IDENTIFICATION OF LINEARITY AND NONLINEARITY OF DRAINAGE BASINS

A DISSERTATION

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

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MASTER OF TECHNOLOGY

in

WATER RESOURCES DEVELOPMENT

By

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CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the dissertation entitled **"IDENTIFICATION OF LINEARITY AND NONLINEARITY OF DRAINAGE BASINS"**, in partial fulfillment of the requirements for the award of the degree of **Master of Technology** in **Water Resources Development (Civil)**, submitted in the Water Resources Development Training Centre, Indian Institute of Technology Roorkee, is an authentic record of my own work carried out during the period July 16th 2002 to 30th November 2002 under the supervision of **Dr. U. C. Chaube**, Professor, WRDTC, Indian Institute of Technology Roorkee (IITR).

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

Date : December 02, 2002

Place : Roorkee

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

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(I Ketut Suarbawa)

ABSTRACT

Drainage basin hydrologic linearity is defined as the condition that exists on a drainage basin when runoff volume is directly proportional to precipitation volume. Hydrologic nonlinear exists when runoff volume is not directly proportional to precipitation volume.

Original standardized peak discharge distribution (OPDD) is defined as the log of peak discharge regressed on log runoff volume. The OPDD and its variants were developed and tested by Rogers (1980, 1982) for basins in the U.S.A. The distributions have been tested for eight drainage basins in Greece (Mimikou, 1983). Singh and Aminian, (1986) proposed peak discharge distribution per unit area.

The inherent assumption in OPDD is that time to peak (T_p) and ratio of time to peak (T_p) and base period (T_b) are constant.

In the present study, peak discharge, time and volume relationship (PDTVR) defined as relation between peak discharge (Q_p), time to peak (T_p) and runoff volume (V) in logarithm space has also been investigated in addition to above mentioned distribution. Regression analysis in log space shows strong correlation between time to peak (Tp) and base period (Tb) and weak correlation between Qp & Tp and between V & Tp. Ratio Tb/Tp is not a fixed value 2.67 as assumed by Mockus. For linear basin, it is nearly 2.67 but for nonlinear basins, it is different and not necessarily constant.

For testing the applicability of OPDD and its variants to other regions, various peak discharge distributions are developed and analyzed for six drainage basins ranging in size from 114 to 904 sq. km in India. From the analysis of 53 flood hydrographs, it is shown that only original peak discharge distribution (OPDD) is sufficient for identification of degree of nonlinearity and prediction of peak discharge. Out of six basins analyzed in the present study only one basin can be assumed to be linear (3f sub-zone of Godavari basin).

Hydrologic design should be based on identification of the degree of basin hydrologic nonlinearity and selection of appropriate method for flood estimation. Use of linear method to nonlinear basins (such as Gola, Umar, Teriya etc) can result in serious design errors by over estimate or under estimate of design flood. The variation in the peak

discharge distribution's intercept b is significantly explained by the logarithm of any of the two basin morphological indices AS/L and A/L with A the drainage area in sq. km, L the length of main river in km and S the slope of river bed in % for the basins excluding Gola basin.

Peak discharge distribution provides a more reliable method for estimation of flood in nonlinear drainage basins where application of unit hydrograph theory is not valid. Peak discharge, time and volume relationship (PDTVR) may be more useful in prediction peak discharge in highly nonlinear basins.

Peak discharge distribution per unit area can be utilized for ungaged catchments in a variety of hydrologic analyses such as estimation of peak discharge in combination with SCS-CN method, estimation of flooding potential; identification of drainage basin similarity, estimation of sediment yields and derivation of unit hydrograph as illustrated in this study.

The application of peak discharge distribution per unit area in combination with SCS-CN method is successfully validated for estimation of flood in 3f sub-zone of lower Godavari basin in India.

This study does not confirm the finding of Singh and Aminian (1986) that basin area alone can be used to explain variance of intercept b. Basins with similar area but in different regions such as Himalayan region and central Indian region will produce significantly different magnitude of peak discharge per unit runoff volume (log inverse of b) as shown in the study. Therefore it is recommended that separate relationship between b and A/L or between b and AS/L should be evolved for different geomorphological regions.

Further study for several basins in India is recommended particularly for establishing usefulness of peak discharge distribution in ungaged catchments, and for analyzing the influence of pattern of rainfall on the peak discharge distributions.

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LIST OF NOTATION

A	= Catchment Area in sq. km
Af	= Forest Area in sq. km.
AMC	= Antecedent Moisture Condition
AOV	= Analysis of Variance
b	= Arithmetic Intercept
CN	= Curve Number
df	= Degree of Freedom
dI/dt	= Change of inflow with Respect to Time
dQ/dt	= Change of outflow with Respect to Time
dS/dt	= Change of Storage with Respect to Time
E	= Mean Basin Elevation
Fa	= Continuing Abstraction
I	= Inflow
Ia	= Initial Abstraction,
K	= Erodibility Factor
L	= Length of Main River in km
m	= Slope of the Line Best Fitting the Data
P	= Total Rainfall
Pe	= Rainfall Excess,
Q	= Outflow
Q_p	= Peak Discharge in m^3/s
Q_T	= Annual Peak Discharge with a Return Periode T
q_p	= Peak Discharge per Unit Area in cm/hr
R	= Multiple Correlation Coefficient
r	= Coefficient Correlation
r^2	= Coefficient of Determination
rms	= regression mean square
S	= Slope of River Bed in %

S = Potential Retention

s^2 = Error Mean Squared

SCS = Soil Conservation Service

S_d = Standard Deviation

S_e = Standard Error

T_p = Time in hours from Start to Rise to Peak

T_r = Time in hours from Peak to Recession Base of Hydrograph

T_b = Time Base of Hydrograph in hours

tc = Time Concentration

UH = Unit Hydrograph

V = Runoff Volume under the Hydrograph Converted to Centimeter Uniformly Distributed Over the Entire Drainage Basin

V_m = Mean Annual Runoff

X_1 = Dependent Variable

X_2, X_3 = Independent Variables

ϕ = linear operator

α = Slope of the Line Best Fitting the Data in Peak Discharge per Unit Area

CHAPTER 1

INTRODUCTION

1.1. GENERAL

Rainfall occurring over a basin causes flow of water in its streams. Water from these streams passes into mainstream of a basin. At a certain location along the main stream the rise in water level is significant from the point of view of flood damage or surface storage of water. A very important problem in the area of hydrology is to predict the runoff hydrograph at such a location on the basin caused by a particular storm. The exact functional relationship between rainfall and runoff in terms of physiographic and climatic factors is almost impossible to determine or derive, because the variables involved are many and complex in nature (figure 1.1). Chow (1964) gives a detailed list of the factors. The inherent complexities of the problem have made it impossible to set up controlled experiments. Thus one has to rely on the available historical data, which are often scanty and possibly subjected to error. Several methods have been proposed, developed and used for estimation of floods which are briefly discussed here:

1.1.1. Empirical Approach

The early hydrologic investigations were limited to the development of methods, which relate the peak discharge of the runoff hydrograph to one or more of the basin parameters. An exhaustive list of such formula is given by Chow (1988). The major drawback of this approach has been the subjective selection of variables and coefficients, which are to be used with great care. However with the use of computers and mathematical sophistication, more advanced empirical block box models have recently been developed making use of Artificial Neural Networks (ANN) technique (Govindaraju, 2000).

1.1.2. Probability Methods

The phenomena of rainfall-runoff and floods are natural processes, which vary at random in space and time. Hence they cannot be predicted with certainty. Probability theory has been used to analyze the randomness of rainfall-runoff process. In this approach floods are generally characterized by a single parameter, namely the peak discharge or the peak stage and the frequency distribution of parameter is investigated. Probability methods are useful in hydrologic design but they do not give detailed information about floods. The present study does not deal with the probabilistic nature of the process.

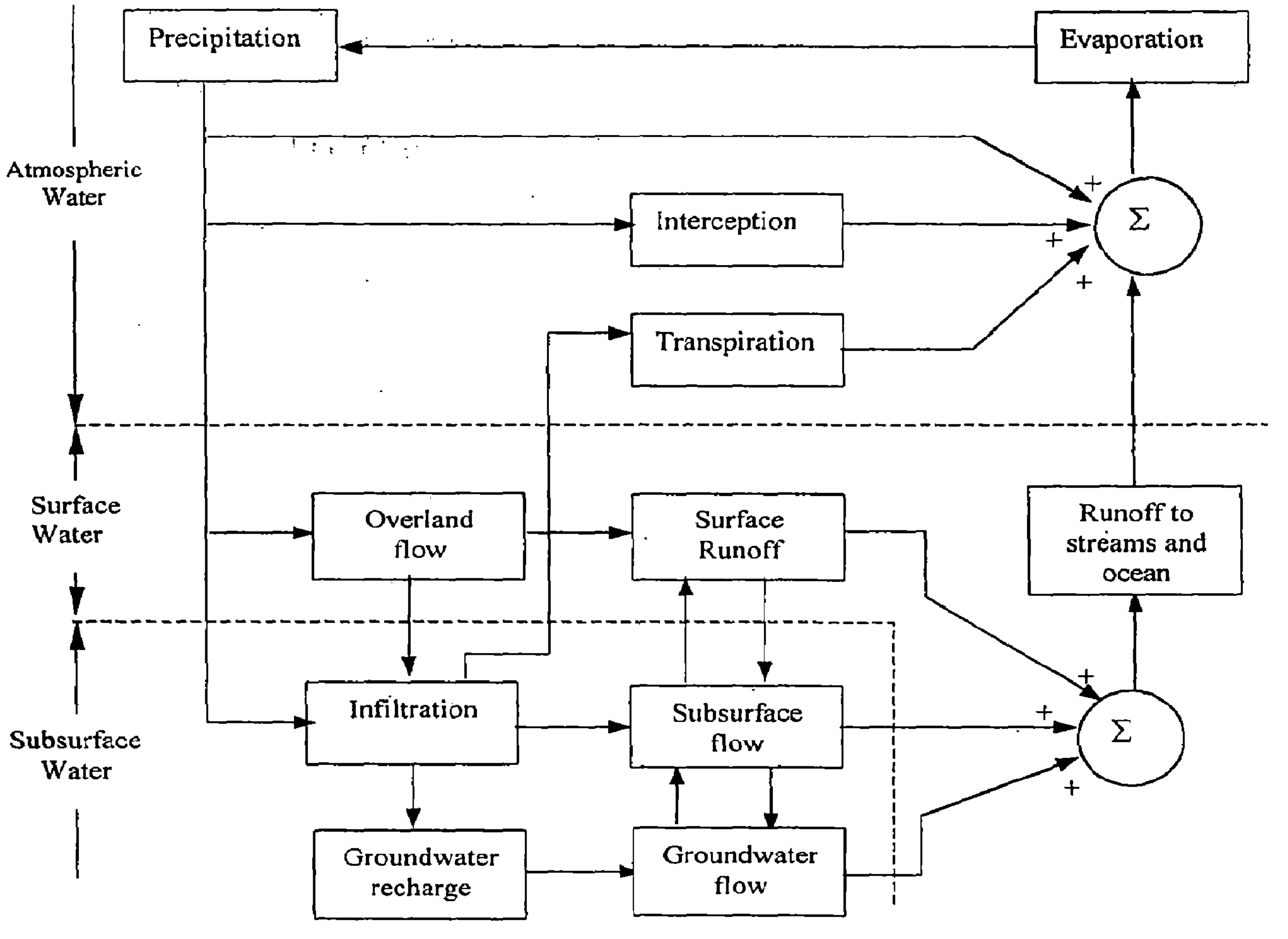


Fig. 1.1 Block Diagram of the Global Hydrologic System

1.1.3. System Model Approach

Natural hydrologic processes are so complex and complicated that they defy exact mathematical analysis. For a practical analysis of complex natural processes, mathematical simulation may be used to develop simplified models whose behavior approximates the real system and which are amenable to mathematical analysis.

The conversion of rainfall into runoff involves various components. A global hydrologic system is depicted in Fig. 1.1, which is self-explanatory. Unless the watershed characteristics are fully known, it is very difficult, if not impossible to establish relationships between various components. However, it is observed that during floods, the direct surface runoff (DSRO) is generally much larger as compared to subsurface and groundwater flow. Generally for the sake of simplicities, the study of rainfall-runoff process is conceived of as a sequence of two problems. The first part deals with the separation of abstractions (interception and infiltration) from give rainfall (to get effective rainfall) and separation of subsurface and ground water flow (in combined form i.e. base flow) from total run off (to get DSRO) and then in the second part the relationship between effective rainfall and DSRO is analyzed. In the study of effective rainfall-DSRO process, it has been generally assumed that the empirical methods adopted for abstraction and separations of base flow are satisfactory, which is not correct.

The following definitions are relevant for further discussion:

- If the characteristics of the basin change with time then it is said to be a time variant system and otherwise the system is called time invariant system.
- If the spatial variations of the internal and external influences are considered in a system then the system is said to be a distributed system. The governing equations of such systems are partial differential equations. However, if it is assumed that the spatial variations of the influences are ignored, the system is said to be lumped system and the governing equations of such systems are ordinary differential equations.
- The available rainfall data are in the form of station rainfall value. After abstraction of the losses, these can either be considered as multiple inputs into the system or can be lumped into a single input given by the weighted average of effective rainfall evaluated at each of the station. The effective rainfall, subsurface flow and ground water flow may also be considered as multiple inputs into the system.
- A drainage basin is said to be hydrologically linear when runoff volume is directly proportional to rainfall volume.

Using the concepts of system approach various models have been proposed for the basin. They can be classified in the following manner:

1. Linear vs. nonlinear model
2. Time invariant vs. time variant model
3. Lumped vs. distributed parameter model
4. Single input model vs. multiple input model

Of these, the linear time invariant single input, single output, lumped parameter model has been investigated extensively. Various methods and models exist for the analysis of linear hydrologic system. Linearity and nonlinearity of hydrologic system is discussed in chapter 2.

Dooge (1973) observed that the hydrologic basin is a heavily damped system. The basin is relatively insensitive with respect to the transformation of effective rainfall to DSRO. Hence all the above-mentioned methods predict the output fairly well from given effective rainfall data. Thus satisfactory prediction of output cannot be used as the sole criteria for testing the particular method of analysis and model.

1.2. BACKGROUND OF THE RESEARCH PROBLEM

Two definitions of basin nonlinearity have been discussed by Sivapalan et al (2002). The first definition of nonlinearity is with respect to the dynamical property such as the rainfall-runoff response of a catchment, and nonlinearity in this sense refers to a nonlinear dependence of the storm response on the magnitude of the rainfall inputs (Minshall, 1960; Rogers, 1980; Wang et al., 1981). The second definition of nonlinearity (Goodrich et al., 1997) is with respect to the dependence of a catchment statistical property, such as the mean annual flood, on the area of the catchment. The change of nonlinearity with area (scale) has been an important motivation for hydrologic research. While both definitions are correct mathematically, they refer to hydrologically different concepts. Sivapalan et al, (2002) has shown that nonlinearity in the dynamical sense and that in the statistical sense can exist independently of each other (i.e., can be unrelated). If not carefully distinguished, the existence of these two definitions can lead to a catchment's response being described as being both linear and nonlinear at the same time.

Both of the above definitions conform to the standard mathematical definition of linearity: function $y = f(x)$ is linear with respect to the input variable x if and only if $f(c_1x_1 + c_2x_2) = c_1f(x_1) + c_2f(x_2)$, where c_1 and c_2 are arbitrary constants. However, both the input and output variables in the above definitions are different. In the first case it is a pair of dynamical variables rainfall R(t) and outflow Q(t), while in the second case it is a pair of variables A and Q_T (or V_m), where A is a geomorphological characteristic of the catchment, whereas Q_T is annual peak discharge with a return period T and V_m is the mean annual runoff both being statistical characteristics.

In the present study hydrologic linearity is defined as the condition that exists on a basin when runoff volumes are directly proportional to rainfall volumes. Hydrologic nonlinearity exists when runoff volumes are not directly proportional to rainfall volumes. Serious error in hydrologic design can occur by over estimating or under estimating design discharge when a drainage basin is assumed to be linear while in fact it is nonlinear. Chow (1964) has reviewed the work done by various researchers on nonlinearity of runoff distribution. The wide spread and long lasting usage of the unit hydrograph model (Sherman, 1932), which is based on the assumption of hydrologic linearity, makes more intensive the need for developing criteria for checking the applicability of the method and, thus, the linearity and nonlinearity in the rainfall-runoff process. One of the most important attempts on this subject has been made in the U.S.A., where a family of three different relations between peak discharge and volume of runoff termed as peak discharge distribution has been developed and studied in detail by Rogers, (1980, 1982); Rogers and Zia, (1982). It was found that, slopes of these distributions are related to the drainage basin runoff characteristics. The slopes of these relationships were proposed as criteria for indicating the degree of drainage basin hydrologic nonlinearity. Predicted peak discharges by the unit hydrograph method for nonlinear basins were found to be seriously overestimated. Several other important conclusions concerning rainfall-runoff linearity and nonlinearity have been drawn from this research as well.

Mimikou (1983) tested the applicability of the peak discharge distributions in eight drainage basins in Greece. He found that distributions slope have no geographic, climatic or basin morphological influence. Mimikou found that only the original peak discharge distribution is necessary and quite sufficient by itself for checking basin hydrologic linearity and accurately predicting peak discharges.

Singh and Aminian (1986) proposed linear two parameter relation in log space between direct runoff volume per unit area and peak discharge of direct runoff per unit area.

The above mentioned discussion shows that before using linear model such as unit hydrograph or Instantaneous Unit Hydrograph, it necessary to identify linearity/nonlinearity of the basin, otherwise serious error in estimation of flood may occur. Further, since a basin is heavily damped system (Dooge, 1973) prediction of similar output by different linear models cannot be considered as a satisfactory criterion for validation of a proposed linear model without ascertaining hydrologic linearity of a basin.

The present study deal with development and analysis of peak discharge distribution for several basins in India.

1.3. OBJECTIVE OF STUDY

1. To develop and apply relationship between peak discharge and runoff volume for various catchments in India with a view to identify drainage basin linearity and nonlinearity. Such identification is useful in adopting an appropriate rainfall-runoff model for estimation of peak discharge and thus is expected to lead to more reliable hydrologic design.
2. Application study of relationship between peak discharge (Q_p) and runoff volume (V) in the hydrologic design.
3. To develop multiple linear relations between peak discharge (Q_p), time to peak (T_p) and runoff volume (V) and compare with other peak discharge distributions.
4. To relate peak discharge per unit runoff with catchment characteristics.
5. To explore use of peak discharge distribution in variety of hydrologic analysis such as (i) flood estimation in ungaged catchments, (ii) determination of sediment yield, (iii) identification of drainage basin similarity, (iv) derivation of unit hydrograph and (v) estimation of flooding potential.

CHAPTER 2

LINEAR AND NONLINEAR HYDROLOGIC SYSTEM

As discussed in section 1.2 of chapter 1 two uses of the terms "linearity" and "nonlinearity" appear in literature. Sivapalan et al, (2002) provides discussion of the two alternative definitions.

This dissertation work is concerned with analysis of the first definition of hydrologic nonlinearity, that is, runoff response to storm rainfall input. In the following sections, system theory as applied to modeling of rainfall-runoff process in a basin is presented followed by description of a model to examine nonlinearity of basin.

2.1. HYDROLOGIC SYSTEM

A hydrologic system is defined as a structure or volume in space, surrounded by a boundary, that accepts water and other inputs, operates on them internally, and produces them as outputs (Fig. 2.1). The structure (for surface and subsurface flow) or volume in space (for atmospheric moisture flow) is the totality of the flow paths through which the water may pass as throughout from the point it enters the system to the point it leaves. The boundary is a continuous surface defined in three dimensions enclosing the volume or structure. A working medium enters the system as input, interact with the structure and other media, and leaves as output. Physical, chemical, and biological processes operate on the working media within the system; the most common working media involved in hydrologic analysis are water, air, and heat energy.

A watershed also known as basin or catchment is area of land draining into a stream at a given location. The watershed divide is a line dividing land whose drainage flows toward the given stream from land whose drainage flows away from that stream. The system boundary is drawn around the watershed by projecting the watershed divide vertically upwards and downwards to horizontal planes at the top and bottom. Rainfall is input, distributed in space over the upper plane; stream flow is the output, concentrated in space at the watershed outlet. Evaporation and subsurface flow could also be considered as outputs, but they are small compared with stream flow during a storm. The structure of the system is the set of flow paths over or through the soil and includes the tributary streams, which eventually merge to become stream flow at the watershed outlet.

As the rainfall-runoff process is complicated, very often a simpler process of effective rainfall-direct surface runoff (DSRO) is studied. The DSRO may be considered as the response of the basin system to the input of effective rainfall. The basin system may be linear and nonlinear.

2.2. GENERAL HYDROLOGIC SYSTEM MODEL

A general model as given in Chow et al., (1988) is explained below:

The amount of water storage in a hydrologic system, S may be related to the rates of inflow I and outflow Q by the integral equation of continuity.

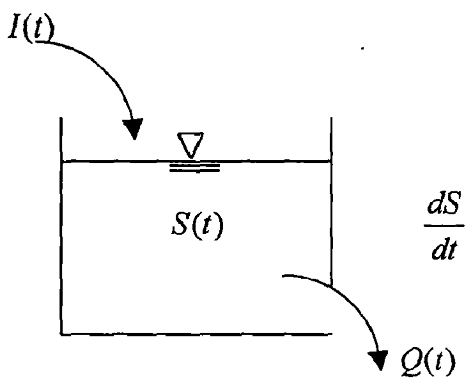
$$\frac{dS}{dt} = I - Q \qquad (2.1)$$

Imagine that the water is storage in a hydrologic system, such as a reservoir (Fig. 2.1), in which the amount of storage rises and falls with time in response to I and Q and their rates of change with respect to time: dI/dt, $d^2I/dt^2, \ldots d^nI/dt^n$, dQ/dt, $d^2Q/dt^2, \ldots d^nQ/dt^n$. Thus, the amount of storage at any time can be expressed by storage function as:

$$S = f\left(I, \frac{dI}{dt}, \frac{d^2I}{dt^2}, \ldots \frac{d^nI}{dt^n}, \quad Q, \frac{dQ}{dt}, \frac{d^2Q}{dt^2}, \ldots \frac{d^nQ}{dt^n} \right) \qquad (2.2)$$

The function f is determined by the nature of the hydrologic system being examined. For example, a linear reservoir as a model for base flow in streams relates storage and outflow by $S = kQ$, where k is constant.

The continuity equation (2.1) and storage function equation (2.2) must be solved simultaneously so that the output Q can be calculated given the input I, where Q and I are both functions of time. This can be done in two ways: by differentiating the storage function and substituting the result for dS/dt in Eq. (2.1), then solving the resulting differential equation in I and Q by integration: or by applying the finite difference method directly to Eqs. (2.1) and (2.2) to solve them recursively at discrete points in time.



$$\frac{dS}{dt} = I(t) - Q(t)$$

Fig. 2.1. Continuity of Water Stored in a Hydrologic System

Let storage function S be approximated by,

$$S = a_1 Q + a_2 \frac{dQ}{dt} + a_3 \frac{d^2Q}{dt^2} + \dots\dots + a_n \frac{d^{n-1}}{dt^{n-1}}$$

$$+ b_1 I + b_2 \frac{dI}{dt} + b_3 \frac{d^2 I}{dt^3} + \dots\dots + bm \frac{d^{m-1} I}{dt^{m-1}} \qquad (2.3)$$

in which $a_1, a_2, \dots a_n, b_1, b_2, \dots b_m$ are coefficient and derivatives of higher order than those shown are neglected. If coefficients are not function of time then system is time invariant i.e. the way the system process input into output does not change with time.

Differentiating Eq. (2.3) with respect to time and substituting the result for *dS/dt* in Eq. (2.1), and rearranging yields,

$$a_n \frac{d^n Q}{dt^n} + a_{n-1} \frac{d^{n-1} Q}{dt^{n-1}} + \dots\dots + a_2 \frac{d^2 Q}{dt^2} + a_1 \frac{dQ}{dt} + Q =$$

$$I - b_1 \frac{dI}{dt} - b_2 \frac{d^2 I}{dt^2} - \dots\dots - b_{m-1} \frac{d^{m-1} I}{dt_{m-1}} - b_m \frac{d^m I}{dt^m} \qquad (2.4)$$

which may be rewritten in the more compact form

$$N(D)Q = M(D)I \qquad (2.5)$$

Where D = *d/dt* and *N(D)* and *M(D)* are the differential operators

$$N(D) = a_n \frac{d^n}{dt^n} + a_{n-1} \frac{d^{n-1}}{dt_{n-1}} + \dots\dots + a_1 \frac{d}{dt} + 1$$

and $$M(D) = -b_m \frac{d^m}{dt^m} - b_{m-1} \frac{d^{m-1}}{dt^{m-1}} - .... - b_1 \frac{d}{dt} + 1$$

Solving Eq. (2.5) for Q yields

$$Q(t) = \frac{M(D)}{N)D)} I(t) = \Omega I(t) \tag{2.6}$$

The function Ω or M(D)/N(D) is called the transfer function of the system; it describes the response of the output to a given input sequence.

Equation (2.4) was presented by Chow and Kulandaiswamy (1971, as mentioned in Chow et al., 1988) as a general hydrologic system model. It describes a lumped system because it contains derivatives with respect to time alone and not spatial dimensions. Chow and Kulandaiswamy showed that many of the previously proposed models of lumped hydrologic systems were special cases of this general model. For example, for a linear reservoir, the storage function Eq. (2.3) has $a_1 = k$ and all other coefficients zero, so Eq. (2.4) becomes

$$k \frac{dQ}{dt} + Q = I \tag{2.7}$$

2.3. LINEAR SYSTEM

A system is said to be linear if it satisfies the following definition:

Let X_1 and X_2 be two inputs for which the outputs of the system are $Y_1 = \phi$ (X_1) and $Y_2 = \phi$ (X_2) respectively then the system is said to be linear if the following two relations are satisfied :

$Y_1 + Y_2 = f(X_1 + X_2)$ (Superposition)

and $\phi(CX) = C\,\phi(X)$ (Homogeneity)

where ϕ is a linear operator

When the runoff volume (output) from drainage basin is directly proportional to the precipitation volume (input) for a range precipitation of volumes, the drainage basin is said to exhibit linear runoff or is said to be hydrologically linear.

The physical condition occurring on a drainage basin which result in linear runoff is that the combined effect of hydrologic variables, namely infiltration, interception, depression storage, evaporation and transpiration, must be reasonably uniform throughout the drainage basin. Such a condition will permit uniform distribution of runoff depth to occur throughout the drainage basin if the basin is covered with uniform precipitation.

Drainage basin linearity is a basic assumption of the unit hydrograph (Sherman, 1932), which also assumes that peak discharge is directly proportional to the runoff volume. This assumption is a natural consequence of foregoing discussion.

The nearly ideal example of a linear drainage basin is a drainage basin with a surface consisting only of uniform, impermeable material. Such a drainage basin would have no infiltration losses. By also assuming this drainage basin to be of water repellant material on a uniform slope, and by assuming the air to be saturated, all precipitation falling on this basin would become runoff (Rogers, 1982).

Because there are no hydrologic losses from this idealized drainage basin, it can be seen that, this drainage basin will produce the same runoff volume (output) as the precipitation (input), which falls on the drainage basin. Thus, output is proportional to input (in this case output actually equals input) and the drainage basin is, therefore, linear.

It should be noted in this example of an idealized linear drainage basin that linearity of runoff volume does not depend on rainfall distribution. Any distribution of rainfall can occur on this idealized drainage basin, and yet the runoff volume will be directly proportional to the precipitation volume.

The idealized hydrologically linear drainage basin meeting the unit hydrograph conditions will have linear peak discharge. This condition can be demonstrated by routing uniform rainfall of different volumes from the idealized drainage basin to produce elemental runoff hydrograph (Rogers, 1982). Mathematical linearity should not be confused with hydrologic linearity, even though runoff data from a hydrologically linear or nonlinear drainage basin may also be mathematically linear.

If all hydrologic losses are distributed uniformly, then the runoff volume must equal the precipitation volume minus a constant loss. In other words, output must be directly proportional to input and the drainage basin is hydrologically linear.

A linear hydrologic basin can be represented in terms of UH and IUH as they satisfy the principle of superposition and homogeneity or proportionality. The DSRO hydrograph resulting from a given pattern of effective rainfall can be built by multiplying and superposing the UH or IUH ordinates with the given rainfall excess values.

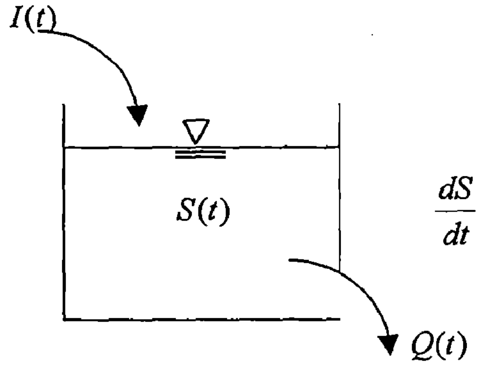
2.4. NONLINEAR SYSTEM

Two definitions of "linearity" and "nonlinearity" appear in literature (Sivapalan et al., 2002). This dissertation work deals with analysis of hydrologic nonlinearity of a basin with respect to runoff response to storm rainfall event.

A nonlinear analysis of hydrologic system by means of a generalized functional series was originally introduced by Wiener (as given in Chow, 1964) for mathematical analysis of nonlinear system. This functional series may be viewed as a conceptual model containing a number of operating systems in parallel, all receiving input function and producing output function. The output from the first system is obtained by an ordinary convolution integral containing the input and a fixed kernel function. The output from the second system is obtained by a generalized convolution integral containing the input, and two dimensional kernel function. The outputs from other systems are obtained in a similar manner. The final output is the sum of all outputs from the system in parallel.

Drainage basins exhibiting hydrologically nonlinear runoff do not have a runoff volume that is directly proportional to the rainfall volume. Quantitative evidence to support the explanation for this condition is lacking, but qualitative relationships exist, which do support it (Rogers, 1980). Drainage basins, which show measured or obvious increase in infiltration capacities toward the margins of the drainage basin, also exhibit hydrologic nonlinearity.

The condition of higher peak discharge from a storm which occurs near the gage than from a similar storm, which occurs in the upper portion of the drainage basin is caused by a condition of hydrologic nonlinearity. This nonlinear condition is mainly due to increasing infiltration losses toward the drainage divide (Rogers, 1980). These infiltration losses can become so great that a given storm occurring towards the drainage divide will produce a lower volume of runoff and therefore a lower peak discharge than a similar storm occurring near the gage where infiltration rate are lower. The degree of nonlinearity is related to the magnitude of the difference in the infiltration rates along the main channel equation in I and Q by integration: or by applying the finite difference method directly to Eqs. (2.1) and (2.2) to solve them recursively at discrete points in time.



$$\frac{dS}{dt} = I(t) - Q(t)$$

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Let storage function S be approximated by,

$$S = a_1 Q + a_2 \frac{dQ}{dt} + a_3 \frac{d^2Q}{dt^2} + ... + a_n \frac{d^{n-1}}{dt^{n-1}}$$

$$+ b_1 I + b_2 \frac{dI}{dt} + b_3 \frac{d^2 I}{dt^3} + ... + bm \frac{d^{m-1} I}{dt^{m-1}} \qquad (2.3)$$

in which $a_1, a_2, ...a_n, b_1, b_2, ...b_m$ are coefficient and derivatives of higher order than those shown are neglected. If coefficients are not function of time then system is time invariant i.e. the way the system process input into output does not change with time.

Differentiating Eq. (2.3) with respect to time and substituting the result for *dS/dt* in Eq. (2.1), and rearranging yields,

$$a_n \frac{d^n Q}{dt^n} + a_{n-1} \frac{d^{n-1} Q}{dt^{n-1}} + ... + a_2 \frac{d^2 Q}{dt^2} + a_1 \frac{dQ}{dt} + Q =$$

$$I - b_1 \frac{dI}{dt} \div b_2 \frac{d^2 I}{dt^2} - .... - b_{m-1} \frac{d^{m-1} I}{dt_{m-1}} - b_m \frac{d^m I}{dt^m} \qquad (2.4)$$

which may be rewritten in the more compact form

$$N(D)Q = M(D)I \qquad (2.5)$$

Where D = *d/dt* and *N(D)* and *M(D)* are the differential operators

$$N(D) = a_n \frac{d^n}{dt^n} + a_{n-1} \frac{d^{n-1}}{dt_{n-1}} + ... + a_1 \frac{d}{dt} + 1$$

and $$M(D) = -b_m \frac{d^m}{dt^m} - b_{m-1} \frac{d^{m-1}}{dt^{m-1}} - .... - b_1 \frac{d}{dt} + 1$$

Solving Eq. (2.5) for Q yields

$$Q(t) = \frac{M(D)}{N)D)} I(t) = \Omega I(t) \tag{2.6}$$

The function Ω or M(D)/N(D) is called the transfer function of the system; it describes the response of the output to a given input sequence.

Equation (2.4) was presented by Chow and Kulandaiswamy (1971, as mentioned in Chow et al., 1988) as a general hydrologic system model. It describes a lumped system because it contains derivatives with respect to time alone and not spatial dimensions. Chow and Kulandaiswamy showed that many of the previously proposed models of lumped hydrologic systems were special cases of this general model. For example, for a linear reservoir, the storage function Eq. (2.3) has $a_1 = k$ and all other coefficients zero, so Eq. (2.4) becomes

$$k \frac{dQ}{dt} + Q = I \tag{2.7}$$

2.3. LINEAR SYSTEM

A system is said to be linear if it satisfies the following definition:

Let X_1 and X_2 be two inputs for which the outputs of the system are $Y_1 = \phi$ (X_1) and $Y_2 = \phi$ (X_2) respectively then the system is said to be linear if the following two relations are satisfied :

$Y_1 + Y_2 = f(X_1 + X_2)$ (Superposition)

and $\phi(CX) = C\,\phi(X)$ (Homogeneity)

where ϕ is a linear operator

When the runoff volume (output) from drainage basin is directly proportional to the precipitation volume (input) for a range precipitation of volumes, the drainage basin is said to exhibit linear runoff or is said to be hydrologically linear.

The physical condition occurring on a drainage basin which result in linear runoff is that the combined effect of hydrologic variables, namely infiltration, interception, near the gage and on and toward the basin divides. If this difference is large, the drainage basin will be highly nonlinear.

In the present study, degree of nonlinearity of a basin is proposed to be identified through analysis of relationship between peak discharge and runoff volume as suggested by Rogers (1980). The procedure is discussed below.

2.5. VOLUME – PEAK DISCHARGE RELATION

This method is also known as the Peak discharge-rating curve (Singh, 1994) or a standardized peak discharge distribution (Rogers, 1980) for drainage basin in which peak discharge is plotted against runoff volume (in centimeters or inches uniformly distributed over the entire drainage basin) rather than stage. This method relies on measured discharge data from a stream gage or weir.

The standardized peak discharge distribution is based on the fact that, this peak discharge runoff volume relation is a transformation of the stream gage rating curve. The stream gage rating curve is a plot of the discharge versus stage (in meters or feet), whereas the standardized peak discharge distribution is a plot of the peak discharge versus the distributed depth of the total hydrograph volume over the entire drainage basin.

The development of the standardized peak discharge distribution was motivated by the established and accepted UH method. Both methods plot peak discharge against runoff volume (in centimeters or inches). This relation is desirable because it is customary to relate the amount of rainfall (in centimeters or inches) to the runoff volume from a drainage basin (in centimeters or inches) uniformly distributed over the entire drainage basin. This relation is logical and can be applied to compute design flood events from runoff volumes from a drainage basin. An important advantage of the standard peak discharge distribution is that it does not require base flow separation, (Rogers, 1980) which is an uncertain procedure.

2.5.1. Data Requirements

The standardized peak discharge distribution is constructed from records of flood hydrographs ordinarily kept for any stream gage site. The essential information that is required is the following:

1. The stream gage rating curve or rating table that is valid for the time period during which the recorded hydrograph were obtained.
2. Several recorded hydrographs
3. The drainage area, length of mainstream and slope.

The number of hydrographs used is less important than the quality of the hydrographs. The hydrograph should be simple hydrographs type that is the result of single rainfall event although complex hydrographs can be used and simplified if it is determined that those hydrographs were the result of multiple rainfall events. The hydrograph should not be complicated by upstream obstructions or down stream obstructions. Obstructions can affect the shape of the hydrograph, so that each hydrograph should be carefully examined to see if there has been significant distortion of the hydrograph caused by some external influence. Useful hydrographs for this method or any other method can be only those that result from the influence of drainage basin characteristics and not influenced by some alien condition. The number of hydrographs chosen usually consists of the number that is reasonably available. The larger the number, the more reliable the rating curves. It is useful to choose hydrographs that cover the range from small to large if possible.

2.5.2 Construction and Meaning of the Standardized Peak Discharge Distribution

The Standardized peak discharge distribution is defined as the distribution of the logarithm of peak discharge Q_p (m^3s^{-1}) plotted against the logarithm of the runoff volume V (cm) of the total hydrograph producing that peak discharge. An equation for this plot can be determined using the least square method and a measure of the fit can be determined. The equation takes the form:

$$Q_p = aV^m$$

or

$$\text{Log } Q_p = \text{b} + \text{m} \log V \qquad (2.8)$$

where :

b $= \text{Log } a$

Q_p = peak discharge in m^3s^{-1}

V = runoff volume under the hydrograph converted to centimeter uniformly distributed over the entire drainage basin

b = intercept

m = slope of the line best fitting the data

Based on the above equation, Rogers (1980) developed the peak discharge distribution using runoff data of 43 drainage basins ranging in size from 5 to 700 km^2 in United States. The study was then extended to larger basins up to about 23,000 km^2 (Rogers and Zia, 1982). A family of three peak discharge distributions was finally developed as follows:

(i) Original peak discharge distribution (OPDD): This distribution is given by equation (2.8).

(ii) The first order standardized peak discharge distribution (FSPDD) is the logarithm of peak discharge divided by runoff volume to the first power and distribution is represented by the general equation:

$$\text{Log}\,(Q_p/V) = b + (m\text{-}1)\log V. \qquad (2.9)$$

(iii) The second order standardized peak discharge distribution (SSPDD) is the logarithm of peak discharge divided by runoff volume to the second power, it is modeled by the general equation:

$$\text{Log}\,(Q_p/V^2) = b + (m\text{-}2)\log V. \qquad (2.10)$$

It was found that no base flow separation was necessary for the development of these distributions.

It is easy to see that in deriving Eqs. (2.9) and (2.10) from Eq. (2.8), the intercept b remains unchanged, whereas the slope m is reduced by one unit for the FSPDD and by two units for the SSPDD. Rogers (1980, 1982) observed that for the hydrologically linear basins meeting the unit hydrograph conditions where hydrograph ordinates and therefore peak discharge are directly proportional to runoff volume, the slope of Eq. (2.8) must be equal to 1.0, the slope of Eq. (2.9) equal to 0 and the slope of Eq. (2.10) equal to –1.0. Smaller slope indicate hydrologic nonlinearity, which is found to be related to the non-uniform distribution of infiltration capacities (and generally of hydrologic losses) on drainage basin.

Among the most important conclusions drawn from the study of the peak discharge distribution is that the degree of hydrologic nonlinearity is indicated by the magnitude of the negative slope of the SSPDD, also that linear design methods are not appropriate for nonlinear basins. The significant improvement in the coefficient of determination for Eq.

(2.10) over either Eq. (2.8) or Eq. (2.9) and the consistently very high correlation coefficient associated with Eq. (2.10) were the main reasons that Rogers (1980, 1982) preferred the SSPDD as the reference distribution.

Based on study in Greece by Mimikou (1983), the most important conclusions drawn from the study of the peak discharge distribution is that only the original peak discharge distribution (OPDD) is necessary and quite sufficient by itself for checking basin hydrologic linearity and predicting peak discharges. Its slope m being 1.0 for linear and less than 1.0 for nonlinear basins, is a criterion indicating (by its magnitude) the degree of drainage basin hydrologic nonlinearity.

2.6. PEAK DISCHARGE, TIME TO PEAK AND VOLUME RELATIONSHIP

Peak discharge is known to depend not only on volume of runoff but also on time to peak (Mockus, 1957). Therefore relation between peak discharge (Q_p), time to peak (T_p) and runoff volume (V) has been developed in present study and termed as Peak Discharge, Time and Volume Relationship (PDTVR). The three variables relationship is developed in logarithm space and represented by general equation as:

$$\text{Log } Q_p = b' + b'' \text{ Log } T_p + m' \text{ Log } V \qquad (2.11)$$

where b', b" and m' are constants.

