regional method as standard, a comparison has been presented in Table 7.8. In general the models which use SCS curve number method to represent infiltration performed better than other models.

7.4.5 Performance of KW Theory based Models on Other Watersheds

In the following sections, comparison of two KW theory based models has been discussed for Ralston Creek and Santa Anita Creek watersheds (Cadavid et al., 1991) with a aim to judge the performance on common data base. One model represents infiltration losses by Philip's equation while other uses SCS curve number method.

Ralston Creek Watershed

Cadavid et al.(1991) applied their model on Ralston Creek watershed. The details of this watershed are presented in section 5.4.1 and section 6.5. Parameters of stochastic rainfall model and KW model were kept same. Parameters of Philip's infiltration equation were used for KW - Philip model. SCS curve number was estimated from available information and later on optimised by visual fitting.

Table 7.7 Extrapolated 50 and 100 years quantiles computed by various models

RETURN PERIOD	GcIUH BASED MODELS			KW THE. BASED MODELS			AT SITE REG.	REG. ONLY
	PHI	PHILIP	SCS	PHI	PHILIP	SCS		
	TAIRHIA WATERSHED							
50	329.4	436.8	681.5	328.9	436.6	681.6	607.8	761.7
100	360.5	486.4	837.4	359.9	486.2	837.6	688.5	862.8
	PAUSAR WATERSHED							
50	298.4	402.2	586.0	294.2	399.0	587.0	579.0	596.0
100	326.4	447.5	720.3	322.4	444.5	720.5	655.9	675.2
	LAKHORA WATERSHED							
50	570.9	752.8	1058.6	551.3	736.7	1147.0	648.6	973.1
100	626.3	840.5	1322.0	607.5	825.4	1450.0	734.7	1102.3
	KHARANALA WATERSHED							
50	112.1	135.6	250.8	107.1	131.6	252.0	381.4	452.2
100	122.7	151.2	307.7	117.8	147.4	307.7	432.0	512.3
	SUK TAWA WATERSHED							
50	573.1	653.7	918.9	568.2	638.5	1000.0	1555.1	1073.7
100	627.2	729.9	1125.4	623.0	714.9	1415.0	1761.6	1216.3

Table 7.8 Per cent error in 50 and 100 years quantiles

RETURN PERIOD	GcIUH BASED MODELS			KW THE. BASED MODELS			REG. ONLY
	PHI	PHILIP	SCS	PHI	PHILIP	SCS	
	TAIRHIA WATERSHED						
50	-45.8	-28.1	12.1	-45.9	-28.2	12.1	25.3
100	-47.6	-29.4	21.6	-47.7	-29.4	21.7	25.3
	PAUSAR WATERSHED						
50	-48.5	-30.5	1.2	-49.2	-31.1	1.4	2.9
100	-50.2	-31.8	9.8	-50.8	-32.2	9.8	2.9
	LAKHORA WATERSHED						
50	-12.0	16.1	63.2	-15.0	13.6	76.8	50.0
100	-14.8	14.4	79.9	-17.3	12.3	97.4	50.0
	KHARANALA WATERSHED						
50	-70.6	-64.4	-34.2	-71.9	-65.5	-33.9	18.6
100	-71.6	-65.0	-28.8	-72.7	-65.9	-28.8	18.6
	SUK TAWA WATERSHED						
50	-63.1	-58.0	-40.9	-63.5	-58.9	-35.7	-31.0
100	-64.4	-58.6	-36.1	-64.6	-59.4	-19.7	-31.0

Fig. 7.33 shows the frequency curves for the two models. The outputs of the two models are quite similar for the range of the plotted data.

Santa Anita Creek Watershed

The second application watershed of Cadavid et al.(1991) is Santa Anita Creek. The parameters of rainfall, KW and infiltration models were given in sections 5.4.2 and 6.5. The SCS curve number for the watershed was optimised for KW - SCS model by visual fitting of the frequency curves. As depicted in Fig. 7.34, the frequency curve predicted by KW - SCS model and KW-Philip model are quite close. The quantiles given by KW-SCS method are higher as compared to KW-Philip method.

7.5 EFFECT OF CORRELATION

The model developed in Chapter 4 (GcIUH-ϕ-index Correlated) has been used to study the effect of correlation between rainfall intensity and duration on flood frequency estimates. Data of Davidson watershed (Table 6.18) has been used for this purpose. As given in section 6.6 Diaz-Granados et al.(1983) produced a reasonable fit to observed data by using 50 per cent contributing area and a ϕ - index of 0.72 cm/hr. A contributing area of 50 per cent and ϕ-index of 0.72 cm/hr have been taken to study the effect of correlation. Other parameters were kept same. The value of γ was varied from 0 to 1 with an interval of 0.2. The corresponding correlation coefficients are given in Table 7.9.

The return periods corresponding to various discharges are given in Table 7.9 for different values of γ or ρ. The discharges correspond to selected values of intensities of rainfall. The same are plotted in Fig.7.35.

As depicted in the Fig. 7.35 a maximum correlation coefficient of -0.404 will estimate a quantile of 247 cumec for 100 years return period as compared to 334 cumec when a DFFD model with independent rainfall intensity and duration (zero

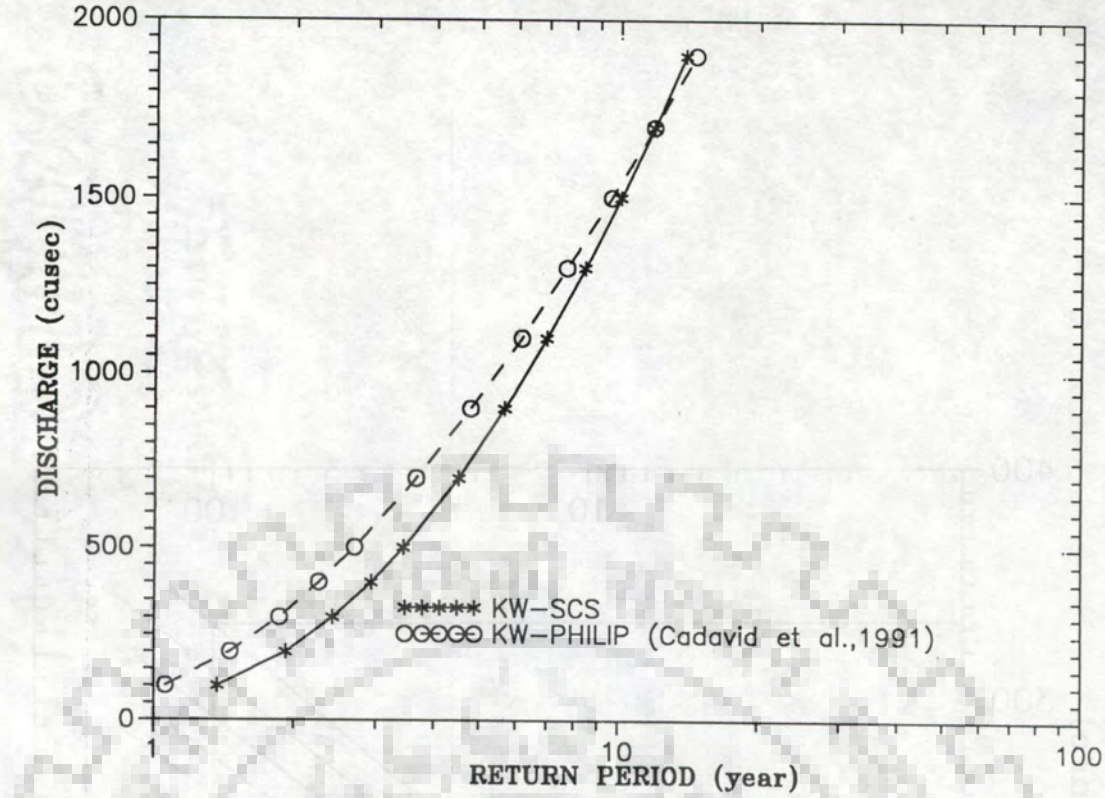

FIG. 7.33—COMPARISON OF FLOOD FREQUENCIES AT RALSTON CREEK WATERSHED.

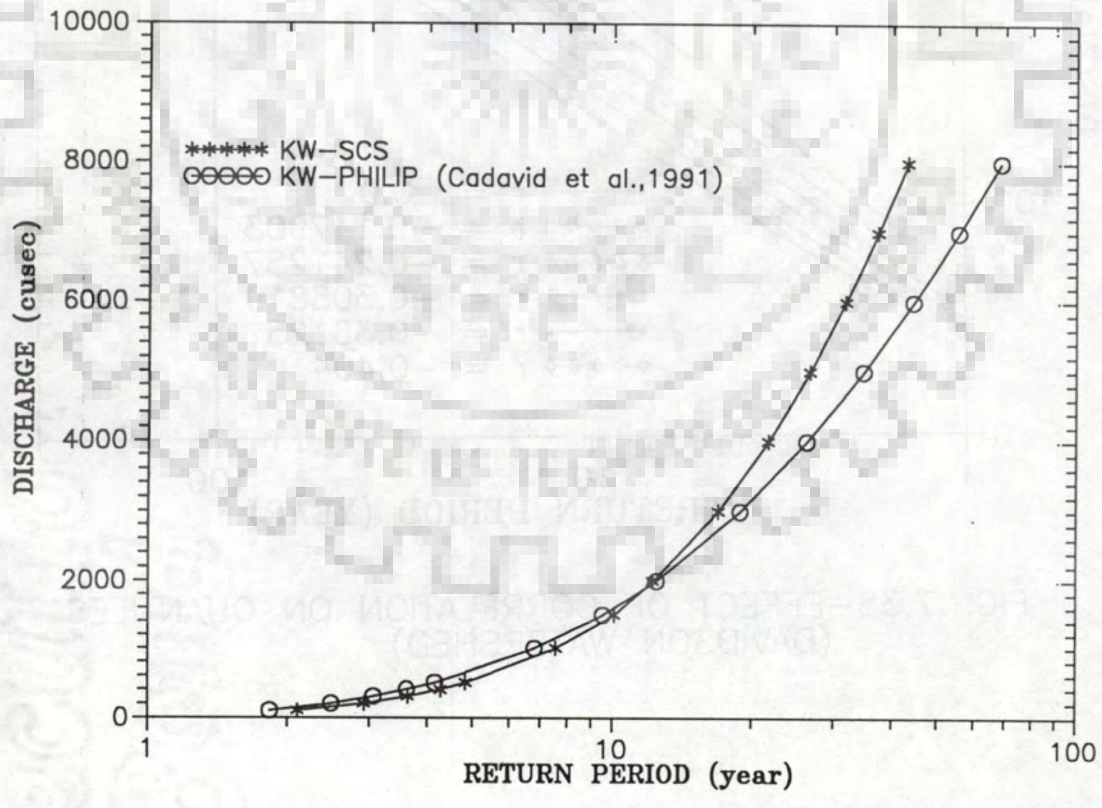

FIG. 7.34—COMPARISON OF FLOOD FREQUENCIES AT SANTA ANITA CREEK WATERSHED.