From the above equation we can also develop linear relation between log Q_p and Log V with assuming that the time to peak (T_p) of flood hydrograph is constant, general equation as:

$$\text{Log } Q_p = b_1 + m' \text{ Log } V \qquad (2.12)$$

where $b_1 = b' + b''$ Log T_p.

2.7. POTENTIAL APPLICATIONS OF PEAK DISCHARGE DISTRIBUTION

Because the standardized peak discharge distribution deals both with peak discharge and runoff volume, it has several applications of interest.

The strength of the standardized peak discharge distribution procedure is that it relies on real measured data and not upon synthetic data. The relation between peak discharge and runoff volume is so good that reasonably accurate results can be expected even when using few data points (Singh, 1994).

Equation (2.8) can be used to predict peak discharge from an anticipated runoff volume for a drainage basin, for any rainfall distribution and duration, and for any antecedent moisture condition.

The standardized peak discharge distribution data can be extrapolated within reason in order to predict peak discharges at other locations along the same drainage system (Singh, 1994). If the location of interest is upstream of the gage, then it can usually be assumed that the drainage characteristics of the drainage basin are approximately the same and, therefore, the value of m for a standardized peak discharge distribution at that point would be approximately the same. If the location of interest is down stream of the gage, then it must be ensured that the drainage characteristics of the added drainage area in the down stream direction are not significantly different from the characteristics of the drainage area above the gage. This decision is subjective and should be done carefully. Examination of urbanization, geology, and soils should indicate whether or not characteristic of the added drainage area are significantly different.

Assuming that the added down stream drainage area is reasonably similar, then it can be assumed that the slope m of Eq. (2.8) for the down stream point of interest would not be significantly different from the upstream gage if a gage were in fact present at the new location of interest. The actual volume of runoff from any drainage basin area is the equivalent depth, in equation (2.8), multiplied by the drainage basin area. Therefore, the difference in runoff volume for the same depth of runoff between two drainage basins with like nonlinearity, m, is simply the ratio of the drainage area.

To extend the work of Rogers (1982) and Mimikou (1983), Singh and Aminian (1986) developed a relation between volume and peak of direct runoff by employing a large number (134) of drainage basins from the United States, Australia, Italy and Greece:

$$\text{Log}\, q_p = b + \alpha\, \text{Log}\, V \qquad (2.13)$$

In which q_p is peak discharge of direct runoff per unit area (cm/hr), V is the direct runoff volume per unit area (cm), b is the intercept (cm/hr) and α is dimensionless slope. Subtracting 2 Log V from both side of Eq. (2.13) following Rogers (1980)

$$\text{Log}\,(q_p/V^2) = \text{b} + \text{m Log}\,V \qquad (2.14)$$

where $\text{m} = \alpha - 2$.

Equation (2.14) can be meaningfully applied to a variety of hydrologic analysis as follow,

1. Rainfall volume – direct runoff peak relation: A relation between volumes of direct runoff and rainfall on a unit area basis has been developed by Soil Conservation Service (1972), popularly known as the SCS-CN method. Using the SCS curve number (SCS-CN) method and Eq. (2.14) we can predict peak discharge resulting from storm rainfall in a catchment for which peak discharge distribution is known from observed data or for which hydrologic similarity with other basin (with known m) is assumed. Application study is presented in chapter 5.
2. Derivation of unit hydrograph: If the D-hour unit hydrograph (UH) is represented by a triangle as proposed by Soil Conservation Service (SCS, 1972) then knowing q_p from Eq. (2.14) will suffice to specify the UH. Duration D and volume V of effective rainfall are assumed known. The duration of recession from the time to peak is taken as approximately 1.67 times the duration of rise, T_p. Application is examined in chapter 6.
3. Flooding potential: Equation (2.14) can be combined with the SCS hypothesis of representing the flood hydrograph by a triangle in exactly the same way as the UH derived above. This allows determination of not only the flood peak, but also the flood duration and flood volume. Application is illustrated in chapter 6.
4. Determination of sediment yield: Singh and Chen (1982) found that relationship between sediment yield and volume of direct runoff is linear in log space. It can be used to estimate sediment yield. Volume of direct runoff can be estimated using SCS-CN method. Application is illustrated in chapter 6.
5. Drainage basin similarity: Parameter m can be considered as measure of hydrologic similarity of drainage basins. For a family of similar basins only one value of m would suffice which can be obtained for the basins having rainfall-runoff records and transferred to those members of the family not having such records. This application is examined in chapter 6.

CHAPTER 3

THE RIVER BASINS AND DATA

3.1. GENERAL

For this study, following six drainage basins in India ranging in size from 114 sq. km to 904 sq. km have been chosen:

1. Temur Sub-basin of Narmada basin up to bridge no. 249
2. Umar Sub-basin of Narmada basin up to bridge no. 930
3. Teriya Sub-basin of Narmada basin up to bridge no. 253
4. Kolar Sub-basin of Narmada basin up to satrana
5. The 3f Sub-zone watershed of Lower Godavari Basin up to bridge no. 807
6. Gola Sub-basin up to Dam site near Kathgodam.

Predominant flow in these basins is surface flow. The general location of these basins is shown in figure 3.1. The area A (km^2) of the drainage basins, the river main course length L (km) and average bed slope S (%) from the divide of the basins to the outlet stations and number of hydrographs are given in table 3.1. Source of data for the study basins are also indicated in the table 3.1.

3.2. STUDY AREA

3.2.1. Upper Narmada basin

The upper Narmada Basin lies between east longitudes 76° 12' to 81° 45' and north latitudes of 20° 10' to 23° 45' lying in the northern extremity of the Deccan plateu, the sub-zone covers the states of Madya Pradesh and Maharashtra. The Sub-zone is bounded by Chambal basin 1(b) Betwa basin 1(c) and Sone basin 1(d) on the north, Lower Narmada and Tapi Sub-zone 3(b) on the west, Lower Godavari sub-zone 3(f) on the south and Mahanadi sub-zone 3(d) on the east. Important cities and towns within the sub-zone are Mandla, Jabalpur, Narsinghpur, Itarsi, Betul, Hoshangabad, Akola and Amravati.

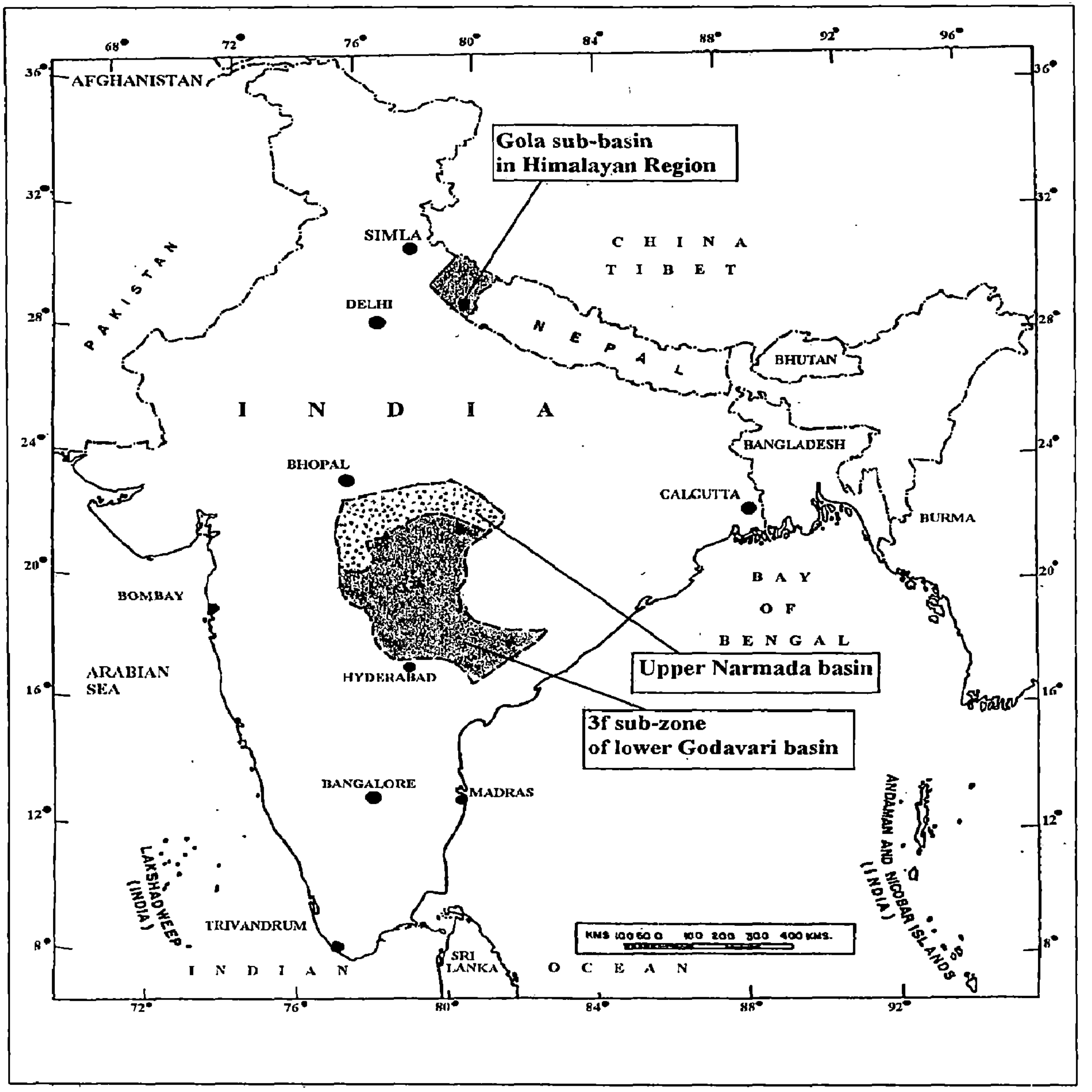

Fig. 3.1. General Location of the Basins

The Upper Narmada has a complex relief. High range of above 900 m exists over a small area near the source of Narmada river at Amarkantak. Areas varying in height between 600 m to 900 m lie along the eastern and middle portion of the boundary. About 60 % of the sub-zone varies in height from 300 m to 600 m. Areas varying in height from 150 m to 300 m lie in patches near the western boundary.

For this study, we use four sub-basins in Narmada basin i.e. Temur sub-basin, Teriya sub-basin, Umar sub-basin and Kolar sub-basin. Temur sub-basin has a catchment area 518.67 km^2, length of main river 56.52 km and slope of river bed 0.303%. Teriya sub-basin has a catchment area 114.22 km^2, length of main river 35.42 km and slope of river bed 0.321%. Umar sub-basin has a catchment area 226.27 km^2, length of main river 33.60 km and slope of river bed 0.250%. And the Kolar sub-basin has a catchment area 903.88 km^2, length of main river 75.35 km and slope of river bed 0.53%.

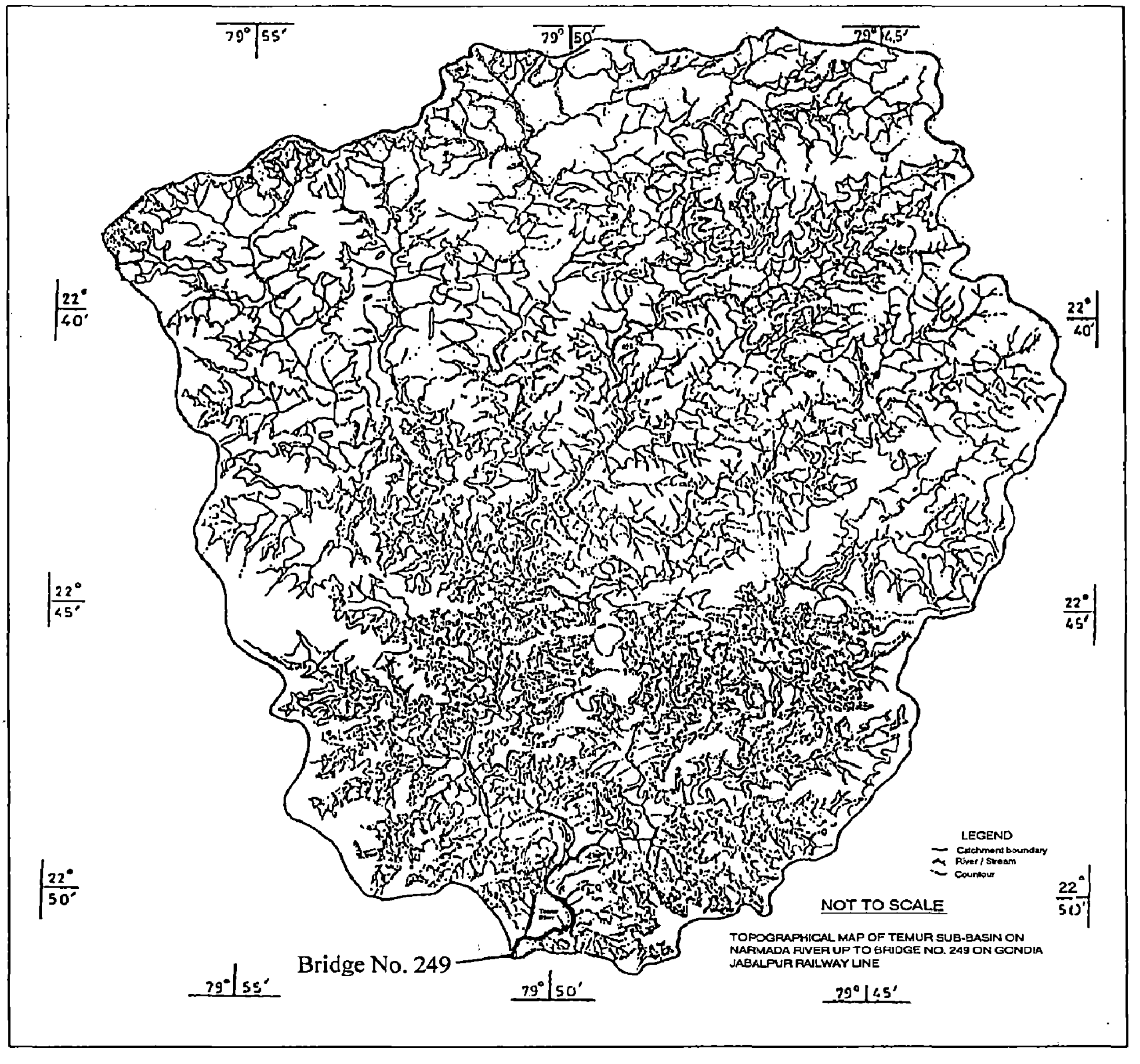

Fig. 3.2. Map of Temur Sub-Basin

Figs. (3.2), (3.3), (3.4) and (3.5) show drainage of Temur sub-basin, Teriya sub-basin, Umar sub basin and Kolar sub-basin respectively.

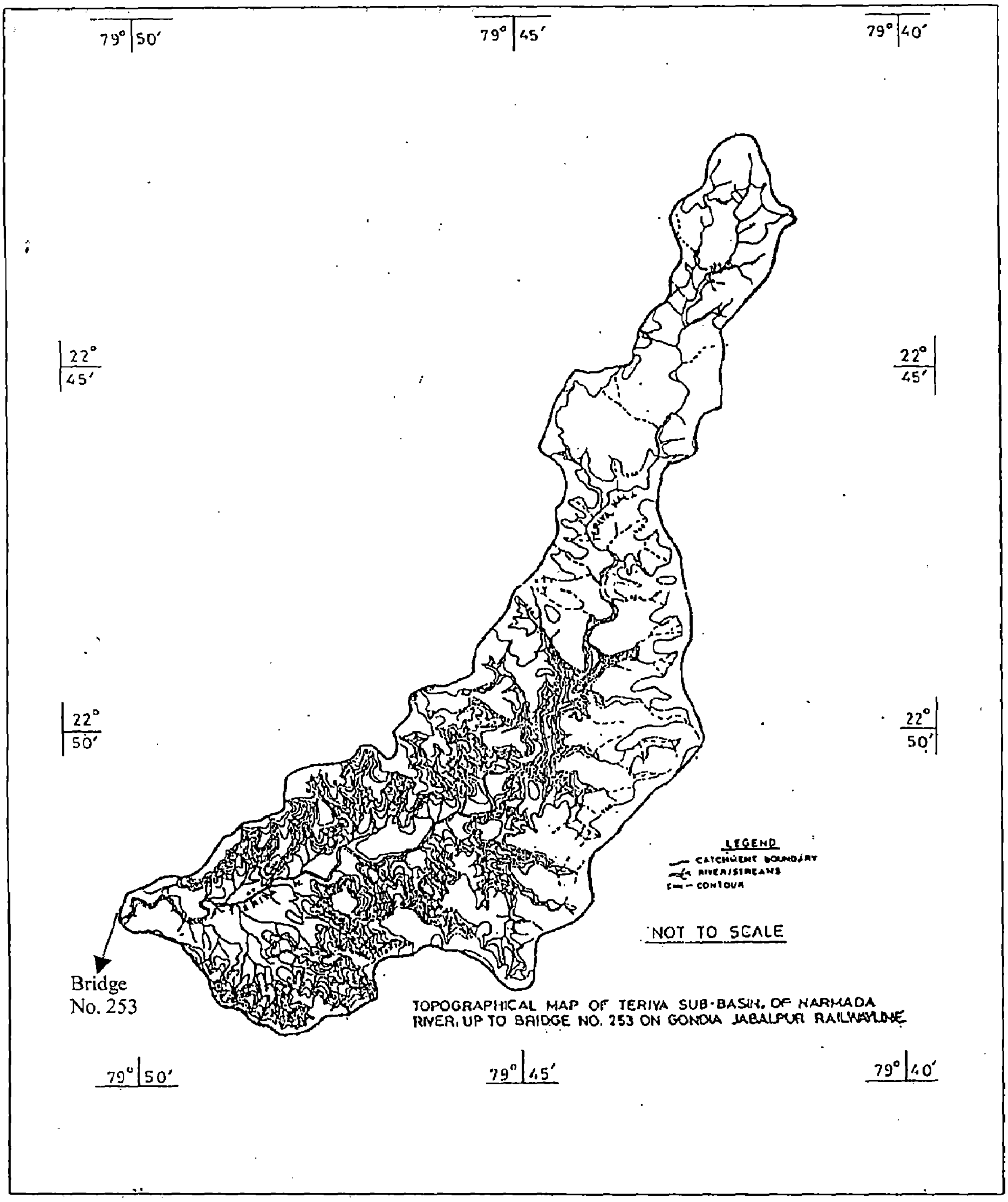

Fig. 3:3. Map of Teriya Sub-Basin

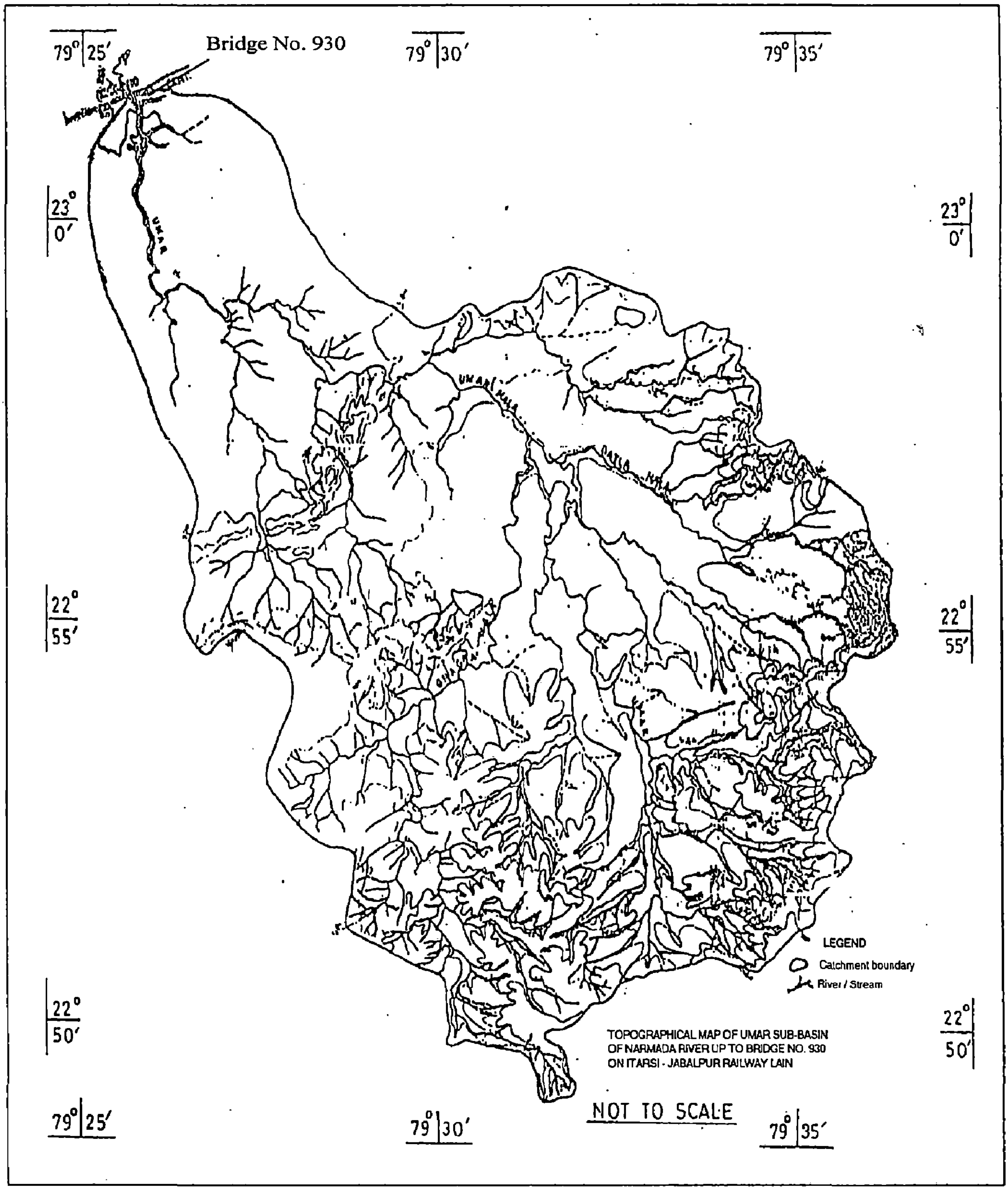

Fig. 3.4. Map of Umar Sub-Basin

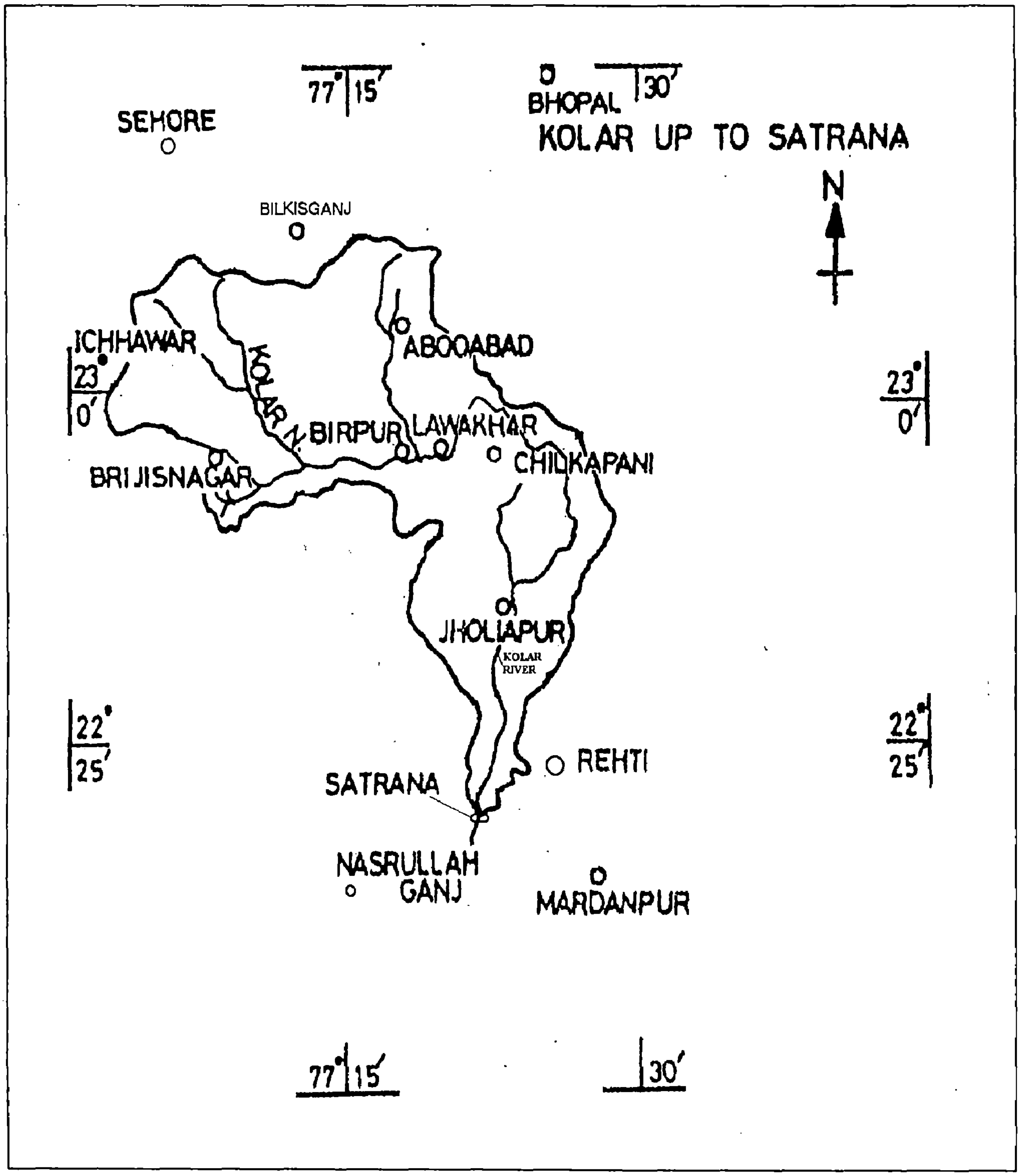

Fig. 3.5. Map of Kolar Sub-Basin

3.2.2. Lower Godavari Basin

Lower Godavari sub-zone extends from longitudes 76° to 83° east and latitudes 17° to 23° north. The sub-zone is bounded by Upper Narmada and Tapi sub-zone 3(c) on the north and northwest, Upper Godavari sub-zone 3(e) on the west, Krishna and Pennar sub-zone 3(h) on the south, Upper eastern coast sub-zone 4(a) on the southeast, Mahanadi sub-zone 3(d) and Indravati sub-zone 3(g) on the east.

The region includes the states of Maharashtra, Madya Pradesh, Andhra Pradesh, and Orissa. Nagpur, Chandrapur, Wardha, Gondia, Nizamabad, Kazipet and Adilabad are some of the important cities and towns located in the zone.

The Lower Godavari sub-zone has a complex relief. Plains of medium height up to 150 m exist near main Godavari river in its Lower reaches. Higher plains between heights of 150 to 300 m cover most of the upper reaches. The western part of the subzone and north of Nagpur is the zone of the low plateaus in the range of 300 to 600 m. The southeast and northwest portions of the sub-zone cover high plateaus in the range of 300 to 900 m and there are hills and higher plateaus ranges from 900 to 1350 m in the southeastern part of the sub-zone.

The 3f sub-zone watershed of lower Godavari basin extends up to Bridge No. 807 in humid region of India. Sub-zone 3f of Godavari and its tributaries encompass about 56% of the total catchment area (=823.62 km^2) of the main Godavari river in the State of Maharasthra, Andhra Pradesh and Orissa. The length of main river is 61.08 km and river bed slope is 0.124%. Map of 3f sub-zone of Lower Godavari Basin can be shown at Fig. 3.6.

3.2.3. Gola Sub-Basin

Gola sub-basin is located in Nainital District in Uttaranchal State, India. This basin has a catchment area about 450 km^2, length of main river is 23,5 km and slope of the river bed is 1.4%. Map of Gola Basin is shown in Fig. 3.7.

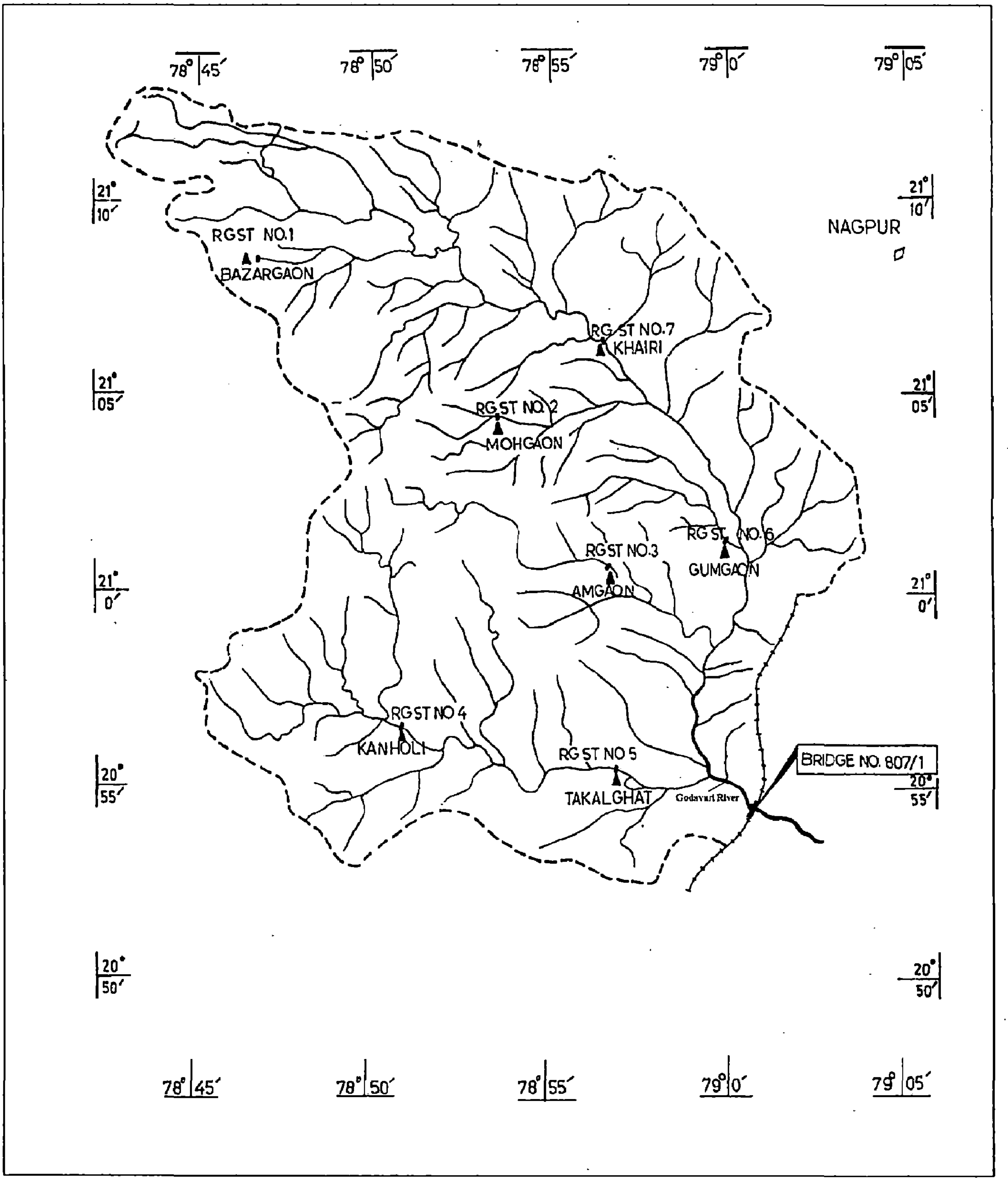

Fig. 3.6. Map of 3f Sub-Zone up to Bridge no. 807 of Lower Godavari Basin

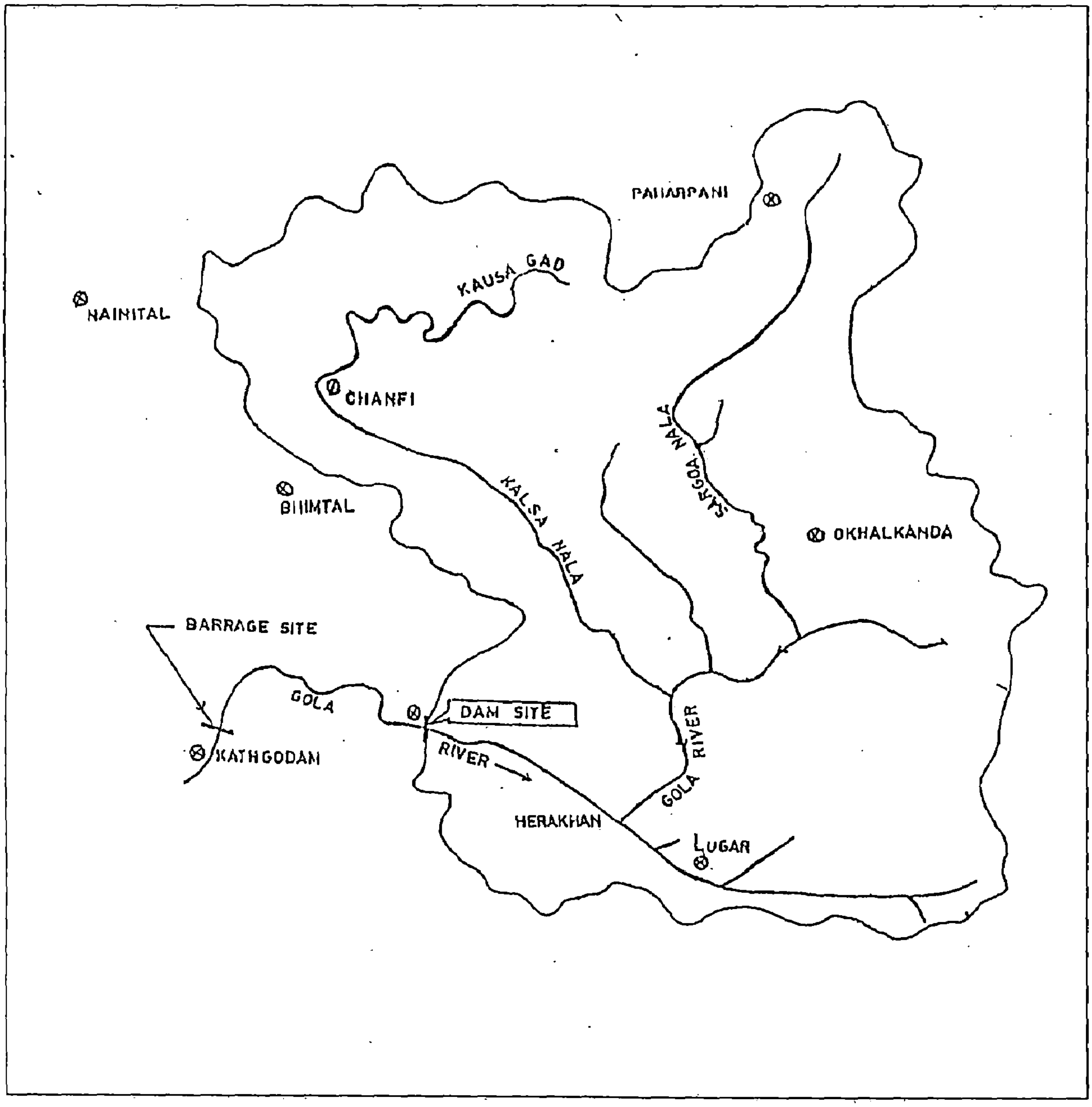

Fig. 3.7. Map of Gola Sub-Basin

3.3. METEOROLOGY AND CLIMATOLOGY

3.3.1. Upper Narmada Basin

The sub-zone has a continental type of climate. It is very hot in summer and cold in winter and receives most of the rainfall from the Southwest monsoon from June to October. Mean annual rainfall of the sub-zone varies approximately from 800 to 1600 mm.

About 50% of the sub-zone on eastern side is having mean annual temperature of 22.5° C to 25° C, while the western side is having mean annual temperature of 25° C to 27° C. The maximum temperature has been recorded in the month of May and minimum temperature has been recorded in the month of December.

The main soil group of the sub-zone is black soil comprising of different varieties viz., deep black soil, medium black soil and shallow black soil. In addition, mixed red and black soil, red and yellow soil and skeletal soil are also observed in pocket.

The sub-zone is having extensive area of about 55% under arable land, 40% of area under forest and remaining under wasteland, grassland, etc.

3.3.2 Lower Godavari Basin

The annual rainfall over major portion of the sub-zone is between 900 mm to 1600 mm. The annual rainfall is the lowest in the Western and southwestern parts of the sub-zone and increases northeastwards and eastwards. The centers of low rainfall are around Chandur (809 mm) in district Amraoti of Maharasthra on the west and around Siddipet in district Medak of Andhra Pradesh on the southwest. The centers of high rainfall are around Tamia (1787 mm) in district Chhidwara of Madya Pradesh on the northwest, around Lanji (1857 mm) in district Balaghat of M.P. on the northeast and around Jeypore (1940 mm) in district Koraput of Orissa on the southeast. The maximum rainfall is in the month of July at all representative stations, except Jeypore where the maximum rainfall is in the month of August.

The mean daily temperatures are slightly below 23°C over southeastern parts of sub-zone in Koraput district in Orissa State and over northwestern parts adjoining Pachmarhi in M.P. The highest mean daily temperature is slightly above 28°C over Karimnagar district of Andhra Pradesh. Major parts of the sub-zone experience mean daily temperatures between 25°C and 28°C.

The broad soil groups in the sub-zone are red soils and black soils. The red soils are of the red sandy, red loamy and red yellow type. Black soils are of deep black, clayey in texture. The textures of the red soils vary considerably from place to place.

More than 50% of the area is covered by forest. Arable land is of the order of 25%.

3.3.2. Gola Sub-Basin

The region has a dry season from October till May and wet season from June to September when approximately 90% of the average rainfall is received. The relative humidity in the area varies from 20% to 90%, highest in July to August and lowest in March to June. The temperature during the year varies from 13°C to 32°C. The maximum and minimum wind velocities observed in area are 268.0 km/hr and 21.0 km/hr respectively.

3.4. DATA AVAILABILITY FOR THE STUDY

For this study 53 flood hydrographs of six drainage basins have been analysis. Basin characteristics viz., area of basin (A in km^2), Length of the main river (L in km), slope of the river bed (S in %) and source of data are indicated in table 3.1.

For some of the drainage basins hydrograph (direct runoff, flood discharge including base flow) are available whereas for others only peak discharge and runoff volume are available. Data available for each of the six drainage basins are given in Appendix A to H.

Tabel 3.1. Drainage Basin Characteristics and Source of Data

No	River	Station	Area, A (km^2)	River Length, L (km)	Slope of River bed S, (%)	Number of Hydrographs	Source of Data
1	Temur	Bridge no. 249	518.67	56.62	0.303	7	CWC, 1983; NIH, 1995; Jain et al, 1995
2	Teriya	Bridge no. 253	114.22	35.42	0.321	11	CWC, 1983; NIH, 1995
3	Umar	Bridge no. 930	223.77	33.60	0.250	6	CWC, 1983; NIH, 1995
4	Kolar	Satrana	903.88	75.34	0.530	6	Jain et al, 1995
5	Godavari	Bridge no. 807	823.62	61.08	0.124	13	CWC, 1995; Mishra , 1998; Tyagi, 1995; Kumar et al, 2001
6	Gola	Dam site	450.00	23.50	1.40	10	WRDTC, 2002

3.5. PREPARATION OF DATA

Appendix A to H shows preparation of input data i.e. Q_p (m^3/sec), Q_p/V (m^3/sec/cm), Q_p/V^2 (m^3/sec/cm^2) and T_p (hours) for each of the storm event in the six drainage basins. A general procedure for preparation of data is explained in the following paragraphs.

The hydrographs to be used to prepare the standardized peak discharge distribution must be taken from the recorded data provided by the stream gage. These data, whether they are digital printouts of some type or recorded charts, will be in the form of stage height versus time. The hydrograph is indicated simply by a recorder increase or decrease in stage over time. These stages are converted to discharge by use of the stream gage rating curve. The time interval used to convert the stage hydrograph to a discharge hydrograph should be short enough so that substantial errors are not introduced by leaving out part of the hydrograph. In other words, the time interval available on the stage hydrograph might be shorter than required to produce a reasonable discharge hydrograph so that it is not necessary to use all the detail available.

Because the standardized peak discharge distribution method requires the runoff volume under the hydrograph, it is necessary to define the hydrograph in a manner that is consistent so that all hydrographs will be comparable. Choosing the point of the beginning of the hydrograph is simply to choose the point at which the hydrograph begins to rise from the antecedent stream flow. This antecedent stream flow is usually a gentle recession so that identification of the point of rise is easy.

Whereas choosing the point of rise on the hydrograph is usually easy, determining the end of hydrograph is not. Because it is not yet really known as to how to determine the end of hydrograph, it must be done on some consistent basis so that each hydrograph can be compared to other hydrograph on the same basis. The peak discharge can be used as reference point, because the end of hydrograph occurs at some time after peak discharge. An empirical equation has been developed by Rogers and Zia (1982) to identify the termination discharge of the hydrograph as follow.

$$Q_t = Q_o + Q_P^{0.6} \tag{3.1}$$

where Q_t is the discharge in cfs (0.0283 m^3/s) identifying the termination discharge of the hydrograph, Q_p is the peak discharge of the hydrograph in cfs (0.0283 m^3/s), and Q_o is the base flow discharge in cfs (0.0283 m^3/s) immediately prior to the hydrograph rise .

Once the hydrograph is identified, the next step is to determine the volume of water, *V*, under this hydrograph. This is a simple procedure and is easily done from tabulated hydrograph discharge. This volume of water is then distributed uniformly over the entire drainage area to determine the value of *V* (in inches or centimeters).

CHAPTER 4

ANALYSIS OF DRAINAGE BASIN LINEARITY AND NONLINEARITY

4.1. GENERAL

Based on review of work done by Rogers (1980), Rogers and Zia (1982), Mimikou (1983), and Singh and Aminian (1986), following important points emerge:

i) Mathematical linearity should not be confused with hydrologic linearity; even though runoff data from a hydrologically linear or nonlinear drainage basin may also be mathematically linear. Hydrologic linearity or nonlinearity can be determined by linear relationship between peak discharge and volume of runoff in log space.

ii) Three peak discharge distributions have been attempted,

   a) Original Peak Discharge Distribution (OPDD)

      Log $Q_p = b + m \log V$

   b) First Order Standardized Peak Discharge Distribution (FSPDD)

      Log $(Q_p/V) = b + (m - 1) \log V$

   c) Second Order Standardized Peak Discharge Distribution (SSPDD)

      Log $(Q_p/V^2) = b + (m - 2) \log V$

   For idealized hydrologically linear drainage basin, m = 1 and UH or IUH theory can be applied. Smaller the value of m or (m – 1) or (m – 2) more nonlinear is the drainage basin hydrologically.

iii) The volume (cm) – peak discharge (m^3/s) relationships rely on measured stream discharge data

iv) It is not necessary to separate base flow which requires use of an uncertain procedure

v) The data required for developing such relationships are several recorded flood hydrographs, drainage area, length, and slope values for each of the drainage basin considered in analysis.

vi) The number of hydrographs used is less important than the quality of hydrographs. Simple hydrographs are desirable though complex hydrographs can also be used and simplified. It is useful to choose hydrographs that cover the range from small area to large area if possible.

vii) Only Original Peak Discharge Distribution is necessary and quite sufficient by itself for checking drainage basin linearity (Mimikou, 1983)

viii) Standardized peak discharge distribution can be meaningfully applied to a variety of hydrologic analysis.

4.2. METHOD OF STUDY

A sample of six basins in India ranging in size from 114 sq. km to 904 sq. km for which several observed hydrographs were readily available has been used. Total 53 hydrographs have been used in the present study (Table. 3.1 in chapter 3). Characteristics of the six drainage basins and data availability have been discussed in chapter 3.

Runoff volumes for different floods were determined for (a) The natural hydrographs, which included base flow and (b) for the surface runoff hydrographs where base flow was already separated from hydrograph. Peak discharges were converted to uniform reference values for comparison. Accordingly, peak discharge were standardized with two standardized procedure dividing by (i) runoff volume in cm or by (ii) runoff volume in cm squared. This standardization does not consider specifically duration of rainfall excess, which is considered in unit hydrograph theory.

4.2.1. Volume – Peak Discharge Relationships

Original peak discharge (Q_p), first order standardized peak discharge (Q_p/V) and second order standardized peak discharge (Q_p/V^2) were then regressed on runoff volume (cm) in log space for each drainage basin and the coefficient of determination was computed. Results are shown in table 4.1.

4.2.2. Peak Discharge, Time to Peak and Volume Relationship

A basis exists to show that peak discharge is dependent not only on volume of runoff but also on time to peak (T_p). Mockus (1957) has given following relationship,

$$q_i = \frac{2V}{T_p + T_r} \quad ; T_b = T_p + T_r \tag{4.4}$$

where, q_i: peak discharge in cm/hour, V: total runoff in cm, T_p : time in hours from rise to peak, T_r: time in hours from peak to recession base of hydrograph, T_b: time base of hydrograph in hours

Let $y = \frac{q_i}{V^2}$. Substituting it in Eq. (4.4) above and become

$$y = \frac{2\,V}{V^2\,T_b} = \frac{2}{T_b\,V}$$

Mockus (1957) assumed T_b = 2.67 T_p for triangular unit hydrograph. In general, assuming that ratio of time base (T_b) to rise time (T_p) is constant i.e.

$$T_b = c\,T_p$$

Therefore $y = \frac{2}{c\,T_p\,V}$

By taking common logarithm of both sides

$$\text{Log } y = -\log(c\;T_p) - \log V$$

or $$\text{Log } q = -\log(c\;T_p) + \log V \tag{4.5}$$

Since q is peak discharge per unit area, it can be written as $\frac{Q_p}{A}$ with proper conversion to cm/hr. Therefore Q_p is related to rise time (T_p) and runoff (V) in log space. In the present study following relationship has been attempted for regression analysis

$$\text{Log } Q_p = b' + b'' \log T_p + m' \log V \tag{4.6}$$

b', b" and m' are regression constants. Equation (4.6) is termed as *"Peak Discharge, Time and Volume Relationship (PDTVR)"*. m' is equal to one for linear catchment. The results of peak discharge, time and volume relationship (PDTVR) for each drainage basin are shown in Table 4.2.

The three variables relation (one dependent variable and two independent variables) in Eq. (4.6) can be simplified by assuming that T_p is constant for a drainage basin.

$$\text{Log } Q_p = b_1 + m' \log V \tag{4.7}$$

where, $b_1 = b' + b'' \log T_p$

Equation (4.7) is same as OPDD. Assumption inherent in OPDD is that rise tie (T_p) is constant and ratio of base period (T_b) to rise time (T_p) is constant. Therefore, it is necessary to verify validity of these inherent assumptions in OPDD before use of OPDD in various applications.

Table 4.2 shows results of regression analysis (parameters and coefficient of determination) for relationship in log space between peak discharge (Q_p) as dependent variable and time to peak (T_p) and runoff volume (V) as independent variables. Results of OPDD (slope m and coefficient of determination r^2) are also shown for comparison.

An attempt was made to verify whether T_p and ratio T_b/T_p for a drainage basin are constant which are the inherent assumptions in OPDD. Regression analysis between (i) T_p and T_b, (ii) Q_p and T_p and (iii) T_p and V were carried out in log space. Coefficient of determination and average T_p, average T_b, average T_b/T_p are given in table 4.3.

4.2.3. Relation between Volume and Peak Discharge per Unit Area

In addition to regressing Q_p on V in log space, (Q_p/A) has also been regressed on V in log space. A is area of drainage basin in sq. km and the results are compared with original peak distribution in Table 4.4.

$$\text{Log}\,(Q_p/A) = b + m\,\text{Log}\,V \quad (4.8)$$

Table 4.1. Standardized Peak Discharge Distribution for Drainage Basins in India.

No	Basin	Station	Intercept (b)	OPDD		FSPDD		SSPDD	
				m	r^2	m-1	r^2	m - 2	r^2
1	Temur, Upper Narmada	Bridge No. 249	2.196	0.720	0.974	-0.280	0.852	-1.280	0.992
2	Teriya, Upper Narmada	Bridge No. 253	1.816	0.723	0.852	-0.277	0.458	-1.277	0.947
3	Umar, Upper Narmada	Bridge No. 930	2.005	0.647	0.903	-0.353	0.735	-1.353	0.976
4	Kolar, Upper Narmada	Satrana	2.729	0.667	0.953	-0.333	0.835	-1.333	0.988
5	Sub-Zone 3f, Lower Godavari	Bridge No. 807	2.588	0.800	0.894	-0.200	0.343	-1.200	0.950
6	Gola, Nainital	Dam site	2.091	0.644	0.696	-0.356	0.412	-1.356	0.910
Average coefficient of determination (r^2)					0.879		0.606		0.961

OPDD: Original Peak Discharge Distribution, FSPDD: First Order Standardized Peak Discharge Distribution, SSPDD: Second Order Standardized Peak Discharge Distribution

Table 4.2. Peak Discharge, Time and Volume Relationship (PDTVR) and OPDD in Log Space.

No.	River	Station	Peak discharge, time and volume relationship (PDTVR)						OPDD	
			b'	b"	m'	Partial coeff. of determination: $\log Q_p$ & $\log T_p$	Partial coeff. of determination: $\log Q_p$ & $\log V$	Multiple coeff. of determination	m	r^2
1	Temur, Upper Narmada	Br. No. 249	2.177	0.021	0.718	0.002	0.971	0.975	0.720	0.974
2	Teriya, Upper Narmada	Br. No. 253	2.048	-0.324	0.750	0.140	0.872	0.873	0.723	0.852
3	Umar, Upper Narmada	Br. No. 930	2.353	-0.400	0.777	0.838	0.983	0.984	0.647	0.903
4	Kolar, Upper Narmada	Satrana	2.858	-0.189	0.727	0.187	0.939	0.962	0.667	0.953
5	Sub-Zone 3f	Br. No. 807	2.870	-0.330	0.759	0.339	0.915	0.930	0.800	0.894
6	Gola, Nainital	Dam site	2.283	-0.347	0.737	0.792	0.931	0.937	0.644	0.696

Table 4.3. Coefficient of Determination in Log Space and Average T_p, T_b and ratio T_b/T_p

No	River	Station	Coeff. of determination (r^2)			Average T_p (hrs)	Average T_b (hrs)	Average T_b/T_p (hrs)
			T_p vs T_b	Q_p vs T_p	T_p vs V			
1	Temur, Upper Narmada	Br. No. 249	0.793	0.100	0.234	7.83	26.00	3.32
2	Teriya, Upper Narmada	Br. No. 253	0.589	0.004	0.048	5.82	21.27	3.65
3	Umar, Upper Narmada	Br. No. 930	0.650	0.092	0.321	12.83	29.17	2.27
4	Kolar, Upper Narmada	Satrana	0.402	0.375	0.485	9.00	31.67	3.52
5	Sub-Zone 3f,	Br. No. 807	0.548	0.181	0.066	7.38	20.23	2.74
6	Gola, Nainital	Dam site,	0.634	0.078	0.056	13.30	52.20	3.92

Table 4.4. Linear Relation between Volume and Peak Discharge per Unit Area

No	River	Station	Intercept b	OPDD		SSPDD	
				Slope, α	r^2	Slope, M	r^2
1	Temur, Upper Narmada	Br. No. 249	-0.9625	0.720	0.974	-1.280	0.992
2	Teriya, Upper Narmada	Br. No. 253	-0.6852	0.723	0.852	-1.277	0.947
3	Umar, Upper Narmada	Br. No. 930	-0.7848	0.645	0.902	-1.355	0.976
4	Kolar, Upper Narmada	Satrana	-0.6713	0.667	0.953	-1.333	0.988
5	Sub-Zone 3f, Lower Godavari	Br. No. 807	-0.7665	0.808	0.900	-1.192	0.952
6	Gola, Nainital	Dam site, Kathgodam	-1.0062	0.644	0.696	-1.356	0.910

4.3. ANALYSIS OF PEAK DISCHARGE DISTRIBUTION

4.3.1. General.

The peak discharge distributions for drainage basins in India are graphically depicted in Figs. 4.1. to 4.6. Their intercept, slope and coefficient of determination r^2 are given in Table 4.1. Scatter in the plotted data, as indicated by the r^2, represent data errors, i.e. from gauging, back water etc., and also the basin runoff condition. The magnitude of the data scatters around the regression line in Figs. 4.1. to. 4.6 have been statistically checked for each sample by analysis of variance and 95% confidence limits. All r^2 values of the original peak distribution and the SSPDD are found to be significant at the 95 % confidence level. Analysis of variance and 95% confidence limit can be shown in Appendices I and L.

4.3.2. Significance of the Coefficient of Determination

The high values of the coefficient of determination in OPDD and SSPDD indicate that on an average 87.9% of the variation in original peak discharge distribution (OPDD) and 96.1% in second order peak discharge distribution (SSPDD) can be explained by runoff volume alone.

The successive subtraction from each of the original distribution variables in Eq. (4.1) of the independent variable log *V*, i.e. subtracted once for obtaining the FSPDD and twice for the SSPDD, can improve the original correlation provided that the derived distribution's slope is not milder than the original slope m. Otherwise, the correlation of the derived distribution is forced to be worse than the original one, e.g. for linear basins with slope m = 1.0 in Eq. (4.1) and m-1 = 0 in Eq. (4.2) , the subtraction of log *V* from the r.h.s. of Eq. (4.1) results in the complete disappearance of the independent variable and, thus, in less r^2-value and significant data scatter for the FSPDD. From Figs. 4.1 – 4.6 for six drainage basins under study one can see that the condition for the steeper than original slope is always met for the SSPDD, whose consistent improvement in r^2-values and in reduction of data scatter over the original distribution occurs with decreasing degree of linear association between the original variables. All r^2-values of the original peak discharge distribution were found to be significant at the 95% confidence level.

SSPDD has higher r^2 compared to OPDD but intercept b remains same. There is a consistent and systematic improvement in r^2 values and in data scatter for the SSPDD over the OPDD. Improvement is significant in case of nonlinear basins such as Gola basin (m = 0.644, r^2 = 0.696 for OPDD and r^2 = 0.910 for SSPDD). The deterioration in r^2 and in data error from OPDD to FSPDD is significant for linear basins such as sub-zone 3f of lower Godavari (m=0.800, r^2 = 0.894 for OPDD and r^2 = 0.343 for FSPDD). This corroborates the finding of Mimikou (1983).

Some qualitative explanation of first and second order r^2 differences may be indicated by an apparent relation between the distribution of rainfall on a drainage basin and the degree of linearity of first order standardized peak discharge distribution as shown by the coefficient of determination, (Rogers, 1980). In the case of Gola drainage basin, r^2 value for FSPDD is 0.412 whereas it is 0.910 for SSPDD. Probably rainfall distribution over the basin being mountainous is not uniform. In contrast to this in Temur and Kolar drainage basins difference in r^2 for FSPDD and SSPDD is small. The hydrographs of Temur and Kolar basins might have resulted from storms that were similar in type and therefore there is little or no precipitation variability reflected in the r^2 values for FSPDD.

There is strong linear relation between peak discharge (m^3/s) and runoff volume (cm) in logarithm term in drainage basins in the present study.

4.3.3. Slope of standardized peak discharge distribution

The slope of the line that best fits the standardized peak discharge data is an indicator of the nonlinearity of the runoff distribution. Although it has been suggested that spatial distribution of rainfall on a drainage basin might be an element affecting the coefficient of determination of standardized peak discharge data, discussion here is limited to the factors that appear to determine the slope of the standardized peak discharge distribution.

A nonlinear standardized peak discharge distribution slope is defined as any negative slope value for first order distribution and a slope value less than negative one for second order distribution. Because second order standardized peak discharge distribution always has a higher r^2, this distribution was used as the reference distribution by Rogers, (1980). However, Mimikou (1983) has recommended that only original peak discharge distribution is necessary and sufficient by itself for checking drainage basin linearity. The original peak discharge distribution has slope m being 1.0 for linear and less than 1.0 for nonlinear basins.

Theoretically, runoff from an impervious, or uniformly pervious surface is linear and the second order standardized peak discharge distribution will have a slope of –1.0. This linear condition can be confirmed by applying the rational formula to such a surface for different rainfall intensities and therefore, different volumes of runoff.

It is apparent from the above that the most positive slope possible for second order peak discharge distribution is –1.0 and 1.0 for original peak discharge distribution and this slope represents linear runoff distribution.

In this study, the slopes encountered for original peak discharge distribution, are less than 1 (one) and they have range 0.644 to 0.800. It therefore appears that these drainage basins have hydrologic nonlinearity. Rogers (1980) in U.S.A and Mimikou (1983) in Greece found that most drainage basins had hydrologic nonlinearity. Only one drainage basin can be assumed as hydrologic linearity i.e. 3f Sub-Zone of Lower Godavari Basin up to bridge no. 807, and all other drainage basins exhibit nonlinearity.

There appears to be little or no correlation between area and other conventional drainage basin physical characteristics and standardized peak discharge distribution slopes.

Precipitation data available was insufficient to test the effect of rainfall distribution on slope, although the effect of this rainfall factor might be important.

Rogers (1980) collected some evidence, which suggests that drainage basin nonlinearity and therefore the slope of the standardized peak discharge data is related to non-uniform infiltration capacity distribution condition, which commonly exists on many drainage basins. This condition was identified when infiltration rates on drainage basins were measured. It was found from these infiltration experiments that highest infiltration capacities occurred on the drainage basin divides. Further, the infiltration capacities typically decreased down topographic slope, and were lowest in the main drainage valley bottoms (Rogers, 1970b, in Rogers, 1980).

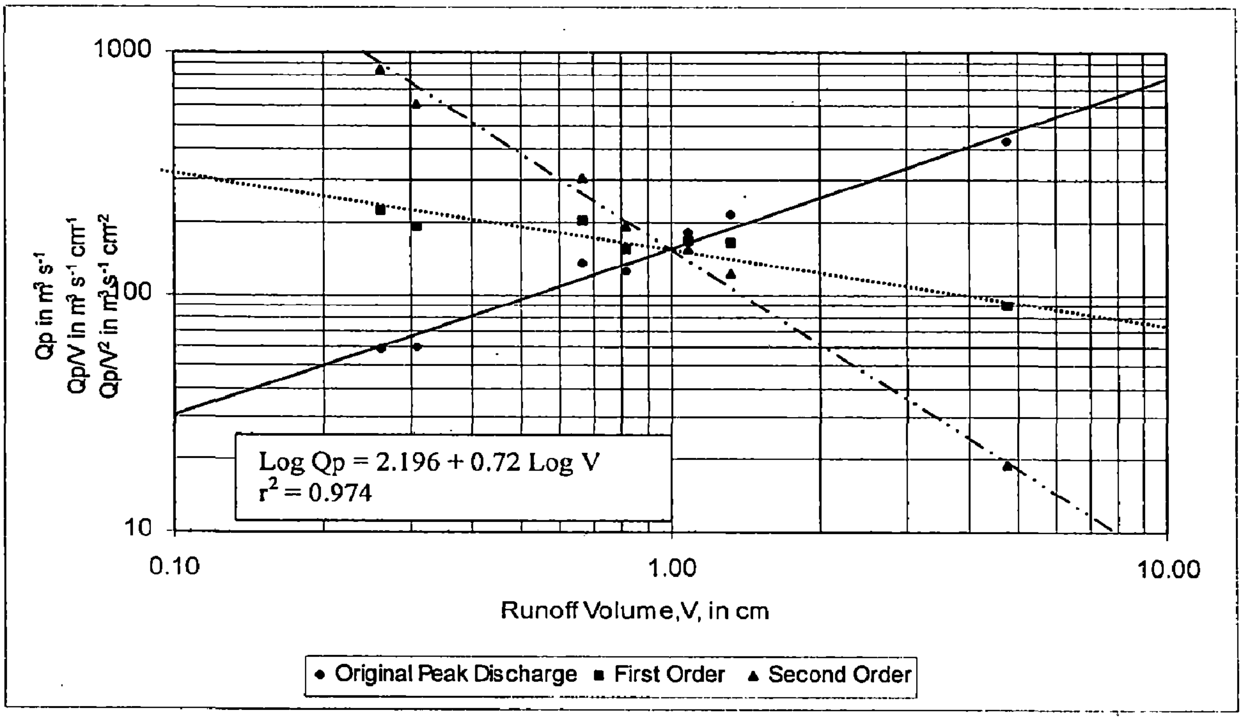

Fig. 4.1. Peak Discharge Distribution for Temur Sub-Basin (Upper Narmada Basin)

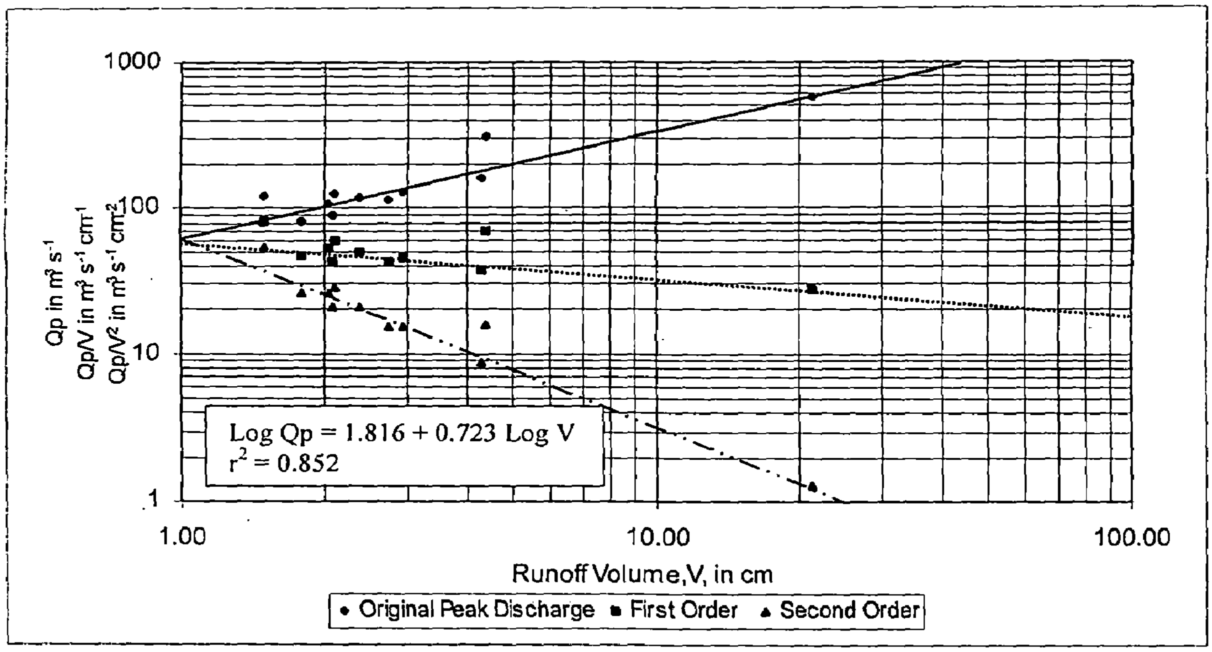

Fig. 4.2. Peak Discharge Distribution for Teriya Sub-Basin (Upper Narmada Basin)

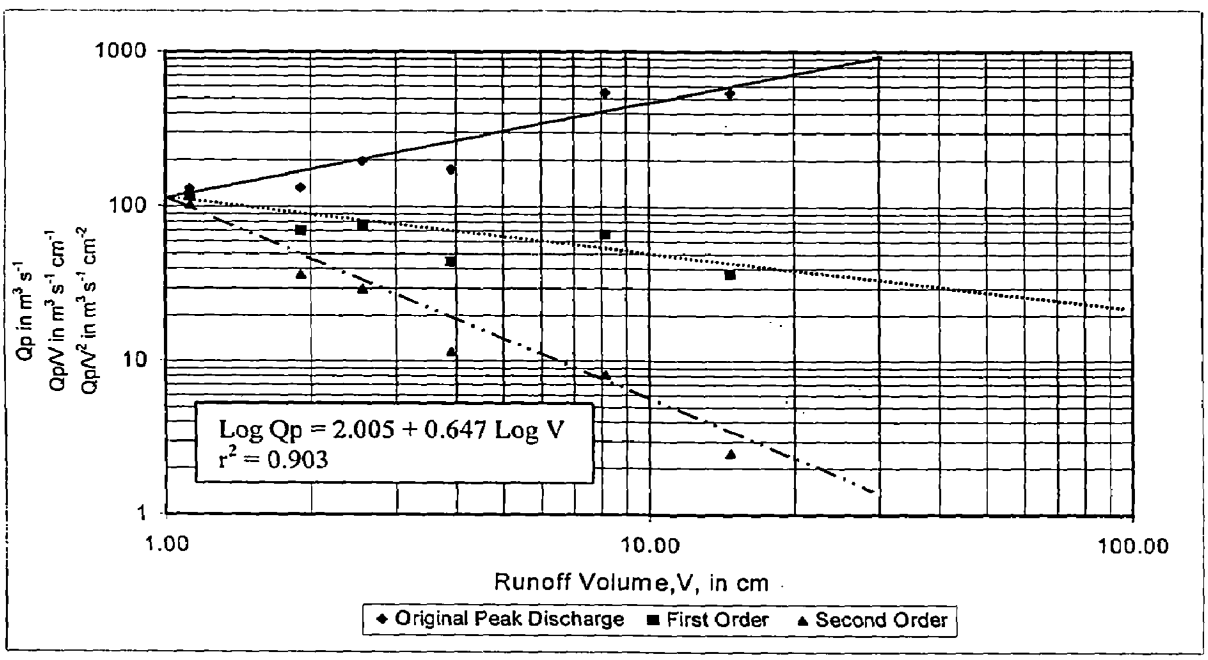

Fig. 4.3. Peak Discharge Distribution for Umar Sub-Basin (Upper Narmada Basin)

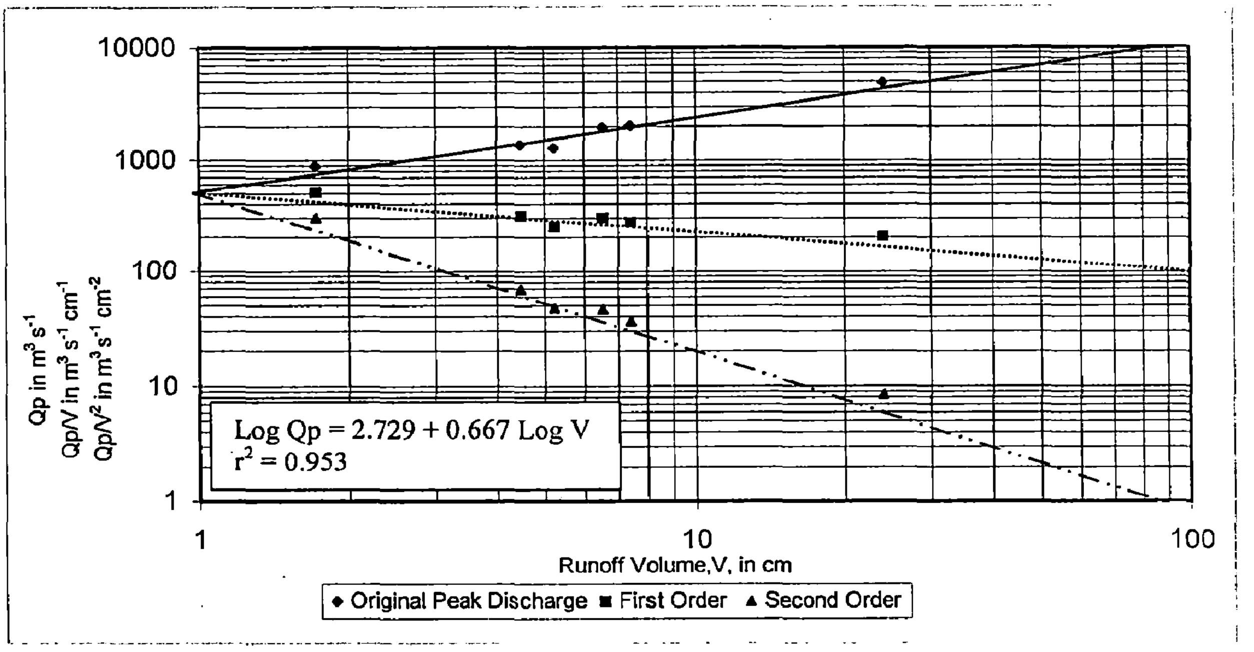

Fig. 4.4. Peak Discharge Distribution for Kolar Sub-Basin (Upper Narmada Basin)

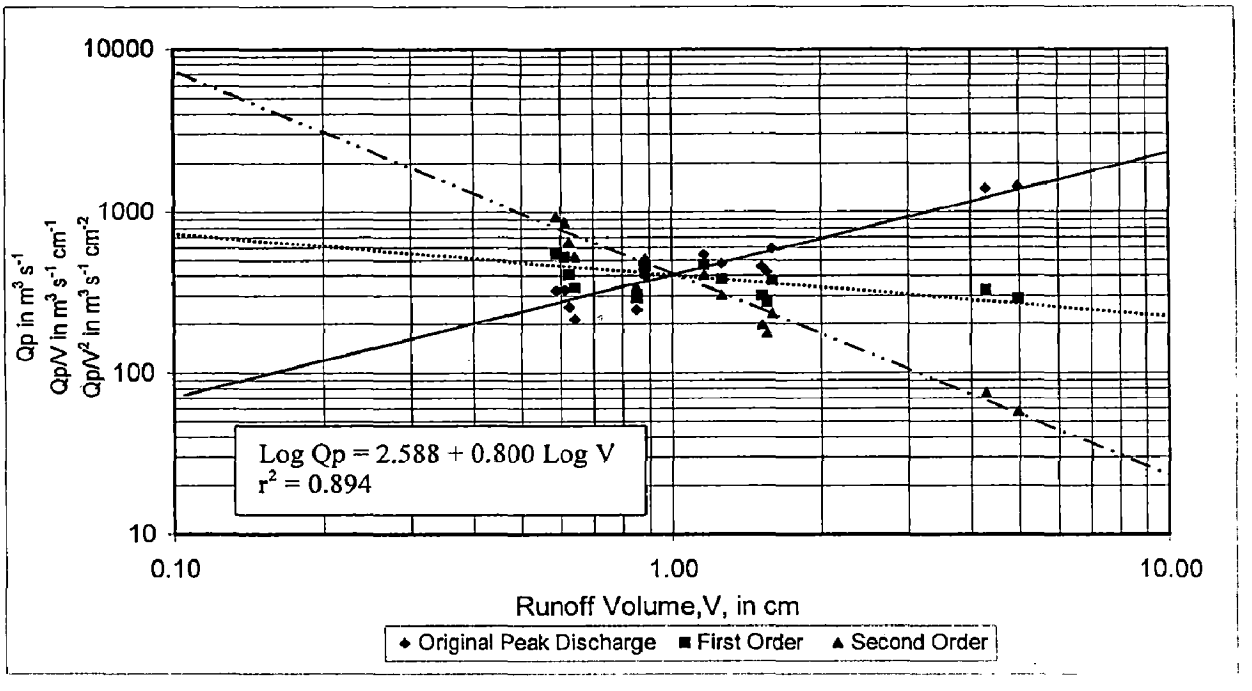

Fig. 4.5. Peak Discharge Distribution for 3f Sub-Zone (Lower Godawari Basin)

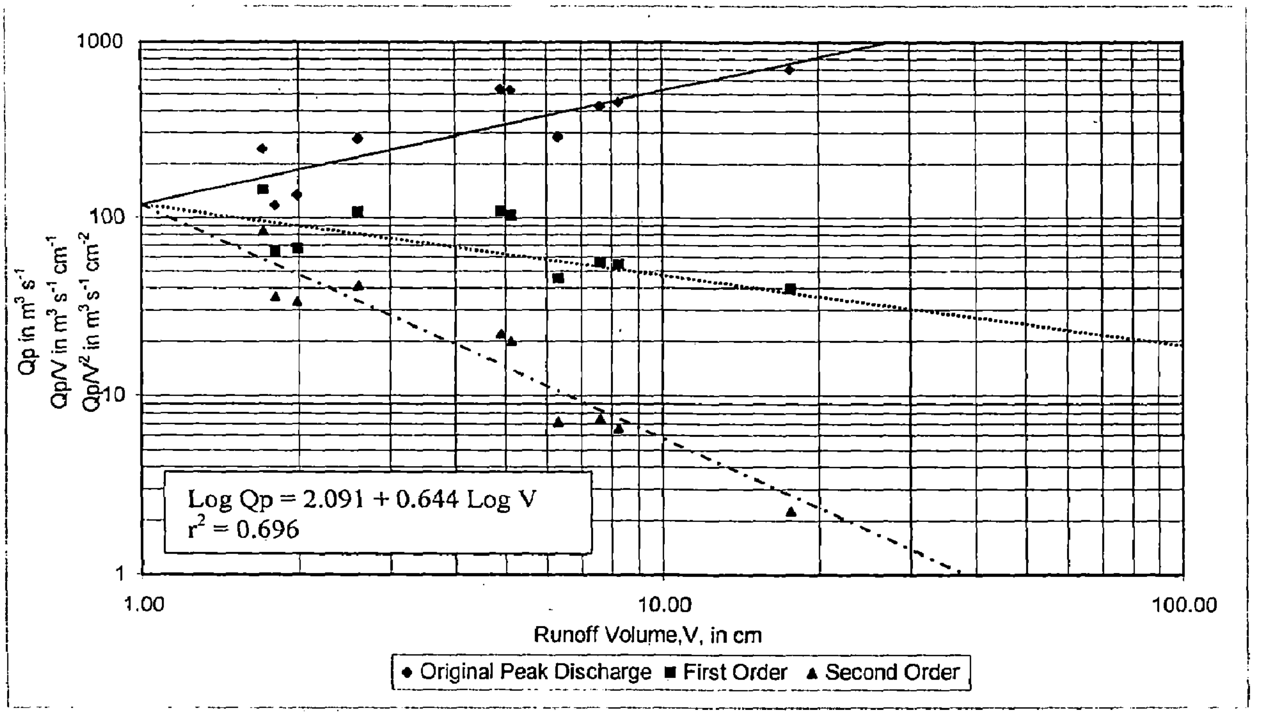

Fig. 4.6. Peak Discharge Distribution for Gola Sub-Basin (Western Himalayan)

4.4. PEAK DISCHARGE, TIME AND VOLUME RELATIONSHIP (PDTVR)

It has been explained in section 4.2.2 that relation exists between time to peak (T_p, in hours), peak discharge (Q_p, in m^3/s) and runoff volume (V, in cm), (equation 4.6). Also it has been explained that OPDD is obtained from peak discharge, time and volume relationship (PDTVR) with substitution of average time to peak (Eq. 4.7). Regression analysis was carried out in the logarithm terms. Regression analysis shows strong correlation between peak discharge and runoff volume (r^2 = 0.872 to 0.984 for the six basins). Further, correlation between peak discharge and time to peak is stronger for Umar sub-basin (r = -0.915, r^2 = 0.837) and Gola sub-basin (r = -0.890, r^2 = 0.792) but very weak in other basins, as seen in table 4.2. It is interesting to note that Gola and Umar sub-basins are most nonlinear basins out of the six basins. For Gola basin, partial coefficients of determination r^2 between log Q_P and log V in PDTVR is significantly higher (0.931) compared to coefficient of determination in OPDD (0.696). Therefore use of PDTVR might be more appropriate for prediction of peak discharge in highly nonlinear basin. Average multiple coefficient of determination (r^2) for PDTVR is 0.943.

Q_p is inversely proportional to T_p in log space. Values of m (i.e. slope related with log V) improve over corresponding values in OPDD for all the basins but only marginally.

As seen in table 4.3, T_p has a stronger correlation with T_b as compared to correlation with Q_p or V. For 3f sub-zone of Godavari basin, the average ratio T_b/T_p is 2.74 (Table 4.3) which is nearly close to 2.67 assumed by Mockus (1957) for triangular unit hydrograph (see section 4.2.2) applicable to linear basin and this study also shows that 3f sub-zone is nearly linear. Since T_p does not depend on Q_p and V, it can be assumed to be constant.

Regression analysis between Tp and geomorphological indexes (A/L, AS/L, A) in log space was also carried out. Correlations were found to be very weak.

Flood hydrographs of several other basins need to be analyzed for comparative performance of OPDD and peak discharge, time and volume relationship (PDTVR). Regression analysis of peak discharges, time to peak and runoff volume relation is shown in Appendix J. Results and comparison with original peak discharge distributions are given in Table 4.2.

4.5. RELATION BETWEEN VOLUME AND PEAK DISCHARGE PER UNIT AREA

This relation is extension of the peak discharge distribution. Equations (4.8) is proposed relation between discharge per unit basin area and direct runoff volume (cm) in log space. The results of regression analysis are shown in table 4.4. Figure 4.7 to 4.12 show the relationship between runoff volume and peak discharge per unit area in log space for the six drainage basins in India.

The regression analysis shows that Temur and Teriya sub-basins are hydrologically similar as indicated by similar value of slope m. Similarly Umar and Gola sub-basins are also hydrologically similar in term of runoff characteristics. However as discussed later Umar and Gola sub-basins differ in terms of relation between intercept b and catchment characteristics.

Table 4.3 shows that the coefficient of determination r^2 varies from 0.696 to 0.974 in the original relation and varies from 0.910 to 0.992 in the second order relation. This indicates strong relation between peak discharge per unit area (cm/hr) and direct runoff volume per unit area (cm). Parameter b varies from –0.6713 to –1.0062 and the slope m of above relation varies from 0.644 to 0.808 for original relation and varies from –1.192 to

−1.356 for second order relation. The slopes of this relation also indicate nonlinearity of hydrologic basin.