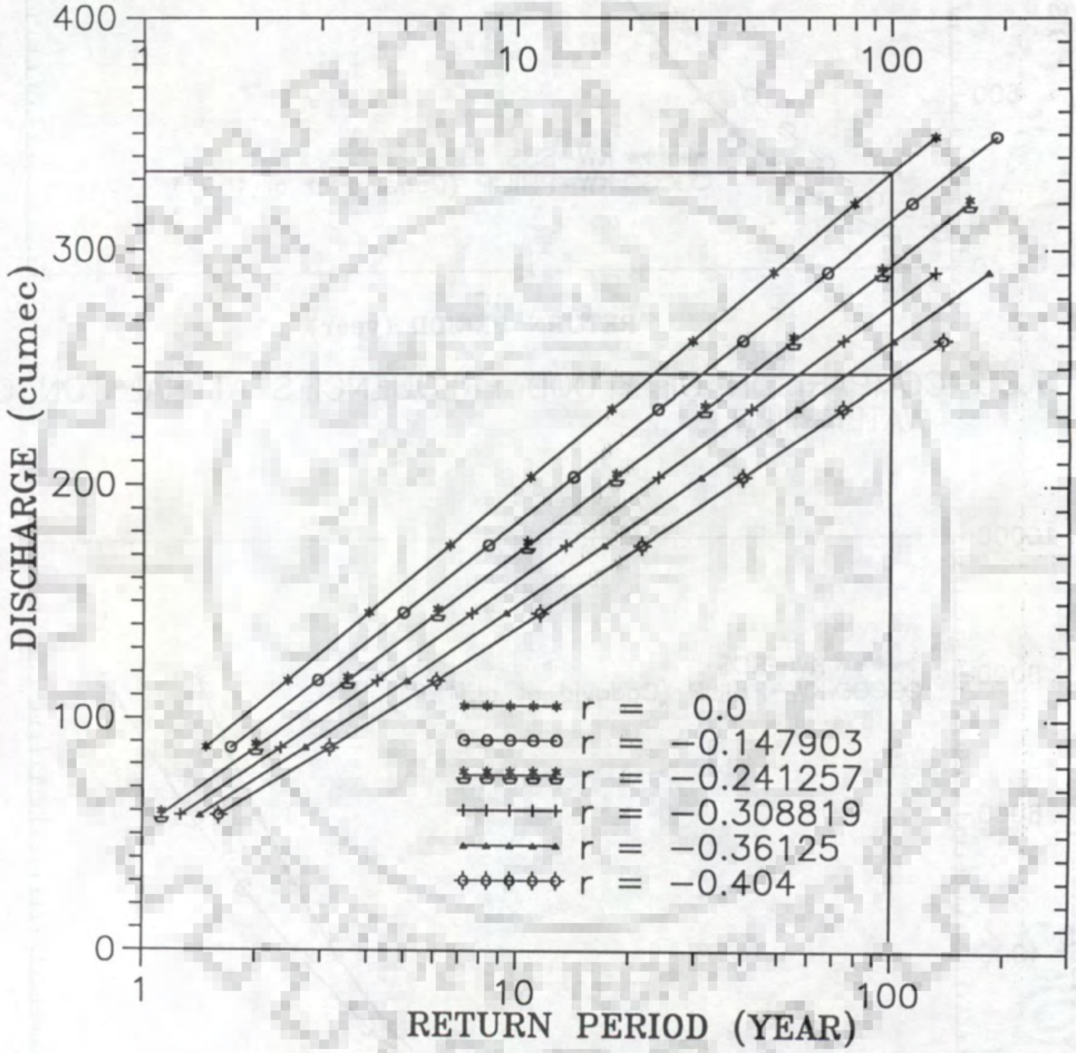

FIG. 7.35–EFFECT OF CORRELATION ON QUANTILES (DAVIDSON WATERSHED)

Table 7.9 - Effect of correlation on quantiles (Davidson watershed)

DISCHARGE	RETURN PERIOD					
	γ=0.0	γ=0.2	γ=0.4	γ=0.6	γ=0.8	γ=1.0
	ρ=0.0	ρ=-0.1479	ρ=-0.24126	ρ=-0.30882	ρ=-0.36125	ρ=-0.404
58.11	-	-	1.13	1.27	1.42	1.60
87.17	1.48	1.72	2.01	2.34	2.72	3.16
116.22	2.44	2.94	3.53	4.24	5.08	6.09
145.28	4.02	4.98	6.15	7.60	9.37	11.53
174.33	6.62	8.41	10.67	13.53	17.12	21.62
203.39	10.89	14.18	18.44	23.96	31.07	40.21
232.44	17.89	23.87	31.80	42.29	56.14	74.36
261.50	29.41	40.13	54.70	74.42	101.02	136.83
290.56	48.32	67.42	93.95	130.61	-	-
319.61	79.41	113.21	161.11	-	-	-
348.67	130.49	-	-	-	-	-

correlation coefficient) is used for estimation. It may be concluded that the quantiles estimated by the DFFD models which consider rainfall intensity to be independent of their durations are higher than the flood quantiles estimated by the proposed model which accounts for the negative correlation between these variables.

CHAPTER 8

CONCLUSIONS

8.1 GENERAL OBSERVATIONS

Physically based flood frequency models or derived flood frequency distribution (DFFD) models have been developed in the present work. All the DFFD models use joint PDF of exponentially distributed rainfall intensity and duration. In the present study, infiltration process has been represented by three different models i.e. ϕ - index, Philip's equation and SCS curve number method. Two effective rainfall-runoff models based on KW theory and GcIUH concepts have been used. Kinematic wave theory based model has a physical basis whereas GcIUH is a conceptual model. For the first time an attempt has been made to develop and use a stochastic rainfall model which accounts for correlation between rainfall intensity and duration. Based on this study, the following general observations can be made.

1. The physically based flood frequency models provide a potentially attractive and alternative solution to ungauged watersheds.

2. The impact of watershed changes on flood magnitudes and frequencies can be studied through DFFD models. This would require calibration of DFFD model for current catchment characteristics and application for the changed scenarios.

3. The models developed and studied could not meet our expectations because of many constraints. The main constraint being the lack of long term reliable rainfall and runoff data. If we are to understand better the physical factors that control the probability distribution of floods we need to collect continuous discharge and rainfall data at multiple locations. This would be slightly expensive and inconsistent with the usual goals of official stream gauging networks. But this is unavoidable if the usual design procedures are to be

replaced by new improved methods.

8.2 SPECIFIC CONCLUSIONS

On the basis of application of DFFD models to five Indian watersheds and three watersheds from U.S.A., the following conclusions can be drawn.

1. Parameters of stochastic rainfall model are most sensitive input to the DFFD models and therefore, should be estimated carefully.

2. Out of the three infiltration models used, the parameter of SCS curve number model can be estimated quite easily with reasonable accuracy. The DFFD models based on this infiltration model perform better than the other models.

3. GcIUH and KW theory based effective rainfall-runoff models perform equally well in DFFD models.

4. The quantiles estimated by the DFFD models which consider rainfall intensity to be independent of their durations are higher than the flood quantiles estimated by the proposed model which accounts for the negative correlation between these variables. DFFD models for positively correlated case still need to be developed.

8.3 SUGGESTIONS FOR FUTURE WORK

Physically based flood frequency models are relatively new in the field of hydrology, and are under development stage. There is a need for application of these models to more watersheds before recommending them for field use. New DFFD models which can take into account the positive correlation between intensity and duration also need to be developed.

The technique of derived distributions is a powerful tool. In the field of hydrology, information is needed for the cumulative effect of many random variables on the hydrologic system. The information on water yield, sediment yield, the chemicals transported to a site due to runoff events are vital for the planning of hydrologic projects. The derived distribution technique can be used to develop new models in the above fields.

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GcIUH -ϕ-INDEX MODEL

```
C***********************************************************************
C       PROGRAMMME COMPUTES DERIVED FLOOD FREQUENCY CURVE
C       USING BIVARIATE EXPONENTIAL DISTRIBUTION OF
C       RAINFALL INTENSITY AND DURATION (FOR INDEPENDENT CASE (GAMA=0)
C       AND CORRELATED CASE(0<GAMA<=1)) WITH INFILTRATION LOSSES
C       REPRESENTED BY PHI-INDEX AND EFFECTIVE RAINFALL-RUNOFF
C       BY GEOMORPHOCLIMATIC IUH
C
C
C       THIS PROGRAMME IS MODIFIED FOR CORRELATED CASE BY R.S.KUROTHE,
C       RESEARCH SCHOLAR, DEPT. OF HYDROLOGY, UNIVERSITY OF ROORKEE,
C       ROORKEE, INDIA FROM THE PROGRAMME WRITTEN BY MARIO A. DIAZ-
C       GRANADOS, AT THE PARSONS LABORATORY FOR WATER RESOURCES AND
C       HYDRODYNAMICS, M.I.T., CAMBRIDGE, MASS. 02139.
C***********************************************************************
C       PARAMETERS AND VARIABLES
C       ************************
C       BETA1 = MEAN AREAL RAINFALL INTENSITY (cm/hr)
C       DELTA1 = MEAN DURATION OF THE STORM (hr)
C       GAMA = PARAMETER DEFINING CORRELATION BETWEEN INTENSITY AND
C              DURATION (BACCHI et al., 1994; SAME AS DELTA OF EQ. 4.1)
C       MNU = AVERAGE NO. OF STORMS/YEAR
C       A = AREA OF BASIN (sq km)
C       RL = LENGTH RATIO OF THE BASIN
C       XL = LENGTH OF HIGHEST ORDER STREAM (km)
C       ALFA = KINEMATIC WAVE PARAMETER OF HIGHEST ORDER STREAM
C              OF THE BASIN (1/(sm**1/3))
C       PHI = PHI INDEX FOR THE BASIN (cm/hr)
C       Q = DISCHARGE (cumec)
C       NQ = NO. OF DISCHARGE VALUES USED
C       FQ = COMPUTED CDF OF PEAK DISCHARGE
C       T = COMPUTED RETURN PERIOD (year)
C       TW = RETURN PERIOD GIVEN BY WEIBULL FORMULA (year)
C       TG = RETURN PERIOD GIVEN BY GRINGORTON FORMULA (year)
C*********************************************************************
        EXTERNAL F1
        DIMENSION Q(40),FQ(40),T(40),TITLE(80)
        DIMENSION TW(40),TG(40)
        REAL MNU
        COMMON/H1/BETA,DELTA,GAMA,PHI,QP,XK1,S
        OPEN(UNIT=1,FILE='DDCORD.DAT',STATUS='OLD')
        OPEN(UNIT=2,FILE='DDCORD.OUT')
C
C       READS AND WRITES INPUT DATA
C*****************************************
C
        READ(1,2)TITLE
        READ(1,*)NQ
        READ(1,*)(Q(I),I=1,NQ)
        WRITE(2,2)TITLE
2       FORMAT(80A1)
        READ(1,*)A,RL,XL,ALFA,PHI,GAMA
```

```
      WRITE(2,4)A,RL,XL,ALFA,PHI,GAMA
 3    FORMAT(/'MIR(cm/hr)=',F7.3,2X,'MTR(hr)=',F7.2,2X,'MNU=',F5.1/)
 4    FORMAT(/'AREA(sq.km)=',F6.1,2X,'RL=',F6.3,2X,'XL(km)=',F5.2,2X,
     1 'ALPHA(1/(sm**(1/3)))='F5.2/'PHI(cm/hr)='F5.2,2X,'GAMA=',F7.5)
 5    FORMAT(2X,'FREQUENCY',9X,'DISCHARGE',2X,
     1 'RECURRENCE INTERVAL',2X,'T(WEIBULL)',4X,'T(GRING)')
 6    FORMAT(21X,'(cumec)',10X,'(year)',9X,'(year)',7X,'(year)'/)
      READ(1,*)BETA1,DELTA1,MNU
      WRITE(2,3)BETA1,DELTA1,MNU
      WRITE(2,5)
      WRITE(2,6)
C
C     COMPUTES PARAMETERS OF RAINFALL MODEL
C*****************************************
C
      BETA=1./BETA1
      DELTA= 1./DELTA1
C
C     COMPUTES A PARAMETER FOR EXPRESSIONS OF GcIUH PEAK AND
C     TIME TO PEAK
C**************************************************************
      XK1=(A*RL)**0.4*ALFA**0.6/XL
C
C     COMPUTES CDF OF PEAK DISCHARGES
C***********************************
      DO 52 K=1,NQ
C
C     COMPUTES LOWER LIMIT OF INTEGRATION
C***************************************
      QP=0.36*Q(K)/A
      XINF=QP
C
C     COMPUTES INTEGRATION AREA FROM LOWER LIMIT TO UPPER LIMIT
C     UPPER LIMIT SHOULD BE INFINITY BUT INTEGRATION IS DONE TO
C     A MAXIMUM VALUE FOR A GIVEN TOLERANCE
C*****************************************************************
      AREA=0.
      NV=0
103   XSUP=XINF+5
      EDEL=0.001
      ARE=TEGRAL(F1,XINF,XSUP,EDEL)
      NV=NV+1
      AREA=AREA+ARE
      IF(NV.EQ.1)GO TO 104
      IF(ABS((AREA-ARE1)/AREA).LE.0.00001)GO TO 105
104   ARE1=AREA
      XINF=XSUP
      GO TO 103
C
C     COMPUTES CDF AND RETURN PERIODS FOR WEIBUL, GRINGORTON
C     AND BY DFFD MODEL
C**************************************************************
105   FQ(K)=1.-BETA*EXP(-BETA*PHI)*AREA
      AA=0.0
      TW(K)=(NQ+1-2*AA)/(K-AA)
      AA=0.44
      TG(K)=(NQ+1-2*AA)/(K-AA)
      IF(FQ(K).GE.1.)THEN
      T(K)=10000
```

```
      GO TO 106
      ENDIF
      T(K)=1./(MNU*(1.-FQ(K)))
106   WRITE(2,7)FQ(K),Q(K),T(K),TW(K),TG(K)
7     FORMAT(F12.8,2X,F12.0,8X,F10.2,7X,F8.2,2X,F10.2)
52    CONTINUE
      STOP
      END
C
C
      FUNCTION F1(X)
C
C     COMPUTES THE ARGUMENT OF THE INTEGRAL
C*******************************************
      COMMON/H1/BETA,DELTA,GAMA,PHI,QP,XK1,S
      F1=0.0
      TE=2.*(1.-SQRT(1.-QP/X))/(0.871*XK1*X**0.4)
      ARG=BETA*X+(DELTA+BETA*DELTA*GAMA*PHI+BETA*DELTA*GAMA*X)*TE
      F1=(1.+DELTA*GAMA*TE)*EXP(-ARG)
      RETURN
      END
C
C
      FUNCTION TEGRAL(FI,A,B,EDEL)
C
C     THIS FUNCTION USES THE ROMBERG INTEGRATION METHOD TO INTEGRATE
C     FI FROM A TO B
C
      EXTERNAL FI
      DIMENSION T(30,30)
      T(1,1)=(B-A)*(FI(A)+FI(B))/2
      T(1,2)=T(1,1)/2+(B-A)*FI((A+B)/2)/2
      T(2,1)=(4*T(1,2)-T(1,1))/3
      J=3
5     DX=(B-A)/2**(J-1)
      X=A-DX
      N=2**(J-2)
      SUM=0.0
      DO 10 I=1,N
      X=X+2.*DX
      SUM=SUM+FI(X)
10    CONTINUE
      T(1,J)=T(1,J-1)/2+DX*SUM
      DO 20 L=2,J
      K=J+1-L
      T(L,K)=(4**(L-1)*T(L-1,K+1)-T(L-1,K))/(4**(L-1)-1)
20    CONTINUE
      TT=ABS((T(J,1)-T(J-1,1))/T(J,1))
      IF(TT.LE.EDEL)GO TO 30
      J=J+1
      IF(J.GT.30)THEN
      WRITE(*,*)'WARNING TEGRAL:MATRIX DIMENSION > 30'
      END IF
      GO TO 5
30    TEGRAL=T(J,1)
      RETURN
      END
C*****************************************************************
```