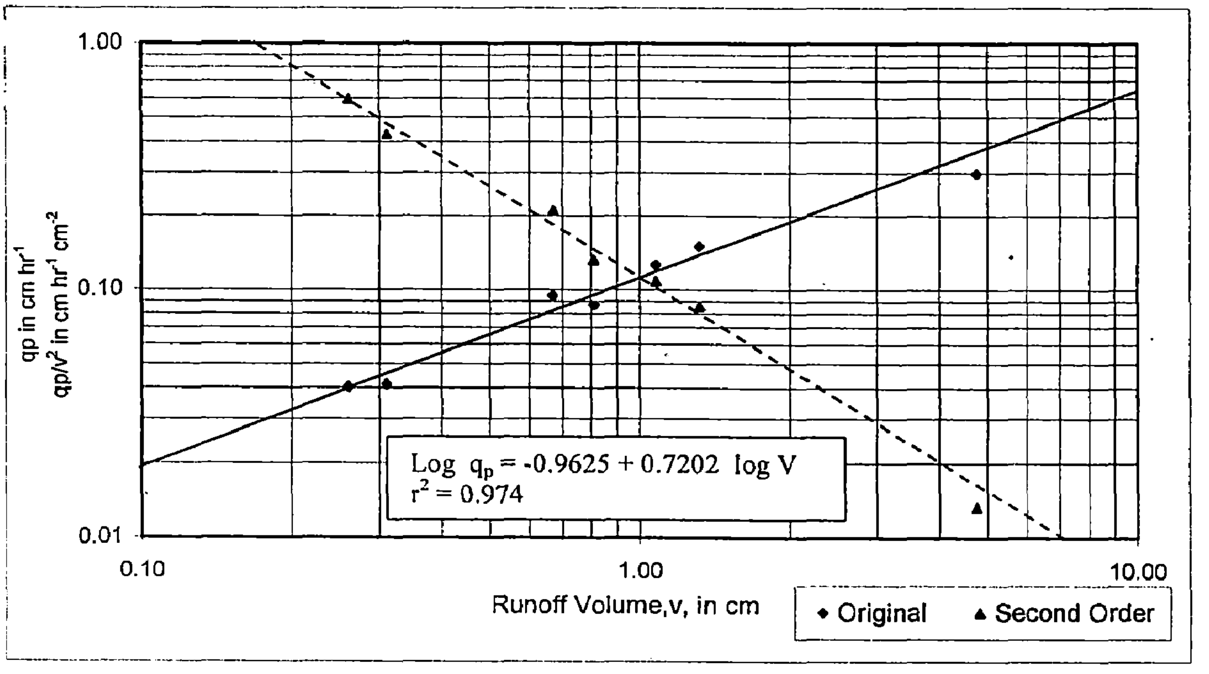

Fig. 4.7. Peak Discharge Distribution per Unit Area for Temur Sub-Basin (Upper Narmada Basin)

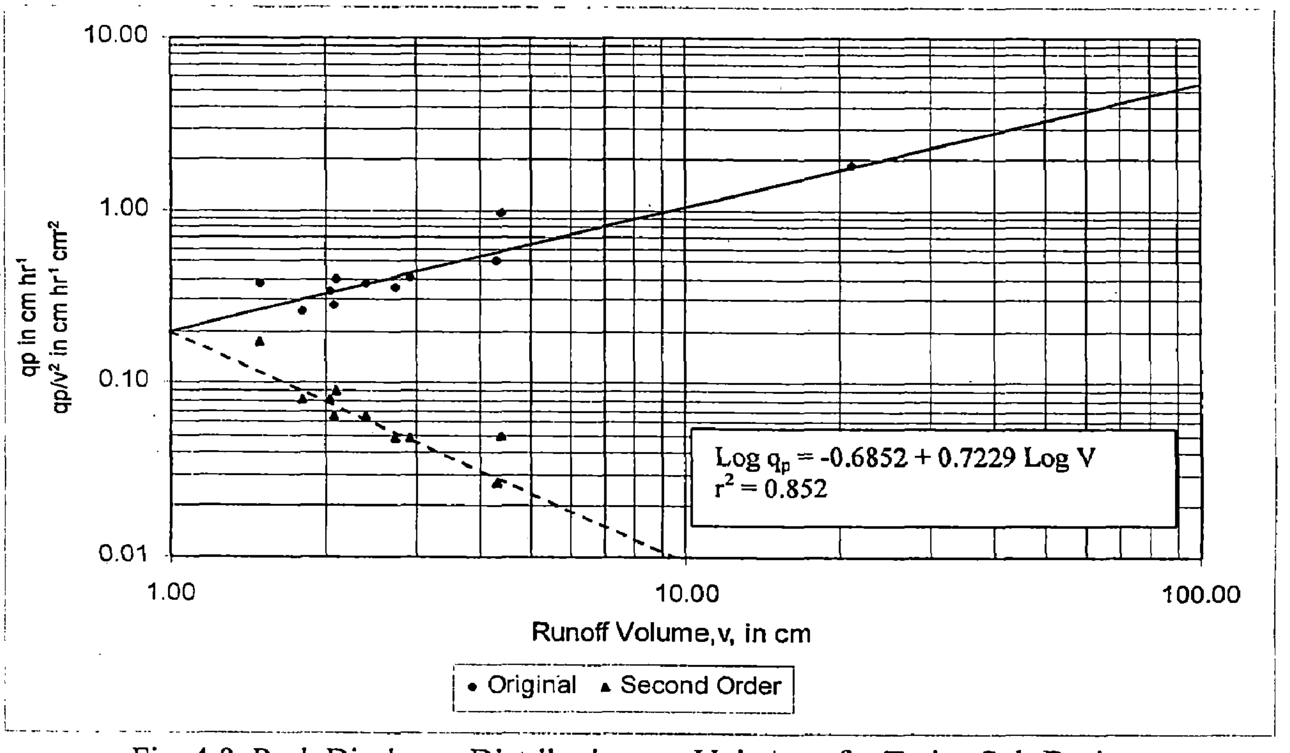

Fig. 4.8. Peak Discharge Distribution per Unit Area for Teriya Sub-Basin (Upper Narmada Basin)

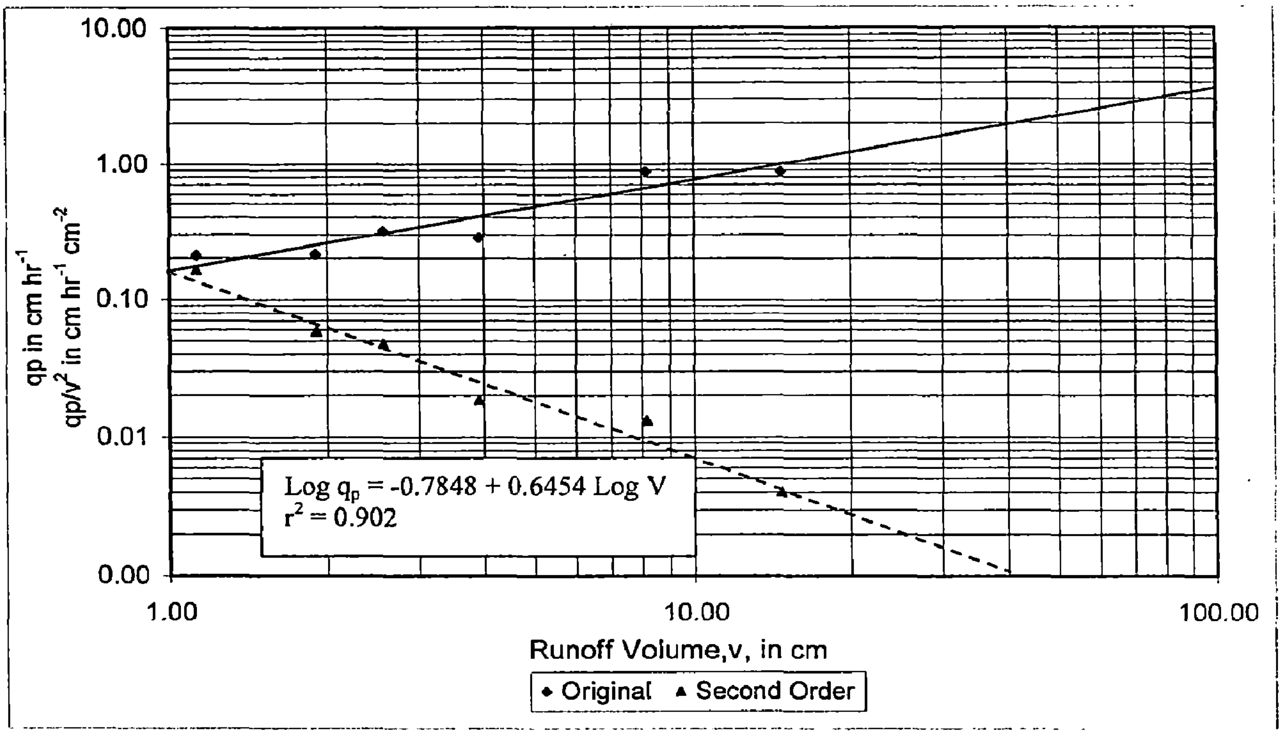

Fig. 4.9. Peak Discharge Distribution per Unit Area for Umar Sub-Basin (Upper Narmada Basin)

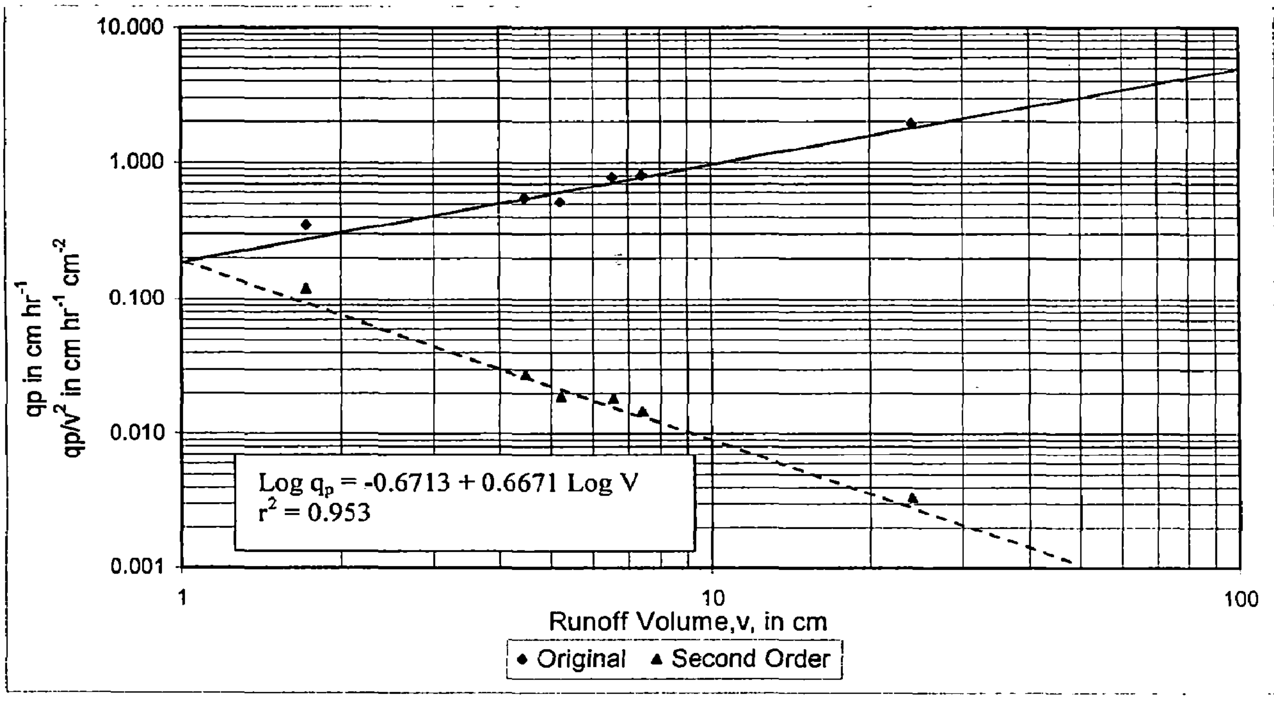

Fig. 4.10. Peak Discharge Distribution per Unit Area for Kolar Sub-Basin (Upper Narmada Basin)

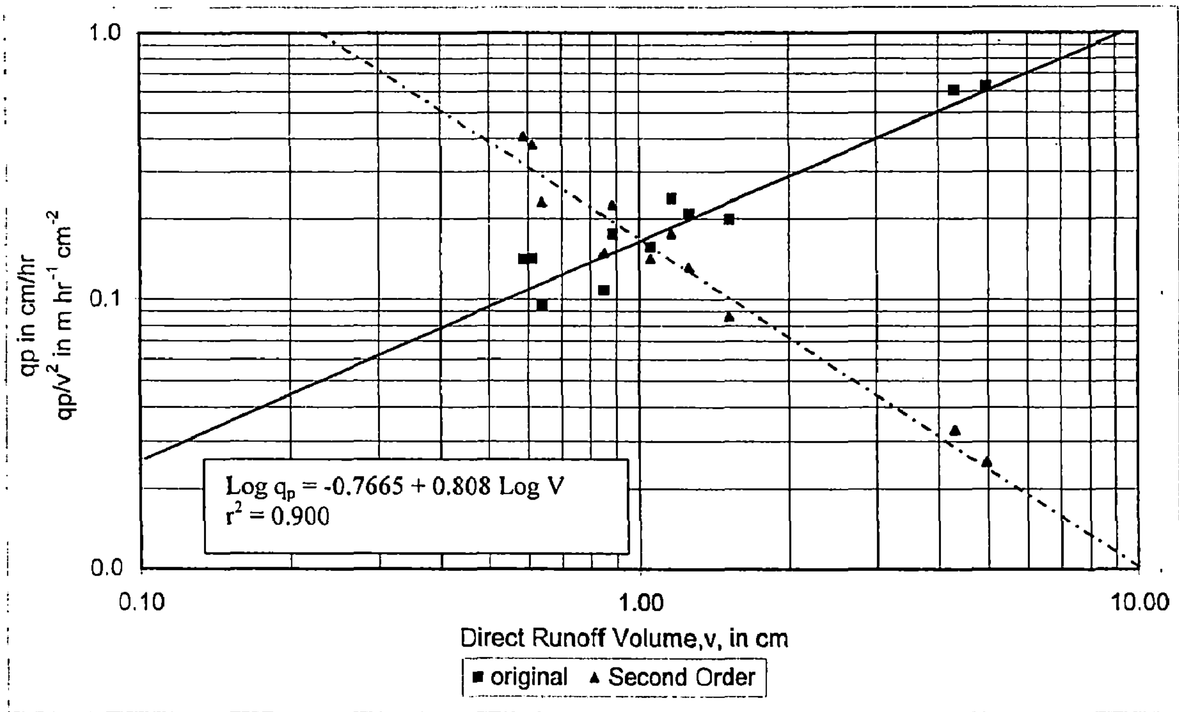

Fig. 4.11. Peak Discharge Distribution per Unit Area for 3f Sub-Zone (Lower Godavari Basin)

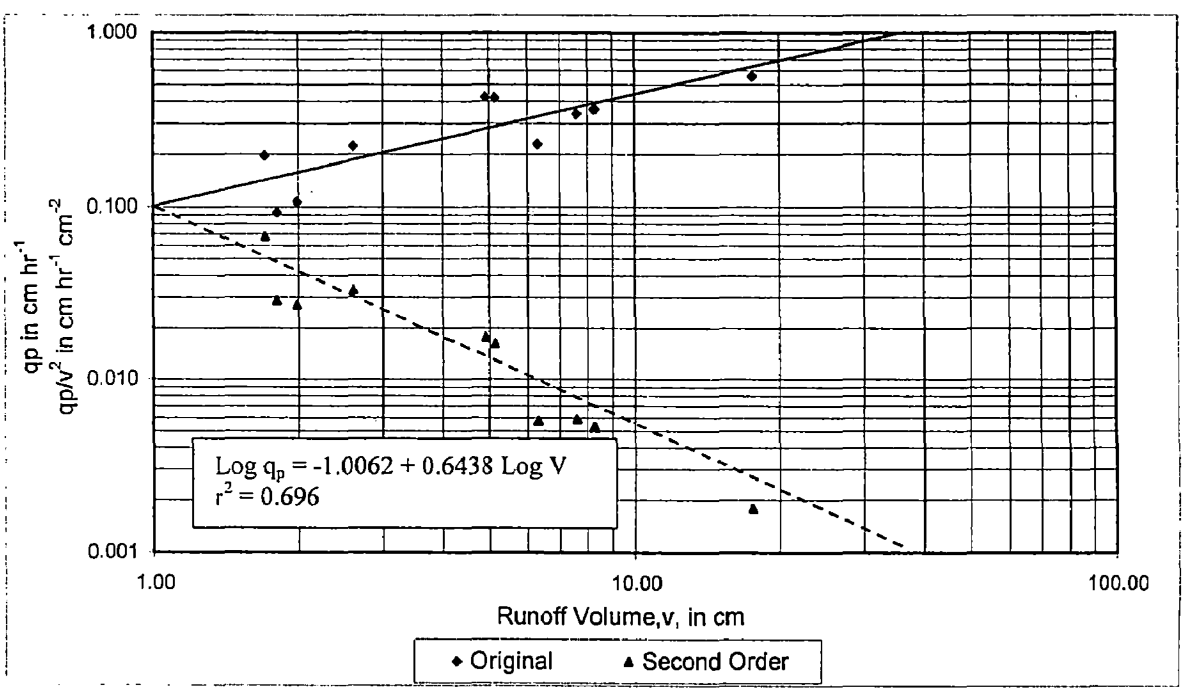

Fig. 4.12. Peak Discharge Distribution per Unit Area for Gola Sub-Basin (Western Himalayan)

4.6. PEAK DISCHARGE PREDICTION

The prediction of peak discharge is very important for the design of hydraulic structure. The most commonly used method for estimation of design peak discharges is based on a rainfall frequency analysis and the use of a unit hydrograph of the drainage basin, indicating its runoff response. The use of this method in basins with nonlinear runoff characteristics becomes questionable and inaccurate. The peak discharge distribution in Eq. (4.1) constitutes a reliable and very simple peak discharge prediction method, provided that the total input runoff volume is known. The method is applicable to both hydrologically linear and nonlinear basins, unlike hydrograph method, which is applicable only for hydrologically linear basins. The surface input (runoff volume in centimeters) can be estimated either by rainfall measurement (for flood forecasting), or from rainfall frequency analysis or from rainfall maximizing methods (for estimation of design flood), by subtracting from the rainfall an appropriately estimated average drainage basin infiltration rate. A base flow and surface hydrograph runoff volume analysis of the available hydrographs, produced from rainfalls with durations similar to the selected rainfall duration, can provide an estimation of the average percentage of surface runoff increase due to base flow. This increase of the estimated surface runoff volume provides the total input flood runoff volume in centimeters uniformly distributed over the basin.

4.6.1. Comparison with Observed Flood Peaks

In order to verify the accuracy of Eq. (4.1) to predict peak discharges, a comparison has been made between observed flood peaks outside those used for the calibration of the distributions and their estimates from Eq. (4.1). Two examples are given; one for the nonlinear Umar sub-basin (m = 0.647) and another for approximately linear 3f sub-zone of lower Godavari basin (m = 0.800).

In Umar sub-basin, a 4-hr rainfall with runoff volume 2.54 cm produced a peak discharge of 195.5 m^3/sec as observed on June 23, 1971. Its estimate from Eq. (4.1) by using the runoff volume 2.54 cm is 185 m^3/s, which is close (5.37% error) to the actual discharge.

In 3f sub-zone of Lower Godavari basin, a 2-hr rainfall with runoff volume 1.05 cm produced a observed peak discharge equal to 360.3 m^3/s. Its estimate from Eq. (4.1) by using the runoff volume 1.05 cm is 402 m^3/s, which is not so close (11.57% error) to the actual discharge

The coefficient of determination (r^2) for Umar sub-basin is higher than for sub-zone of lower Godavari basin. Hence there is less prediction error in peak discharge for Umar sub-basin. These two examples illustrate the potential use of peak discharge distribution in prediction of peak flood for linear and nonlinear hydrologic basins.

4.6.2. Comparison with UH Method in Estimation of PMF

Three examples are given in Table 4.5 for comparative study of original peak discharge distribution and unit hydrograph application in prediction of Probable Maximum Flood (PMF) in linear basin (lower Godavari) and nonlinear basins (Umar and Temur). Hydrologic data and 1-hour synthetic unit hydrograph for these basins are taken from CWC (1995) and CWC (1983). The estimated runoff volume and peak flood using OPDD and unit hydrograph method are shown in Table 4.5.

Table 4.5. Probable Maximum Flood Using OPDD and Unit Hydrograph

Basin	Hydrologic character	PMP (cm)	PMP duration (hrs)	Infiltration rate (cm/hr)	Base flow ($m^3/s/km^2$)	Runoff volume (cm)	Probable Max. Flood (m^3/s)		
							Using OPDD	Using 1-hr UH	Error (%)
Sub-zone 3f, lower Godavari	Nearly linear m=0.800	14.61	10	0.2	0.05	12.22	2909.4	2943.6	1.16
Temur, Upper Narmada	Nonlinear m=0.720	13.11	7	0.3	0.05	11.01	909.8	2300.7	60.5
Umar, Upper Narmada	Nonlinear m=0.647	14.03	5	0.3	0.05	12.53	530.4	1413.4	62.5

PMP : Probable Maximum Precipitation
1 hour UH: One hour unit hydrograph derived by Snyder's method.

Difference in prediction by two methods is less in the case of sub-zone 3f (lower Godavari basin) because the basin is nearly linear and therefore UH theory can be applied. The difference in prediction by the two methods is very high in Umar and Temur sub-basins because the basins are nonlinear ($m = 0.647$ and $m = 0.720$) and application of unit hydrograph theory is not valid.

These examples demonstrate that the hydrologic linearity of a basin should be checked before using the linear design methods. Application of unit hydrograph theory in nonlinear basins can result in serious design errors. For estimation of design peak discharge, only the peak discharge distribution in Eq. (4.1) is needed. It can be successfully used in both hydrologically linear and nonlinear basin. Calculation PMF by UH method can be seen in Appendix N.

4.7. RELATIONSHIP OF b AND m WITH CATCHMENT CHARACTERISTICS

4.7.1. Relationship Between b and Catchment Characteristics

Intercept 'b' in the original peak distribution (Eq. 4.1) has a physical significance. Usefulness of developing such relationship lies in: i) predicting peak discharge per unit of direct runoff in application of unit hydrograph theory and ii) developing peak discharge distribution from known catchment characteristics. Intercept b is equal to log Q_p when runoff volume V is equal to 1 cm. Its prediction from catchment characteristics can help in applying peak discharge distribution in similar ungaged catchments.

Base on study in Greece, Mimikou, (1983) found that variation in the intercept b is significantly explained by the logarithm of any of the two basin morphological indices AS/L and A/L; with A (km^2) the drainage area, L (km) the main coarse length and S (%) average bed slope of the river, respectively, as given in Table 3.1.

Singh and Aminian (1986) studied 134 drainage basins and found that basin area alone explains variance of b by more than 86 percent ($r^2 = 0.861$). Inclusion of bed slope S and stream length increased r^2 marginally to 86.9%. Singh and Aminian (1986) therefore concluded that relationship between b and catchment area A alone is satisfactory.

In the present study, regional intercept prediction equation has been developed by using A, L, S data of the six drainage basins in India. The developed regional intercept prediction equations, calibrated with the least square method are the following,

$$b = -0.1905 + 0.9217 \log A, \qquad r^2 = 0.750 \tag{4.9}$$

$$b = 1.0024 + 1.4046 \, \text{Log} \, (A/L), \qquad r^2 = 0.828 \tag{4.10}$$

and

$$b = 1.9381 + 0.9647 \text{ Log (AS/L)}, \quad r^2 = 0.565 \tag{4.11}$$

The relationship in Eqs. (4.9), (4.10) and (4.11) are linear in semi log as given in figure 4.13 (b vs log A/L), figure 4.14 (b vs log AS/L) and figure 4.15 (b vs log A).

Since log $\dot{Q}_p$ = b for V =1, Equations. (4.9), (4.10), (4.11) can be rewritten as

$$Q_p' = 10^{-0.1905} (A)^{0.9217} \tag{4.12}$$

$$Q_p' = 10^{1.0024} (A/L)^{1.4046} \tag{4.13}$$

and

$$Q_p' = 10^{1.9381} (AS/L)^{0.9647} \tag{4.14}$$

where Q_p' is peak discharge when runoff volume is one unit.

It is important to note that point relating Gola basin appears to be an outlier as seen in figure 4.13 and figure 4.14, and was therefore excluded. Therefore data for only five basins have been used in developing relationship between b and A/L and between b and AS/L, (excluding Gola basin).

As suggested by Singh and Aminian (1986) relationship between b and A was also attempted as shown in figure 4.15. Now point relating to Gola basin is not an outlier. Gola basin is in Himalayan region with different geomorphological characteristic (steep slope, etc) compared to other five basins, which are in relatively flatter terrain. By considering catchment area alone as an independent variable influencing b the distinguishing feature of regional geomorphology (L, S) are not incorporated which would lead to incorrect evaluation of b. Therefore it is necessary that separate relationship between b and A/L or between b and AS/L should be evolved for Himalayan region and flat region encompassing remaining five basins. It would be incorrect to relate b with A only.

Based on highest coefficient of determination for b vs A/L, it is recommended for estimating of b. However further study is required for investigation of dependence of b on catchment characteristics.

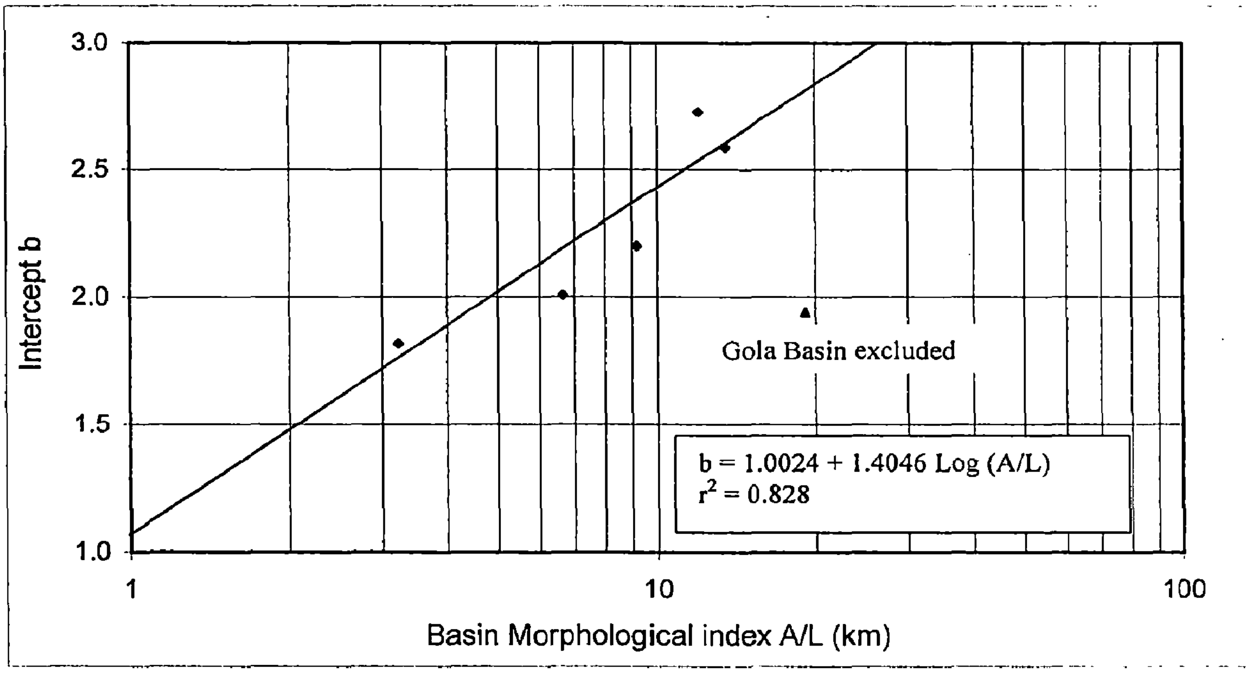

Fig. 4.13. Relationship between the Intercept b and the Basin Morphological Index A/L

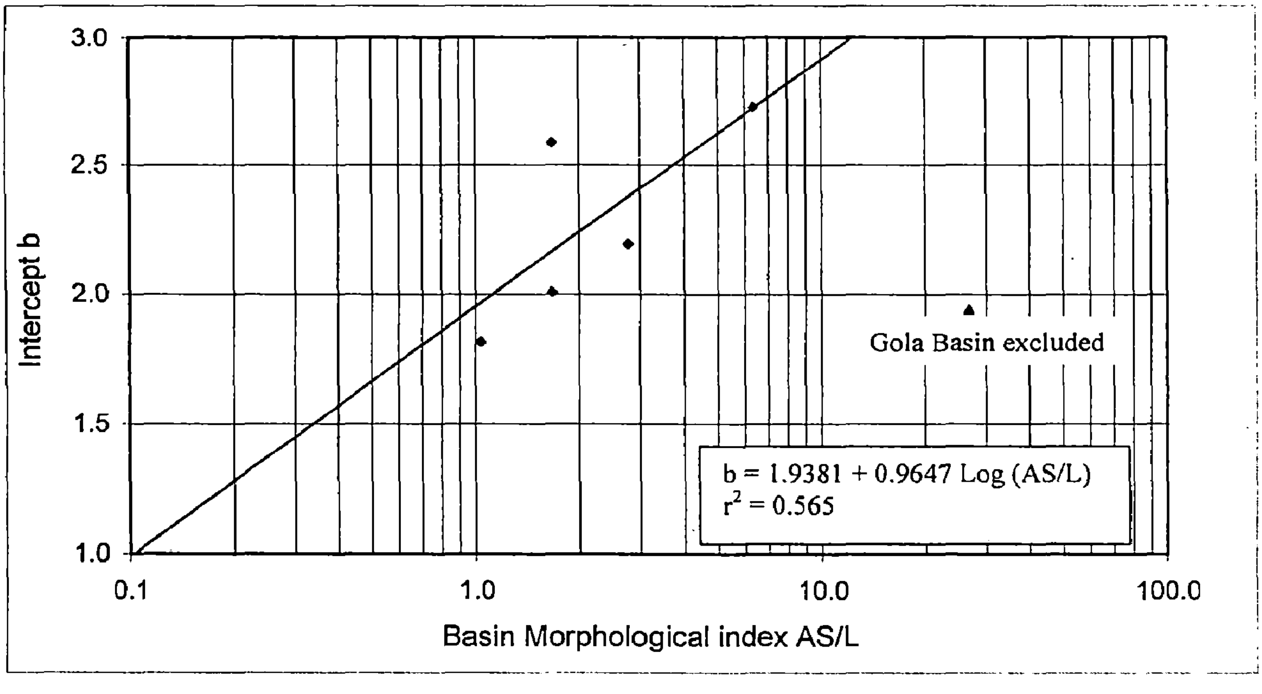

Fig. 4.14. Relationship between the Intercept b and the Basin Morphological Index AS/L

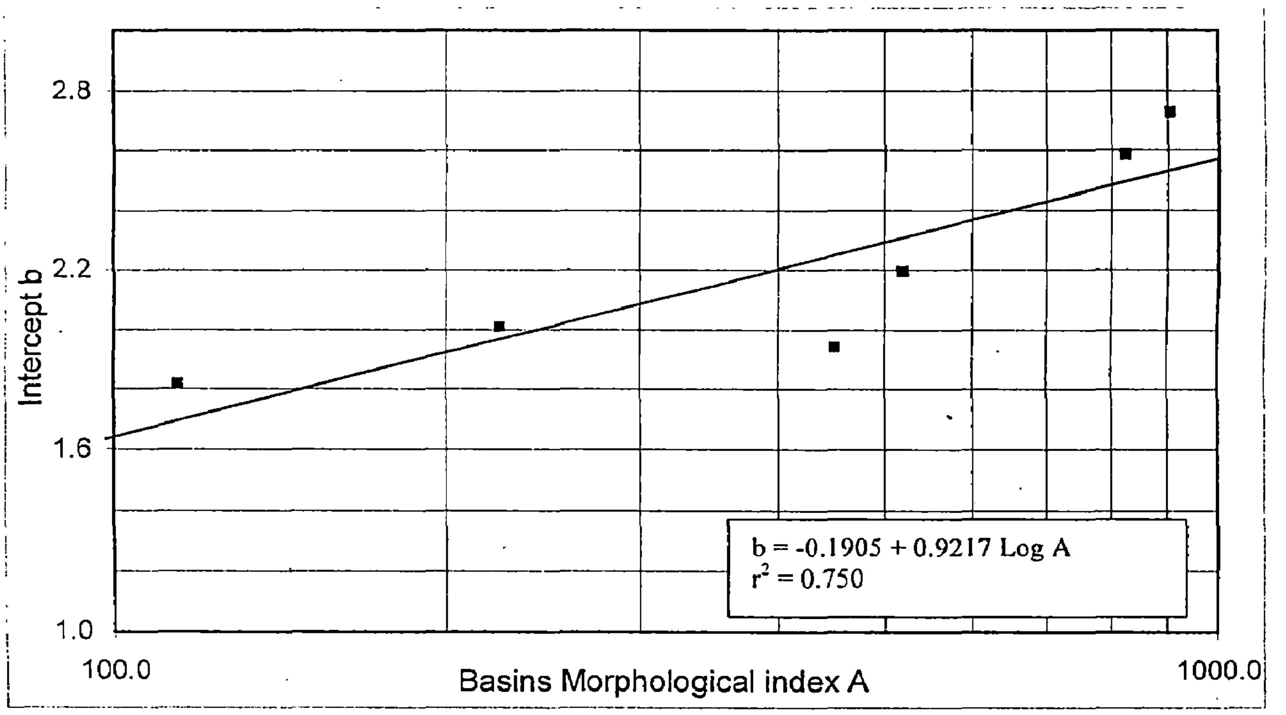

Fig. 4.15. Relationship between the Intercept b and the Basin Morphological Index A

4.7.2. Relationship of m with Catchment Characteristics

m is the slope of original peak discharge distribution. Regression analysis was carried out between m and A/L and between m and AS/L. Correlation was found to be poor. Studies carried out by Mimikou (1983) and Singh and Aminian (1986) have also shown that there is not significant correlation between parameter m and basin characteristics A, L, S. Therefore basins having similar A/L or AS/L may not necessary have same m.

CHAPTER 5

APPLICATION OF PEAK DISCHARGE DISTRIBUTION IN COMBINATION WITH SCS METHOD

5.1. GENERAL

The relation between volume and peak of direct runoff is of fundamental importance in a wide variety of hydrologic process, especially where hydrologic data are scarce. It has immediate application in hydraulic design and water resources planning. Such as for quantifying the hydrologic effect of land use changes needed for systematically planned urban development, designing flood protection and irrigation projects, planning drainage and detention facilities etc.

Rogers (1980) was perhaps the first to have proposed a linear relation (in logarithmic domain) between volumes and peak of runoff and calibrated it on 42 drainage basins ranging in size 5 to 700 square kilometers. Further, Rogers (1982) and Rogers and Zia (1982) applied this relation to basins as large as 23,000 square kilometers. However no attempt was reported on correlating parameters of the relationship with physically measurable basin characteristics. Mimikou (1983) attempted to relate slope (m) and intercept (b) in this relation to characteristics of eight basins in Greece. It was found that correlation between model slope and geographic, climatic, or basin characteristic was not significant. However the intercept was highly correlated (with explained variance equal to 91 percent) with drainage area (A), mainstream length (L) and average bed slope (S). A shortcoming of Mimikou's work was that a small sample of only eight basins in Greece was used.

To extend the work of Rogers (1982) and Mimikou (1983), Singh and Aminian (1986) developed a relation between volume and peak of direct runoff by employing a large number (134) of drainage basins from the United States, Australia, Italy and Greece. Singh and Aminian (1986) have confirmed validity of linear relation between peak and volume of direct runoff in log space.

This chapter explains use of peak discharge distribution in combination with SCS method for estimation of peak discharge due to storm in ungaged catchment.

The procedure consists of adopting a peak discharge distribution for the ungaged catchment by establishing similarity with adjacent catchments for which peak discharge distribution is known. By establishing similarity, slope m of the peak discharge distribution (for ungaged catchment) can be estimated. Intercept b can be found from relationship between b and A, L, S (Figs. 4.13 or 4.14 in chapter 4). Direct runoff is estimated using SCS method and peak discharge is estimated using the peak discharge distribution.

5.2. SCS – CURVE NUMBER METHOD

The Soil Conservation Service (SCS 1972) of U.S. Department of Agricultural developed a method for computing abstractions from storm rainfall. For the storm as a whole, the depth of excess precipitation or direct runoff P_e is always less than or equal to the depth of precipitation; likewise after runoff begins, the additional depth of water retained in the watershed, F_a is less than or equal to some potential maximum retention S. There is some amount of rainfall I_a (initial abstraction before ponding) for which no runoff will occur, so the potential runoff is $P - I_a$. The hypothesis of the SCS method is that the ratios of the two actual to the two potential quantities are equal, that is,

$$\frac{F_a}{S} = \frac{P_e}{P - I_a} \tag{5.1}$$

From the continuity principle

$$P = P_e + I_a + F_a \tag{5.2}$$

where :

I_a is initial abstraction, P_e is rainfall excess, F_a is continuing abstraction

P is total rainfall

Combining Eqs. (5.1) and (5.2) to solve for P_e gives

$$P_e = \frac{(P - I_a)^2}{P - I_a + S} \tag{5.3a}$$

or

$$V = \frac{(P - I_a)^2}{P - I_a + S} \tag{5.3b}$$

where, V is direct runoff volume expressed in equivalent depth over the catchment area. Equation (5.3) is the basic equation for computing the depth of excess rainfall or direct runoff from a storm by the SCS-CN method.

An exact determination of I_a is very difficult. However, for practical purpose, I_a can be related to S. Base on analysis of data from a large number of small watersheds, an empirical relation was developed.

$$I_a = \lambda S \qquad (5.4)$$

On this basis Eq. (5.3b) becomes,

$$V = \frac{(P - \lambda S)^2}{P + (1-\lambda)S}, \qquad \text{for } P > \lambda S \qquad (5.5)$$

and $V = o$ for $P \le \lambda S$

But the value of λ can range between 0 to ∞ (Mishra and Singh, 1999a and 1999b, as mentioned in Mishra and Garg, 1998). In conventional applications λ is usually assumed to be 0.2.

The curve number (CN) and S are related by

$$S = \frac{1000}{CN} - 10, \qquad \textit{where } S \text{ in inch} \qquad (5.6a)$$

or

$$S = \frac{2540}{CN} - 25.4, \qquad \textit{where } S \text{ in centimeters} \qquad (5.6b)$$

The CN value is determined from (a) soil type and land use, and (b) antecedent moisture condition (AMC). The Soil is classified into four soil groups:

Group A : Deep sand, deep loess, aggregated silts

Group B : Shallow loess, sandy loam

Group C : Clay loams, Shallow sandy loam, soils low in organic content, and soils usually high in clay

Group D : Soils that swell significantly when wet, heavy plastic clays and certain saline soils.

The range of antecedent moisture condition for each class is shown in Table 5.1.

Table 5.1. Classification of antecedent moisture condition (AMC) for the SCS-CN method

AMC group	Total 5 – days antecedent rainfall (cm)	
	Dormant season	**Growing season**
I	Less than 1.270	Less than 3.556
II	1.270 to 2.794	3.556 to 5.334
III	Over 2.794	Over 5.334

The values of CN for various land uses on these soil types for normal antecedent moisture condition II (AMC II) are given in Appendix M. For dry conditions (AMC I) or wet conditions (AMC III), equivalent curve numbers can be computed by

$$CN(I) = \frac{4.2CN(II)}{10 - 0.058CN(II)} \tag{5.7}$$

and

$$CN(III) = \frac{23CN(II)}{10 + 0.13CN(II)} \tag{5.8}$$

For watershed made up of several soil types and land uses, a composite CN can be calculated.

5.3. RAIFALL VOLUME – DIRECT RUNOFF PEAK RELATION

Equations (2.14) and (5.3b) can be meaningfully applied to a rainfall volume – direct runoff peak relation. By replacing V from Eq. (5.3b) to Eq. (2.14), an explicit relation for q_p directly in term of P, I_a, and S is given as:

$$Log\left(\frac{q_p}{V^2}\right) = b + m\ Log\ \frac{(P - I_a)^2}{P - I_a + S} \tag{5.9}$$

or

$$q_p = 10^b \left[\frac{(P - I_a)^2}{P - I_a + S}\right]^{m+2} \tag{5.10}$$

If $I_a = 0.2S$, the expression becomes,

$$q_p = 10^b \left[\frac{(P-0.2S)^2}{P+0.8S} \right]^{m+2} \qquad (5.11)$$

The value of S can be computed by Eq. (5.6a) in inch terms or by Eq. (5.6b) in centimeters term. Slope of the peak discharge distribution (m) for ungaged catchment can be assumed to be same as that for hydrologically similar gauged catchment for which peak discharge distribution is available. Intercept b of the peak discharge distribution can be found from known catchment characteristics A, L, S of the ungaged catchment as relation between b and A, L, S is already established for similar gauged catchments.

5.4. APPLICATION STUDY

The reliability of the method is demonstrated by its application study to the 3f sub-zone up to bridge no. 807 of Lower Godavari basin. The catchment area is 823.62 km^2. The main soils types of the watershed are red loamy sand and black soil; 50% of watershed area is covered by forest, 25% by cultivated land, and the remaining 25% by barren land. Soil moisture condition in AMC II and AMC III.

Four storm event which were not used in the development of peak discharge distribution have been used for the purpose of this application study. Table 5.2 shows the observed rainfall and discharge data for these events (Mishra, 1998 and Tyagi, 1995).

Table 5.2 Observed Rainfall and Discharge Data for 3f Sub-Zone of Lower Godavari Basin

Event No	**Rainfall* (cm)**	**Observed discharge* (m^3/s)**	**Base flow* (m^3/s)**	**Direct runoff volume* (cm)**
1	2.723	600	6.64	1.595
2	4.085	440	15.60	1.570
3	2.664	255	0.00	0.653
4	4.351	490	23.00	1.226

**source : Tyagi, 1995 and Mishra, 1998*

While using SCS method for computation of direct runoff (or excess rainfall) it is necessary to know antecedent moisture condition of basin. Unfortunately AMC conditions for these events are not known. However observed rainfall and discharge data (Table 5.2) shows that event no. 1 produced relatively higher peak discharge even though storm rainfall is relatively less compared to event no. 2, event no.3 and event no. 4.

It is assumed that wet catchment condition (AMC III) was prevailing when event no. 1 occurred and AMC II was prevailing when events no. 2, 3, 4 occurred. While applying SCS method, I_a has been assumed to be equal to 0.1S, 0.2S, 0.1S, and 0.2S for storm events 1, 2, 3 and 4 respectively in consideration of wetness of the basin just preceding the events.

a. Determine of composite curve number (CN)

Soils of the lower Godavari basin belong to Group C.

Table 5.3. Calculation of Curve Number in AMC II

No	Land use	% calculation of soil group	CN	Weighted Product of CN
1	Forest	50 % x 100%	77	38.50
2	Cultivated	25% x 100%	86	21.50
3	Barren	25% x 100%	88	22.00
Weighted CN (II)				82.00

Wet catchment condition (AMC III) was prevailing when event no. 1 occurred, so the curve number becomes:

$$CN(III) = \frac{23CN(II)}{(10 + 0.13CN(II)}$$

$$= \frac{23 \, x \, 82}{(10 + (0.13x82))} = 91.29$$

b. Calculation of retention of water by drainage basin (S) in cm

From Eq. (5.6b) we get the value of S as follow,

$$S = \frac{2540}{91.29} - 25.4$$

$$= 2.423 \text{ cm}$$

c. Calculation peak of direct runoff (V).

Initial abstraction $I_a = 0.1S$,

$= 0.10 \times 2.423$

$= 0.2423$ cm

Total depth of rainfall = 2.723 cm

From the Eq. (5.3b) we get,

$$V = \frac{(2.723 - 0.2423)^2}{2.723 - 0.2423 + 2.423}$$

$= 1.255$ cm

Apply equation (2.14); relation between volume and peak of direct runoff for 3f sub-zone of lower Godavari Basin

$$\text{Log}\,(q_p/V^2) = -0.7665 - 1.1923 \log V$$

or

$$q_p/V^2 = 10^{-0.7665}\,(V^{-1.1923})$$

or

$$q_p = 10^{-0.7665}\,(V^{0.8077})$$

$= 0.206$ cm/hr

Similarly q_p were estimated for other storm events (event 2, event 3 and event 4) and the results are shown in table 5.4.

Table 5.4. Observed and Predicted Peak Discharge for 3f Sub-zone of Lower Godavari Basin.

Event No	Rainfall depth* (cm)	Direct Runoff (cm)	Observed Peak* (cm/hr)	Predicted Peak (cm/hr)	Relative error (%)
1	2.723	1.255	0.259	0.206	20.46
2	4.085	1.032	0.185	0.176	4.86
3	2.664	0.580	0.111	0.110	1.00
4	4.351	1.190	0.204	0.197	3.43

* *source: Mishra, 1998 and Tyagi, 1995*

5.5. DISCUSSION

From the results of application study for four events in lower Godavari basin (Table 5.4) it is seen that all events have significantly small error (except event no. 1) between predicted and observed flood peak. Observed peak discharge for storm event no. 1 does not appear to be reliable. Based on above analysis, it is concluded that Eq. (2.14) can be used to estimate peak of direct runoff in combination with SCS-curve number method in ungaged catchments.

It is important to note that accurate determination of initial abstraction (I_a) based on soil moisture condition (AMC) of the catchments is necessary. It is recommended that further study should be carried out for different basins and by considering a large number of events for validation under different catchment wetness conditions prevailing before storm events.

CHAPTER 6

OTHER POTENTIAL APPLICATIONS OF PEAK DISCHARGE DISTRIBUTION

6.1. GENERAL

In chapter 2 several applications of the peak discharge distributions have been indicated. Following applications have been studied in previous chapter:

i) Identification of hydrological linearity / nonlinearity and quantification of nonlinearity of the basin on the basis of slope of peak discharge distribution (chapter 4)

ii) Estimation of peak flood of linear or nonlinear basins for which peak discharge distributions are established using observed runoff and peak discharge data (chapter 4)

iii) Estimation of peak discharge per unit runoff (b) from basin characteristics A, L, S (chapter 4)

iv) Estimation of peak flood in ungaged catchment using assumed peak discharge distribution in combination with SCS method (chapter 5)

In this chapter some other applications of peak discharge distributions are explored. These are i) identification of drainage basin similarity, ii) derivation of triangular unit hydrograph, iii) estimation of flooding potential and iv) estimation of sediment yield. The purpose is to explore possibility of using peak discharge distribution in data scarce basins.

6.2. DRAINAGE BASIN SIMILARITY

When Equation (2.14) is developed and plotted for several basins having observed flood hydrographs, families of straight lines may be identified such that each family has more or less parallel lines but with differing intercepts. It is reasoned that each family represents similar drainage basins. This implies that parameter m can also be considered as a measure of basin similarity, and that for a family of hydrologic similar basins, only one value of parameter m would suffice. This value of parameter m can be obtained for the basins having rainfall-runoff records and transferred to those members of the family not having such records. This concept can be gainfully employed to assume a peak discharge distribution for ungaged catchment belonging to a family of similar basins having known peak discharge distribution. For the example study can be shown as follow,

Example in determines of drainage basin similarity, we consider six basins in the present study as mentioned in chapter 3. The data for six basins are given below,

Tabel 6.1. Direct Runoff Volume (V) and Peak of Direct Runoff per V^2 for Six Drainage Basins

Gola Sub-basin, Nainital		Umar sub-basin, upper Narmada basin		Temur sub-basin, upper Narmada basin		Teriya sub-basin, upper Narmada basin		Kolar sub-basin, upper Narnada basin		3f sub-zone, lower Godawari	
V (cm)	$\frac{q_p}{V^2}$ $\frac{cm/hr}{cm^2}$	V (cm)	$\frac{q_p}{V^2}$ $\frac{cm/hr}{cm^2}$	V (cm)	$\frac{q_p}{V^2}$ $\frac{cm/hr}{cm^2}$	V (cm)	$\frac{q_p}{V^2}$ $\frac{cm/hr}{cm^2}$	V (cm)	$\frac{q_p}{V^2}$ $\frac{cm/hr}{cm^2}$	V (cm)	$\frac{q_p}{V^2}$ $\frac{cm/hr}{cm^2}$
8.26	0.005	2.56	0.048	4.75	0.010	4.32	0.027	24.01	0.003	1.05	0.140
7.6	0.006	3.91	0.018	1.08	0.110	1.49	0.171	7.44	0.015	1.16	0.180
6.32	0.006	1.91	0.059	0.67	0.210	1.79	0.081	5.22	0.019	1.26	0.130
1.99	0.027	1.00	0.169	1.32	0.090	2.72	0.048	4.47	0.028	0.61	0.380
1.80	0.029	1.13	0.165	0.81	0.130	2.38	0.066	6.54	0.018	0.88	0.220
4.90	0.018	8.15	0.013	0.26	0.600	2.92	0.048	1.72	0.119	0.85	0.150
1.71	0.067	14.73	0.004	0.31	0.430	2.07	0.065			4.29	0.030
5.13	0.016					2.04	0.081			0.59	0.410
17.62	0.002					4.40	0.050			1.52	0.090
2.60	0.033					21.21	0.004			0.64	0.230
						2.10	0.090			4.98	0.030

From direct runoff volume (V) in cm and peak discharge per unit area per direct runoff square (q_p/V^2) data in Table 6.1 for each drainage basin we get graph in logarithm term as follows,

From the graph (Figure 6.1) it can be seen that some of drainage basins in the present study have nearly parallel regression best-fit lines. These drainage basins have hydrologic similarity. The drainage basin hydrologic similarity exists between Gola sub-basin located in Western Himalayan region, Umar and Kolar sub-basins located in Upper Narmada. Also Teriya sub-basin has hydrologic similarity with Temur sub-basin. Both basins lie in upper Narmada basin.

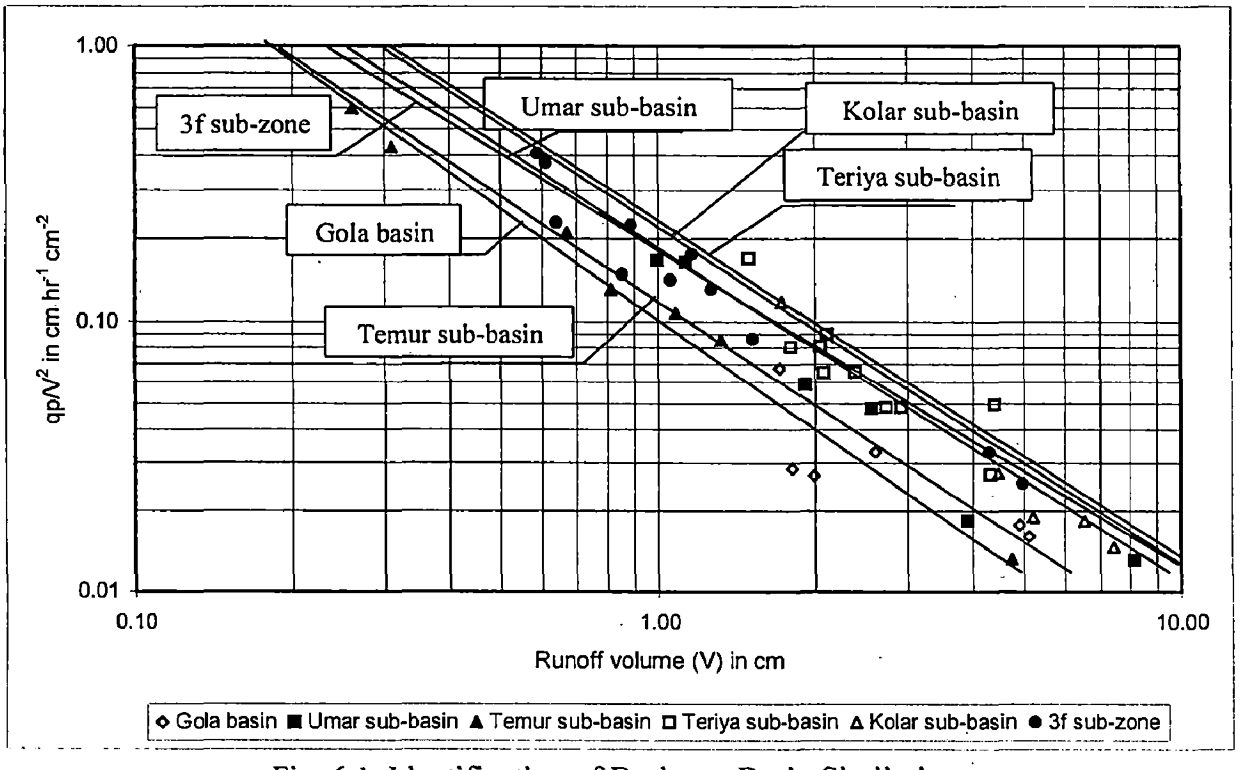

Fig. 6.1. Identification of Drainage Basin Similarity

Teriya and Temur sub-basins also lie in upper Narmada basin but do not have hydrologic similarity with Kolar and Umar sub-basins. It should be noted that, peak discharge shows strong correlation with time to peak also for highly nonlinear basins (Gola and Umar) as shown in Table 4.2 in chapter 4. Therefore similarity should be established with regard to regression constant related with time to peak also, for application of peak discharge, time to peak, and volume relationship (PDTVR) in nonlinear ungaged catchments.

It is also important to note that 3f sub-zone of lower Godavari basin does not have hydrologic similarity with other basins. Out of the six basins, only 3f sub-zone can be considered to be nearly linear hydrologic basin.

6.3. DERIVATION OF TRIANGULAR UNIT HYDROGRAPH

Before deriving unit hydrograph for a basin it would be necessary to establish its hydrologic linearity by methods discussed in chapter 2.

D-hour unit hydrograph is represented by a triangle as proposed by Soil Conservation Service (1972). Knowing q_p from equation (2.14) will suffice to specify the unit hydrograph (UH). The duration D and volume V of effective rainfall are assumed to be known. The duration of recession from the time to peak is taken as approximately 1.67 times the duration of rise, T_p. The illustration of triangle UH is given in Fig. 6.2.

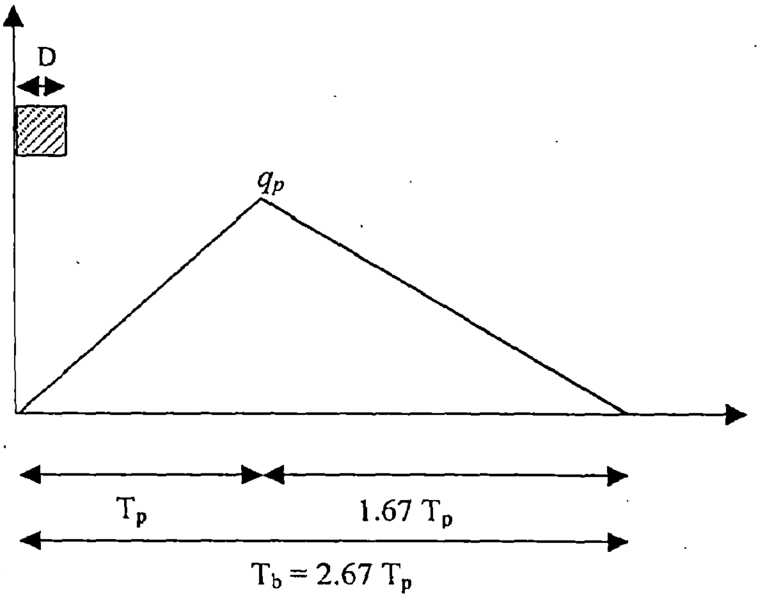

Fig. 6.2. Triangle Unit Hydrograph

Equation (2.13), log q_p = b + m log V, since V = 1 unit of direct runoff by definition of unit hydrograph, therefore log q_p = b.

From Fig. 6.2 it can be seen that,

$$T_p = \frac{2}{2.67\, q_p} \tag{6.1}$$

Furthermore, T_p is related to the duration of effective rainfall D through time concentration tc (Soil Conservation Service, 1972) as,

$$tc + D = 1.7\ Tp \tag{6.2}$$

and

$$\frac{D}{2} + 0.6\, t_c = T_p \tag{6.3}$$

Equations (6.2) and (6.3) yield

$$D = 0.133\, t_c \tag{6.4}$$

and

$$T_p = \frac{D}{2} + 0.6\, t_c = \frac{2}{3}\, t_c \tag{6.5}$$

Thus, a D-hour UH can be completely specified. Note that Eq. (2.13) directly specifies q_p of the UH as Log q_p = b with V=1.

For the application in derivation of unit hydrograph, consider 3f sub-zone of lower Godavari basin with catchment characteristics as follow,

- Catchment Area (A) : 823.62 sq. km
- Length of main course (L) : 61.08 km
- River bed slope (S) : 0.124 %

Volume of direct runoff (V) = 1 cm for unit hydrograph, , so q_p directly specifies peak of the unit hydrograph (UH) and duration of rainfall (D) = 1 hr.

From Eq. (2.14) for example basin (3f sub-zone of lower Godavari) peak of direct runoff is calculated as follows,

$$Log\ \frac{q_p}{V^2} = -0.7665 - 1.1923\ Log\quad V \tag{6.6}$$

or

$$\frac{q_p}{V^2} = 10^{-0.7665}\ (V^{-1.1923}) \tag{6.7}$$

or

$$q_p = 10^{-0.7665}\ (V^{0.8077}) \tag{6.8}$$

Substituting $V = 1$ into Eq. (6.8),

$$q_p = 10^{-0.7665}\ (1^{0.8077})$$

or

$$q_p = 0.1712 \text{ cm/hr}$$

and

$$Q_p = \frac{(q_p \ x \ A)}{0.36} \qquad (6.9)$$

$$Q_p = \frac{(0.1712 \ x \ 823.62)}{0.36}$$

$$= 391.677 \text{ m}^3/\text{s}$$

From Eq. (6.1) we get,

$$T_p = \frac{2}{2.67 \ q_p}$$

$$T_p = \frac{2}{2.67 \ x \ 0.1712}$$

$$= 4.375 \text{ hrs}$$

say 4.5 hrs

Apply Eq. (6.2) and we get,

$$t_c + D = 1.7 \ Tp$$

$$t_c = 1.7 \ T_p - D$$

$$= (1.7 \text{ x } 4.5) - 1$$

$$= 6.65 \text{ hrs}$$

Check duration of rainfall (D) with Eq. (6.4),

$$D = 0.133 \ t_c$$

$$= 0.133 \text{ x } 6.65 \text{ hr}$$

$$= 0.89 \text{ hr}$$

say 1 hr OK

Time base of unit Hydrograph (T_b) = $2.67 \ x \ T_p$

$= 2.67 \text{ x } 4.5$

$= 12.015$ hrs

say 12 hrs.

Table 6.2 Ordinate of 1 – hr Triangular Unit Hydrograph

Time (hrs)	**Ordinate (m^3/s)**
0	0
1	78.33
2	156.67
3	235.00
4	313.34
5	391.68
6	335.72
7	279.77
8	223.82
9	167.86
10	111.91
11	55.95
12	0

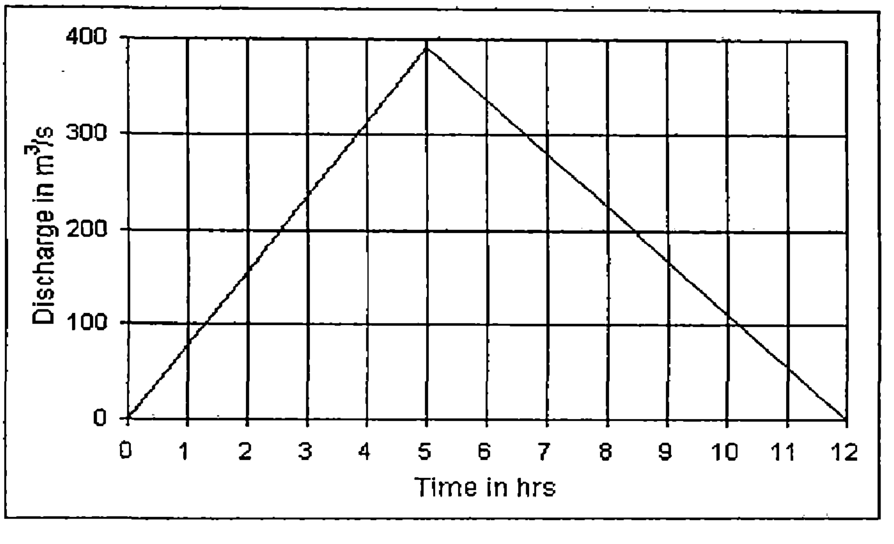

Fig. 6.3 1-Hour Triangle Unit Hydrograph

6.3. FLOODING POTENTIAL

Equation (2.14) can be combined with the SCS hypothesis of representing the flood hydrograph by a triangle in exactly the same way as the UH derived above. This allows determination of not only the flood peak, but also the flood duration and flood volume.

It is important to note that in the development and application of unit hydrograph for estimation of peak flood and flooding potential, it is implied that the drainage basin is hydrologically linear. Therefore it would be necessary to first check the linearity of the basin.

For the application of flooding potential, consider 3f sub-zone of lower Godavari basin. And for this example we take event no.2 in chapter 5.

From chapter 5 we get following characteristics of this basin,

- Catchment Area (A) : 823.62 sq. km
- Total Depth of Rainfall (P) : 4.085 cm
- Volume for direct runoff (V) : 1.032 cm

From Eq. (2.14) for 3f sub-zone of lower Godavari basin we calculate peak of direct runoff as follows,

$$Log\ \frac{q_p}{V^2} = -0.7665 - 1.1923\ Log\quad V$$

or

$$\frac{q_p}{V^2} = 10^{-0.7665}\ (V^{-1.1923})$$

or

$$q_p = 10^{-0.7665}\ (V^{0.8077})$$

for runoff volume (V) = 1.032 cm,

$$q_p = 10^{-0.7665}\ (1.032^{0.8077})$$

$$q_p = 0.17561 \text{ cm/hr}$$

and

$$Q_p = \frac{(q_p \ x \ A)}{0.36}$$

$$Q_p = \frac{(0.17561 \ x \ 823.62)}{0.36}$$

$$= 401.77 \text{ m}^3/\text{s}$$

From Eq. (6.1) we get,

$$T_p = \frac{2}{2.67 \ q_p}$$

$$T_p = \frac{2}{2.67 \ x \ 0.17561}$$

$$= 4.270 \text{ hrs}$$

say 4.5 hrs

Apply Eq. (6.2) and we get,

$$t_c + D = 1.7 \ Tp$$

$$t_c = 1.7 \ T_p - D$$

$$= (1.7 \text{ x } 4.5) - 1$$

$$= 6.65 \text{ hrs}$$

Time base of unit Hydrograph (T_b) = $2.67 \ x \ T_p$

= 2.67 x 4.5

= 12.015 hrs

say 12 hrs

From table 6.3 and figure 6.4 we determine:

- Flood Peak : 401.77 m^3/s
- Flood Duration : 12 hrs
- Flood Volume : 2410.62 x 60 x 60 m^3
  = 8678232 m^3

Table 6.3. Ordinate of Direct Runoff Hydrograph for 3f Sub-Zone of Lower Godavari Basin for Event no. 2

Time (hrs)	Ordinate (m^3/s)
0	0
1	80.35
2	160.71
3	241.06
4	321.41
5	401.77
6	344.37
7	286.98
8	229.58
9	172.18
10	114.79
11	57.39
12	0

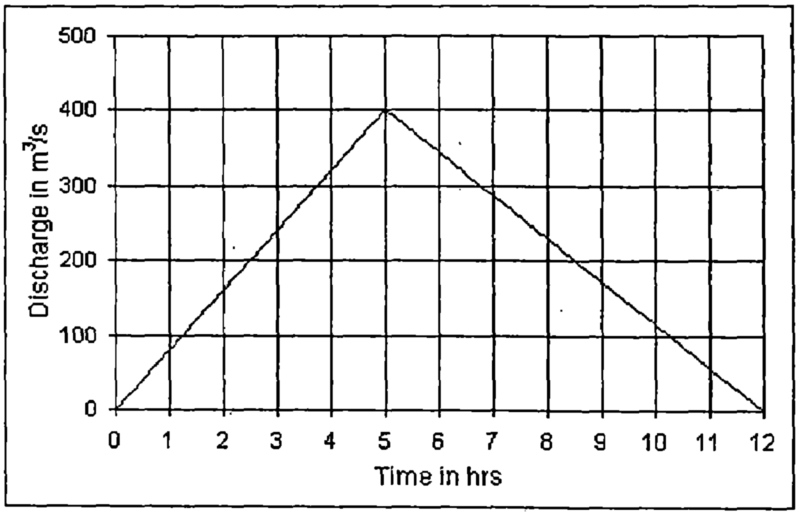

Fig. 6.4 Direct Runoff Hydrograph for 3f Sub-Zone of Lower Godavari Basin (Event no. 2)

6.4. ESTIMATION OF SEDIMENT YIELD

Singh and Chen (1982) related sediment yield Y in tons per unit area to volume of direct runoff (V) in cm for a storm event on a watershed as

$$Y = aV^c \quad (6.10)$$

or

$$Log\ Y = Log\ a + c\ Log\ V \quad (6.11)$$

where Log a is the intercept and c the slope of the line. Y was taken in metric tons and V in cm; both are based on a unit area. Here by knowing V from equation (5.3b), we can determine Y from Eq. (6.11) if a and c are known for a given watershed. In equation (6.11) a varies from watershed to watershed and c varies from one family of similar watershed (in the context of sediment production) to another. Singh and Chen (1982) were successful in determining 'a' from basin characteristics. Analogous to the parameter 'm' in Eq. (2.14), the correlation between parameter c and basin characteristics was weak.

Parameter a and c were both correlated with watershed characteristics, including watershed area A_d (km^2), main channel length L (km), main channel slope S (m/km), mean basin elevation E (m), erodibility factor K, shape factor R, forest area A_f and lake storage area S_a (%).

Geomorphic data from 21 watersheds in USA were analysis to develop following empirical relation

Based on the study done by Singh and Chen (1982), the regression equations obtained is:

$$\text{Log}\ a = -2.5628 + 0.0006A_d - 0.0075L - 0.1543S - 0.00055Af + 0.0029E + 0.0004S_a + 8.6996K - 0.0033R \quad (6.12)$$

with correlation coefficient of 0.9393 and standard error of estimate of 0.2977. Eq. (6.12) explained 88.23% of the variation in log a. With reduced geomorphic characteristics the equation is:

$$\text{Log}\ a = -2.337 - 0.003L - 0.1415S - 0.0004Af + 0.004E + 7.576K \quad (6.13)$$

Having a correlation coefficient of 0.9107 and a standard error of estimate of 0.3205.

Similarly,

$$\text{Log } c = 0.2163 - 0.00002A_d + 0.00053L + 0.00019S + 0.00008A_f$$
$$- 0.00045E - 0.0238A_a + 0.0923K - 0.000003R \quad (6.14)$$

with correlation coefficient of 0.7422 and standard error of estimate of 0.0695. Eq. (6.13) explained 55.08% of the variation in log c. With reduced geomorphic characteristic the equation is:

$$\text{Log } c = 0.2625 - 0.00001L + 0.00007A_f - 0.0004E$$
$$- 0.0251S - 0.0000007R \quad (6.15)$$

having a correlation coefficient of 0.7384 and a standard error of estimate of 0.06205.

Utilizing the straight-line relationship with parameters estimated in the above manner, sediment yield can be predicted. However, observed data on sediment yield and volume of storm runoff for several events and several watersheds are needed to develop empirical relation for finding *a* and c. As such data was not available for the watershed in present study these relationships could not be developed. In the following example, empirical relations based on study by Singh and Chen (1982) has been used only to illustrate the potential application of the method.

For this application we consider 3f sub-zone of lower Godavari Basin in event no. 2 with volume of direct runoff is 1.032 cm.

Other characteristics of this basin are as follows,

- Catchment Area (A) : 823.62 sq. km
- Main channel length (L) : 61.08 km
- Main channel slope (S) : 1.24 m/km
- Mean basin elevation (E) : 300 m
- Erodibility factor (K), assumed : 0.32
- Shape factor (R) : 0.221
- Forest area (Af) : 411.81 sq. km

Furthermore we apply Eqs. (6.9) and (6.11), (reduced geomorphic characteristic) as follows,

$$\text{Log } a = -2.337 - (0.003 \times 61.08) - (0.1415 \times 1.24) - (0.0004 \times 411.81) + (0.004 \times 300) + (7.576 \times 0.32) = 0.7639$$

$$\text{Log } c = 0.2625 - (0.00001 \times 61.08) + (0.00007 \times 411.81) - (0.0004 \times 300) - (0.0251 \times 1.24) - (0.0000007 \times 0.221) = 0.1396$$

or $c = 1.3791$

Apply Eq. (6.7), so the sediment yield Y can be expressed as

$$\text{Log } Y = 0.7639 + 1.3791 \text{ Log } V$$

for direct runoff volume (V) = 1.032 cm,

$$\text{Log } Y = 0.7639 + (1.3791 \times \text{Log } 1.032) = 0.7828$$

or $Y = 6.0645$ metric tons per unit area.

CHAPTER 7

SUMMARY AND CONCLUSIONS

7.1. SUMMARY OF STUDY

Two uses of term "nonlinearity" have appeared in recent literature. The first definition of nonlinearity is with respect to dynamic property such as rainfall-runoff process of a catchment and the second definition is with respect to statistical dependence of a catchment property such as mean annual flood on catchment area.

In the present study, basin linearity is defined as the condition that exists on a basin when runoff volumes are directly proportional to rainfall volumes. Hydrologic nonlinearity exists when runoff volumes are not directly proportional to rainfall volumes. Serious error in hydrologic design can occur by over estimating or under estimating design discharge when a drainage basin is assumed to be linear while in fact it is nonlinear. The wide spread and long lasting usage of the unit hydrograph model (Sherman, 1932), which is based on the assumption of hydrologic linearity, makes more intensive the need for developing criteria for checking the applicability of the unit hydrograph model and, thus, the linearity and nonlinearity in the rainfall-runoff process. One of the most important attempts on this subject has been made by Rogers, (1980, 1982); Rogers and Zia, (1982) in the U.S.A., where a family of three peak discharge distributions has been developed and studied in detail. Predicted peak discharges by the unit hydrograph method for nonlinear basins were found to be seriously overestimated. Mimikou (1983) used the family of the peak discharge distributions for checking nonlinearity of drainage basins in Greece.

Singh and Aminian (1986) proposed linear two parameter relation in log space between direct runoff volume per unit area and peak discharge of direct runoff per unit area.

Based on review of work done by Rogers (1980), Rogers and Zia (1982), Mimikou (1983), and Singh and Aminian (1986), following important points emerge:

i) Mathematical linearity should not be confused with hydrologic linearity; even though runoff data from a hydrologically linear or nonlinear drainage basin may also be mathematically linear. Hydrologic linearity or nonlinearity can be determined by linear relationship between peak discharge and volume of runoff in log space.

ii) Three peak discharge distributions have been attempted,

a) Original Peak Discharge Distribution (OPDD)

$\text{Log } Q_p = \text{b} + \text{m} \log V$

b) First Order Standardized Peak Discharge Distribution (FSPDD)

$\text{Log } (Q_p/V) = \text{b} + (\text{m} - 1) \log V$

c) Second Order Standardized Peak Discharge Distribution (SSPDD)

$\text{Log } (Q_p/V^2) = \text{b} + (\text{m} - 2) \log V$

For idealized hydrologically linear drainage basin, m = 1 and UH or IUH theory can be applied. Smaller the value of m or (m – 1) or (m – 2) more nonlinear is the drainage basin hydrologically.

iii) The volume (cm) – peak discharge (m^3/s) relationships rely on measured stream discharge data

iv) It is not necessary to separate base flow which requires use of an uncertain procedure

v) The data required for developing such relationships are several recorded flood hydrographs, drainage area, length, and slope values for each of the drainage basin considered in analysis.

vi) The number of hydrographs used is less important than the quality of hydrographs. Simple hydrographs are desirable though complex hydrographs can also be used and simplified. It is useful to choose hydrographs that cover the range from small area to large area if possible.

vii) Only Original Peak Discharge Distribution is necessary and quite sufficient by itself for checking drainage basin linearity (Mimikou, 1983)

viii) Standardized peak discharge distribution can be meaningfully applied to a variety of hydrologic analysis.

In this study, peak discharge distributions have been developed and applied for identification of linearity and nonlinearity of six drainage basins in India i.e. Umar, Temur, Teriya and Kolar sub-basins of upper Narmada basin, 3f sub-zone of lower Godavari basin and Gola sub-basin in western Himalayan region. The drainage basins ranged in size from 114 sq. km to 904 sq. km and in total 53 flood hydrographs were analyzed.

Peak discharge distribution, peak discharge distribution per unit area and also relationship between peak discharge, time to peak and volume (PDTVR) have been analyzed in the present study for six drainage basins in India. Following applications have been investigated:

- Prediction of probable maximum flood for linear and nonlinear drainage basins and comparison with unit hydrograph method.
- Estimation of peak discharges per unit runoff (log inverse intercept b) from basin characteristics A, L, S.
- Application of peak discharge distribution in combination with SCS-CN method to determine peak discharge in ungaged catchment.
- Application of peak discharge distribution for identification of drainage basins similarity, derivation of unit hydrograph, estimation of flooding potential and estimation of sediment yields.

Percentage variations explained by various relations are given below,

	Peak discharge distributions			PDTVR	Peak disch. dist per unit area	
	OPDD	FSPDD	SSPDD		OPDD	SSPDD
Average r^2	0.879	0.606	0.961	0.943*	0.879	0.961

***multiple coefficient of determination with time to peak and runoff volume as independent variable.**

OPDD per unit area and SSPDD per unit area may not be necessary for identification of nonlinearity. SSPDD has higher r^2 compared to OPDD but intercept b remains same. There is a consistent and systematic improvement in r^2 values and in data scatter for the SSPDD over the OPDD. Improvement is significant in case of nonlinear basins such as Gola basin ($m = 0.644$, $r^2 = 0.696$ for OPDD and $r^2 = 0.910$ for SSPDD). The deterioration in r^2 and in data error from OPDD to FSPDD is significant for linear basins such as sub-zone 3f of lower Godavari ($m=0.800$, $r^2 = 0.894$ for OPDD and $r^2 = 0.343$ for FSPDD). This corroborates the finding of Mimikou (1983).

7.2. CONCLUSIONS

The conclusions drawn from this study are as given below,

i) There is no geographic, climatic or morphological influence on slopes of various relations, as these relations have been tested for basins in different regions of the world including basins in India in the present study. Percentage variations explained by various relations are given below,

	Peak discharge distributions			PDTVR	Peak disch. dist per unit area	
	OPDD	FSPDD	SSPDD		OPDD	SSPDD
Average r^2	0.879	0.606	0.961	0.943*	0.879	0.961

**multiple coefficient of determination with time to peak and runoff volume as independent variable.*

OPDD per unit area and SSPDD per unit area may not be necessary for identification of nonlinearity. SSPDD has higher r^2 compared to OPDD but intercept b remains same. There is a consistent and systematic improvement in r^2 values and in data scatter for the SSPDD over the OPDD. Improvement is significant in case of nonlinear basins such as Gola basin ($m = 0.644$, $r^2 = 0.696$ for OPDD and $r^2 = 0.910$ for SSPDD). The deterioration in r^2 and in data error from OPDD to FSPDD is significant for linear basins such as sub-zone 3f of lower Godavari ($m=0.800$, $r^2 = 0.894$ for OPDD and $r^2 = 0.343$ for FSPDD). This corroborates the finding of Mimikou (1983).

ii) Original peak discharge distribution constitutes a reliable method for identification of nonlinear/linear dependence of storm response on the magnitude of rainfall input and for predicting peak discharges; its slope being 1.0 for linear and less than 1.0 for nonlinear basins.

iii) Multiple linear regression analysis between peak discharge, time to peak and volume in log space (PDTVR) shows that partial correlation between log Q_p and log T_p is stronger for highly nonlinear basins and weak for other basins. Flood hydrographs of several other basins need to be analyzed to arrive at a definite conclusion with regard to comparative performance of OPDD and PDTVR in prediction of peak discharge in highly nonlinear basins.

iv) The inherent assumption in OPDD is that T_p is constant and ratio of T_b/T_p is constant. The regression analysis in log space shows stronger correlation of T_p

with T_b and weak correlation with Q_p and V. Therefore T_p can be assumed to be independent of Q_p and V as required in OPDD. For 3f sub-zone of Godavari basin the average ratio T_b/T_p is 2.74, which is close to 2.67 assumed by Mockus in triangular unit hydrograph (UH) applicable to linear basins. The OPDD analysis also shows that the 3f sub-zone is nearly linear and thus validates the assumption that T_b/T_p is 2.67 for linear basins. However, for nonlinear basins ratio T_b/T_p could be different than 2.67 and not necessarily constant.

v) Application of linear UH theory for estimation of peak flood in nonlinear basins is not valid. Prediction error increases with increasing nonlinearity of the basin.

vi) Peak discharge distribution per unit area can be used for estimation of peak discharge in combination with SCS-CN method, identification of drainage basin similarity, estimation of flooding potential, estimation of sediment yields and derivation of unit hydrograph as illustrated in this study. The application of peak discharge distribution per unit area in combination with SCS-CN method is successfully validated for estimation of flood in 3f sub-zone of lower Godavari basin in India. Peak discharge, time and volume relationship (PDTVR) may be more useful for identification of basin similarity when the basins are highly nonlinear.

vii) Estimation of b from catchment characteristics is useful in development of peak discharge distribution for ungaged catchment. Incorrect results are obtained if intercept b is explained only by the logarithm of drainage basin area A. Basins with similar area but in different regions such as Himalayan region and central Indian region will produce significantly different magnitude of peak discharge per unit runoff volume (log inverse of b) as shown in the study. Therefore separate relationship between b and A/L or between b and AS/L need to be evolved for different geomorphological regions.

viii) Further study for several basins in India is recommended particularly for establishing usefulness of peak discharge distributions in ungaged catchments and for analyzing the influence of pattern of rainfall on the peak discharge distributions.