GcIUH - PHILIP MODEL

```
C*********************************************************************
C      THIS PROGRAMME CALCULATES THE FLOOD FREQUENCY CURVE
C      USING BIVARIATE EXPONENTIAL DISTRIBUTION
C      OF RAINFALL INTENSITY AND DURATION, INFILTRATION LOSSES
C      REPRESENTED BY PHILIP'S EQUATION AND THE GEOMORPHOCLIMATIC
C      IUH AS EFFECTIVE RAINFALL-RUNOFF MODEL
C
C      THIS PROGRAMME IS MODIFIED BY R.S.KUROTHE, RESEARCH SCHOLAR,
C      DEPT. OF HYDROLOGY, UNIVERSITY OF ROORKEE, ROORKEE, INDIA
C      FROM THE PROGRAMME WRITTEN BY MARIO A. DIAZ-GRANADOS, AT
C      THE PARSONS LABORATORY FOR WATER RESOURCES AND
C      HYDRODYNAMICS, M.I.T., CAMBRIDGE, MASS. 02139.
C*********************************************************************
C      PARAMETERS AND VARIABLES
C      ************************
C      BETA1 = MEAN AREAL RAINFALL INTENSITY (cm/hr)
C      DELTA1 = MEAN DURATION OF THE STORM (hr)
C      MNU = AVERAGE NO. OF STORMS/YEAR
C      DELTAI = INCREMENT OF INTENSITY (cm/hr)
C      A = AREA OF BASIN (SQ KM)
C      RL = LENGTH RATIO OF THE BASIN
C      XL = LENGTH OF HIGHEST ORDER STREAM (km)
C      ALFA = KINEMATIC WAVE PARAMETER OF HIGHEST ORDER STREAM
C             OF THE BASIN (1/(sm**1/3))
C      A0 = GRAVITATIONAL INFILTRATION RATE (cm/hr)
C      S = AVERAGE SORPTIVITY OF THE BASIN (cm/hr**1/2)
C      Q = DISCHARGE (cumec)
C      NQ = NO. OF DISCHARGE VALUES USED
C      FQ = COMPUTED CDF OF PEAK DISCHARGE
C      T = COMPUTED RETURN PERIOD (year)
C      TW = RETURN PERIOD GIVEN BY WEIBULL FORMULA (year)
C      TG = RETURN PERIOD GIVEN BY GRINGORTON FORMULA (year)
C      NCUR = NO. OF CURVES TO BE COMPUTED
C********************************************************************
C
       EXTERNAL F1,F2
       DIMENSION FQ(40,20),Q(40),T(40,20),TW(40),TG(40)
       DIMENSION TITLE(80),AI(4),BI(4)
       REAL MNU
       COMMON/H1/BETA,DELTA,QP,XK1,S
       COMMON/H2/II
C*******************************************************************
C      COEFFICIENTS FOR INTEGRALS
C*******************************************************************
       DATA AI/0.0001,0.1235,0.5033,1.2216/
       DATA BI/0.1235,0.5033,1.2216,2.2962/
       OPEN(UNIT=1,FILE='DDGVBD.DAT',STATUS='OLD')
       OPEN(UNIT=2,FILE='DDGVBD.OUT')
C*******************************************************************
C      READS AND WRITES INPUT DATA
C*******************************************************************
```

```
104     READ(1,*)NCUR
        IF(NCUR.EQ.-99)GO TO 105
        READ(1,1)TITLE
        READ(1,*)NQ
        READ(1,*)(Q(I),I=1,NQ)
1       FORMAT(80A1)
        WRITE(2,1)TITLE
        READ(1,*)A0,S
3       FORMAT(/'MIR(cm/hr)=',F6.3,3X,'MTR(hr)=',F8.3,3X,'MNU=',F5.1/)
4       FORMAT(/'AREA(sq.km)=',f6.1,2x,'RL=',F6.3,2x,'XL(km)=',F6.3,2X
     1  'ALPHA(1/(sm**(1/3)))=',F6.3)
        READ(1,*)A,RL,XL,ALFA
        WRITE(2,4)A,RL,XL,ALFA
        WRITE(2,5)A0,S
5       FORMAT('A0(cm/hr)=',F6.3,2x,'S(cm/hr**0.5)=',F6.3/)
6       FORMAT(2x,'FREQUENCY',7X,'Q(cumec)',7X,'T(comp.)',7X,'T(Weib)
     1',1X,'T(Gring)'/)
C***********************************************************************
C       COMPUTES PARAMETERS OF RAINFALL MODEL, K1, SIGMA AND
C       CONSTANT OF INTEGRATION
C***********************************************************************
        DO 51 I=1,NCUR
        READ(1,*)BETA1,DELTA1,MNU
        WRITE(2,3)BETA1,DELTA1,MNU
        WRITE(2,6)
        BETA=1./BETA1
        DELTA= 1./DELTA1
        XK1=(A*RL)**0.4*ALFA**0.6/XL
        SIGMA=DELTA*(S*BETA/(2.8284*DELTA))**(2./3.)
        SIGMA1=SIGMA+1.
        CALL GAMMA(SIGMA1,GAM)
        CONST=DELTA*EXP(-BETA*A0)*EXP(-2.*SIGMA)*GAM/SIGMA**SIGMA
C
C       EVALUATES INTEGRALS
C*******************************************************************
        DO 52 K=1,NQ
C
C       COMPUTES RETURN PERIOD BY WEIBUL AND GRINGORTON FORMULA
C*****************************************************************
        AA=0.0
        TW(K)=(N+1-2*AA)/(K-AA)
        AA=0.44
        TG(K)=(N+1-2*AA)/(K-AA)
C
C       COMPUTES LOWER LIMIT OF INTEGRATION
C*****************************************
        QP=0.36*Q(K)/A
        XINF=2.2962/QP**0.4/XK1
C
C       COMPUTES AREA OF INTEGRAL I
C**************************************
        AREA=0.
        NV=0
101     XSUP=XINF+5
        EDEL=0.001
        ARE=TEGRAL(F1,XINF,XSUP,EDEL)
        NV=NV+1
        AREA=AREA+ARE
        IF(NV.EQ.1)GO TO 102
```

```
      IF(ABS((AREA-ARE1)/AREA).LE.0.0000001)GO TO 103
102   ARE1=AREA
      XINF=XSUP
      GO TO 101
C
C     COMPUTES AREA OF INTEGRALS J
C***********************************
103   AREA1=0.
      DO 53 II=1,4
      XINF=AI(II)/QP**0.4/XK1
      XSUP=BI(II)/QP**0.4/XK1
      EDEL=0.001
      ARE=TEGRAL(F2,XINF,XSUP,EDEL)
      AREA1=AREA1+ARE
53    CONTINUE
      AREAT=AREA+AREA1
C
C     COMPUTES FREQUENCY AND RECURRENCE INTERVALS
C*****************************************************************
      FQ(K,I)=1.-CONST*AREAT
      T(K,I)=1./(MNU*(1.-FQ(K,I)))
      WRITE(2,7)FQ(K,I),Q(K),T(K,I),TW(K),TG(K)
7     FORMAT(F12.8,2X,F8.0,7X,F12.2,2(5X,F8.2))
52    CONTINUE
51    CONTINUE
      GO TO 104
105   STOP
      END
C
C     COMPUTES ARGUMENT OF FIRST INTEGRAL
C******************************************************************
      FUNCTION F1(X)
      COMMON/H1/BETA,DELTA,QP,XK1,S
      F1=0.0
      ARG=DELTA*X+1.4434*BETA*S**0.1558*QP**0.8442/X**.0779
      IF(ARG.GT.-88.)F1=EXP(-ARG)
      RETURN
      END
C
C     COMPUTES ARGUMENT OF SECOND INTEGRAL
C*****************************************************************
      FUNCTION F2(X)
      DIMENSION CI(4),DI(4),EI(4)
      COMMON/H1/BETA,DELTA,QP,XK1,S
      COMMON/H2/II
C
C     COEFFICIENTS OF INTEGRAL J
C************************************
      DATA CI/0.5,0.65295,0.80482,1.0/
      DATA DI/1.0,1.10812,1.36396,3.1358/
      DATA EI/1.4,1.50812,1.76396,3.5358/
      C=CI(II)
      D=DI(II)
      E=EI(II)
      ARG=1.4434*BETA*S**0.1558/X**0.0779
      ARG=ARG*(2.*(C*QP)**D/(0.871*XK1*X))**(0.8442/E)
      ARG=DELTA*X + ARG
      F2=EXP(-ARG)
      RETURN
```

```
      END
C*********************************************************************
C
      FUNCTION TEGRAL(FI,A,B,EDEL)
C     (AS GIVEN IN APPENDIX - I)
C
      SUBROUTINE GAMMA(X,GAM)
C************************************************************
C     THIS SUBROUTINE COMPUTES GAMMA FUNCTION
C************************************************************
      IER=999
      IF(X.LT.0.0) RETURN
      IER=0.0
      IF(X.LE.20.0) GO TO 10
      Y=1./(X*X)
      P=(0.77783067E-3*Y-0.277765545E-2)*Y+0.8333333309E-1
      P=P/X
      GAM=(X-0.5)*ALOG(X)-X+0.9189385+P
      GAM=EXP(GAM)
      RETURN
10    Y=AINT(X)
      N=Y-2.
      Y=X-Y
      GAM=(((0.1082985985E-1*Y-0.3427052255E-2)*Y+0.77549276E-1)
     1*Y)
      GAM=(((GAM+0.8017824769E-1)*Y+0.4121029027)*Y+0.4227663678)*Y
      GAM=GAM+1.000000199
      T1=1.0
      YP2=Y+2.0
      IF(N) 40,70,60
40    CONTINUE
C  NEGATIVE N
      N=IABS(N)
      DO 45 I=1,N
45    T1=T1*(YP2-I)
      T1=1.0/T1
      GO TO 70
60    CONTINUE
C  POSITIVE N
      N=N-1
      DO 65 I=0,N
65    T1=T1*(YP2+I)
70    GAM=GAM*T1
      RETURN
      END
C*********************************************************************
```