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APPENDICES

APPENDIX - A.1

Runoff Events of Temur Sub-basin

1. Event 1

No Serie	Time (Hours)	Discharge (cumec)	Direct Runoff Volume
1	0.00	0.00	0.00
2	1.00	1.00	1800.00
3	2.00	2.90	7020.00
4	3.00	5.90	15840.00
5	4.00	11.76	31788.00
6	5.00	19.85	56898.00
7	6.00	28.66	87354.00
8	7.00	45.60	133704.00
9	8.00	67.65	203850.00
10	9.00	175.00	436770.00
11	10.00	181.23	641214.00
12	11.00	172.06	635922.00
13	12.00	150.00	579708.00
14	13.00	132.40	508320.00
15	14.00	110.29	436842.00
16	15.00	88.23	357336.00
17	16.00	67.65	280584.00
18	17.00	58.82	227646.00
19	18.00	48.53	193230.00
20	19.00	38.97	157500.00
21	20.00	30.90	125766.00
22	21.00	26.50	103320.00
23	22.00	20.60	84780.00
24	23.00	17.65	68850.00
25	24.00	14.70	58230.00
26	25.00	11.76	47628.00
27	26.00	11.40	41688.00
28	27.00	9.50	37620.00
29	28.00	7.35	30330.00
30	29.00	5.50	23130.00
31	30.00	0.00	9900.00
Total Runoff Volume (m3)			5624568.00
Total Runoff Depth in cm			1.08

2. Event 2

No Serie	Time (Hours)	Discharge (cumec)	Direct Runoff Volume
1	0.00	0.00	0.00
2	1.00	1.70	3060.00
3	2.00	5.56	13068.00
4	3.00	12.22	32004.00
5	4.00	27.40	71316.00
6	5.00	122.22	269316.00
7	6.00	135.92	464652.00
8	7.00	130.50	479556.00
9	8.00	95.56	406908.00
10	9.00	80.50	316908.00
11	10.00	68.50	268200.00
12	11.00	60.00	231300.00
13	12.00	50.00	198000.00
14	13.00	43.00	167400.00
15	14.00	36.50	143100.00
16	15.00	28.50	117000.00
17	16.00	22.10	91080.00
18	17.00	15.50	67680.00
19	18.00	11.10	47880.00
20	19.00	8.70	35640.00
21	20.00	5.56	25668.00
22	21.00	5.00	19008.00
23	22.00	3.33	14994.00
24	23.00	2.22	9990.00
25	24.00	0.00	3996.00
Total Runoff Volume (m3)			3497724.00
Total Runoff Depth in cm			0.67

3. Event 3

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	13.00	23400.00
3	2.00	25.50	69300.00
4	3.00	57.00	148500.00
5	4.00	126.50	330300.00
6	5.00	168.00	530100.00
7	6.00	186.50	638100.00
8	7.00	206.50	707400.00
9	8.00	214.93	758574.00
10	9.00	200.00	746874.00
11	10.00	164.40	655920.00
12	11.00	124.40	519840.00
13	12.00	93.30	391860.00
14	13.00	66.60	287820.00
15	14.00	48.50	207180.00
16	15.00	42.10	163080.00
17	16.00	31.00	131580.00
18	17.00	27.50	105300.00
19	18.00	20.00	85500.00
20	19.00	18.00	68400.00
21	20.00	17.50	63900.00
22	21.00	15.00	58500.00
23	22.00	11.10	46980.00
24	23.00	10.00	37980.00
25	24.00	7.40	31320.00
26	25.00	6.50	25020.00
27	26.00	4.44	19692.00
28	27.00	2.22	11988.00
29	28.00	0.00	3996.00
Total Runoff Volume (m3)			6868404.00
Total Runoff Depth in cm			1.32

4. Event 4

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	1.00	1800.00
3	2.00	2.90	7020.00
4	3.00	11.00	25020.00
5	4.00	14.70	46260.00
6	5.00	22.06	66168.00
7	6.00	29.41	92646.00
8	7.00	45.59	135000.00
9	8.00	105.90	272682.00
10	9.00	124.59	414882.00
11	10.00	123.50	446562.00
12	11.00	117.65	434070.00
13	12.00	98.50	389070.00
14	13.00	76.47	314946.00
15	14.00	70.60	264726.00
16	15.00	61.03	236934.00
17	16.00	50.00	199854.00
18	17.00	42.60	166680.00
19	18.00	34.50	138780.00
20	19.00	27.94	112392.00
21	20.00	24.27	93978.00
22	21.00	22.00	83286.00
23	22.00	17.60	71280.00
24	23.00	14.70	58140.00
25	24.00	9.50	43560.00
26	25.00	8.60	32580.00
27	26.00	4.40	23400.00
28	27.00	2.90	13140.00
29	28.00	1.50	7920.00
30	29.00	0.00	2700.00
Total Runoff Volume (m3)			4195476.00
Total Runoff Depth in cm			0.81

APPENDIX - A.1 Continued

5. Event 5

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	1.00	1800.00
3	2.00	2.00	5400.00
4	3.00	5.00	12600.00
5	4.00	9.00	25200.00
6	5.00	20.00	52200.00
7	6.00	30.00	90000.00
8	7.00	48.00	140400.00
9	8.00	58.16	191088.00
10	9.00	53.00	200088.00
11	10.00	27.00	144000.00
12	11.00	26.00	95400.00
13	12.00	21.00	84600.00
14	13.00	16.00	66600.00
15	14.00	11.50	49500.00
16	15.00	10.50	39600.00
17	16.00	8.50	34200.00
18	17.00	8.00	29700.00
19	18.00	7.00	27000.00
20	19.00	6.00	23400.00
21	20.00	5.00	19800.00
22	21.00	3.00	14400.00
23	22.00	2.10	9180.00
24	23.00	1.20	5940.00
25	24.00	1.00	3960.00
26	25.00	0.00	1800.00
Total Runoff Volume (m3)			1367856.00
Total Runoff Depth in cm			0.26

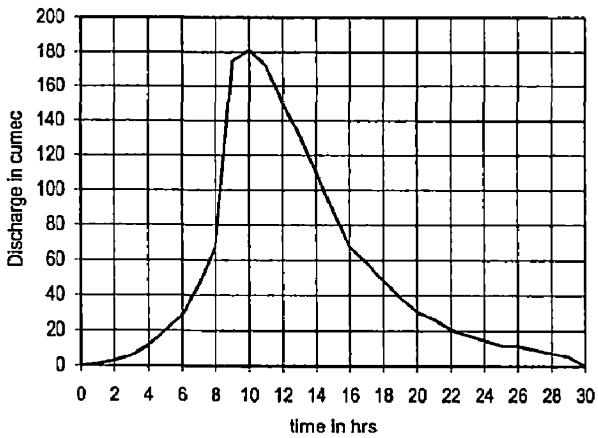
6. Event 6

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	1.40	2520.00
3	2.00	2.80	7560.00
4	3.00	15.40	32760.00
5	4.00	32.90	86940.00
6	5.00	56.00	160020.00
7	6.00	59.50	207900.00
8	7.00	56.00	207900.00
9	8.00	50.00	190800.00
10	9.00	35.70	154260.00
11	10.00	30.80	119700.00
12	11.00	27.50	104940.00
13	12.00	21.90	88920.00
14	13.00	17.00	70020.00
15	14.00	12.90	53820.00
16	15.00	9.30	39960.00
17	16.00	6.80	28980.00
18	17.00	4.40	20160.00
19	18.00	2.20	11880.00
20	19.00	0.55	4950.00
21	20.00	0.00	990.00
Total Runoff Volume (m3)			1594980.00
Total Runoff Depth in cm			0.31

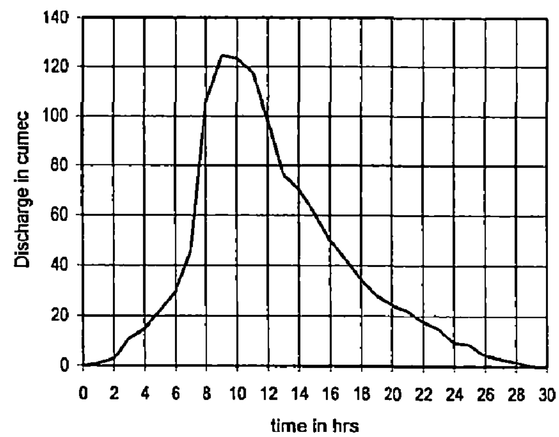
APPENDIX - A.2

Direct Runoff Hydrograph of Temur Sub-basin

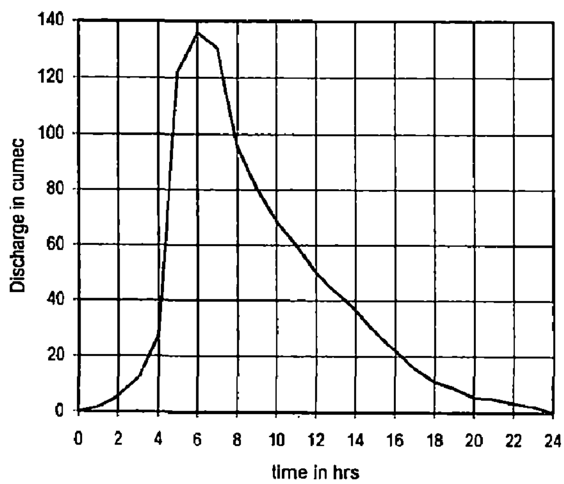
Direct Runoff Hydrograph event 1



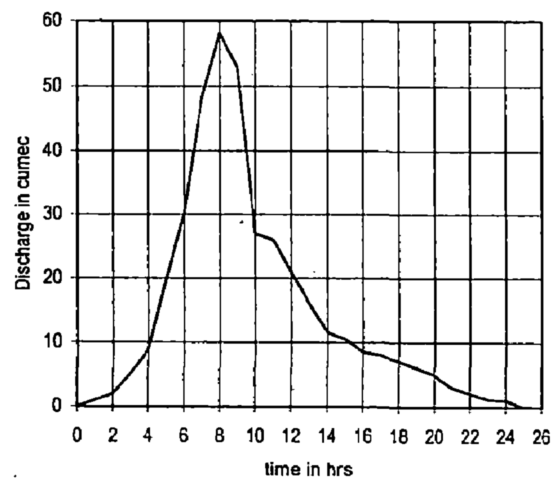
Direct Runoff Hydrograph event 4



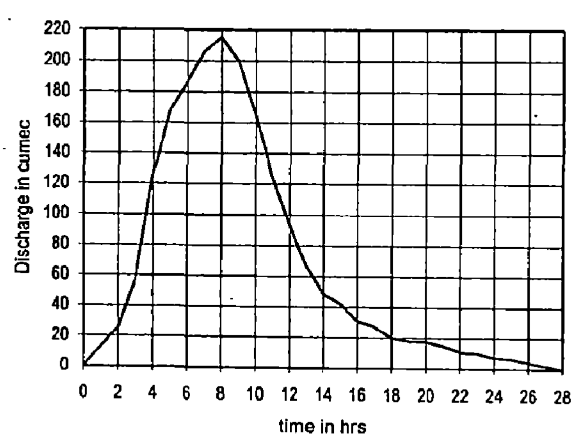
Direct Runoff Hydrograph event 2



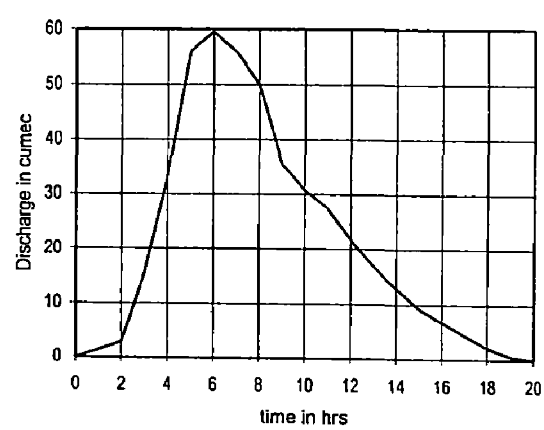
Direct Runoff Hydrograph event 5



Direct Runoff Hydrograph event 3



Direct Runoff Hydrograph event 6



APPENDIX - B.1

Runoff Events of Teriya Sub-basin

1. Event 1, (31.07.1967)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	22.00	39600.00
3	2.00	37.00	106200.00
4	3.00	79.00	208800.00
5	4.00	151.00	414000.00
6	5.00	161.00	561600.00
7	6.00	144.00	549000.00
8	7.00	126.00	486000.00
9	8.00	106.00	417600.00
10	9.00	86.00	345600.00
11	10.00	72.00	284400.00
12	11.00	61.00	239400.00
13	12.00	52.00	203400.00
14	13.00	46.00	176400.00
15	14.00	39.00	153000.00
16	15.00	34.00	131400.00
17	16.00	30.00	115200.00
18	17.00	26.00	100800.00
19	18.00	22.00	86400.00
20	19.00	18.00	72000.00
21	20.00	15.00	59400.00
22	21.00	13.00	50400.00
23	22.00	10.00	41400.00
24	23.00	8.00	32400.00
25	24.00	6.00	25200.00
26	25.00	4.00	18000.00
27	26.00	2.00	10800.00
28	27.00	0.00	3600.00
Total Runoff Volume (m^3)			4932000.00
Total Runoff Depth in cm			4.32

2. Event 2, (13.08.1970)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	1.00	1800.00
3	2.00	3.00	7200.00
4	3.00	15.00	32400.00
5	4.00	78.00	167400.00
6	5.00	120.00	356400.00
7	6.00	82.00	363600.00
8	7.00	54.00	244800.00
9	8.00	36.00	162000.00
10	9.00	26.00	111600.00
11	10.00	21.00	84600.00
12	11.00	15.00	64800.00
13	12.00	11.00	46800.00
14	13.00	7.00	32400.00
15	14.00	3.00	18000.00
16	15.00	0.00	5400.00
Total Runoff Volume (m^3)			1699200.00
Total Runoff Depth in cm			1.49

3. Event 3, (16.08.1970)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	4.00	7200.00
3	2.00	39.50	78300.00
4	3.00	58.00	175500.00
5	4.00	70.50	231300.00
6	5.00	82.50	275400.00
7	6.00	77.50	288000.00
8	7.00	50.00	229500.00
9	8.00	40.00	162000.00
10	9.00	33.50	132300.00
11	10.00	28.50	111600.00
12	11.00	23.00	92700.00
13	12.00	18.50	74700.00
14	13.00	14.00	58500.00
15	14.00	10.00	43200.00
16	15.00	7.50	31500.00
17	16.00	5.50	23400.00
18	17.00	3.50	16200.00
19	18.00	2.00	9900.00
20	19.00	0.50	4500.00
21	20.00	0.00	900.00
Total Runoff Volume (m^3)			2046600.00
Total Runoff Depth in cm			1.79

4. Event 4, (28.08.1970)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	10.50	18900.00
3	2.00	13.50	43200.00
4	3.00	31.00	80100.00
5	4.00	69.00	180000.00
6	5.00	102.00	307800.00
7	6.00	114.00	388800.00
8	7.00	100.50	386100.00
9	8.00	81.50	327600.00
10	9.00	65.00	263700.00
11	10.00	55.00	216000.00
12	11.00	42.00	174600.00
13	12.00	35.50	139500.00
14	13.00	31.50	120600.00
15	14.00	27.00	105300.00
16	15.00	23.00	90000.00
17	16.00	19.50	76500.00
18	17.00	16.00	63900.00
19	18.00	12.50	51300.00
20	19.00	8.00	36900.00
21	20.00	5.00	23400.00
22	21.00	2.00	12600.00
23	22.00	0.00	3600.00
Total Runoff Volume (m^3)			3110400.00
Total Runoff Depth in cm			2.72

APPENDIX - B.1 Continued

5. Event 5, (02.08.1971)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	3.00	5400.00
3	2.00	5.00	14400.00
4	3.00	19.00	43200.00
5	4.00	39.00	104400.00
6	5.00	62.00	181800.00
7	6.00	89.00	271800.00
8	7.00	118.00	372600.00
9	8.00	99.00	390600.00
10	9.00	77.00	316800.00
11	10.00	55.00	237600.00
12	11.00	46.00	181800.00
13	12.00	35.00	145800.00
14	13.00	26.00	109800.00
15	14.00	21.00	84600.00
16	15.00	18.00	70200.00
17	16.00	13.00	55800.00
18	17.00	10.00	41400.00
19	18.00	8.00	32400.00
20	19.00	6.00	25200.00
21	20.00	4.00	18000.00
22	21.00	2.00	10800.00
23	22.00	1.00	5400.00
24	23.00	0.00	1800.00
Total Runoff Volume (m^3)			2721600.00
Total Runoff Depth in cm			2.38

6. Event 6, (31.08.1971)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	19.00	34200.00
3	2.00	38.00	102600.00
4	3.00	65.00	185400.00
5	4.00	86.00	271800.00
6	5.00	109.00	351000.00
7	6.00	131.00	432000.00
8	7.00	120.00	451800.00
9	8.00	88.00	374400.00
10	9.00	65.00	275400.00
11	10.00	50.00	207000.00
12	11.00	40.00	162000.00
13	12.00	30.00	126000.00
14	13.00	24.00	97200.00
15	14.00	19.00	77400.00
16	15.00	16.00	63000.00
17	16.00	11.00	48600.00
18	17.00	8.00	34200.00
19	18.00	5.00	23400.00
20	19.00	2.00	12600.00
21	20.00	0.00	3600.00
Total Runoff Volume (m^3)			3333600.00
Total Runoff Depth in cm			2.92

7. Event 7, (17.10.1971)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	2.00	3600.00
3	2.00	5.00	12600.00
4	3.00	13.00	32400.00
5	4.00	35.00	86400.00
6	5.00	53.00	158400.00
7	6.00	78.00	235800.00
8	7.00	89.00	300600.00
9	8.00	79.00	302400.00
10	9.00	63.00	255600.00
11	10.00	50.00	203400.00
12	11.00	40.00	162000.00
13	12.00	32.00	129600.00
14	13.00	26.00	104400.00
15	14.00	22.00	86400.00
16	15.00	19.00	73800.00
17	16.00	15.00	61200.00
18	17.00	12.00	48600.00
19	18.00	9.00	37800.00
20	19.00	7.00	28800.00
21	20.00	5.00	21600.00
22	21.00	3.00	14400.00
23	22.00	1.00	7200.00
24	23.00	0.00	1800.00
Total Runoff Volume (m^3)			2368800.00
Total Runoff Depth in cm			2.07

8. Event 8, (28.08.1972)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0	0
2	1.00	17	30600.00
3	2.00	24	73800.00
4	3.00	38	111600.00
5	4.00	54	165600.00
6	5.00	80	241200.00
7	6.00	108	338400.00
8	7.00	115	401400.00
9	8.00	108.00	401400.00
10	9.00	99.00	372600.00
11	10.00	78.00	318600.00
12	11.00	59.00	246600.00
13	12.00	47.00	190800.00
14	13.00	41.00	158400.00
15	14.00	35.00	136800.00
16	15.00	30.00	117000.00
17	16.00	25.00	99000.00
18	17.00	21.00	82800.00
19	18.00	17.00	68400.00
20	19.00	13.00	54000.00
21	20.00	9.00	39600.00
22	21.00	6.00	27000.00
23	22.00	3.00	16200.00
24	23.00	0.00	5400.00
Total Runoff Volume (m^3)			2334600.00
Total Runoff Depth in cm			2.04

APPENDIX - B.1 Continued

9. Event 9, (21.07.1973)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	0.00	0.00
3	2.00	8.00	14400.00
4	3.00	87.00	171000.00
5	4.00	225.00	561600.00
6	5.00	305.00	954000.00
7	6.00	227.00	957600.00
8	7.00	155.00	687600.00
9	8.00	110.00	477000.00
10	9.00	70.00	324000.00
11	10.00	57.00	228600.00
12	11.00	45.00	183600.00
13	12.00	37.00	147600.00
14	13.00	22.00	106200.00
15	14.00	18.00	72000.00
16	15.00	11.00	52200.00
17	16.00	9.00	36000.00
18	17.00	7.00	28800.00
19	18.00	2.00	16200.00
20	19.00	0.00	3600.00
Total Runoff Volume (m^3)			5022000.00
Total Runoff Depth in cm			4.40

10. Event 10, (30.08.1973)

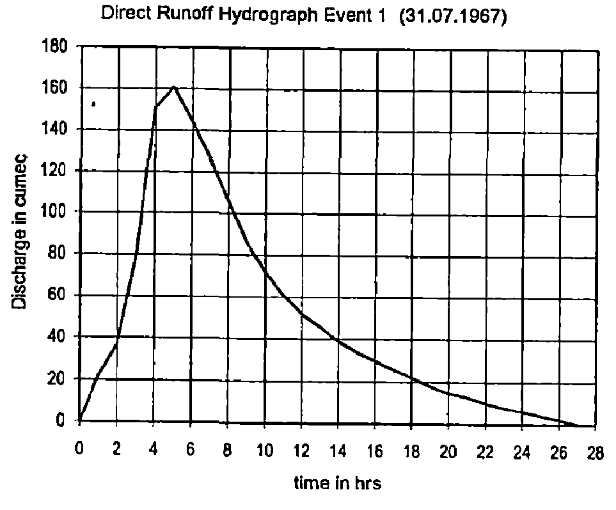
No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	6.00	10800.00
3	2.00	15.10	37980.00
4	3.00	24.24	70812.00
5	4.00	303.00	589032.00
6	5.00	472.70	1396260.00
7	6.00	557.60	1854540.00
8	7.00	575.75	2040030.00
9	8.00	557.60	2040030.00
10	9.00	524.20	1947240.00
11	10.00	490.90	1827180.00
12	11.00	463.64	1718172.00
13	12.00	424.24	1598184.00
14	13.00	381.82	1450908.00
15	14.00	333.33	1287270.00
16	15.00	303.00	1145394.00
17	16.00	266.67	1025406.00
18	17.00	242.42	916362.00
19	18.00	200.00	796356.00
20	19.00	175.75	676350.00
21	20.00	145.45	578160.00
22	21.00	103.00	447210.00
23	22.00	66.67	305406.00
24	23.00	39.40	190926.00
25	24.00	18.20	103680.00
26	25.00	12.00	54360.00
27	26.00	11.50	42300.00
28	27.00	6.60	32580.00
29	28.00	6.00	22680.00
30	29.00	3.00	16200.00
31	30.00	0.00	5400.00
Total Runoff Volume (m^3)			24227208.00
Total Runoff Depth in cm			21.21

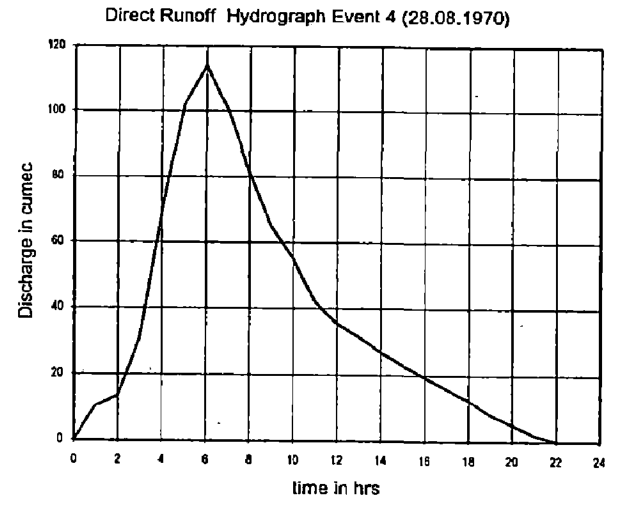
11. Event 11, (05.09.1973)

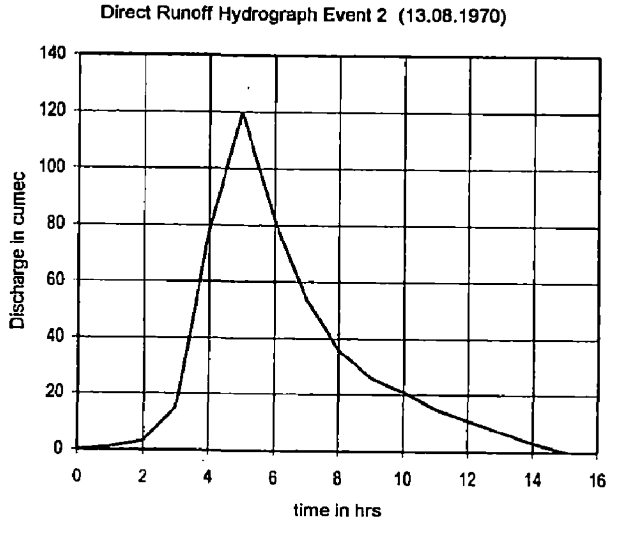
No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	18.00	32400.00
3	2.00	92.10	198180.00
4	3.00	127.00	394380.00
5	4.00	121.30	446940.00
6	5.00	104.50	406440.00
7	6.00	73.00	319500.00
8	7.00	50.00	221400.00
9	8.00	34.80	152640.00
10	9.00	23.60	105120.00
11	10.00	15.70	70740.00
12	11.00	7.30	41400.00
13	12.00	0.00	13140.00
Total Runoff Volume (m^3)			2402280.00
Total Runoff Depth in cm			2.10

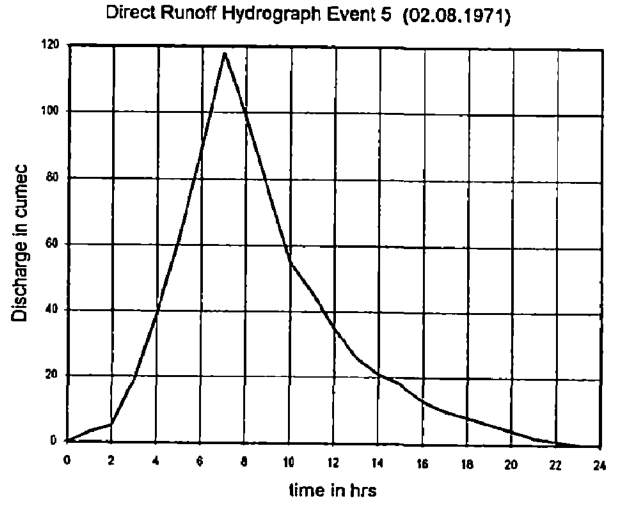
APPENDIX - B.2

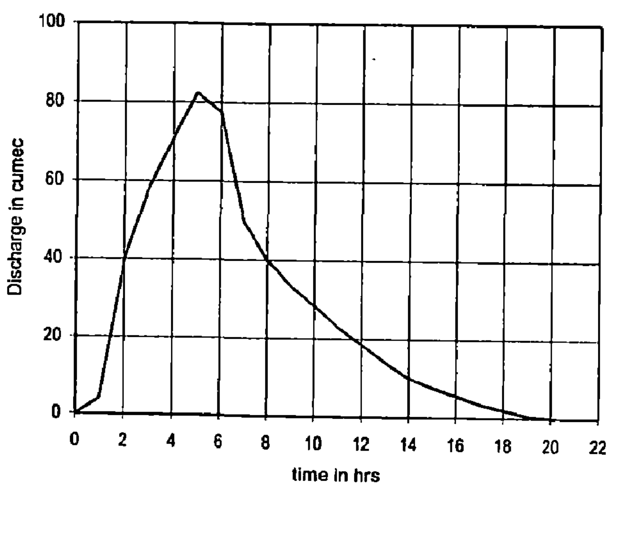
Direct Runoff Hydrograph of Teriya Sub-basin

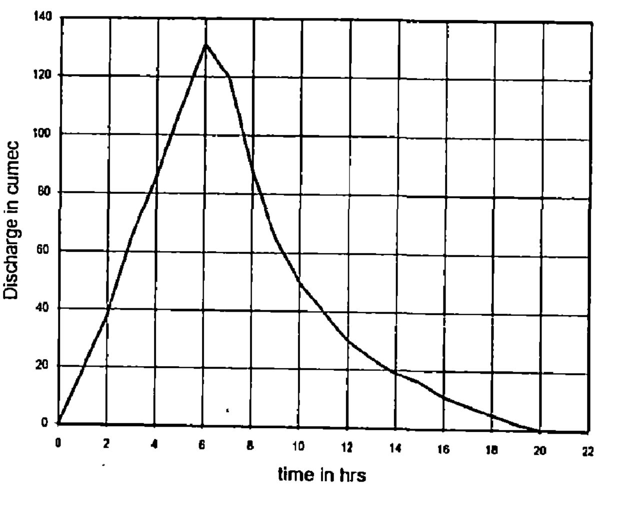




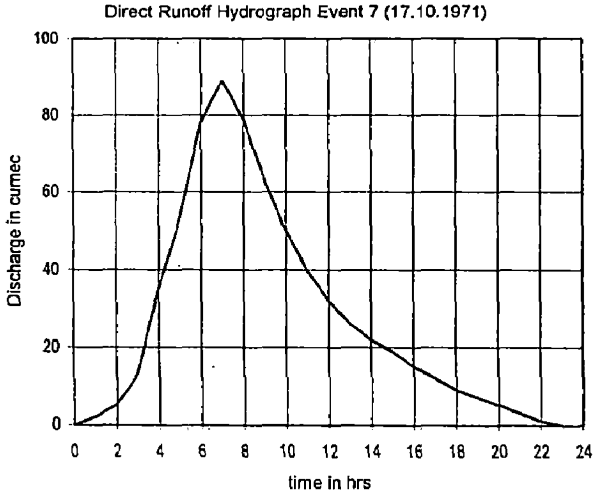


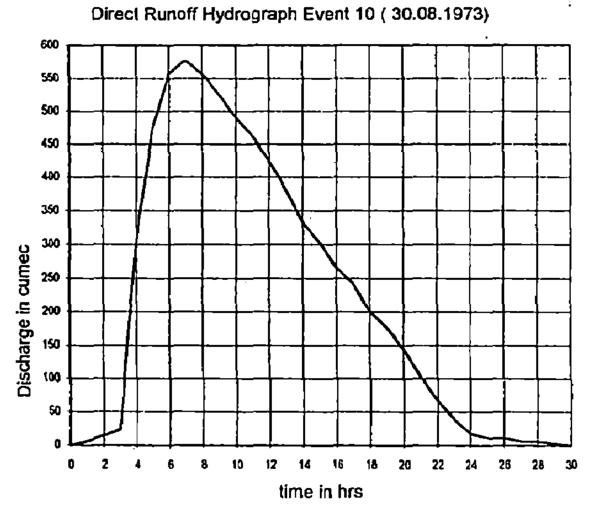


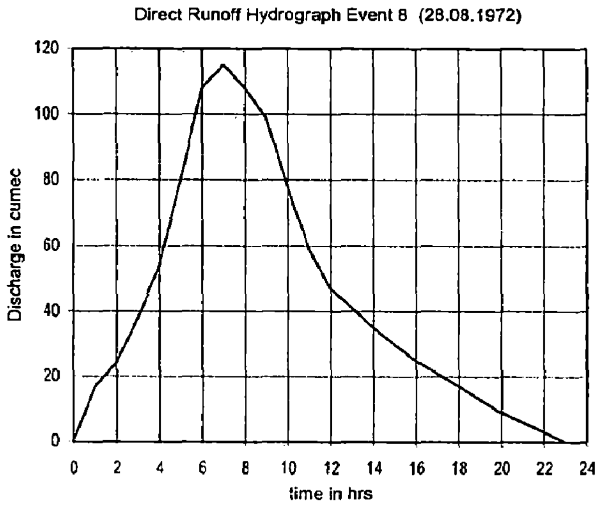


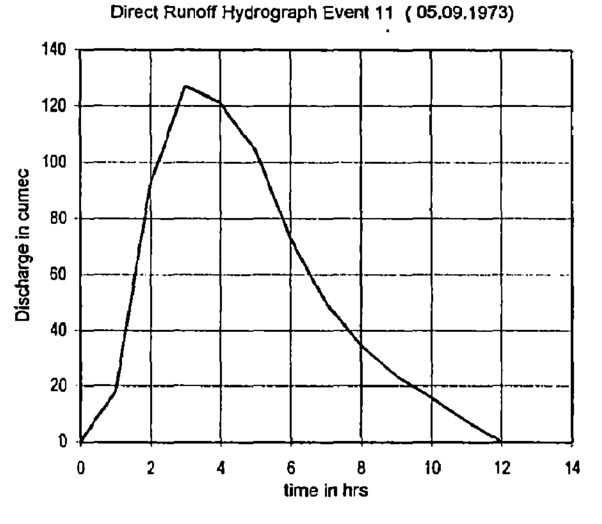


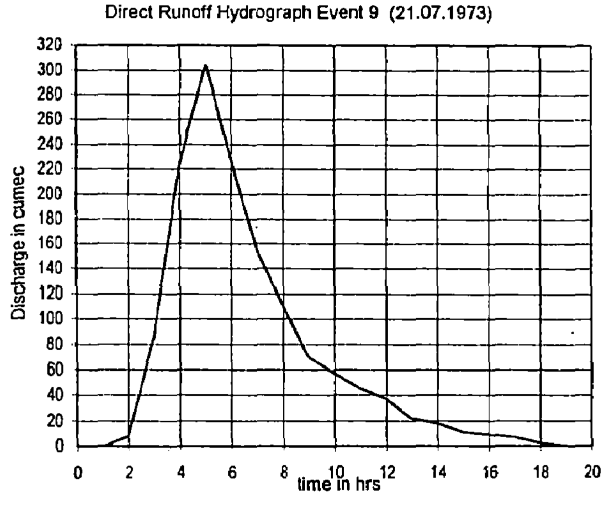
APPENDIX - B.2 Continued











APPENDIX - C.1

Runoff Events of Umar Sub-basin

1. Event 1

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	3.00	5400.00
3	2.00	6.00	16200.00
4	3.00	9.00	27000.00
5	4.00	15.00	43200.00
6	5.00	16.40	56520.00
7	6.00	19.40	64440.00
8	7.00	22.40	75240.00
9	8.00	30.50	95220.00
10	9.00	45.50	136800.00
11	10.00	52.20	175860.00
12	11.00	62.70	206820.00
13	12.00	73.80	245700.00
14	13.00	79.80	276480.00
15	14.00	96.30	316980.00
16	15.00	97.00	347940.00
17	16.00	97.00	349200.00
18	17.00	100.00	354600.00
19	18.00	103.00	365400.00
20	19.00	110.40	384120.00
21	20.00	126.10	425700.00
22	21.00	131.30	463320.00
23	22.00	156.70	518400.00
24	23.00	175.40	597780.00
25	24.00	158.20	600480.00
26	25.00	140.30	537300.00
27	26.00	103.00	437940.00
28	27.00	82.00	333000.00
29	28.00	71.60	276480.00
30	29.00	61.20	239040.00
31	30.00	61.20	220320.00
32	31.00	41.80	185400.00
33	32.00	40.00	147240.00
34	33.00	27.00	120600.00
35	34.00	15.00	75600.00
36	35.00	0.00	27000.00
Total Runoff Volume (m^3)			8748720.00
Total Runoff Depth in cm			3.91

2. Event 2

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	1.70	3060.00
3	2.00	3.50	9360.00
4	3.00	5.50	16200.00
5	4.00	11.70	30960.00
6	5.00	20.50	57960.00
7	6.00	50.00	126900.00
8	7.00	72.20	219960.00
9	8.00	100.00	309960.00
10	9.00	124.40	403920.00
11	10.00	130.50	458820.00
12	11.00	133.30	474840.00
13	12.00	117.80	451980.00
14	13.00	86.70	368100.00
15	14.00	60.00	264060.00
16	15.00	38.90	178020.00
17	16.00	28.90	122040.00
18	17.00	23.30	93960.00
19	18.00	18.90	75960.00
20	19.00	16.10	63000.00
21	20.00	14.20	54540.00
22	21.00	12.10	47340.00
23	22.00	12.10	43560.00
24	23.00	10.00	39780.00
25	24.00	8.90	34020.00
26	25.00	8.90	32040.00
27	26.00	8.90	32040.00
28	27.00	8.80	31860.00
29	28.00	8.80	31680.00
30	29.00	8.30	30780.00
31	30.00	7.80	28980.00
32	31.00	7.70	27900.00
33	32.00	6.40	25380.00
34	33.00	5.50	21420.00
35	34.00	4.40	17820.00
36	35.00	3.30	13860.00
37	36.00	2.80	10980.00
38	37.00	1.70	8100.00
39	38.00	0.00	3060.00
Total Runoff Volume (m^3)			4264200.00
Total Runoff Depth in cm			1.91

3. Event 3

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	2.20	3960.00
3	2.00	4.50	12060.00
4	3.00	5.50	18000.00
5	4.00	7.80	23940.00
6	5.00	17.80	46080.00
7	6.00	23.30	73980.00
8	7.00	36.70	108000.00
9	8.00	46.70	150120.00
10	9.00	90.00	246060.00
11	10.00	103.90	349020.00
12	11.00	100.00	367020.00
13	12.00	76.70	318060.00
14	13.00	38.90	208080.00
15	14.00	27.80	120060.00
16	15.00	18.30	82980.00
17	16.00	13.10	56520.00
18	17.00	5.50	33480.00
19	18.00	0.00	9900.00
Total Runoff Volume (m^3)			2227320.00
Total Runoff Depth in cm			1.00

4. Event 4

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	18.90	34020.00
3	2.00	32.20	91980.00
4	3.00	50.00	147960.00
5	4.00	130.30	324540.00
6	5.00	123.90	457560.00
7	6.00	102.20	406980.00
8	7.00	81.10	329940.00
9	8.00	64.40	261900.00
10	9.00	42.20	191880.00
11	10.00	28.90	127980.00
12	11.00	17.80	84060.00
13	12.00	7.80	46080.00
14	13.00	0.00	14040.00
Total Runoff Volume (m^3)			2518920.00
Total Runoff Depth in cm			1.13

APPENDIX - C.1 Continued

5. Event 5

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	19.50	35100.00
3	2.00	39.00	105300.00
4	3.00	100.00	250200.00
5	4.00	300.00	720000.00
6	5.00	390.00	1242000.00
7	6.00	439.00	1492200.00
8	7.00	500.00	1690200.00
9	8.00	541.50	1874700.00
10	9.00	531.70	1931760.00
11	10.00	463.00	1790460.00
12	11.00	390.20	1535760.00
13	12.00	312.20	1264320.00
14	13.00	253.70	1018620.00
15	14.00	195.10	807840.00
16	15.00	141.50	605880.00
17	16.00	97.60	430380.00
18	17.00	73.00	307080.00
19	18.00	48.80	219240.00
20	19.00	46.30	171180.00
21	20.00	36.60	149220.00
22	21.00	33.20	125640.00
23	22.00	24.40	103680.00
24	23.00	22.00	83520.00
25	24.00	17.00	70200.00
26	25.00	14.60	56880.00
27	26.00	13.60	50760.00
28	27.00	10.20	42840.00
29	28.00	5.00	27360.00
30	29.00	4.40	16920.00
31	30.00	0.00	7920.00
Total Runoff Volume (m^3)			18227160.00
Total Runoff Depth in cm			8.15

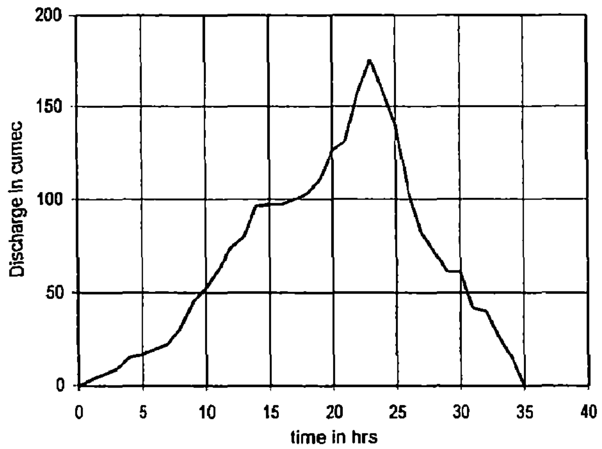
7. Event 6

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	1.00	1800.00
3	2.00	2.20	5760.00
4	3.00	3.50	10260.00
5	4.00	4.40	14220.00
6	5.00	5.30	17460.00
7	6.00	8.90	25560.00
8	7.00	11.10	36000.00
9	8.00	22.20	59940.00
10	9.00	44.40	119880.00
11	10.00	68.90	203940.00
12	11.00	97.70	299880.00
13	12.00	115.50	383760.00
14	13.00	151.10	479880.00
15	14.00	182.20	599940.00
16	15.00	200.00	687960.00
17	16.00	231.10	775980.00
18	17.00	288.90	936000.00
19	18.00	333.30	1119960.00
20	19.00	426.70	1368000.00
21	20.00	466.70	1608120.00
22	21.00	544.40	1819980.00
23	22.00	533.30	1939860.00
24	23.00	493.30	1847880.00
25	24.00	471.10	1735920.00
26	25.00	466.50	1687680.00
27	26.00	457.80	1663740.00
28	27.00	448.90	1632060.00
29	28.00	431.10	1584000.00
30	29.00	408.90	1512000.00
31	30.00	393.30	1443960.00
32	31.00	373.30	1379880.00
33	32.00	351.10	1303920.00
34	33.00	311.10	1191960.00
35	34.00	240.00	991980.00
36	35.00	204.40	799920.00
37	36.00	155.50	647820.00
38	37.00	106.70	471960.00
39	38.00	66.70	312120.00
40	39.00	22.20	160020.00
41	40.00	8.90	55980.00
42	41.00	0.00	16020.00
Total Runoff Volume (m^3)			32952960.00
Total Runoff Depth in cm			14.73

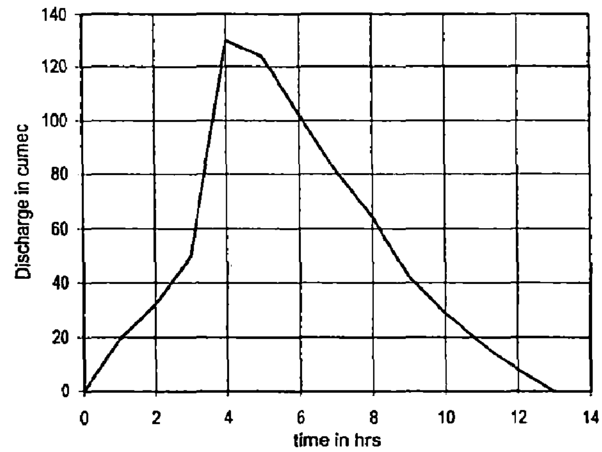
APPENDIX - C.2

Direct Runoff Hydrograph of Umar Sub-basin

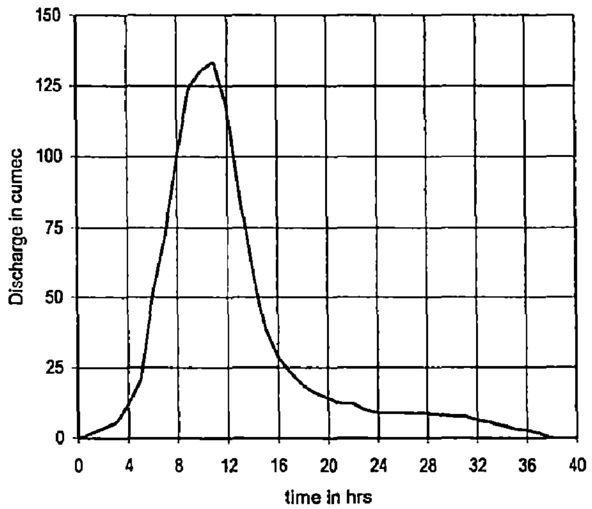
Direct Runoff Hydrograph event 1



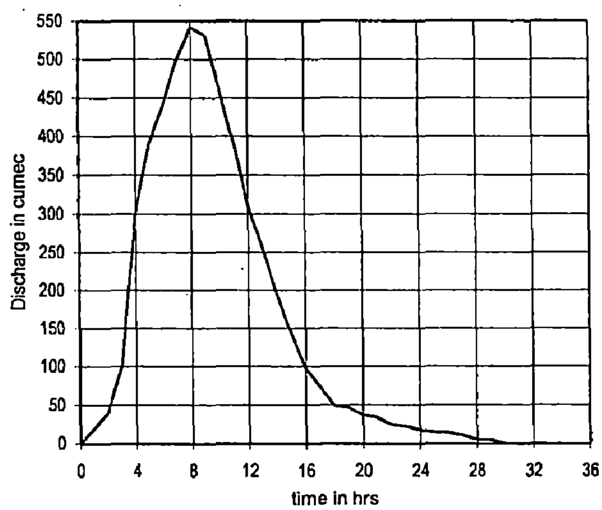
Direct Runoff Hydrograph event 3



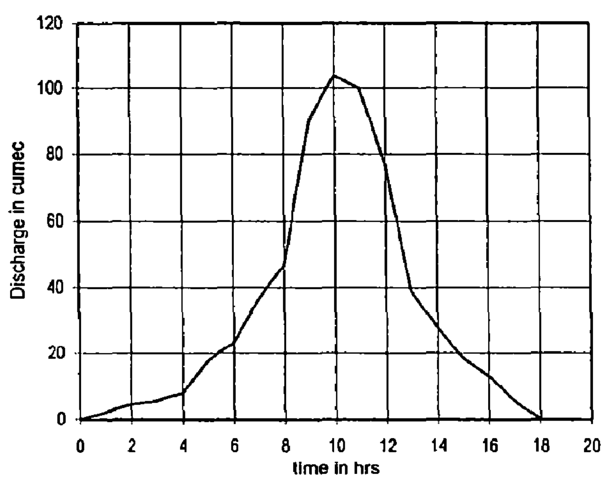
Direct Runoff Hydrograph event 2



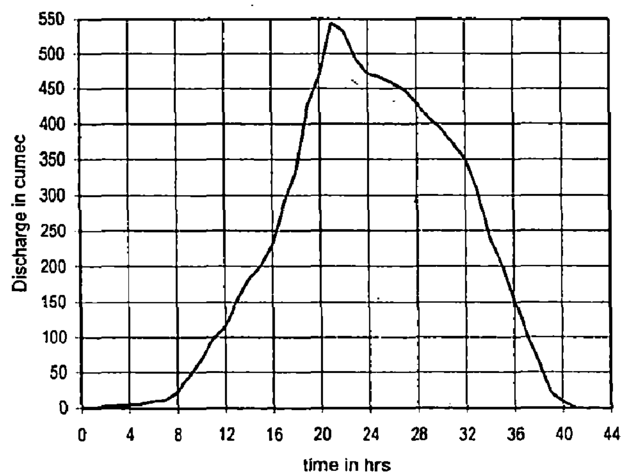
Direct Runoff Hydrograph event 5



Direct Runoff Hydrograph event 3



Direct Runoff Hydrograph event 6



APPENDIX - D.1

Runoff Events of Kolar Sub-basin

1. Event 1, (28-8-1983)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	5.00	9000.00
3	2.00	15.50	36900.00
4	3.00	145.40	289620.00
5	4.00	362.00	913320.00
6	5.00	690.00	1893600.00
7	6.00	2800.00	6282000.00
8	7.00	4035.00	12303000.00
9	8.00	4724.00	15766200.00
10	9.00	4795.00	17134200.00
11	10.00	4871.40	17399520.00
12	11.00	4744.00	17307720.00
13	12.00	4617.00	16849800.00
14	13.00	4472.00	16360200.00
15	14.00	4326.00	15836400.00
16	15.00	4180.00	15310800.00
17	16.00	4000.00	14724000.00
18	17.00	3890.50	14202900.00
19	18.00	3600.00	13482900.00
20	19.00	1960.00	10008000.00
21	20.00	980.00	5292000.00
22	21.00	760.00	3132000.00
23	22.00	630.00	2502000.00
24	23.00	500.00	2034000.00
25	24.00	400.00	1620000.00
26	25.00	325.00	1305000.00
27	26.00	276.00	1081800.00
28	27.00	232.00	914400.00
29	28.00	218.00	810000.00
30	29.00	167.00	693000.00
31	30.00	150.00	570600.00
32	31.00	145.00	531000.00
33	32.00	123.00	482400.00
34	33.00	109.00	417600.00
35	34.00	87.00	352800.00
36	35.00	80.00	300600.00
37	36.00	0.00	144000.00
Total Runoff Volume (m^3)			228293280.00
Total Runoff Depth in cm			25.26