GcIUH - SCS MODEL

```
C*************************************************************************
C
C       THIS PROGRAMME CALCULATES THE FLOOD FREQUENCY CURVE
C       USING BIVARIATE EXPONENTIAL DISTRIBUTION
C       OF RAINFALL INTENSITY AND DURATION, THE INFILTRATION LOSSES
C       REPRESENTED BY SCS CURVE NUMBER METHOD (RAINES & VALDES 1993)
C       AND THE GEOMORPHOCLIMATIC IUH AS EFFECTIVE RAINFALL-RUNOFF
C       MODEL
C
C       THIS PROGRAMME IS WRITTEN BY R.S.KUROTHE, RESEARCH SCHOLAR,
C       DEPT. OF HYDROLOGY, UNIVERSITY OF ROORKEE, ROORKEE, INDIA
C*************************************************************************
C       PARAMETERS AND VARIABLES
C       ************************
C       BETA1 = MEAN AREAL RAINFALL INTENSITY (cm/hr)
C       DELTA1 = MEAN DURATION OF THE STORM (hr)
C       MNU = AVERAGE NO. OF STORMS/YEAR
C       A = AREA OF BASIN (sq km)
C       RL = LENGTH RATIO OF THE BASIN
C       XL = LENGTH OF HIGHEST ORDER STREAM (km)
C       ALFA = KINEMATIC WAVE PARAMETER OF HIGHEST ORDER STREAM
C              OF THE BASIN (1/(sm**1/3))
C       CN = WEIGHTED CURVE NUMBER FOR THE BASIN
C       Q = DISCHARGE (cumec)
C       N = NO. OF DISCHARGE VALUES USED
C       FQ = COMPUTED CDF OF PEAK DISCHARGE
C       T = COMPUTED RETURN PERIOD (year)
C       TW = RETURN PERIOD GIVEN BY WEIBULL FORMULA (year)
C       TG = RETURN PERIOD GIVEN BY GRINGORTON FORMULA (year)
C       NCUR = NO. OF CURVES TO BE COMPUTED
C*********************************************************************
C
        EXTERNAL F1,F2
        DIMENSION FQ(40,10),Q(40),TW(40),TG(40),T(40,10)
        DIMENSION TITLE(40),,AI(4),BI(4)
        REAL MNU
        COMMON/H1/BETA,DELTA,QP,XK1,S
        COMMON/H2/II
C
C   A & B COEFFICIENTS OF INTEGRAL J
C*************************************************************************
        DATA AI/0.0000,0.1235,0.5033,1.2216/
        DATA BI/0.1235,0.5033,1.2216,2.2962/
        OPEN(UNIT=1,FILE='DDRVD.DAT',STATUS='OLD')
        OPEN(UNIT=2,FILE='DDRVD.DAT')
C
C READS AND WRITES INPUT DATA
C*************************************************************************
101     READ(1,*)NCUR
        IF(NCUR.EQ.-99)GO TO 102
        READ(1,1)TITLE
        READ(1,*)N
        READ(1,*)(Q(I),I=1,N)
```

```
      WRITE(2,1)TITLE
1     FORMAT(40A1)
C     WRITE(2,2)NCUR
2     FORMAT(/'NUMBER OF CURVES='I2/)
3     FORMAT(/'Mir(cm/hr)=',F6.3,2x,'Mtr(hr)=',
    1 F6.3,2x,'MNU=',f5.1)
4     FORMAT('AREA(km**2)=',F8.2,2x,'RL=',F6.3,2X,'XL(km)='F6.3/
    1 'ALPHA(1/(s.m**(1/3)))=',F6.3,2x,'CURVE NO.=',F5.2/)
      READ(1,*)A,RL,XL,ALFA,CN
      WRITE(2,4)A,RL,XL,ALFA,CN
5     format(/'FREQUENCY',2X,'Q(cumec)',2X,'T(comp.)',2X,
    1 'T(Weib) ',2X,'T(Gring)'/)
C
C COMPUTES THE INTEGRALS I AND J
C*********************************************************************
      DO 51 I=1,NCUR
      READ(1,*)BETA1,DELTA1,MNU
      WRITE(2,3)BETA1,DELTA1,MNU
      WRITE(2,5)
C
C COMPUTES INVERSE OF MEAN AREAL RAINFALL INTENSITY
C*********************************************************************
      BETA=1./BETA1
      DELTA=1./DELTA1
C
C COMPUTES MAX. POTENTIAL RETENTION, K1 AND CONSTANT
C*********************************************************************
      S=2540/CN-25.4
      SIGMA=DELTA*(0.2*S*BETA/DELTA)**0.5
      SIGMA1=SIGMA+1.
      CALL GAMMA(SIGMA1,GAM)
      CONST=DELTA*EXP(-SIGMA)*GAM/SIGMA**SIGMA
      XK1=(A*RL)**0.4*ALFA**0.6/XL
C
C COMPUTES THE INTEGRAL I
C***************************
      DO 52 K=1,N
      AA=0.0
      TW(K)=(N+1-2*AA)/(K-AA)
      AA=0.44
      TG(K)=(N+1-2*AA)/(K-AA)
      QP=0.36*Q(K)/A
      XINF=2.2962/QP**0.4/XK1
      EDEL=0.0001
      AREA=0.
      NV=0
103   XSUP=XINF+5
      ARE=TEGRAL(F1,XINF,XSUP,EDEL)
      NV=NV+1
      AREA=AREA+ARE
      IF(NV.EQ.1)GO TO 104
      IF(ABS((AREA-ARE1)/AREA).LE.0.0000001)GO TO 105
104   ARE1=AREA
      XINF=XSUP
      GO TO 103
105   AREA1=0.
C
C COMPUTES THE INTEGRALS  J
C***************************
```

```
      DO 53 II=1,4
      XINF=AI(II)/QP**0.4/XK1
      XSUP=BI(II)/QP**0.4/XK1
      EDEL=0.001
      ARE=TEGRAL(F2,XINF,XSUP,EDEL)
      AREA1=AREA1+ARE
53    CONTINUE
C
C COMPUTES FREQUENCY AND RECURRENCE INTERVAL
C**************************************************************
      AREAT=AREA+AREA1
      FQ(K,I)=1.-CONST*AREAT
      T(K,I)=1./(MNU*(1.-FQ(K,I)))
      WRITE(2,6)FQ(K,I),Q(K),T(K,I),TW(K),TG(K)
6     FORMAT(F7.5,2X,F8.0,3(2X,F8.2))
52    CONTINUE
51    CONTINUE
      GO TO 101
102   STOP
      END
C
      FUNCTION F1(X)
C
C COMPUTES ARGUMENT OF INTEGRAL I
C**************************************************************
      COMMON/H1/BETA,DELTA,QP,XK1,S
      F1=0.0
      ARG=DELTA*X+1.39047*BETA*S**0.44161*QP**0.55839/X**0.44161
      IF(ARG.GT.103.)go to 5
      F1=EXP(-ARG)
5     RETURN
      END
C
      FUNCTION F2(X)
C
C COMPUTES ARGUMENT OF INTEGRAL J
C**************************************************************
      DIMENSION CI(4),DI(4),EI(4)
      COMMON/H1/BETA,DELTA,QP,XK1,S
      COMMON/H2/II
C
C COEFFICIENTS OF INTEGRAL J
C*******************************************
      DATA CI/0.5,0.65295,0.80482,1.0/
      DATA DI/1.0,1.10812,1.36396,3.1358/
      DATA EI/1.4,1.50812,1.76396,3.5358/
      F2=0.0
      IF(X.EQ.0.)GO TO 5
      C=CI(II)
      D=DI(II)
      E=EI(II)
      ARG=1.39047*BETA*S**0.44161
      ARG=ARG*(2.*(C*QP)**D/(0.871*XK1*X))**(0.55839/E)/X**0.44161
      ARG=DELTA*X + ARG
      IF(ARG.GT.103.)GO TO 5
      F2=EXP(-ARG)
5     RETURN
      END
C*******************************************************
```

KW - PHILIP MODEL

```
C*************************************************************************
C
C        THIS PROGRAMME CALCULATES THE FLOOD FREQUENCY CURVE
C        FOR A GIVEN BASIN USING BIVARIATE EXPONENTIAL DISTRIBUTION
C        OF RAINFALL INTENSITY AND DURATION, THE INFILTRATION
C        LOSSES REPRESENTED BY PHILIP'S INFILTRATION EQUATION
C        AND THE KINEMATIC WAVE THEORY AS EFFECTIVE
C        RAINFALL-RUNOFF MODEL
C
C        THIS PROGRAMME IS WRITTEN BY R.S.KUROTHE, RESEARCH SCHOLAR,
C        DEPT. OF HYDROLOGY, UNIVERSITY OF ROORKEE, ROORKEE, INDIA
C*************************************************************************
C        PARAMETERS AND VARIABLES
C*************************************
C        LC = LENGTH OF MAIN CHANNEL (ft)
C        W = WIDTH OF OVERLAND PLANE (ft)
C        SP AND SC ARE SLOPE OF OVERLAND PLANE AND CHANNEL RESPECTIVELY
C        NP AND NC ARE MANNING'S ROUGHNESS COEFFICIENT FOR OVERLAND
C        PLANE AND CHANNEL RESPECTIVELY
C        AA AND BB ARE COEFFICIENT AND EXPONENT OF HYDRAULIC RADIUS
C        AND FLOW CROSS-SECTIONAL AREA RELATIONSHIP
C        BETA1 = MEAN AREAL RAINFALL INTENSITY (in/hr)
C        DELTA1 = MEAN DURATION OF STORM (hr)
C        MNU = AVERAGE NO. OF STORMS/YEAR
C        KS = GRAVITATIONAL INFILTRATION RATE (in/hr)
C        S = AVERAGE SORPTIVITY OF THE BASIN (in/hr**1/2)
C        K1 = 0.1558
C        Q = DISCHARGE (cusec)
C        NQ = NO. OF DISCHARGE VALUES USED
C        FQ = COMPUTED CDF OF PEAK DISCHARGE
C        T = COMPUTED RETURN PERIOD (year)
C        TW = RETURN PERIOD GIVEN BY WEIBULL FORMULA (year)
C        TG = RETURN PERIOD GIVEN BY GRINGORTON FORMULA (year)
C*********************************************************************
C
      EXTERNAL F1,F2,F3,F4
      DIMENSION Q(50),FQ(50),T(50),TW(50),TG(50),TITLE(40)
      REAL KS,K1,MNU,IE12,IE24,IE43,LC,NP,NC
      COMMON/H1/BETA,DELTA,S,SI,CC,K1,P,BP,H
      COMMON/H2/IE12,IE24,IE43,TE12,TE24,TE43
      COMMON/H3/C,D,F,BC,TT
      COMMON/H4/G,E,U
      COMMON/H5/LC,W,AP
      OPEN(UNIT=1,FILE='RSCOSPN.DAT',STATUS='OLD')
      OPEN(UNIT=2,FILE='RSCOSPN.OUT')
C READS AND WRITES INPUT DATA
C*************************************************************************
      READ(1,1)TITLE
      WRITE(2,1)TITLE
1     FORMAT(40A1)
2     FORMAT(/'LC(m)=',F9.1,2x,'W(m)=',F9.1,2X,'SP=',F8.4,2X,'SC=',
     1 F6.4/'NP=',F6.4,2X,'NC=',F6.3,2X,'AA=',F5.3,2X,'BB=',F5.3)
```

```
3       FORMAT('MIr(cm/hr)=',F6.3,2X,'MTr(hr)=',f8.2,2x,'MNU=',F7.1)
4       FORMAT('KS(cm/hr)=',F5.2,2X,'S(cm/hr**0.5)=',F6.3,2X,'K1=',
     1  F6.4/)
C
C***********************************************************************
        READ(1,*)LC,W,SP,SC,NP,NC,AA,BB,BETA1,DELTA1,MNU,KS,S,K1
        READ(1,*)NQ
        READ(1,*)(Q(J),J=1,NQ)
C
C       LC AND W ARE CONVERTED IN METRE
C       BETA1 AND KS ARE CONVERTED IN cm/hr
C       S IS CONVERTED IN cm/hr**1/2 FOR OUTPUT FILE
C******************************************************
        WRITE(2,2)LC/3.2808,W/3.2808,SP,SC,NP,NC,AA,BB
        WRITE(2,3)BETA1*2.54,DELTA1,MNU
        WRITE(2,4)KS*2.54,S*2.54,K1
5       FORMAT(/2X,'FREQUENCY',5X,'Q(cumec)',7X,
     1  'T(COMP.)',7X,'T(WEIB)',2X,'T(GRING)'/)
C
C       COMPUTES KW PARAMETERS AND COEFFICIENTS
C*******************************************************************
        AP=1.486*SQRT(SP)/NP
        BP=5./3.
        AC=1.486*AA**(2./3.)*SQRT(SC)/NC
        BC=1.+2.*BB/3.
        C=(W/AP)**(1./BP)
        D=(1.-BP)/BP
        E=W/(AP*BP)
        F=(LC/(AC*(2.*W)**(BC-1.)))**(1./BC)
        G=(LC/(AC*(2.*AP)**(BC-1.)))**(1./BC)
        R=AP/W
C
C COMPUTES INVERSE OF MEAN AREAL RAINFALL INTENSITY AND DURATION
C******************************************************************
        BETA=1./BETA1
        DELTA=1./DELTA1
C
C       COMPUTES CONSTANT, PNR AND CC
C***********************************
        SIGMA=DELTA*(S*BETA/(2.8284*DELTA))**(2./3.)
        SIGMA1=SIGMA+1.
        CALL GAMMA(SIGMA1,GAM)
        CONST=DELTA*EXP(-BETA*KS)*EXP(-2.*SIGMA)*GAM/SIGMA**SIGMA
        PNR=1.-CONST/DELTA
        WRITE(2,*)'PNR(ANALYTICAL)=',PNR
        CC=DELTA*(1.-PNR)
C
C COMPUTES THE INTEGRALS 1 TO 4
C***********************************************************************
        SI=1.4434*BETA*S**K1
        WRITE(2,5)
        DO 51 J=1,NQ
C
C       DISCHARGE IS CONVERTED IN cusec AS IT WAS READ IN cumec
C******************************************************************
        Q(J)=35.314*Q(J)
C
C   COMPUTES COORDINATES OF INTERSECTION POINTS
C*******************************************
```

```
C
        CALL LIMINT(J,Q)
C
C   KW CONSTANTS AND COEFFICIENTS
C**********************************
        H=Q(J)/(2*LC*W)
        P=Q(J)/(2*LC*AP)
        TT=Q(J)/(0.02*LC*W)
        U=Q(J)/(0.02*LC*AP)
C
        IF(J.GE.2)GO TO 104
        AREA1=0.0
        NV=0
        XL1=TE12/3600.
105     XL2=XL1+2.
        EDEL=0.001
        ARE=TEGRAL(F1,XL1,XL2,EDEL)
        NV=NV+1
        AREA1=AREA1+ARE
        IF(NV.EQ.1)GO TO 106
        TEMAX=XL2
        IF(ABS((AREA1-ARE1)/AREA1).LE.0.00000001)GO TO 107
106     ARE1=AREA1
        XL1=XL2
        GO TO 105
104     AREA1=0.0
        XL1=TE12/3600.
108     XL2=XL1+2.
        IF(XL2.LT.TEMAX)THEN
        ARE=TEGRAL(F1,XL1,XL2,EDEL)
        AREA1=AREA1+ARE
        XL1=XL2
        GO TO 108
        ENDIF
107     XL1=TE24/3600.
        XL2=TE12/3600.
        AREA2=TEGRAL(F2,XL1,XL2,EDEL)
        XL1=TE43/3600.
        XL2=TE24/3600.
        AREA3=TEGRAL(F3,XL1,XL2,EDEL)
        XL1=0.0/3600.
        XL2=TE43/3600.
        AREA4=TEGRAL(F4,XL1,XL2,EDEL)
C
C COMPUTES FREQUENCY AND RECURRENCE INTERVAL
C***********************************************************************
        FQ(J)=PNR+AREA1+AREA2+AREA3+AREA4
        IF(FQ(J).GE.1.)THEN
        T(J)=10000
        GO TO 109
        ENDIF
        T(J)=1./(MNU*(1.-FQ(J)))
109     AAA=0.0
        TW(J)=(NQ+1-2*AAA)/(J-AAA)
        AAA=0.44
        TG(J)=(NQ+1-2*AAA)/(J-AAA)
6       FORMAT(2X,F9.7,4X,F8.2,4X,F12.2,4x,F8.2,3X,F8.2)
C
```

```
C       DISCHARGE IS CONVERTED IN cumec FOR OUTPUT FILE
C*****************************************************
        WRITE(2,6)FQ(J),Q(J)/35.314,T(J),TW(J),TG(J)
51      CONTINUE
        STOP
        END
C
C
        FUNCTION F1(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 1
C*****************************************************
        REAL K1
        COMMON/H1/BETA,DELTA,S,SI,CC,K1,P,BP,H
C
        B=SI*(H*12.*3600.)**(1.-K1)
        F1=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**0.0779))
        RETURN
        END
C
        FUNCTION F2(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 2
C*****************************************************
        REAL K1,IE1
        COMMON/H1/BETA,DELTA,S,SI,CC,K1,P,BP,H
C
        CALL SUB2(X,IE1)
        B=SI*IE1**(1.-K1)
        F2=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**0.0779))
        RETURN
        END
C
        FUNCTION F3(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 3
C*****************************************************
        REAL K1,IE2
        COMMON/H1/BETA,DELTA,S,SI,CC,K1,P,BP,H
C
        CALL SUB3(X,IE2)
        B=SI*IE2**(1.-K1)
        F3=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**0.0779))
        RETURN
        END
c
        FUNCTION F4(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 4
C*****************************************************
        REAL K1
        COMMON/H1/BETA,DELTA,S,SI,CC,K1,P,BP,H
C
        IF(X.EQ.0.)THEN
        F4=CC
        GO TO 100
        ENDIF
        AIE=(P/(X*3600.)**BP)**(1./BP)*3600.*12.
        B=SI*AIE**(1.-K1)
```

```
      F4=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**0.0779))
100   RETURN
      END
C
      FUNCTION TEGRAL(FI,A,B,EDEL)
C
C     AS GIVEN IN APPENDIX - I
C
      SUBROUTINE SUB2(TE,IE1)
C*******************************************************************
C     THIS SUBROUTINE COMPUTES VALUES OF EFFECTIVE RAINFALL
C     INTENSITIES (ie) FOR A GIVEN VALUE OF EFFECTIVE RAINFALL
C     DURATION (te) USING SECANT METHOD TO SOLVE THE
C     NON-LINEAR EQUATIONS FOR CASE 2 (CADAVID et al.; 1991)
C
      DIMENSION FX(3),IE(3)
      REAL IE,IE12,IE24,IE1
      COMMON/H2/IE12,IE24,IE43,TE12,TE24,TE43
      COMMON/H3/C,D,F,BC,TT
C
      TE=TE*3600.
      IF(TE.EQ.TE12)THEN
      IE1=IE12*12.*3600.
      TE=TE/3600.
      RETURN
      ENDIF
      IF(TE.EQ.TE24)THEN
      IE1=IE24*12.*3600.
      TE=TE/3600.
      RETURN
      ENDIF
      IE(1)=IE12
      IE(2)=IE24
101   DO 51 I=1,2
      TC=C*IE(I)**D
      TS=F/IE(I)**((BC-1.)/BC)
      Y1=(TT/IE(I)+129.697)/49.878
      Y2=ALOG((100*TE)/(TC+TS))
      FX(I)=Y1-Y2
      IF(FX(I).EQ.0.0)THEN
      IE(I)=IE(3)
      GO TO 102
      ENDIF
51    CONTINUE
      IF(FX(1)*FX(2).GE.0.)THEN
      WRITE(*,*)FX(1),FX(2),IE(1),IE(2)
      WRITE(*,*)'ENTER IE(1),IE(2)-SUB2'
      READ(*,*)IE(1),IE(2)
      GO TO 101
      ENDIF
      N=1000
      DO 52 I=1,N
      IE(3)=(FX(2)*IE(1)-IE(2)*FX(1))/(FX(2)-FX(1))
      TC=C*IE(3)**D
      TS=F/IE(3)**((BC-1.)/BC)
      Y1=(TT/IE(3)+129.697)/49.878
      Y2=ALOG((100*TE)/(TC+TS))
      FX(3)=Y1-Y2
      IF(ABS(FX(3)).LE.0.00001) GO TO 102
```

```
      IF(FX(3).EQ.FX(1).OR.FX(3).EQ.FX(2))THEN
      WRITE(*,*)IE(1),IE(2),IE12,IE24,'SUB2',FX(3),IE(3),TE,TE12,TE24
      GO TO 102
      ENDIF
      IF(FX(3)*FX(1).LT.0.)THEN
      IE(2)=IE(3)
      FX(2)=FX(3)
      ELSE
      IE(1)=IE(3)
      FX(1)=FX(3)
      ENDIF
52    CONTINUE
102   IE1=IE(3)*12.*3600.
      TE=TE/3600.
      RETURN
      END
C*******************************************************************
      SUBROUTINE SUB3(TE,IE1)
C*******************************************************************
C     THIS SUBROUTINE COMPUTES VALUES OF EFFECTIVE RAINFALL
C     INTENSITIES (ie) FOR A GIVEN VALUE OF EFFECTIVE RAINFALL
C     DURATION (te) USING BISECTION METHOD TO SLOVE THE
C     NON-LINEAR EQUATIONS FOR CASE 4 (CADAVID et al.; 1991)
C*******************************************************************
      DIMENSION FX(3),IE(3)
      REAL IE,IE24,IE43,IE1
      COMMON/H1/BETA,DELTA,S,SI,CC,K1,P,BP,H
      COMMON/H2/IE12,IE24,IE43,TE12,TE24,TE43
      COMMON/H3/C,D,F,BC,TT
      COMMON/H4/G,E,U
C
      TE=TE*3600.
      IF(TE.EQ.TE24)THEN
      IE1=IE24*12.*3600.
      TE=TE/3600.
      RETURN
      ENDIF
      IF(TE.EQ.TE43)THEN
      IE1=IE43*12.*3600.
      TE=TE/3600.
      RETURN
      ENDIF
      IE(1)=IE24
      IE(2)=IE43
101   DO 51 i=1,2
      TSS=G/(IE(I)*TE)**(BP*(BC-1.)/BC)
      TP=-D*TE+E/(IE(I)*TE)**(BP-1.)
      Y1=(U/(IE(I)*TE)**BP+118.552)/47.458
      Y2=ALOG((100.*TP)/(TE+TSS))
      FX(I)=Y1-Y2
      IF(FX(I).EQ.0.0)THEN
      IE(I)=IE(3)
      GO TO 102
      ENDIF
51    CONTINUE
      IF(FX(1)*FX(2).GE.0.)THEN
      WRITE(*,*)FX(1),FX(2),IE(1),IE(2)
      WRITE(*,*)'ENTER IE(1),IE(2)-SUB3'
      READ(*,*)IE(1),IE(2)
```

```
      GO TO 101
      ENDIF
      N=1000
      DO 52 I=1,N
      IE(3)=(IE(1)+IE(2))/2.
      TSS=G/(IE(3)*TE)**(BP*(BC-1.)/BC)
      TP=-D*TE+E/(IE(3)*TE)**(BP-1.)
      Y1=(U/(IE(3)*TE)**BP+118.552)/47.458
      Y2=ALOG((100.*TP)/(TE+TSS))
      FX(3)=Y1-Y2
      IF(ABS(FX(3)).LE.0.001) GO TO 102
      IF(FX(3).EQ.FX(1).OR.FX(3).EQ.FX(2))THEN
      WRITE(*,*)IE(1),IE(2),IE24,IE43,'SUB3',FX(3),IE(3),TE,TE24,TE43
      GO TO 102
      ENDIF
      IF(FX(3)*FX(1).LT.0.)THEN
      IE(2)=IE(3)
      FX(2)=FX(3)
      ELSE
      IE(1)=IE(3)
      FX(1)=FX(3)
      ENDIF
52    CONTINUE
102   IE1=IE(3)*12.*3600.
      TE=TE/3600.
      RETURN
      END
C*****************************************************************
      SUBROUTINE LIMINT(J,Q)
C
C     COMPUTES LIMITS OF INTEGRATION IN TERMS OF
C     IE12,TE12,IE24,TE24,IE43 AND TE43 FOR DIFFERENT
C     DICHARGES
C
C******************************************************************
C
      DIMENSION FX(10),IE(10),Q(50)
      REAL LC,IE,IE12,IE24,IE12Q1,IE12Q2
      REAL IE24Q4,IE24Q2,IE43Q4,IE43Q3,IE43
      COMMON/H1/BETA,DELTA,S,SI,CC,K1,P,BP,H
      COMMON/H2/IE12,IE24,IE43,TE12,TE24,TE43
      COMMON/H3/C,D,F,BC,TT
      COMMON/H4/G,E,U
      COMMON/H5/LC,W,AP
C
C     COMPUTATION OF IE12
C**********************************
      IE12Q1=Q(J)/(2.*LC*W)
      IE12Q2=Q(J)/(0.02*LC*W*(-129.697+49.878*ALOG(100.)))
      IE12=(IE12Q1+IE12Q2)/2.
      WRITE(*,*)IE12Q1,IE12Q2,IE12
C
C     COMPUTATION OF TE12
C*******************************
      TC=C*IE12**D
      TS=F/IE12**((BC-1.)/BC)
      TE12=TC+TS
```

```
C       COMPUTATION OF IE24
C*********************************
C       EQUATION Q2
C****************************
        IE(1)=1.0E-6
        IE(2)=1.0E-4
        GO TO 101
102     WRITE(*,*)'ENTER TEST VALUES (TWO) OF IE24 -Q2',Q(J)
        READ(*,*)IE(1),IE(2)
101     DO 51 I=1,2
        Y1=(Q(J)/(0.02*LC*W*IE(I))+129.697)/49.878
        TC=C*IE(I)**D
        TE=TC
        TS=F/IE(I)**((BC-1.)/BC)
        Y2=ALOG((100.*TE)/(TC+TS))
        FX(I)=Y1-Y2
51      CONTINUE
        WRITE(*,*)FX(1),FX(2),IE(1),IE(2)
        IF(FX(1)*FX(2).GE.0.)GO TO 102
        N=1000
        DO 52 I=1,N
        IE(3)=(FX(2)*IE(1)-IE(2)*FX(1))/(FX(2)-FX(1))
        Y1=(Q(J)/(0.02*LC*W*IE(3))+129.697)/49.878
        TC=C*IE(3)**D
        TE=TC
        TS=F/IE(3)**((BC-1.)/BC)
        Y2=ALOG((100*TE)/(TC+TS))
        FX(3)=Y1-Y2
        IF(ABS(FX(3)).LE.0.00001) GO TO 103
        IF(FX(3).EQ.FX(2).OR.FX(3).EQ.FX(1))THEN
        WRITE(*,*)IE(1),IE(2),IE(3),FX(3),'IE24Q2'
        GO TO 103
        ENDIF
        IF(FX(3)*FX(1).LT.0.)THEN
        IE(2)=IE(3)
        FX(2)=FX(3)
        ELSE
        IE(1)=IE(3)
        FX(1)=FX(3)
        ENDIF
52      CONTINUE
103     TE24Q2=C*IE(3)**D
        IE24Q2=IE(3)
C
C*******************************
C       EQUATION Q4
C*******************************
        IE(1)=1.0E-6
        IE(2)=1.0E-4
        GO TO 104
105     WRITE(*,*)'ENTER TEST VALUES (TWO) OF IE24 -Q4',Q(J)
        READ(*,*)IE(1),IE(2)
104     DO 53 I=1,2
        TC=C*IE(I)**D
        TE=TC
        TSS=G/(IE(I)*TE)**(BP*(BC-1.)/BC)
        TP=-D*TE+E/(IE(I)*TE)**(BP-1.)
        Y1=(Q(J)/(0.02*LC*AP*(IE(I)*TE)**BP)+118.552)/47.458
        Y2=ALOG((100.*TP)/(TE+TSS))
```

```
      FX(I)=Y1-Y2
53    CONTINUE
      WRITE(*,*)FX(1),FX(2),IE(1),IE(2)
      IF(FX(1)*FX(2).GE.0.)GO TO 105
      N=1000
      DO 54 I=1,N
      IE(3)=(FX(2)*IE(1)-IE(2)*FX(1))/(FX(2)-FX(1))
      TC=C*IE(3)**D
      TE=TC
      TSS=G/(IE(3)*TE)**(BP*(BC-1.)/BC)
      TP=-D*TE+E/(IE(3)*TE)**(BP-1.)
      Y1=(Q(J)/(0.02*LC*AP*(IE(3)*TE)**BP)+118.552)/47.458
      Y2=ALOG((100*TP)/(TE+TSS))
      FX(3)=Y1-Y2
      IF(ABS(FX(3)).LE.0.00001) GO TO 106
      IF(FX(3).EQ.FX(2).OR.FX(3).EQ.FX(1))THEN
      WRITE(*,*)IE(1),IE(2),IE(3),FX(3),'IE24Q4'
      GO TO 106
      ENDIF
      IF(FX(3)*FX(1).LT.0.)THEN
      IE(2)=IE(3)
      FX(2)=FX(3)
      ELSE
      IE(1)=IE(3)
      FX(1)=FX(3)
      ENDIF
54    CONTINUE
106   TE24Q4=C*IE(3)**D
      IE24Q4=IE(3)
      IE24=(IE24Q2+IE24Q4)/2.
      WRITE(*,*)IE24Q2,IE24Q4,IE24
      TE24=(TE24Q2+TE24Q4)/2.
      WRITE(*,*)TE24Q2,TE24Q4,TE24
C
C     COMPUTATION OF IE43
C*******************************
C     EQUATION Q4
C*****************************
      IE(1)=1.0E-6
      IE(2)=1.0E-4
      GO TO 107
108   WRITE(*,*)'ENTER TEST VALUES (TWO) OF IE43 -Q4',Q(J)
      READ(*,*)IE(1),IE(2)
      IF(IE(1).EQ.-1)THEN
      TE43=0.05
      IE(3)=((Q(J)/(2.*LC*AP))**(1./BP))/TE43
      GO TO 112
      ENDIF
107   DO 55 i=1,2
      TE=(Q(J)/(0.02*LC*AP*IE(I)**BP*(-118.552+47.458*ALOG(100.))))
     1 **(1./BP)
      TSS=G/(IE(I)*TE)**(BP*(BC-1.)/BC)
      TP=-D*TE+E/(IE(I)*TE)**(BP-1.)
      FX(I)=TE-TP+TSS
55    CONTINUE
      WRITE(*,*)FX(1),FX(2),IE(1),IE(2)
      IF(FX(1)*FX(2).GE.0.)GO TO 108
      N=1000
      DO 56 I=1,N
```

```
c       IE(3)=(FX(2)*IE(1)-IE(2)*FX(1))/(FX(2)-FX(1))
        IE(3)=(IE(1)+IE(2))/2.
        TE=(Q(J)/(0.02*LC*AP*IE(3)**BP*(-118.552+47.458*ALOG(100.))))
     1  **(1./BP)
        TSS=G/(IE(3)*TE)**(BP*(BC-1.)/BC)
        TP=-D*TE+E/(IE(3)*TE)**(BP-1.)
        FX(3)=TE-TP+TSS
        IF(ABS(FX(3)).LE.0.00001) GO TO 109
        IF(FX(3).EQ.FX(2).OR.FX(3).EQ.FX(1))THEN
        WRITE(*,*)IE(1),IE(2),IE(3),FX(3),'IE43Q4'
        GO TO 109
        ENDIF
        IF(FX(3)*FX(1).LT.0.)THEN
        IE(2)=IE(3)
        FX(2)=FX(3)
        ELSE
        IE(1)=IE(3)
        FX(1)=FX(3)
        ENDIF
56      CONTINUE
109     WRITE(*,*)I,FX(3)
        TE43Q4=TP-TSS
        IE43Q4=IE(3)
C
C*********************************
C       EQUATION Q3
C*********************************
C
        IE(1)=1.0E-6
        IE(2)=1.0E-4
        GO TO 110
111     WRITE(*,*)'ENTER TEST VALUES (TWO) OF IE43 -Q3'
        READ(*,*)IE(1),IE(2)
        IF(IE(1).EQ.-1)THEN
        TE43=0.05
        IE(3)=((Q(J)/(2.*LC*AP))**(1./BP))/TE43
        GO TO 112
        ENDIF
110     DO 57 i=1,2
        TE=(Q(J)/(2.*LC*AP*IE(I)**BP))**(1./BP)
        TSS=G/(IE(I)*TE)**(BP*(BC-1.)/BC)
        TP=-D*TE+E/(IE(I)*TE)**(BP-1.)
        FX(I)=TE-TP+TSS
57      CONTINUE
        WRITE(*,*)FX(1),FX(2),IE(1),IE(2)
        IF(FX(1)*FX(2).GE.0.)GO TO 111
        N=1000
        DO 58 I=1,N
c       IE(3)=(FX(2)*IE(1)-IE(2)*FX(1))/(FX(2)-FX(1))
        IE(3)=(IE(1)+IE(2))/2.
        TE=(Q(J)/(2.*LC*AP*IE(3)**BP))**(1./BP)
        TSS=G/(IE(3)*TE)**(BP*(BC-1.)/BC)
        TP=-D*TE+E/(IE(3)*TE)**(BP-1.)
        FX(3)=TE-TP+TSS
        IF(ABS(FX(3)).LE.0.00001) GO TO 113
        IF(FX(3).EQ.FX(2).OR.FX(3).EQ.FX(1))THEN
        WRITE(*,*)IE(1),IE(2),IE(3),FX(3),'IE43Q3'
        GO TO 113
        ENDIF
```

```
        IF(FX(3)*FX(1).LT.0.)THEN
        IE(2)=IE(3)
        FX(2)=FX(3)
        ELSE
        IE(1)=IE(3)
        FX(1)=FX(3)
        ENDIF
58      CONTINUE
113     WRITE(*,*)I,FX(3)
        TE43Q3=TP-TSS
        IE43Q3=IE(3)
        IE43=(IE43Q3+IE43Q4)/2.
        TE43=(TE43Q3+TE43Q4)/2.
        WRITE(*,*)IE43Q3,IE43Q4,IE43
        WRITE(*,*)TE43Q3,TE43Q4,TE43
        GO TO 114
112     IE43=IE(3)
        WRITE(*,*)IE43,TE43
114     RETURN
        END
```