2. Event 2, (10-8-84)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	0.00	0.00
3	2.00	36.36	65448.00
4	3.00	72.72	196344.00
5	4.00	200.00	490896.00
6	5.00	400.00	1080000.00
7	6.00	618.18	1832724.00
8	7.00	1054.55	3010914.00
9	8.00	1527.27	4647276.00
10	9.00	1981.82	6316362.00
11	10.00	2032.60	7225956.00
12	11.00	1872.73	7029594.00
13	12.00	1636.36	6316362.00
14	13.00	1581.82	5792724.00
15	14.00	1654.55	5825466.00
16	15.00	1363.64	5432742.00
17	16.00	1036.36	4320000.00
18	17.00	854.55	3403638.00
19	18.00	563.64	2552742.00
20	19.00	327.30	1603692.00
21	20.00	200.00	949140.00
22	21.00	72.73	490914.00
23	22.00	36.36	196362.00
24	23.00	18.20	98208.00
25	24.00	0.00	32760.00
Total Runoff Volume (m^3)			68910264.00
Total Runoff Depth in cm			7.62

APPENDIX - D.1 Continued

3. Event 3, (31-7-1985)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	18.20	32760.00
3	2.00	72.73	163674.00
4	3.00	181.82	458190.00
5	4.00	309.09	883638.00
6	5.00	572.73	1587267.00
7	6.00	836.36	2536353.00
8	7.00	1090.91	3469086.00
9	8.00	1293.00	4291038.00
10	9.00	1282.87	4636557.00
11	10.00	1272.73	4600071.00
12	11.00	1258.20	4555674.00
13	12.00	1236.36	4490208.00
14	13.00	1200.00	4385448.00
15	14.00	1145.45	4221810.00
16	15.00	1036.36	3927258.00
17	16.00	472.73	2716362.00
18	17.00	181.82	1178190.00
19	18.00	174.55	641466.00
20	19.00	145.45	576000.00
21	20.00	138.20	510570.00
22	21.00	127.27	477846.00
23	22.00	121.21	447264.00
24	23.00	115.15	425448.00
25	24.00	109.09	403632.00
26	25.00	103.03	381816.00
27	26.00	96.97	360000.00
28	27.00	90.91	338184.00
29	28.00	84.85	316368.00
30	29.00	78.79	294552.00
31	30.00	72.73	272736.00
32	31.00	60.61	240009.00
33	32.00	48.49	196371.00
34	33.00	36.37	152733.00
35	34.00	24.24	109095.00
36	35.00	12.12	65457.00
37	36.00	0.00	21819.00
Total Runoff Volume (m^3)			54364950.00
Total Runoff Depth in cm			6.01

4. Event 4, (13-8-1985)

No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	8.00	14400.00
3	2.00	20.00	50400.00
4	3.00	28.00	86400.00
5	4.00	32.00	108000.00
6	5.00	40.00	129600.00
7	6.00	48.00	158400.00
8	7.00	1120.00	2102400.00
9	8.00	1140.00	4068000.00
10	9.00	1200.00	4212000.00
11	10.00	1220.00	4356000.00
12	11.00	1381.70	4683060.00
13	12.00	1240.00	4719060.00
14	13.00	1040.00	4104000.00
15	14.00	820.00	3348000.00
16	15.00	600.00	2556000.00
17	16.00	400.00	1800000.00
18	17.00	220.00	1116000.00
19	18.00	140.00	648000.00
20	19.00	88.00	410400.00
21	20.00	80.00	302400.00
22	21.00	60.00	252000.00
23	22.00	53.33	204000.00
24	23.00	46.67	180000.00
25	24.00	40.00	156000.00
26	25.00	38.00	140400.00
27	26.00	36.00	133200.00
28	27.00	34.00	126000.00
29	28.00	32.00	118800.00
30	29.00	30.00	111600.00
31	30.00	28.00	104400.00
32	31.00	22.40	90720.00
33	32.00	16.80	70560.00
34	33.00	11.20	50400.00
35	34.00	5.60	30240.00
36	35.00	0.00	10080.00
Total Runoff Volume (m^3)			40750920.00
Total Runoff Depth in cm			4.51

APPENDIX - D.1 Continued

5. Event 5, (15-8-1986)

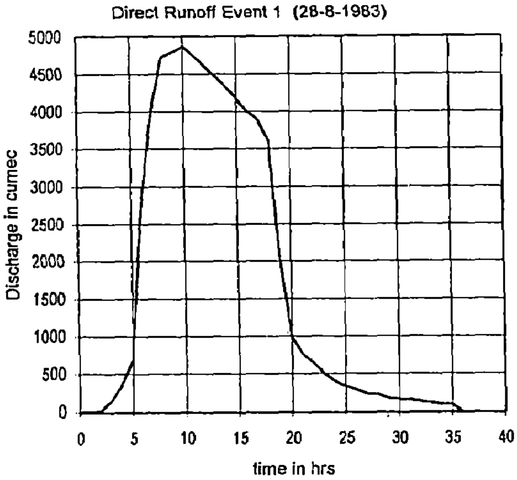
No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	15.00	27000.00
3	2.00	112.00	228600.00
4	3.00	220.00	597600.00
5	4.00	400.00	1116000.00
6	5.00	520.00	1656000.00
7	6.00	680.00	2160000.00
8	7.00	860.00	2772000.00
9	8.00	1080.00	3492000.00
10	9.00	1600.00	4824000.00
11	10.00	1900.00	6300000.00
12	11.00	1968.10	6962580.00
13	12.00	1940.00	7034580.00
14	13.00	1620.00	6408000.00
15	14.00	1000.00	4716000.00
16	15.00	680.00	3024000.00
17	16.00	320.00	1800000.00
18	17.00	248.00	1022400.00
19	18.00	200.00	806400.00
20	19.00	190.00	702000.00
21	20.00	168.00	644400.00
22	21.00	160.00	590400.00
23	22.00	153.33	564000.00
24	23.00	146.67	540000.00
25	24.00	140.00	516000.00
26	25.00	136.67	498000.00
27	26.00	133.33	486000.00
28	27.00	130.00	474000.00
29	28.00	126.67	462000.00
30	29.00	123.33	450000.00
31	30.00	120.00	438000.00
32	31.00	100.00	396000.00
33	32.00	80.00	324000.00
34	33.00	60.00	252000.00
35	34.00	40.00	180000.00
36	35.00	20.00	108000.00
37	36.00	0.00	36000.00
Total Runoff Volume (m^3)			62607960.00
Total Runoff Depth in cm			6.93

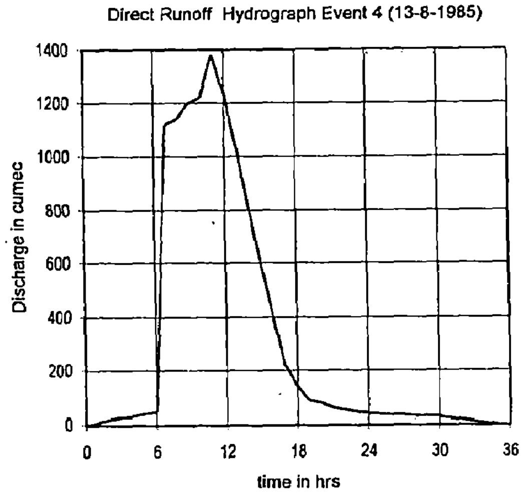
6. Event 6, (27-8-1987)

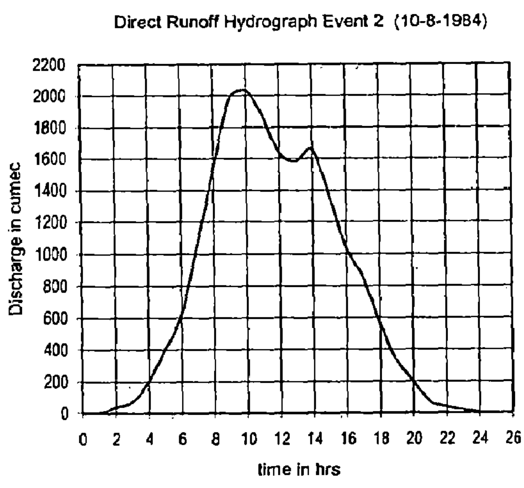
No Serie	Time (Hours)	Discharge (cumec)	Runoff Volume
1	0.00	0.00	0.00
2	1.00	2.00	3600.00
3	2.00	4.00	10800.00
4	3.00	12.00	28800.00
5	4.00	881.30	1607940.00
6	5.00	840.00	3098340.00
7	6.00	770.00	2898000.00
8	7.00	650.00	2556000.00
9	8.00	380.00	1854000.00
10	9.00	290.00	1206000.00
11	10.00	200.00	882000.00
12	11.00	150.00	630000.00
13	12.00	106.00	460800.00
14	13.00	84.00	342000.00
15	14.00	76.00	288000.00
16	15.00	50.00	226800.00
17	16.00	44.00	169200.00
18	17.00	40.00	151200.00
19	18.00	32.00	129600.00
20	19.00	24.00	100800.00
21	20.00	20.00	79200.00
22	21.00	13.33	60000.00
23	22.00	6.67	36000.00
24	23.00	0.00	12000.00
Total Runoff Volume (m^3)			16831080.00
Total Runoff Depth in cm			1.86

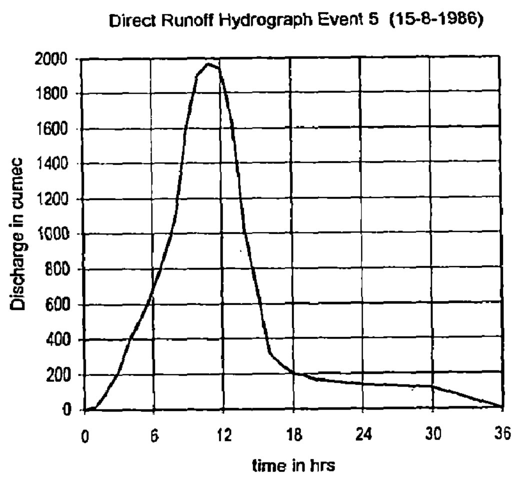
APPENDIX - D.2

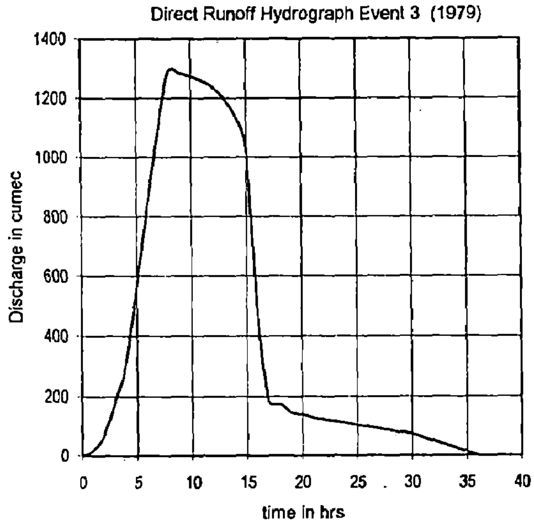
Direct Runoff Hydrograph of Kolar Sub-basin

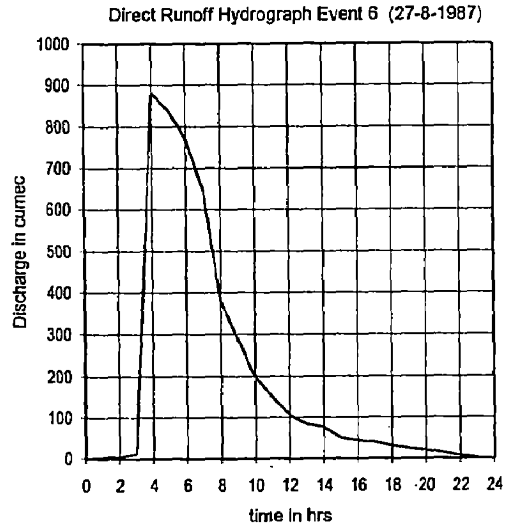












APPENDIX - E.1

Runoff Events of 3f Sub-zone

1. Event 1

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	6.60	11880.00
3	2.00	9.30	28620.00
4	3.00	13.30	40680.00
5	4.00	240.00	455940.00
6	5.00	400.00	1152000.00
7	6.00	543.80	1698840.00
8	7.00	440.00	1770840.00
9	8.00	333.30	1391940.00
10	9.00	266.70	1080000.00
11	10.00	186.70	816120.00
12	11.00	113.30	540000.00
13	12.00	53.30	299880.00
14	13.00	33.30	155880.00
15	14.00	10.70	79200.00
16	15.00	4.00	26460.00
17	16.00	0.00	7200.00
Total Runoff Volume (m^3)			9555480.00
Total Runoff Depth in cm			1.16

2. Event 2

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	5.50	9900.00
3	2.00	11.10	29880.00
4	3.00	72.20	149940.00
5	4.00	94.40	299880.00
6	5.00	478.30	1030860.00
7	6.00	472.20	1710900.00
8	7.00	444.40	1649880.00
9	8.00	344.40	1419840.00
10	9.00	322.20	1199880.00
11	10.00	266.70	1060020.00
12	11.00	166.70	780120.00
13	12.00	88.90	460080.00
14	13.00	83.30	309960.00
15	14.00	22.20	189900.00
16	15.00	11.10	59940.00
17	16.00	0.00	19980.00
Total Runoff Volume (m^3)			10380960.00
Total Runoff Depth in cm			1.26

3. Event 3

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	9.00	16200.00
3	2.00	109.00	212400.00
4	3.00	236.40	621720.00
5	4.00	268.20	908280.00
6	5.00	324.50	1066860.00
7	6.00	227.30	993240.00
8	7.00	127.30	638280.00
9	8.00	63.60	343620.00
10	9.00	36.40	180000.00
11	10.00	0.00	65520.00
Total Runoff Volume (m^3)			5046120.00
Total Runoff Depth in cm			0.61

4. Event 4

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	38.90	70020.00
3	2.00	100.00	250020.00
4	3.00	355.60	820080.00
5	4.00	402.10	1363860.00
6	5.00	316.70	1293840.00
7	6.00	200.00	930060.00
8	7.00	194.40	709920.00
9	8.00	138.90	599940.00
10	9.00	111.10	450000.00
11	10.00	88.90	360000.00
12	11.00	55.60	260100.00
13	12.00	22.20	140040.00
14	13.00	0.00	39960.00
Total Runoff Volume (m^3)			7287840.00
Total Runoff Depth in cm			0.88

5. Event 5

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	2.00	3600.00
3	2.00	2.70	8460.00
4	3.00	4.50	12960.00
5	4.00	13.60	32580.00
6	5.00	27.30	73620.00
7	6.00	100.00	229140.00
8	7.00	190.90	523620.00
9	8.00	247.00	788220.00
10	9.00	231.80	861840.00
11	10.00	227.30	826380.00
12	11.00	218.20	801900.00
13	12.00	195.50	744660.00
14	13.00	154.50	630000.00
15	14.00	105.50	468000.00
16	15.00	100.00	369900.00
17	16.00	63.60	294480.00
18	17.00	40.90	188100.00
19	18.00	22.70	114480.00
20	19.00	0.00	40860.00
Total Runoff Volume (m^3)			7012800.00
Total Runoff Depth in cm			0.85

6. Event 6

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	35.70	64260.00
3	2.00	178.60	385740.00
4	3.00	714.30	1607220.00
5	4.00	1391.00	3789540.00
6	5.00	1250.00	4753800.00
7	6.00	964.30	3985740.00
8	7.00	767.80	3117780.00
9	8.00	732.10	2699820.00
10	9.00	696.40	2571300.00
11	10.00	535.70	2217780.00
12	11.00	410.70	1703520.00
13	12.00	339.30	1350000.00
14	13.00	328.60	1202220.00
15	14.00	321.40	1170000.00
16	15.00	317.80	1150560.00
17	16.00	285.70	1086300.00
18	17.00	257.10	977040.00
19	18.00	214.30	848520.00
20	19.00	71.40	514260.00
21	20.00	0.00	128520.00
Total Runoff Volume (m^3)			35323920.00
Total Runoff Depth in cm			4.29

APPENDIX - E.1 Continued

7. Event 7

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	53.36	96048.00
3	2.00	53.36	192096.00
4	3.00	53.36	192096.00
5	4.00	53.36	192096.00
6	5.00	63.36	210096.00
7	6.00	63.36	228096.00
8	7.00	293.36	642096.00
9	8.00	403.36	1254096.00
10	9.00	593.36	1794096.00
11	10.00	473.36	1920096.00
12	11.00	373.36	1524096.00
13	12.00	323.36	1254096.00
14	13.00	213.36	966096.00
15	14.00	153.36	660096.00
16	15.00	103.36	462096.00
17	16.00	78.36	327096.00
18	17.00	53.36	237096.00
19	18.00	48.36	183096.00
20	19.00	43.36	165096.00
21	20.00	43.36	156096.00
22	21.00	33.36	138096.00
23	22.00	28.36	111096.00
24	23.00	23.36	93096.00
25	24.00	18.36	75096.00
26	25.00	0.00	33048.00
Total Runoff Volume (m^3)			13106304.00
Total Runoff Depth in cm			1.59

8. Event 8

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	0.00	0.00
3	2.00	39.36	70848.00
4	3.00	44.36	150696.00
5	4.00	49.36	168696.00
6	5.00	126.36	316296.00
7	6.00	269.36	712296.00
8	7.00	339.36	1095696.00
9	8.00	354.36	1248696.00
10	9.00	414.36	1383696.00
11	10.00	424.36	1509696.00
12	11.00	269.36	1248696.00
13	12.00	244.36	924696.00
14	13.00	194.36	789696.00
15	14.00	154.36	627696.00
16	15.00	134.36	519696.00
17	16.00	116.36	451296.00
18	17.00	104.36	397296.00
19	18.00	99.36	366696.00
20	19.00	89.36	339696.00
21	20.00	84.36	312696.00
22	21.00	0.00	151848.00
Total Runoff Volume (m^3)			12786624.00
Total Runoff Depth in cm			1.55

9. Event 9

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	1.00	1800.00
3	2.00	1.00	3600.00
4	3.00	1.00	3600.00
5	4.00	1.00	3600.00
6	5.00	1.00	3600.00
7	6.00	1.00	3600.00
8	7.00	1.00	3600.00
9	8.00	65.00	118800.00
10	9.00	130.00	351000.00
11	10.00	230.00	648000.00
12	11.00	255.00	873000.00
13	12.00	190.00	801000.00
14	13.00	120.00	558000.00
15	14.00	80.00	360000.00
16	15.00	55.00	243000.00
17	16.00	55.00	198000.00
18	17.00	35.00	162000.00
19	18.00	35.00	126000.00
20	19.00	35.00	126000.00
21	20.00	35.00	126000.00
22	21.00	27.00	111600.00
23	22.00	22.00	88200.00
24	23.00	15.00	66600.00
25	24.00	15.00	54000.00
26	25.00	15.00	54000.00
27	26.00	10.00	45000.00
28	27.00	0.00	18000.00
Total Runoff Volume (m^3)			5151600.00
Total Runoff Depth in cm			0.63

10. Event 10

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	0.00	0.00
3	2.00	0.00	0.00
4	3.00	0.00	0.00
5	4.00	2.70	4860.00
6	5.00	92.70	171720.00
7	6.00	222.70	567720.00
8	7.00	262.70	873720.00
9	8.00	322.70	1053720.00
10	9.00	222.70	981720.00
11	10.00	122.70	621720.00
12	11.00	62.70	333720.00
13	12.00	32.70	171720.00
14	13.00	2.70	63720.00
15	14.00	0.00	4860.00
16	15.00	0.00	0.00
17	16.00	0.00	0.00
18	17.00	0.00	0.00
19	18.00	0.00	0.00
20	19.00	0.00	0.00
21	20.00	0.00	0.00
22	21.00	0.00	0.00
23	22.00	0.00	0.00
Total Runoff Volume (m^3)			4849200.00
Total Runoff Depth in cm			0.59

APPENDIX - E.1 Continued

11. Event 11

No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	35.00	63000.00
3	2.00	35.00	126000.00
4	3.00	35.00	126000.00
5	4.00	35.00	126000.00
6	5.00	85.00	216000.00
7	6.00	150.00	423000.00
8	7.00	410.00	1008000.00
9	8.00	455.00	1557000.00
10	9.00	380.00	1503000.00
11	10.00	260.00	1152000.00
12	11.00	260.00	936000.00
13	12.00	200.00	828000.00
14	13.00	175.00	675000.00
15	14.00	165.00	612000.00
16	15.00	135.00	540000.00
17	16.00	105.00	432000.00
18	17.00	85.00	342000.00
19	18.00	75.00	288000.00
20	19.00	70.00	261000.00
21	20.00	70.00	252000.00
22	21.00	70.00	252000.00
23	22.00	60.00	234000.00
24	23.00	60.00	216000.00
25	24.00	60.00	216000.00
26	25.00	0.00	108000.00
Total Runoff Volume (m^3)			12492000.00
Total Runoff Depth in cm			1.52

12. Event 12

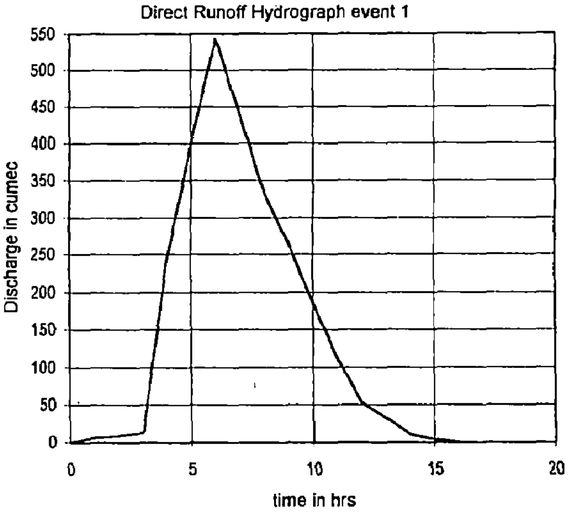
No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	0.00	0.00
3	2.00	0.00	0.00
4	3.00	0.00	0.00
5	4.00	0.00	0.00
6	5.00	0.00	0.00
7	6.00	0.00	0.00
8	7.00	0.00	0.00
9	8.00	80.60	145080.00
10	9.00	160.60	434160.00
11	10.00	215.60	677160.00
12	11.00	195.60	740160.00
13	12.00	190.60	695160.00
14	13.00	180.60	668160.00
15	14.00	155.60	605160.00
16	15.00	120.60	497160.00
17	16.00	70.60	344160.00
18	17.00	65.60	245160.00
19	18.00	25.60	164160.00
20	19.00	2.60	50760.00
21	20.00	0.00	4680.00
22	21.00	0.00	0.00
23	22.00	0.00	0.00
24	23.00	0.00	0.00
25	24.00	0.00	0.00
26	25.00	0.00	0.00
Total Runoff Volume (m^3)			5271120.00
Total Runoff Depth in cm			0.64

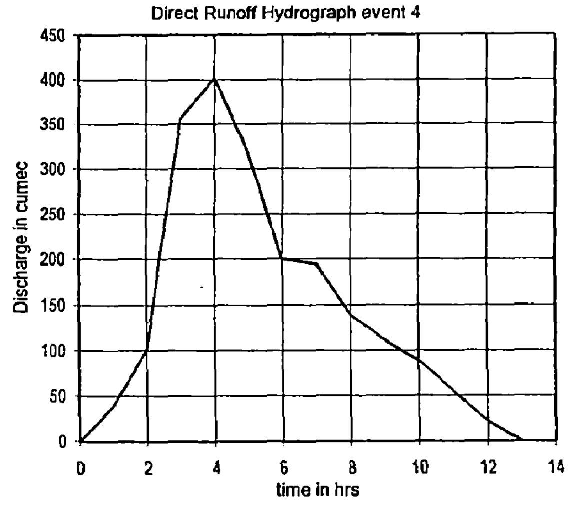
13. Event 13

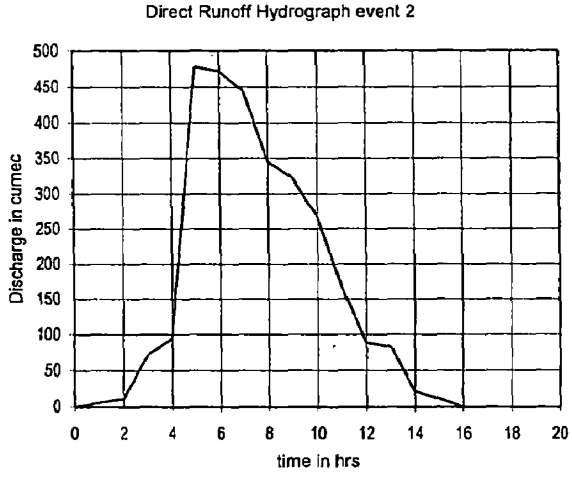
No Serie	Time (Hours)	Discharge (cumec)	Volume Direct Runoff
1	0.00	0.00	0.00
2	1.00	5.00	9000.00
3	2.00	10.00	27000.00
4	3.00	10.00	36000.00
5	4.00	40.00	90000.00
6	5.00	205.00	441000.00
7	6.00	765.00	1746000.00
8	7.00	1432.00	3954600.00
9	8.00	1300.00	4917600.00
10	9.00	1000.00	4140000.00
11	10.00	825.00	3285000.00
12	11.00	800.00	2925000.00
13	12.00	760.00	2808000.00
14	13.00	590.00	2430000.00
15	14.00	480.00	1926000.00
16	15.00	425.00	1629000.00
17	16.00	425.00	1530000.00
18	17.00	425.00	1530000.00
19	18.00	425.00	1530000.00
20	19.00	400.00	1485000.00
21	20.00	375.00	1395000.00
22	21.00	350.00	1305000.00
23	22.00	205.00	999000.00
24	23.00	140.00	621000.00
25	24.00	0.00	252000.00
Total Runoff Volume (m^3)			41011200.00
Total Runoff Depth in cm			4.98

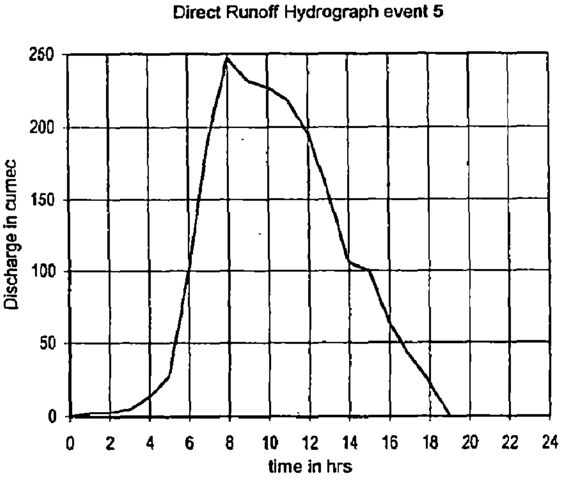
APPENDIX - E.2

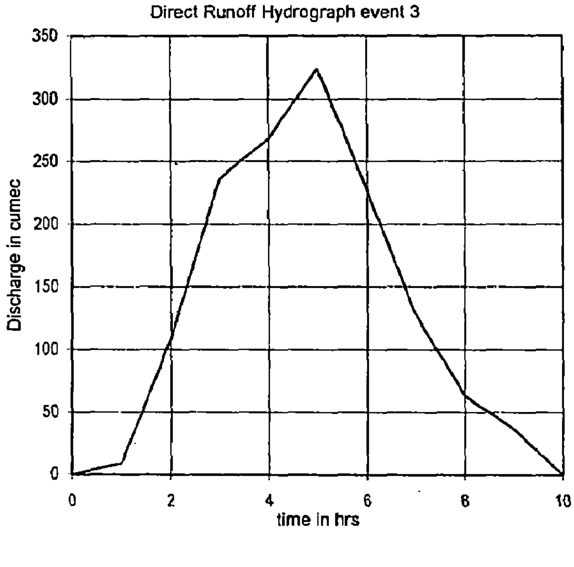
Direct Runoff Hydrograph of 3f Sub-zone

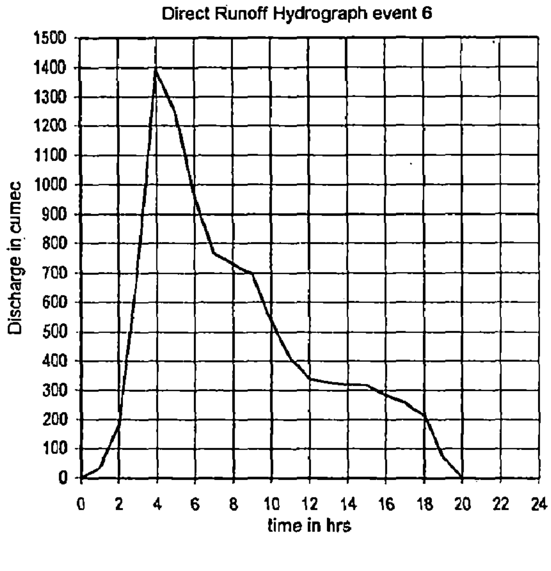






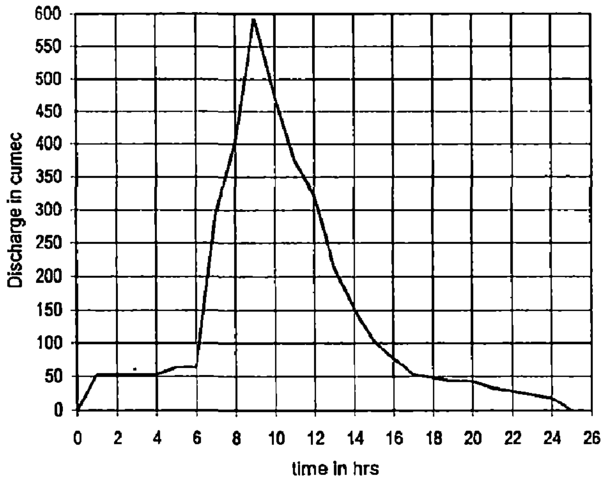




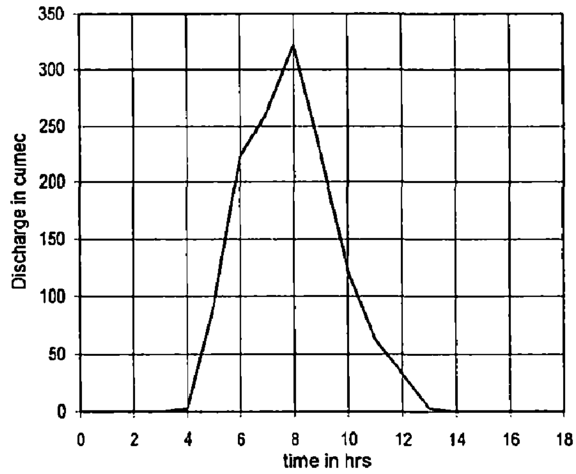


APPENDIX - E.2 Continued

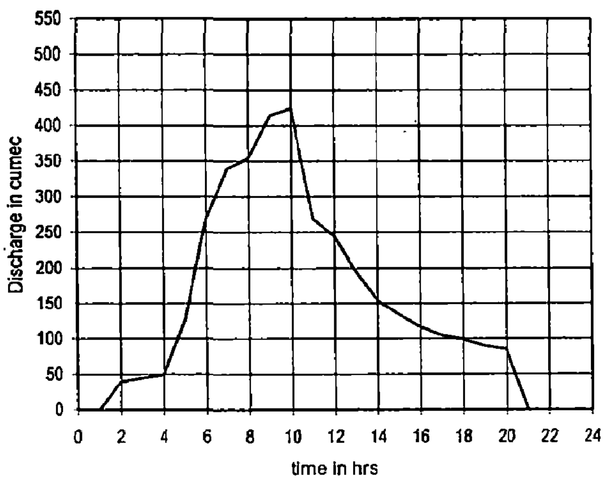
Direct Runoff Hydrograph event 7



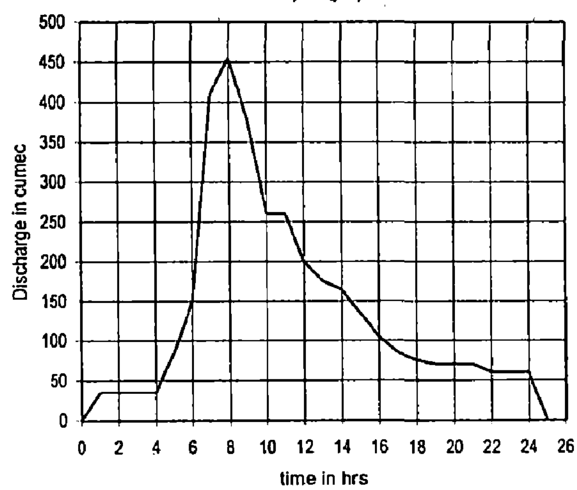
Direct Runoff Hydrograph event 10



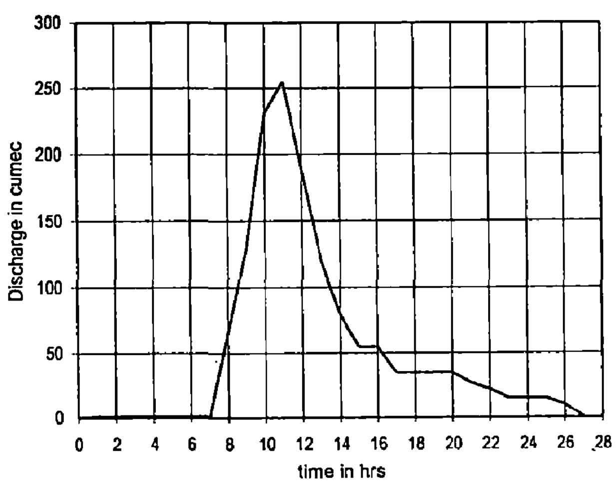
Direct Runoff Hydrograph event 8



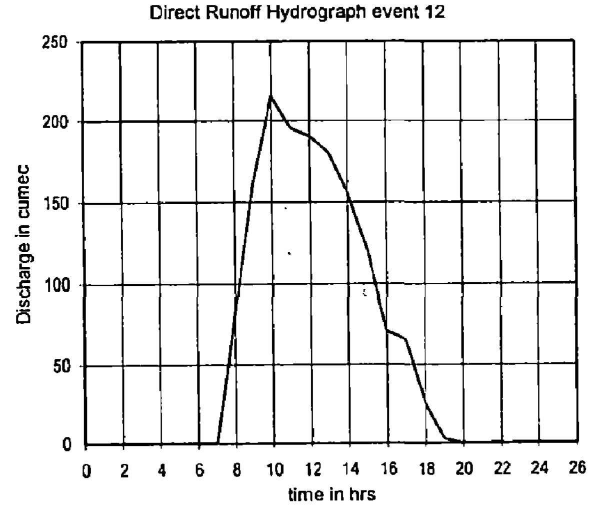
Direct Runoff Hydrograph event 11



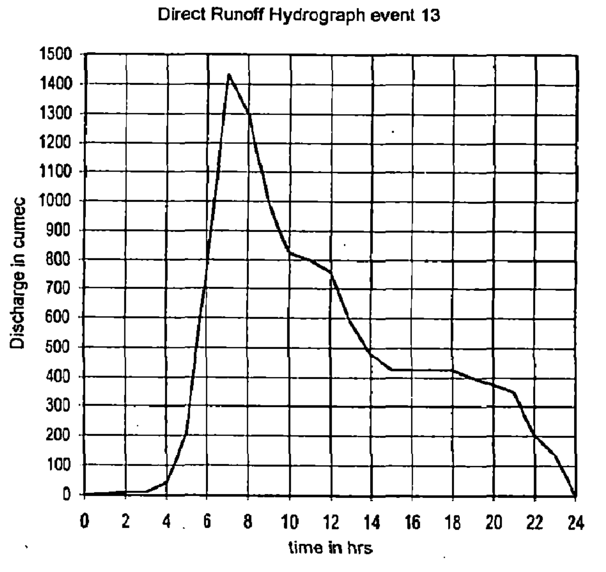
Direct Runoff Hydrograph event 9



Direct Runoff Hydrograph event 12



APPENDIX - E.2 Continued



APPENDIX - F.1

Runoff Events of Gola Sub-basin

1. Event 1, (1977)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	54.00	54.00	0.00	0.00
2	2.00	58.32	54.26	4.06	14618.00
3	4.00	153.30	54.52	98.78	370230.00
4	6.00	153.30	54.78	98.52	710290.00
5	8.00	161.60	55.04	106.56	738302.00
6	10.00	225.00	55.30	169.70	994554.00
7	12.00	418.28	55.56	362.72	1916734.00
8	14.00	506.25	55.82	450.43	2927366.00
9	16.00	506.25	56.08	450.17	3242190.00
10	18.00	418.28	56.34	361.95	2923630.00
11	20.00	266.60	56.59	210.01	2059022.00
12	22.00	266.60	56.85	209.75	1511106.00
13	24.00	245.10	57.11	187.99	1431838.00
14	26.00	239.94	57.37	182.57	1333994.00
15	28.00	239.97	57.63	182.34	1313658.00
16	30.00	213.46	57.89	155.57	1216462.00
17	32.00	210.70	58.15	152.55	1109222.00
18	34.00	197.37	58.41	138.96	1049430.00
19	36.00	194.13	58.67	135.46	987901.00
20	38.00	190.89	58.93	131.96	962687.00
21	40.00	187.64	59.19	128.45	937473.00
22	42.00	184.40	59.45	124.95	912259.00
23	44.00	176.02	59.71	116.31	868550.00
24	46.00	167.64	59.97	107.67	806346.00
25	48.00	164.66	60.23	104.43	763582.00
26	50.00	161.68	60.49	101.19	740258.00
27	52.00	161.68	60.75	100.93	727662.00
28	54.00	161.68	61.01	100.68	725794.00
29	56.00	145.96	61.26	84.70	667334.00
30	58.00	130.24	61.52	68.72	552282.00
31	60.00	130.24	61.78	68.46	493822.00
32	62.00	130.24	62.04	68.20	491954.00
33	64.00	125.89	62.30	63.58	474408.00
34	66.00	121.53	62.56	58.97	441184.00
35	68.00	104.47	62.82	41.64	362204.00
36	70.00	87.40	63.08	24.32	237468.00
37	72.00	75.37	63.34	12.03	130858.00
38	74.00	63.34	63.34	0.00	43308.00
Total Runoff Volume (m3)					37189980.00
Total Runoff Depth in cm					8.26