KW - SCS MODEL

```
C*************************************************************************
C       FPS SYSTEM
C
C       THIS PROGRAMME CALCULATES THE FLOOD FREQUENCY CURVE
C       FOR A GIVEN BASIN USING BIVARIATE EXPONENTIAL DISTRIBUTION
C       OF RAINFALL INTENSITY AND DURATION, THE INFILTRATION
C       LOSSES REPRESENTED BY SCS CURVE NUMBER MEHTOD AND THE
C       KINEMATIC WAVE THEORY AS EFFECTIVE RAINFALL-RUNOFF MODEL
C
C       THIS PROGRAMME IS WRITTEN BY R.S.KUROTHE, RESEARCH SCHOLAR,
C       DEPT. OF HYDROLOGY, UNIVERSITY OF ROORKEE, ROORKEE, INDIA
C*************************************************************************
C       PARAMETERS AND VARIABLES
C***************************************
C       LC = LENGTH OF MAIN CHANNEL (ft)
C       W = WIDTH OF OVERLAND PLANE (ft)
C       SP and SC ARE SLOPE OF OVERLAND PLANE AND CHANNEL RESPECTIVELY
C       NP and NC ARE MANNING'S ROUGHNESS COEFFICIENT FOR OVERLAND
C       PLANE AND CHANNEL RESPECTIVELY
C       AA AND BB ARE COEFFICIENT AND EXPONENT OF HYDRAULIC RADIOUS
C       AND FLOW CROSS-SECTIONAL AREA RELATIONSHIP
C       BETA1 = MEAN AREAL RAINFALL INTENSITY (in/hr)
C       DELTA1 = MEAN DURATION OF STORM (hr)
C       MNU = AVERAGE NO. OF STORMS/YEAR
C       CN = WEIGHTED CURVE NUMBER FOR THE BASIN
C       K1 = 0.44161
C       Q = DISCHARGE (cumec)
C       NQ = NO. OF DISCHARGE VALUES USED
C       FQ = COMPUTED CDF OF PEAK DISCHARGE
C       T = COMPUTED RETURN PERIOD (year)
C       TW = RETURN PERIOD GIVEN BY WEIBULL FORMULA (year)
C       TG = RETURN PERIOD GIVEN BY GRINGORTON FORMULA (year)
C*************************************************************
C
        EXTERNAL F1,F2,F3,F4
        DIMENSION Q(50),FQ(50),T(50),TW(50),TG(50),TITLE(40)
        REAL K1,MNU,LC,NP,NC
        COMMON/H1/DELTA,CC,SI,H,R,P,K1
        COMMON/H3/BP,TT,U,IE12,IE24,IE43,TE12,TE24,TE43
        COMMON/H4/BETA,S
        COMMON/H5/LC,W,SP,SC,NP,NC,AA,BB
        COMMON/H6/AP,AC,BC,C,D,E,F,G
        OPEN(UNIT=1,FILE='DDKGM.DAT',STATUS='OLD')
        OPEN(UNIT=2,FILE='DDKGM.OUT')
C
C READS AND WRITES INPUT DATA
C*************************************************************************
        READ(1,1)TITLE
        WRITE(2,1)TITLE
1       FORMAT(40A1)
2       FORMAT(/'LC(m)=',F9.1,2x,'W(m)=',F9.1,2X,'SP=',F8.4,2X,'SC='
     1  ,F6.4/'NP=',F6.4,4X,'NC=',F6.3,4x,'AA=',F5.3,4X,'BB=',F5.3)
```

```
3       FORMAT('MIr(cm/hr)=',F6.3,2X,'MTr(hr)=',f6.3,2x,'MNU=',F5.1)
4       FORMAT('CN=',F5.2,2X,'K1=',F8.6/)
        READ(1,*)LC,W,SP,SC,NP,NC,AA,BB,BETA1,DELTA1,MNU,CN,K1
        READ(1,*)NQ
        READ(1,*)(Q(I),I=1,NQ)
C
C       LC AND W ARE CONVERTED IN METRE
C       BETA1 AND KS ARE CONVERTED IN cm/hr
C       S IS CONVERTED IN cm/hr**1/2 FOR OUTPUT FILE
C***************************************************
        WRITE(2,2)LC/3.2808,W/3.2808,SP,SC,NP,NC,AA,BB
        WRITE(2,3)BETA1*2.54,DELTA1,MNU
        WRITE(2,4)CN,K1
        WRITE(*,*)LC,W,SP,SC,NP,NC,AA,BB,BETA1,DELTA1,MNU,CN,K1
5       FORMAT(/2X,'FREQUENCY',5X,'Q(cumec)',2X,
     1  'T(COMP)(years)',2X,'T(WEIB)',4X,'T(GRING)'/)
C
C       COMPUTES KW PARAMETERS AND COEFFICIENTS
C       (SEE CADAVID et al. (1991) FOR EQUATIONS. C,D,E,F,G,R ARE
C       USED FOR SIMPLIFING THE COMPUTATION)
C*****************************************************************
        AP=1.486*SQRT(SP)/NP
        BP=5./3.
        AC=1.486*AA**(2./3.)*SQRT(SC)/NC
        BC=1.+2.*BB/3.
        C=(W/AP)**(1./BP)
        D=(1.-BP)/BP
        E=W/(AP*BP)
        F=(LC/(AC*(2.*W)**(BC-1.)))**(1./BC)
        G=(LC/(AC*(2.*AP)**(BC-1.)))**(1./BC)
        R=AP/W
C
C COMPUTES INVERSE OF MEAN AREAL RAINFALL INTENSITY AND DURATION
C********************************************************************
        BETA=1./BETA1
        DELTA=1./DELTA1
C
C  COMPUTES MAX. POTENTIAL RETENTION, SIGMA, CONSTANT, PNR AND CC
C********************************************************************
        S=(1000./CN)-10.
        SIGMA=DELTA*SQRT(0.2*S*BETA/DELTA)
        SIGMA1=SIGMA+1.
        CALL GAMMA(SIGMA1,GAM)
        CONST=EXP(-SIGMA)*GAM/SIGMA**SIGMA
        PNR=1.-CONST
        WRITE(2,*)'PNR(ANALYTICAL)=',PNR
        CC=DELTA*CONST
C
C       COMPUTES THE INTEGRALS 1 TO 4
C********************************************************************
        SI=1.39047*BETA*S**K1
        WRITE(2,5)
        DO 51 J=1,NQ
C       DISCHARGE IS CONVERTED IN cusec AS IT WAS
C       READ IN cumec FROM DATA FILE
C***************************************************
        Q(J)=Q(J)*35.314
C
```

```
C   COMPUTES COORDINATES OF INTERSECTION POINTS
C*******************************************
C
       CALL LIMINT(J,Q)
C
C   COMPUTES KW CONSTANTS AND COEFFICIENTS
C       (SEE CADAVID et al., 1991 FOR EQUATIONS. H,P,TT,U ARE
C       USED FOR SIMPLIFING THE COMPUTATION)
C*************************************************************
       H=Q(J)/(2*LC*W)
       P=Q(J)/(2*LC*AP)
       TT=Q(J)/(0.02*LC*W)
       U=Q(J)/(0.02*LC*AP)
C
C       COMPUTES INTEGRAL 1
C******************************
       IF(J.GE.2)GO TO 107
       AREA1=0.0
       NV=0
       XL1=TE12/3600.
104    XL2=XL1+2.
       EDEL=0.001
       ARE=TEGRAL(F1,XL1,XL2,EDEL)
       NV=NV+1
       AREA1=AREA1+ARE
       IF(NV.EQ.1)GO TO 105
       TEMAX=XL2
       IF(ABS((AREA1-ARE1)/AREA1).LE.0.000000001)GO TO 106
105    ARE1=AREA1
       XL1=XL2
       GO TO 104
107    AREA1=0.0
       XL1=TE12/3600.
108    XL2=XL1+2.
       IF(XL2.LT.TEMAX)THEN
       ARE=TEGRAL(F1,XL1,XL2,EDEL)
       AREA1=AREA1+ARE
       XL1=XL2
       GO TO 108
       ENDIF
C
C       COMPUTES INTEGRAL 2
C*******************************
106    XL1=TE24/3600.
       XL2=TE12/3600.
       EDEL=0.001
       AREA2=TEGRAL(F2,XL1,XL2,EDEL)
C
C       COMPUTES INTEGRAL 3
C***********************************
       XL1=TE43/3600.
       XL2=TE24/3600.
       EDEL=0.001
       AREA3=TEGRAL(F3,XL1,XL2,EDEL)
C
C       COMPUTES INTEGRAL 4
C*********************************
       XL1=0.0/3600.
       XL2=TE43/3600.
```

```
       EDEL=0.001
       AREA4=TEGRAL(F4,XL1,XL2,EDEL)
C
C      COMPUTES FREQUENCY AND RECURRENCE INTERVAL
C******************************************************************
       FQ(J)=PNR+AREA1+AREA2+AREA3+AREA4
       T(J)=1./(MNU*(1.-FQ(J)))
C
C      COMPUTES RETURN PERIODS BY WEIBULL AND GRINGIRTON FORMULA
C******************************************************************
       AAA=0.0
       TW(J)=(NQ+1-2*AAA)/(J-AAA)
       AAA=0.44
       TG(J)=(NQ+1-2*AAA)/(J-AAA)
C*************************************
6      FORMAT(2X,F9.7,4X,F8.2,4X,F8.2,4x,F8.2,3X,F8.2)
       WRITE(2,6)FQ(J),Q(J)/35.314,T(J),TW(J),TG(J)
51     CONTINUE
       STOP
       END
C
       FUNCTION F1(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 1
C*******************************************************************
       REAL K1
       COMMON/H1/DELTA,CC,SI,H,R,P,K1
C
       IF((DELTA*X).GE.104.)THEN
C   SINCE THE COMPILER COMPUTE EXP(-104)=0.0
C
       F1=0.0
       GOTO 101
       ENDIF
       B=SI*(H*12.*3600.)**(1.-K1)
       F1=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**K1))
101    RETURN
       END
C
       FUNCTION F2(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 2
C*******************************************************************
       REAL K1,IE1
       COMMON/H1/DELTA,CC,SI,H,R,P,K1
C
       CALL SUB2(X,IE1)
       B=SI*IE1**(1.-K1)
       F2=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**K1))
       RETURN
       END
C
       FUNCTION F3(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 3
C*******************************************************************
       REAL K1,IE2
       COMMON/H1/DELTA,CC,SI,H,R,P,K1
C
```

```
      CALL SUB3(X,IE2)
      B=SI*IE2**(1.-K1)
      F3=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**K1))
      RETURN
      END
C
      FUNCTION F4(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 4
C***********************************************************************
      REAL K1
      COMMON/H1/DELTA,CC,SI,H,R,P,K1
      COMMON/H3/BP,TT,U,IE12,IE24,IE43,TE12,TE24,TE43
C
      IF(X.EQ.0.) THEN
      F1=CC
      GO TO 101
      ENDIF
      AIE=(P/(X*3600.)**BP)**(1./BP)*3600.*12.
      B=SI*AIE**(1.-K1)
      F4=CC*EXP(-DELTA*X)*(1.-EXP(-B/X**K1))
101   RETURN
      END
C
C***********************************************************************
C
      FUNCTION TEGRAL(FI,A,B,EDEL)
C
C     AS GIVEN IN APPENDIX - I
C
      SUBROUTINE GAMMA(X,GAM)
C
C     AS GIVEN IN APPENDIX - II
C
      SUBROUTINE SUB2(TE,IE1)
C
C     AS GIVEN IN APPENDIX - IV
C
      SUBROUTINE SUB3(TE,IE1)
C
C     AS GIVEN IN APPENDIX - IV
C
      SUBROUTINE LIMINT(J,Q)
C
C     AS GIVEN IN APPENDIX - IV
C
```

KW - φ-INDEX MODEL

```
C***************************************************************************
C
C       THIS PROGRAMME CALCULATES THE FLOOD FREQUENCY CURVE
C       FOR A GIVEN BASIN USING
C       STOCHASTIC RAINFALL MODEL-BIVARIATE EXPONENTIAL (CORRELATED)
C       INFILTRATION MODEL-CONCEPTUAL LOSS RATE (PHI-INDEX)
C       RAINFALL-RUNOFF MODEL-KINEMATIC WAVE THEORY
C
C       THIS PROGRAMME IS WRITTEN BY R.S.KUROTHE, RESEARCH SCHOLAR,
C       DEPT. OF HYDROLOGY, UNIVERSITY OF ROORKEE, ROORKEE, INDIA
C***************************************************************************
C       PARAMETERS AND VARIABLES
C*************************************
C       LC = LENGTH OF MAIN CHANNEL (m)
C       W = WIDTH OF OVERLAND PLANE (m)
C       SP and SC ARE SLOPE OF OVERLAND PLANE AND CHANNEL RESPECTIVELY
C       NP and NC ARE MANNING'S ROUGHNESS COEFFICIENT FOR OVERLAND
C       PLANE AND CHANNEL RESPECTIVELY
C       AA AND BB ARE COEFFICIENT AND EXPONENT OF HYDRAULIC RADIOUS
C       AND FLOW CROSS-SECTIONAL AREA RELATIONSHIP
C       BETA1 = MEAN AREAL RAINFALL INTENSITY (cm/hr)
C       DELTA1 = MEAN DURATION OF STORM (hr)
C       MNU = AVERAGE NO. OF STORMS/YEAR
C       K1 = 0.44161
C       PHI = PHI-INDEX FOR THE BASIN (cm/hr)
C       Q = DISCHARGE (cumec)
C       NQ = NO. OF DISCHARGE VALUES USED
C       FQ = COMPUTED CDF OF PEAK DISCHARGE
C       T = COMPUTED RETURN PERIOD (year)
C       TW = RETURN PERIOD GIVEN BY WEIBULL FORMULA (year)
C       TG = RETURN PERIOD GIVEN BY GRINGORTON FORMULA (year)
C***************************************************************************
C
      EXTERNAL F1,F2,F3,F4
      DIMENSION Q(40),FQ(40),T(40),TW(40),TG(40),TITLE(40)
      REAL MNU,IE12,IE24,IE43,LC,NP,NC
      COMMON/H1/DELTA,H,R,P,BETA,GAMA,PHI
      COMMON/H2/C,D,E,F,G,BC
      COMMON/H3/BP,TT,U,IE12,IE24,IE43,TE12,TE24,TE43
      COMMON/H4/BI,BII,BIII
      COMMON/H5/LC,W,AP
      OPEN(UNIT=1,FILE='RS.DAT',STATUS='OLD')
      OPEN(UNIT=2,FILE='RS.OUT')
C
C READS AND WRITES INPUT DATA
C***************************************************************************
      READ(1,1)TITLE
      WRITE(2,1)TITLE
1     FORMAT(40A1)
2     FORMAT(/'LC(m)=',F9.1,2x,'W(m)=',F9.1,2X,'SP=',F8.4,2X,'SC=',
     1 F6.4/'NP=',F6.4,4X,'NC=',F6.3,4X,'AA=',F5.3,4X,'BB=',F5.3)
3     FORMAT('MIR(cm/hr)=',F6.3,5X,'MTR(hr)=',f8.3,5x,'MNU=',F5.1)
4     FORMAT('PHI(cm/hr)=',F5.2,2X,'GAMA=',F6.3/)
```

```
      READ(1,*)NQ
      READ(1,*)(Q(J),J=1,NQ)
      READ(1,*)LC,W,SP,SC,NP,NC,AA,BB,BETA1,DELTA1,MNU,PHI,GAMA
      WRITE(2,2)LC,W,SP,SC,NP,NC,AA,BB
      WRITE(2,3)BETA1,DELTA1,MNU
      WRITE(2,4)PHI,GAMA
C
C     COMPUTES PARAMETERS OF STOCHASTIC RAINFALL MODEL
C*****************************************************************
      BETA=1./BETA1
      DELTA=1./DELTA1
C
C     COMPUTES CONSTANTS OF INTEGRAL FUNCTION
C*******************************************
C
      BI=1.+BETA*GAMA*PHI
      BII=DELTA*BI
      BIII=BETA*DELTA*GAMA
C
C     COMPUTES KW PARAMETERS AND COEFFICIENTS
C     (SEE CADAVID et al. (1991) FOR EQUATIONS. C,D,E,F,G,R ARE
C     USED FOR SIMPLIFING THE COMPUTATION)
C*****************************************************************
      AP=SQRT(SP)/NP
      BP=5./3.
      AC=AA**(2./3.)*SQRT(SC)/NC
      BC=1.+2.*BB/3.
      C=(W/AP)**(1./BP)
      D=(1.-BP)/BP
      E=W/(AP*BP)
      F=(LC/(AC*(2.*W)**(BC-1.)))**(1./BC)
      G=(LC/(AC*(2.*AP)**(BC-1.)))**(1./BC)
      R=AP/W
C
C     COMPUTES PNR
C*****************************************************************
      PNR=1.-EXP(-BETA*PHI)
      WRITE(2,*)'PNR=',PNR
      WRITE(2,5)
5     FORMAT(2X,'FREQUENCY',10X,'DISCHARGE',2X,'RECURRENCE INTERVAL'
     1 ,2X,'T(WEIB.)',2X,'T(GRING.)')
      WRITE(2,6)
6     FORMAT(22X,'(cumec)',10X,'(year)',9X,'(year)',5X,'(year)'/)
C
C COMPUTES THE INTEGRALS 1 TO 4
C*****************************************************************
      DO 51 J=1,NQ
C
C  COORDINATES OF INTERSECTION POINTS
C***********************************
      CALL LIMINT(J,Q)
C
C  KW CONSTANTS AND COEFFICIENTS
C     (SEE CADAVID et al. (1991) FOR EQUATIONS. H,P,TT,U ARE
C     USED FOR SIMPLIFING THE COMPUTATION)
C*****************************
      H=Q(J)/(2*LC*W)
      P=Q(J)/(2*LC*AP)
      TT=Q(J)/(0.02*LC*W)
```

```
      U=Q(J)/(0.02*LC*AP)
C
C     COMPUTES INTEGRALS 1 TO 4
C
C     XL1 = LOWER LIMIT, XL2 = UPPER LIMIT
C     EDEL IS THE REQUIRED TOLERANCE IN INTEGRATION
C*****************************************************
C
C     INTEGRAL 1
C*****************
      IF(J.GE.2)GO TO 106
      AREA1=0.0
      NV=0
      XL1=TE12/3600.
101   XL2=XL1+2.
      EDEL=0.001
      ARE=TEGRAL(F1,XL1,XL2,EDEL)
      NV=NV+1
      AREA1=AREA1+ARE
      IF(NV.EQ.1)GO TO 102
      TEMAX=XL2
      IF(ABS((AREA1-ARE1)/AREA1).LE.0.0000001)GO TO 103
102   ARE1=AREA1
      XL1=XL2
      GO TO 101
106   AREA1=0.0
      XL1=TE12/3600.
107   XL2=XL1+2.
      IF(XL2.LT.TEMAX)THEN
      ARE=TEGRAL(F1,XL1,XL2,EDEL)
      AREA1=AREA1+ARE
      XL1=XL2
      GO TO 107
      ENDIF
C
C     INTEGRAL 2
C*****************
103   XL1=TE24/3600.
      XL2=TE12/3600.
      EDEL=0.001
      AREA2=TEGRAL(F2,XL1,XL2,EDEL)
C
C     INTEGRAL 3
C*****************
      XL1=TE43/3600.
      XL2=TE24/3600.
      EDEL=0.001
      AREA3=TEGRAL(F3,XL1,XL2,EDEL)
C
C     INTEGRAL 4
C******************
      IF(TE43.EQ.0.05)THEN
      AREA4=0.
      GO TO 104
      ENDIF
      XL1=0.05/3600.
      XL2=TE43/3600.
      EDEL=0.001
      AREA4=TEGRAL(F4,XL1,XL2,EDEL)
```

```
C
C COMPUTES FREQUENCY AND RECURRENCE INTERVAL
C*********************************************************************
C
104     AAA=0.0
        TW(J)=(NQ+1-2*AAA)/(J-AAA)
        AAA=0.44
        TG(J)=(NQ+1-2*AAA)/(J-AAA)
        FQ(J)=PNR+AREA1+AREA2+AREA3+AREA4
        IF(FQ(J).GE.1.)THEN
        T(J)=10000
        GO TO 105
        ENDIF
        T(J)=1./(MNU*(1.-FQ(J)))
105     WRITE(2,7)FQ(J),Q(J),T(J),TW(J),TG(J)
7       FORMAT(2X,F12.10,7X,F8.1,7X,F8.2,7x,F8.2,3x,F8.2)
51      CONTINUE
        STOP
        END
C
        FUNCTION F1(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 1
C***********************************************************************
        COMMON/H1/DELTA,H,R,P,BETA,GAMA,PHI
        COMMON/H4/BI,BII,BIII
C
        B=H*100.*3600.
        F1=DELTA*EXP(-BETA*PHI-BII*X)*(BI
     1  -(BI+BETA*GAMA*B)*EXP(-(BETA+BIII*X)*B))
        RETURN
        END
C
        FUNCTION F2(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 2
C***********************************************************************
        REAL IE1
        COMMON/H1/DELTA,H,R,P,BETA,GAMA,PHI
        COMMON/H4/BI,BII,BIII
C
        CALL SUB2(X,IE1)
        B=IE1
        F2=DELTA*EXP(-BETA*PHI-BII*X)*(BI
     1  -(BI+BETA*GAMA*B)*EXP(-(BETA+BIII*X)*B))
        RETURN
        END
C
        FUNCTION F3(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 3
C***********************************************************************
        REAL IE2
        COMMON/H1/DELTA,H,R,P,BETA,GAMA,PHI
        COMMON/H4/BI,BII,BIII
C
        CALL SUB3(X,IE2)
        B=IE2
        F3=DELTA*EXP(-BETA*PHI-BII*X)*(BI
```

```
     1  -(BI+BETA*GAMA*B)*EXP(-(BETA+BIII*X)*B))
        RETURN
        END
C
        FUNCTION F4(X)
C
C COMPUTES ARGUMENT OF INTEGRAL 4
C*****************************************************************
        COMMON/H1/DELTA,H,R,P,BETA,GAMA,PHI
        COMMON/H3/BP,TT,U,IE12,IE24,IE43,TE12,TE24,TE43
        COMMON/H4/BI,BII,BIII
C
        B=(P/(X*3600.)**BP)**(1./BP)*3600.*100.
        F4=DELTA*EXP(-BETA*PHI-BII*X)*(BI-(BI+BETA*GAMA*B)*
     1  EXP(-(BETA+BIII*X)*B))
        RETURN
        END
C***************************************************************
        FUNCTION TEGRAL(FI,A,B,EDEL)
C
C       AS GIVEN IN APPENDIX - I
C
        SUBROUTINE SUB2(TE,IE1)
C
C       AS GIVEN IN APPENDIX - IV
C
        SUBROUTINE SUB3(TE,IE1)
C
C       AS GIVEN IN APPENDIX - IV
C
        SUBROUTINE LIMINT(J,Q)
C
C       AS GIVEN IN APPENDIX - IV
```

NOTATIONS

A = flow cross-sectional area of channel

= area of the watershed

A_o = gravitational infiltration rate

a = coefficient of flow cross-section and hydraulic radius relationship

b = exponent of flow cross-section and hydraulic radius relationship

CN = Soil Conservation Service curve number

DFFD = derived flood frequency distribution

$F_{I_e,T_e}(i_e,t_e)$ = cumulative distribution function of i_e and t_e

$F_{Q_p}(Q_p)$ = cumulative distribution function of Q_p

f_i = infiltration rate

$f_{I_e,T_e}(i_e,t_e)$ = probability density function of i_e and t_e

GIUH = geomorphologic instantaneous unit hydrograph

GcIUH= geomorphoclimatic instantaneous unit hydrograph

i_e = effective rainfall intensity

i_r = areal rainfall intensity

K_s = hydraulic conductivity

KW = kinematic wave

L_c = length of main channel

L_Ω = length of highest order stream

m_ν = mean number of independent storms per year

n_c = Manning's roughness coefficient for channel

n_p = Manning's roughness coefficient for plane

P = total rainfall depth

PDF = probability density function

P_{NR} = probability of null runoff

Q_p = peak discharge from the catchment

q_L = discharge from unit width of plane entering into channel

q_p = IUH peak

R = excess rainfall depth
R_L = length ratio
Rs = surface runoff
S = maximum potential retention
S_c = channel slope
S_i = infiltration sorptivity
S_p = plane slope
SCS = Soil Conservation Service (USDA)
T = return period
t = time
t_b = IUH time base
t_c = time of concentration for plane
t_e = effective rainfall duration
t_o = time of ponding
t_p = IUH time to peak
t_r = point/areal storm duration
t^* = total time of concentration
W = width of overland plane
y = depth of overland flow
α_c = kinematic wave parameter for channel
α_p = kinematic wave parameter for plane
α_Ω = kinematic wave parameter for highest order stream, $m^{-1/3}s^{-1}$.
β = inverse of mean areal storm intensity
β_c = exponent of area - discharge relationship for channel
β_p = exponent of depth - discharge relationship for plane
γ = parameter describing correlation between intensity and duration
δ = inverse of mean storm duration
ϕ = spatially averaged potential loss rate (ϕ - index)

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