2. Event 2, (1977)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	54.00	54.00	0.00	0.00
2	2.00	58.32	54.27	4.05	14578.00
3	4.00	153.30	54.54	98.76	370110.00
4	6.00	153.30	54.81	98.49	710090.00
5	8.00	161.60	55.08	106.52	738022.00
6	10.00	266.60	55.35	211.25	1143954.00
7	12.00	361.60	55.62	305.98	1862006.00
8	14.00	481.50	55.89	425.61	2633698.00
9	16.00	425.26	56.16	369.10	2860926.00
10	18.00	361.60	56.44	305.17	2427338.00
11	20.00	266.60	56.71	209.89	1854214.00
12	22.00	245.10	56.98	188.12	1432866.00
13	24.00	239.91	57.25	182.66	1334834.00
14	26.00	213.46	57.52	155.94	1218982.00
15	28.00	213.46	57.79	155.67	1121814.00
16	30.00	210.70	58.06	152.64	1109930.00
17	32.00	197.37	58.33	139.04	1050058.00
18	34.00	194.13	58.60	135.53	988449.00
19	36.00	190.89	58.87	132.02	963155.00
20	38.00	187.64	59.14	128.50	937861.00
21	40.00	184.40	59.41	124.99	912567.00
22	42.00	176.02	59.68	116.34	868778.00
23	44.00	167.64	59.95	107.69	806494.00
24	46.00	164.66	60.22	104.44	763650.00
25	48.00	161.68	60.49	101.19	740246.00
26	50.00	161.68	60.76	100.92	727570.00
27	52.00	161.68	61.03	100.65	725622.00
28	54.00	145.96	61.30	84.66	667082.00
29	56.00	130.24	61.58	68.66	551950.00
30	58.00	130.24	61.85	68.39	493410.00
31	60.00	130.24	62.12	68.12	491462.00
32	62.00	125.89	62.39	63.50	473836.00
33	64.00	121.53	62.66	58.87	440532.00
34	66.00	104.47	62.93	41.54	361472.00
35	68.00	87.40	63.20	24.20	236656.00
36	70.00	75.37	63.47	11.90	129966.00
37	72.00	63.34	63.34	0.00	42842.00
Total Runoff Volume (m3)					34207020.00
Total Runoff Depth in cm					7.60

APPENDIX - F.1 Continued

3. Event 3, (1978)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	32.00	32.00	0.00	0.00
2	2.00	100.00	32.33	67.67	243600.00
3	4.00	120.00	32.67	87.33	558000.00
4	6.00	132.00	33.00	99.00	670800.00
5	8.00	145.00	33.33	111.67	758400.00
6	10.00	172.00	33.67	138.33	900000.00
7	12.00	230.00	34.00	196.00	1203600.00
8	14.00	320.00	34.33	285.67	1734000.00
9	16.00	300.00	34.67	265.33	1983600.00
10	18.00	284.38	35.00	249.38	1852950.00
11	20.00	268.75	35.33	233.42	1738050.00
12	22.00	253.13	35.67	217.46	1623150.00
13	24.00	237.50	36.00	201.50	1508250.00
14	26.00	225.63	36.33	189.29	1406850.00
15	28.00	213.75	36.67	177.08	1318950.00
16	30.00	201.88	37.00	164.88	1231050.00
17	32.00	190.00	37.33	152.67	1143150.00
18	34.00	181.00	37.67	143.33	1065600.00
19	36.00	172.00	38.00	134.00	998400.00
20	38.00	158.50	38.33	120.17	915000.00
21	40.00	145.00	38.67	106.33	815400.00
22	42.00	138.50	39.00	99.50	741000.00
23	44.00	132.00	39.33	92.67	691800.00
24	46.00	126.00	39.67	86.33	644400.00
25	48.00	120.00	40.00	80.00	598800.00
26	50.00	110.00	40.33	69.67	538800.00
27	52.00	100.00	40.67	59.33	464400.00
28	54.00	93.00	41.00	52.00	400800.00
29	56.00	86.00	41.33	44.67	348000.00
30	58.00	64.00	41.67	22.33	241200.00
31	60.00	42.00	42.00	0.00	80400.00
Total Runoff Volume (m3)					28418400.00
Total Runoff Depth in cm					6.32

4. Event 4, (1979)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	13.00	13.00	0.00	0.00
2	2.00	17.50	13.21	4.29	15438.46
3	4.00	22.00	13.42	8.58	46315.38
4	6.00	27.50	13.63	13.87	80792.31
5	8.00	33.00	13.85	19.15	118869.23
6	10.00	58.50	14.06	44.44	228946.15
7	12.00	84.00	14.27	69.73	411023.08
8	14.00	98.00	14.48	83.52	551700.00
9	16.00	112.00	14.69	97.31	650976.92
10	18.00	148.00	14.90	133.10	829453.85
11	20.00	145.00	15.12	129.88	946730.77
12	22.00	115.00	15.33	99.67	826407.69
13	24.00	103.33	15.54	87.79	674884.62
14	26.00	91.67	15.75	75.92	589361.54
15	28.00	80.00	15.96	64.04	503838.46
16	30.00	73.58	16.17	57.40	437185.38
17	32.00	67.15	16.38	50.77	389402.31
18	34.00	60.73	16.60	44.13	341619.23
19	36.00	54.30	16.81	37.49	293836.15
20	38.00	49.30	17.02	32.28	251183.08
21	40.00	44.30	17.23	27.07	213660.00
22	42.00	39.30	17.44	21.86	176136.92
23	44.00	34.30	17.65	16.65	138613.85
24	46.00	30.00	17.87	12.13	103610.77
25	48.00	25.00	18.08	6.92	68607.69
26	50.00	22.00	18.29	3.71	38284.62
27	52.00	18.50	18.50	0.00	13361.54
Total Runoff Volume (m3)					8940240.00
Total Runoff Depth in cm					1.99

APPENDIX - F.1 Continued

5. Event 5, (1980)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	20.00	20.00	0.00	0.00
2	2.00	22.83	20.28	2.56	9206.90
3	4.00	25.67	20.55	5.11	27620.69
4	6.00	28.50	20.83	7.67	46034.48
5	8.00	31.33	21.10	10.23	64448.28
6	10.00	34.17	21.38	12.79	82862.07
7	12.00	37.00	21.66	15.34	101275.86
8	14.00	47.50	21.93	25.57	147289.66
9	16.00	58.00	22.21	35.79	220903.45
10	18.00	58.00	22.48	35.52	256717.24
11	20.00	67.33	22.76	44.57	288331.03
12	22.00	75.33	23.03	52.30	348744.83
13	24.00	82.00	23.31	58.69	399558.62
14	26.00	86.33	23.59	62.75	437172.41
15	28.00	90.67	23.86	66.80	466386.21
16	30.00	95.00	24.14	70.86	495600.00
17	32.00	141.00	24.41	116.59	674813.79
18	34.00	125.50	24.69	100.81	782627.59
19	36.00	110.00	24.97	85.03	669041.38
20	38.00	96.00	25.24	70.76	560855.17
21	40.00	82.00	25.52	56.48	458068.97
22	42.00	74.23	25.79	48.43	377692.76
23	44.00	66.45	26.07	40.38	319726.55
24	46.00	58.68	26.34	32.33	261760.34
25	48.00	50.90	26.62	24.28	203794.14
26	50.00	45.68	26.90	18.78	155007.93
27	52.00	43.00	27.17	15.83	124581.72
28	54.00	36.50	27.45	9.05	89565.52
29	56.00	30.00	27.72	2.28	40779.31
30	58.00	28.00	28.00	0.00	8193.10
Total Runoff Volume (m3)					8118660.00
Total Runoff Depth in cm					1.80

6. Event 6, (1982)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	103.00	103.00	0.00	0.00
2	2.00	355.00	103.71	251.29	904658.82
3	4.00	465.00	104.41	360.59	2202776.47
4	6.00	637.00	105.12	531.88	3212894.12
5	8.00	465.00	105.82	359.18	3207811.76
6	10.00	382.00	106.53	275.47	2284729.41
7	12.00	351.50	107.24	244.26	1871047.06
8	14.00	321.00	107.94	213.06	1646364.71
9	16.00	290.50	108.65	181.85	1421682.35
10	18.00	260.00	109.35	150.65	1197000.00
11	20.00	240.00	110.06	129.94	1010117.65
12	22.00	220.00	110.76	109.24	861035.29
13	24.00	200.00	111.47	88.53	711952.94
14	26.00	180.00	112.18	67.82	562870.59
15	28.00	163.75	112.88	50.87	427288.24
16	30.00	147.50	113.59	33.91	305205.88
17	32.00	131.25	114.29	16.96	183123.53
18	34.00	115.00	115.00	0.00	61041.18
Total Runoff Volume (m3)					22071600.00
Total Runoff Depth in cm					4.90

7. Event 7, (1984)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	110.10	110.10	0.00	0.00
2	1.00	351.26	110.36	240.90	433628.69
3	2.00	352.87	110.61	242.26	869696.07
4	3.00	355.87	110.87	245.00	877075.45
5	4.00	265.35	111.12	154.23	718620.83
6	5.00	231.00	111.38	119.62	492936.21
7	6.00	223.29	111.63	111.65	416300.59
8	7.00	215.57	111.89	103.68	387607.97
9	8.00	208.04	112.14	95.90	359248.34
10	9.00	200.64	112.40	88.24	331455.72
11	10.00	193.60	112.65	80.95	304545.10
12	11.00	186.83	112.91	73.92	278768.48
13	12.00	186.83	113.16	73.67	265663.86
14	13.00	183.52	113.42	70.10	258787.24
15	14.00	180.21	113.67	66.54	245952.62
16	15.00	176.90	113.93	62.97	233118.00
17	16.00	171.04	114.18	56.85	215688.88
18	17.00	165.18	114.44	50.74	193665.26
19	18.00	159.31	114.69	44.62	171641.64
20	19.00	153.45	114.95	38.50	149618.02
21	20.00	147.59	115.20	32.38	127594.40
22	21.00	141.73	115.46	26.27	105570.78
23	22.00	135.86	115.71	20.15	83547.16
24	23.00	130.00	115.97	14.03	61523.53
25	24.00	125.00	116.22	8.78	41052.41
26	25.00	122.00	116.48	5.52	25733.79
27	26.00	120.20	116.73	3.47	16175.17
28	27.00	119.00	116.99	2.01	9856.55
29	28.00	118.00	117.24	0.76	4977.93
30	29.00	117.50	117.50	0.00	1359.31
Total Runoff Volume (m3)					7681410.00
Total Runoff Depth in cm					1.71

8. Event 8, (1985)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	35.86	35.86	0.00	0.00
2	1.00	90.81	36.40	54.41	97946.13
3	2.00	133.06	36.93	96.13	270978.39
4	3.00	189.64	37.47	152.17	446944.65
5	4.00	266.35	38.00	228.35	684938.90
6	5.00	329.12	38.54	290.58	934075.16
7	6.00	420.12	39.07	381.05	1208933.42
8	7.00	508.56	39.61	468.95	1529997.68
9	8.00	568.38	40.14	528.24	1794937.94
10	9.00	461.10	40.68	420.42	1707582.19
11	10.00	450.07	41.21	408.86	1492696.45
12	11.00	439.04	41.75	397.29	1451060.71
13	12.00	410.20	42.29	367.92	1377372.97
14	13.00	381.37	42.82	338.55	1271633.23
15	14.00	352.53	43.36	309.17	1165893.48
16	15.00	323.69	43.89	279.80	1060153.74
17	16.00	294.86	44.43	250.43	954414.00
18	17.00	266.02	44.96	221.06	848674.26
19	18.00	237.18	45.50	191.68	742934.52
20	19.00	208.35	46.03	162.31	637194.77
21	20.00	179.51	46.57	132.94	531455.03
22	21.00	169.13	47.11	122.02	458928.29
23	22.00	158.74	47.64	111.10	419614.55
24	23.00	148.36	48.18	100.18	380300.81
25	24.00	137.97	48.71	89.26	340987.06
26	25.00	127.59	49.25	78.34	301673.32
27	26.00	117.20	49.78	67.42	262359.58
28	27.00	106.82	50.32	56.50	223045.84
29	28.00	96.43	50.85	45.58	183732.10
30	29.00	86.05	51.39	34.66	144418.35
31	30.00	75.66	51.92	23.74	105104.61
32	31.00	53.01	53.01	0.00	42723.87
Total Runoff Volume (m3)					23072706.00
Total Runoff Depth in cm					5.13

APPENDIX - F.1 Continued

7. Event 9, (1986)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	20.00	20.00	0.00	0.00
2	6.00	164.00	22.00	142.00	1533600.00
3	12.00	350.00	24.00	326.00	5054400.00
4	18.00	725.00	26.00	699.00	11070000.00
5	24.00	640.00	28.00	612.00	14158800.00
6	30.00	506.00	30.00	476.00	11750400.00
7	36.00	400.00	32.00	368.00	9115200.00
8	42.00	378.00	34.00	344.00	7689600.00
9	48.00	312.00	36.00	276.00	6696000.00
10	54.00	256.00	38.00	218.00	5335200.00
11	60.00	162.00	40.00	122.00	3672000.00
12	66.00	130.00	42.00	88.00	2268000.00
13	72.00	44.00	44.00	0.00	950400.00
Total Runoff Volume (m3)					79293600.00
Total Runoff Depth in cm					17.62

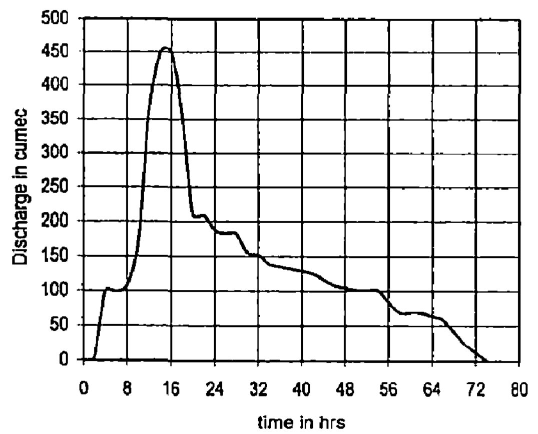
8. Event 10, (1986)

No Serie	Time (Hours)	Discharge (cumec)	Base flow cumecs	DRH ordinate	Direct Runoff Volume
1	0.00	78.15	78.15	0.00	0.00
2	2.00	139.75	78.50	61.25	220483.80
3	4.00	262.95	78.86	184.09	883211.40
4	6.00	358.70	79.21	279.49	1383723.00
5	8.00	269.13	79.57	189.57	1403430.60
6	10.00	231.88	79.92	151.96	1229482.20
7	12.00	211.53	80.28	131.25	1019545.80
8	14.00	196.29	80.63	115.66	888879.69
9	16.00	186.18	80.99	105.19	795074.14
10	18.00	176.06	81.34	94.72	719698.89
11	20.00	165.95	81.70	84.26	644323.63
12	22.00	155.84	82.05	73.79	568948.37
13	24.00	145.72	82.40	63.32	493573.11
14	26.00	135.61	82.76	52.85	418197.86
15	28.00	125.49	83.11	42.38	342822.60
16	30.00	115.38	83.47	31.91	267447.34
17	32.00	105.26	83.82	21.44	192072.09
18	34.00	95.15	84.18	10.97	116696.83
19	36.00	91.85	84.53	7.32	65853.00
20	38.00	88.55	84.89	3.66	39528.60
21	40.00	85.24	85.24	0.00	13180.20
Total Runoff Volume (m3)					11706173.14
Total Runoff Depth in cm					2.60

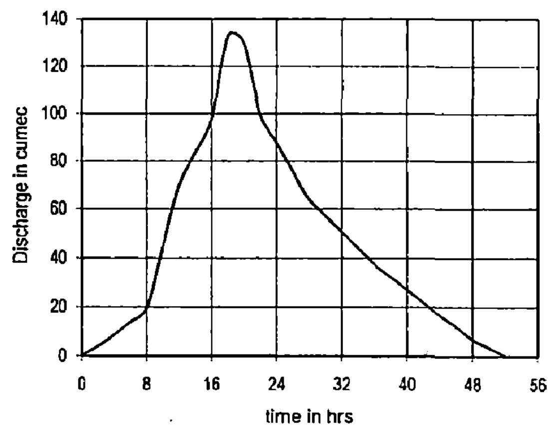
APPENDIX - F.2

Flood Hydrograph of Gola Sub-basin

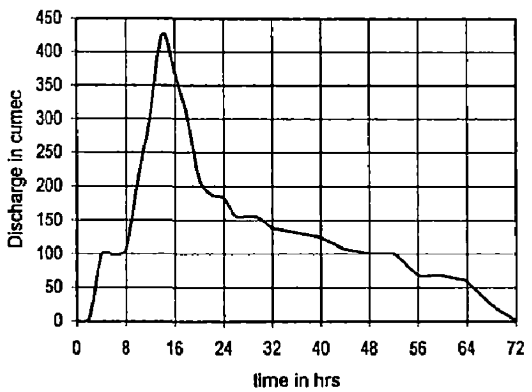
Direct Runoff Hydrograph Event 1



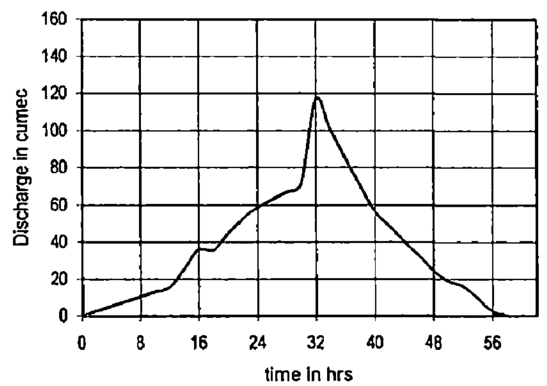
Direct Runoff Hydrograph Event 4



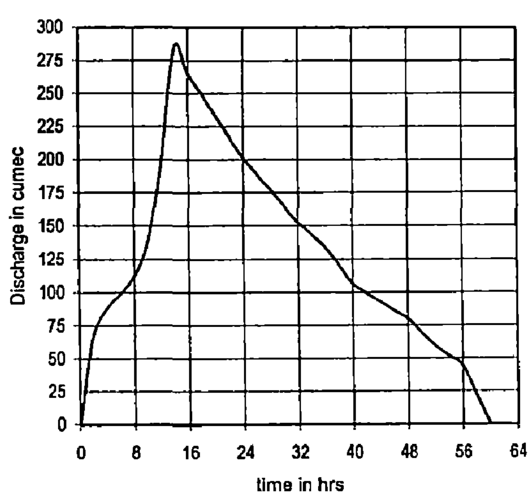
Direct Runoff Hydrograph Event 2



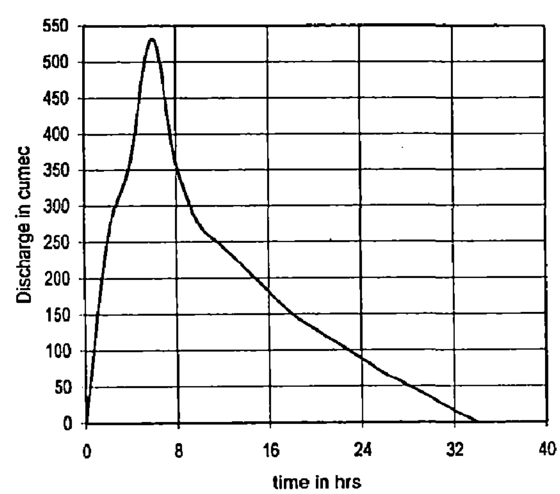
Direct Runoff Hydrograph Event 5



Direct Runoff Hydrograph Event 3

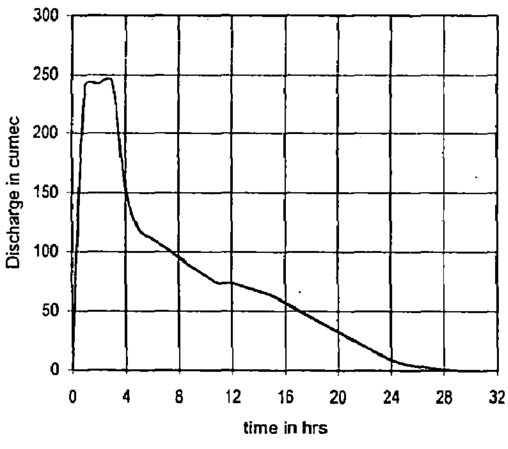


Direct Runoff Hydrograph Event 7

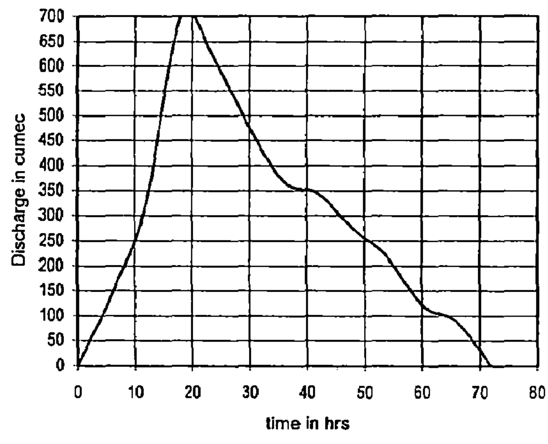


APPENDIX - F.2 Continued

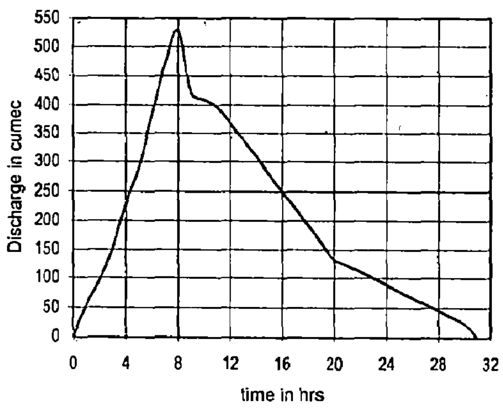
Direct Runoff Hydrograph Event 7



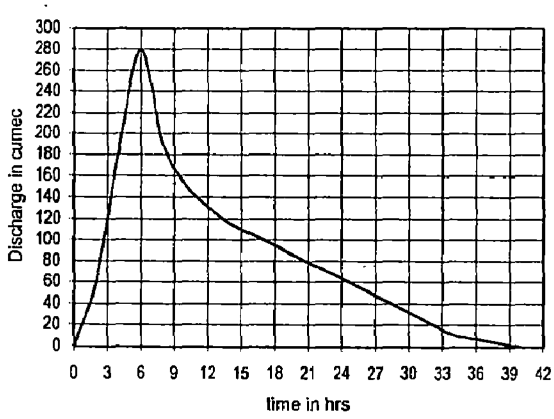
Direct Runoff Hydrograph Event 9



Direct Runoff Hydrograph Event 8



Direct Runoff Hydrograph Event 10



APPENDIX - G

Q_p, Q_p/V, Q_p/V^2 and T_p DATA FOR DEVELOPMENT OF PEAK DISCHARGE DISTRIBUTION

Table G.1. Q_p, Q_p/V, Q_p/V^2 and T_p data for Temur Sub-Basin

No	Event	Catchment Area (km²) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Qp/V (m³/s/cm)	Qp/V² (m³/s/cm²)	Tp (hrs)
1	23.07.1962	518.67	56.62	0.3027	181.23	1.08	167.81	155.38	10.00
2	05.09.1962	518.67	56.62	0.3027	135.92	0.67	202.87	302.78	6.00
3	20.07.1964	518.67	56.62	0.3027	214.93	1.32	162.83	123.35	8.00
4	14.08.1964	518.67	56.62	0.3027	124.59	0.81	153.81	189.89	9.00
5	30.08.1965	518.67	56.62	0.3027	58.16	0.26	223.69	860.36	8.00
6	07.09.1965	518.67	56.62	0.3027	59.50	0.31	191.94	619.15	6.00
7	24.08.1961	518.67	56.52	0.3027	430.39	4.75	90.61	19.08	8.00

Table G.2. Q_p, Q_p/V, Q_p/V^2 and T_p data for Teriya Sub-Basin

No	Event	Catchment Area (km²) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Qp/V (m³/s/cm)	Qp/V² (m³/s/cm²)	Tp (hrs)
1	1977	114.22	35.42	0.321	161.00	4.32	37.29	8.64	5.00
2	1978	114.22	35.42	0.321	120.00	1.49	80.66	54.22	5.00
3	1979	114.22	35.42	0.321	82.50	1.79	46.04	25.70	5.00
4	1980	114.22	35.42	0.321	114.00	2.72	41.86	15.37	6.00
5	1981	114.22	35.42	0.321	118.00	2.38	49.52	20.78	7.00
6	1982	114.22	35.42	0.321	131.00	2.92	44.88	15.38	6.00
7	1984	114.22	35.42	0.321	89.00	2.07	42.91	20.69	7.00
8	1985	114.22	35.42	0.321	108.00	2.04	52.84	25.85	8.00
9	1986	114.22	35.42	0.321	305.00	4.40	69.37	15.78	5.00
10	1986	114.22	35.42	0.321	575.75	21.21	27.14	1.28	7.00
11	1986	114.22	35.42	0.321	127.00	2.10	60.38	28.71	3.00

Table G.3. Q_p, Q_p/V, Q_p/V^2 and T_p data for Umar Sub-Basin

No	Event	Catchment Area (km²) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Qp/V (m³/s/cm)	Qp/V² (m³/s/cm²)	Tp (hrs)
1	23.07.1962	223.77	33.6	0.25039	175.40	3.91	44.86	11.47	23.00
2	05.09.1962	223.77	33.6	0.25039	133.30	1.91	69.95	36.71	11.00
3	20.07.1964	223.77	33.6	0.25039	103.90	1.00	104.38	104.87	10.00
4	14.08.1964	223.77	33.6	0.25039	130.30	1.13	115.75	102.83	4.00
5	30.08.1965	223.77	33.6	0.25039	541.50	8.15	66.48	8.16	8.00
6	07.09.1965	223.77	33.6	0.25039	544.40	14.73	36.97	2.51	21.00

Table G.4. Q_p, Q_p/V, Q_p/V^2 and T_p data for Kolar Sub-Basin

No	Event	Catchment Area (km²) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Qp/V (m³/s/cm)	Qp/V² (m³/s/cm²)	Tp (hrs)
1	28-8-1983	903.88	75.34	0.53	4871.40	24.01	202.89	8.45	10
2	^10-8-1984	903.88	75.34	0.53	2032.63	7.44	273.20	36.72	10
3	31-7-1985	903.88	75.34	0.53	1293.00	5.22	247.70	47.45	8
4	13-8-1985	903.88	75.34	0.53	1381.70	4.47	309.11	69.15	11
5	15-8-1986	903.88	75.34	0.53	1968.10	6.54	300.93	46.01	11
6	27-8-1987	903.88	75.34	0.53	881.35	1.72	512.41	297.91	4

APPENDIX - G Continued

Table G.5. Q_p, Q_p/V, Q_p/V^2 and T_p data for 3f Sub-Zone

No	Event	Catchment Area (km²) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Qp/V (m³/s/cm)	Qp/V² (m³/s/cm²)	Tp (hrs)
1	1	823.62	61.08	0.124	543.80	1.16	468.72	404.01	6.00
2	2	823.62	61.08	0.124	478.30	1.26	379.48	301.08	5.00
3	3	823.62	61.08	0.124	324.50	0.61	529.64	864.48	5.00
4	4	823.62	61.08	0.124	402.10	0.88	454.42	513.56	4.00
5	5	823.62	61.08	0.124	247.00	0.85	290.09	340.70	8.00
6	6	823.62	61.08	0.124	1391.00	4.29	324.33	75.62	4.00
7	7	823.62	61.08	0.124	593.36	1.59	372.88	234.32	10.00
8	8	823.62	61.08	0.124	424.36	1.55	273.34	176.07	10.00
9	9	823.62	61.08	0.124	255.00	0.63	407.69	651.79	11.00
10	10	823.62	61.08	0.124	322.70	0.59	548.09	930.92	8.00
11	11	823.62	61.08	0.124	455.00	1.52	299.99	197.79	8.00
12	12	823.62	61.08	0.124	215.60	0.64	336.88	526.38	10.00
13	13	823.62	61.08	0.124	1432.00	4.98	287.59	57.76	7.00

Table G.6. Q_p, Q_p/V, Q_p/V^2 and T_p data for Gola Sub-Basin

No	Event	Catchment Area (km²) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Qp/V (m³/s/cm)	Qp/V² (m³/s/cm²)	Tp (hrs)
1	1977	450	23.5	1.4	450.43	8.26	54.50	6.59	14.00
2	1977	450	23.5	1.4	425.61	7.60	55.99	7.37	14.00
3	1978	450	23.5	1.4	285.67	6.32	45.23	7.16	14.00
4	1979	450	23.5	1.4	133.10	1.99	66.99	33.72	18.00
5	1980	450	23.5	1.4	116.59	1.80	64.62	35.82	32.00
6	1982	450	23.5	1.4	531.88	4.90	108.44	22.11	6.00
7	1984	450	23.5	1.4	245.00	1.71	143.53	84.08	3.00
8	1985	450	23.5	1.4	528.24	5.13	103.02	20.09	8.00
9	1986	450	23.5	1.4	699.00	17.62	39.67	2.25	18.00
10	1986	450	23.5	1.4	279.49	2.60	107.44	41.30	6.00

APPENDIX - H

q_p/V and q_p/V^2 DATA FOR DEVELOPMENT OF PEAK DISCHARGE DISTRIBUTION PER UNIT AREA

Table H.1. Qp/V and qp/V2 data for Temur Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	qp = Qp/A (cm/hr)	qp/v² (cm/hr/cm²)
1	23.07.1962	518.67	56.62	0.302	181.23	1.08	0.13	0.11
2	05.09.1962	518.67	56.62	0.302	135.92	0.67	0.09	0.21
3	20.07.1964	518.67	56.62	0.302	214.93	1.32	0.15	0.09
4	14.08.1964	518.67	56.62	0.302	124.59	0.81	0.09	0.13
5	30.08.1965	518.67	56.62	0.302	58.16	0.26	0.04	0.60
6	07.09.1965	518.67	56.62	0.302	59.50	0.31	0.04	0.43
7	24.08.1961	518.67	56.52	0.302	430.39	4.75	0.30	0.01

Table H.2. Qp/V and qp/V2 data for Teriya Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	qp = Qp/A (cm/hr)	qp/v² (cm/hr/cm²)
1	1977	114.22	35.42	0.305	161.00	4.32	0.51	0.027
2	1978	114.22	35.42	0.305	120.00	1.49	0.38	0.171
3	1979	114.22	35.42	0.305	82.50	1.79	0.26	0.081
4	1980	114.22	35.42	0.305	114.00	2.72	0.36	0.048
5	1981	114.22	35.42	0.305	118.00	2.38	0.37	0.066
6	1982	114.22	35.42	0.305	131.00	2.92	0.41	0.048
7	1984	114.22	35.42	0.305	89.00	2.07	0.28	0.065
8	1985	114.22	35.42	0.305	108.00	2.04	0.34	0.081
9	1986	114.22	35.42	0.305	305.00	4.40	0.96	0.050
10	1986	114.22	35.42	0.305	575.75	21.21	1.81	0.004
11	1986	114.22	35.42	0.305	127.00	2.10	0.40	0.090

Table H.3. Qp/V and qp/V2 data for Umar Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	qp = Qp/A (cm/hr)	qp/v² (cm/hr/cm²)
1	24.08.1961	223.77	33.6	0.306	195.50	2.56	0.31	0.048
2	23.07.1962	223.77	33.6	0.306	175.40	3.91	0.28	0.018
3	05.09.1962	223.77	33.6	0.306	133.30	1.91	0.21	0.059
4	20.07.1964	223.77	33.6	0.306	103.90	1.00	0.17	0.169
5	14.08.1964	223.77	33.6	0.306	130.30	1.13	0.21	0.165
6	30.08.1965	223.77	33.6	0.306	541.50	8.15	0.87	0.013
7	07.09.1965	223.77	33.6	0.306	544.40	14.73	0.88	0.004

APPENDIX - H Continued

Table H.4. Qp/V and qp/V2 data for Kolar Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	qp = Qp/A (cm/hr)	qp/v^2 (cm/hr/cm^2)
1	28-8-1983	903.88	75.34	0.53	4871.40	24.01	1.94	0.003
2	10-8-1984	903.88	75.34	0.53	2032.63	7.44	0.81	0.015
3	31-7-1985	903.88	75.34	0.53	1293.00	5.22	0.51	0.019
4	13-8-1985	903.88	75.34	0.53	1381.70	4.47	0.55	0.028
5	15-8-1986	903.88	75.34	0.53	1968.10	6.54	0.78	0.018
6	27-8-1987	903.88	75.34	0.53	881.35	1.72	0.35	0.119

Table H.5. Qp/V and qp/V2 data for 3f Sub-Zone

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	qp= Qp/A (cm/hr)	qp/V^2 (cm/hr/cm^2)
1	1	823.62	61.08	0.124	360.30	1.05	0.16	0.14
2	1	823.62	61.08	0.124	543.80	1.16	0.24	0.18
3	2	823.62	61.08	0.124	478.30	1.26	0.21	0.13
4	3	823.62	61.08	0.124	324.50	0.61	0.14	0.38
5	4	823.62	61.08	0.124	402.10	0.88	0.18	0.22
6	5	823.62	61.08	0.124	247.00	0.85	0.11	0.15
7	6	823.62	61.08	0.124	1391.00	4.29	0.61	0.03
8	7	823.62	61.08	0.124	322.70	0.59	0.14	0.41
9	8	823.62	61.08	0.124	455.00	1.52	0.20	0.09
10	9	823.62	61.08	0.124	215.60	0.64	0.09	0.23
11	10	823.62	61.08	0.124	1432.00	4.98	0.63	0.03

Table H.6. Qp/V and qp/V2 data for Gola Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	qp = Qp/A (cm/hr)	qp/v^2 (cm/hr/cm^2)
1	1977	450	23.5	1.4	450.43	8.26	0.36	0.005
2	1977	450	23.5	1.4	425.61	7.60	0.34	0.006
3	1978	450	23.5	1.4	285.67	6.32	0.23	0.006
4	1979	450	23.5	1.4	133.10	1.99	0.11	0.027
5	1980	450	23.5	1.4	116.59	1.80	0.09	0.029
6	1982	450	23.5	1.4	531.88	4.90	0.43	0.018
7	1984	450	23.5	1.4	245.00	1.71	0.20	0.067
8	1995	450	23.5	1.4	528.24	5.13	0.42	0.016
9	1986	450	23.5	1.4	699.00	17.62	0.56	0.002
10	1986	450	23.5	1.4	279.49	2.60	0.22	0.033

APPENDIX - I

ANALYSIS OF VARIANCE (AOV)

I.1. Analysis of Variance (AOV) for Temur Sub-basin

No	Event	Runoff Volume (cm) V	Peak Discharge (cumec) Qp	Log V x	Log Qp y	x_i^2	y_i^2	$(x_i-\bar{x})$	$(x_i-\bar{x})^2$	$(y_i-\bar{y})$	$(y_i-\bar{y})^2$	$x_i y_i$
1	1	4.75	430.39	0.677	2.634	0.458	6.937	0.752	0.566	0.492	0.242	1.782
2	2	1.08	181.23	0.033	2.258	0.001	5.100	0.109	0.012	0.116	0.014	0.075
3	3	0.67	135.92	-0.174	2.133	0.030	4.551	-0.098	0.010	-0.008	0.000	-0.371
4	4	1.32	214.93	0.121	2.332	0.015	5.440	0.196	0.038	0.191	0.036	0.281
5	5	0.81	124.59	-0.092	2.095	0.008	4.391	-0.016	0.000	-0.046	0.002	-0.192
6	6	0.26	58.16	-0.585	1.765	0.342	3.114	-0.510	0.260	-0.377	0.142	-1.032
7	7	0.31	59.50	-0.509	1.775	0.259	3.149	-0.433	0.188	-0.367	0.135	-0.903
		Total		-0.528	14.992	1.113	32.681	0.000	1.073	0.000	0.571	-0.359
		Average		-0.0755	2.1418							

n = 7

$\bar{y}$ = 2.1418

$\bar{x}$ = -0.0755

$$\beta_1 = \frac{\sum x_i y_i - \bar{x}\sum y_i}{\sum x_i^2 - n\bar{x}^2}$$

= 0.7202

$$\beta_0 = \bar{y} - \beta_1\bar{x}$$

= 2.1961

Therefore, the least squares lineal prediction is

$n\bar{x}^2$ = 0.0399

$n\bar{y}^2$ = 32.1099

$\sum_{i=1}^{n} y_i^2$ = 32.6812

$$\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2 = \beta_1^2 \sum (x_i - \bar{x})^2$$

= 0.5567

$$\sum_{i=1}^{n}(y_i - \hat{y}_i)^2 = \sum e_i^2 = \sum_{i=1}^{n} y_i^2 - n\bar{y}^2 - \sum(\hat{y} - \bar{y})^2$$

= 0.0146

$$R^2 = \frac{\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2}$$

= 0.9744

$$s^2 = \frac{(residual.SSQ)}{(n-2)}$$

= 0.0029

$$\hat{y} = 2.1961 + 0.7202\ X$$

AOV Table

Source	df	SSQ	MS	cal F
Total	7	32.6812		
Mean	1	32.1099		
$R(\beta_1 \backslash \beta_0)$	1	0.5567	0.5567	190.602
Residual	5	0.0146	0.0029	

And From table in Appendix - L, we get
$F^*=(1, 5, 0.95)$ = 6.610
cal F = 190.602 > $F^*(1, 5, 0.95)$ = 6.610

Since cal F = 190.602 > $F^*(1, 5, 0.95)$ = 6.610 reject the hypothesis that β_1 = 0 and leave the β_1X term in the linear model.

Since calculated F is more than critical F* at 5% significance level and R2 is nearly one, the linear model :

$$\hat{y} = 2.1961 + 0.7202\ X$$

is satisfactory. The residual S S Q is 0.0146

APPENDIX - I Continued

I.2. Analysis of Variance (AOV) for Teriya Sub-basin

No	Event	Runoff Volume (cm) V	Peak Discharge (cumec) Qp	Log V x	Log Qp y	x_i^2	y_i^2	$(x_i - \bar{x})$	$(x_i - \bar{x})^2$	$(y_i - \bar{y})$	$(y_i - \bar{y})^2$	$x_i y_i$
1	1977	4.32	161.00	0.635	2.207	0.404	4.870	0.157	0.025	0.045	0.002	1.402
2	1978	1.49	120.00	0.173	2.079	0.030	4.323	-0.306	0.093	-0.083	0.007	0.359
3	1979	1.79	82.50	0.253	1.916	0.064	3.673	-0.225	0.051	-0.245	0.060	0.485
4	1980	2.72	114.00	0.435	2.057	0.189	4.231	-0.043	0.002	-0.105	0.011	0.895
5	1981	2.38	118.00	0.377	2.072	0.142	4.293	-0.101	0.010	-0.090	0.008	0.781
6	1982	2.92	131.00	0.465	2.117	0.216	4.483	-0.013	0.000	-0.045	0.002	0.985
7	1984	2.07	89.00	0.317	1.949	0.100	3.800	-0.161	0.026	-0.212	0.045	0.618
8	1985	2.04	108.00	0.310	2.033	0.096	4.135	-0.168	0.028	-0.128	0.016	0.631
9	1986	4.40	305.00	0.643	2.484	0.414	6.172	0.165	0.027	0.323	0.104	1.598
10	1986	21.21	575.75	1.327	2.760	1.760	7.619	0.849	0.720	0.598	0.358	3.662
11	1986	2.10	127	0.323	2.104	0.104	4.426	-0.155	0.024	-0.058	0.003	0.679
		Total		5.258	23.780	3.520	52.024	0.000	1.006	0.000	0.617	12.095
		Average		0.4780	2.1618							

n = 11

$\bar{y}$ = 2.1618

$\bar{x}$ = 0.4780

$$\beta_1 = \frac{\sum x_i y_i - \bar{x}\sum y_i}{\sum x_i^2 - n\bar{x}^2}$$

= 0.7229

$$\beta_0 = \bar{y} - \beta_1 \bar{x}$$

= 1.8162

Therefore, the least squares line[ar] prediction is

$$\hat{y} = 1.8162 + 0.7229\ X$$

$n\bar{x}^2$ = 2.5135

$n\bar{y}^2$ = 51.4066

$\sum_{i=1}^{n} y_i^2$ = 52.0238

$$\sum_{i=1}^{n} (\hat{y}_i - \bar{y})^2 = \beta_1^2 \sum (x_i - \bar{x})^2$$

= 0.5259

$$\sum_{i=1}^{n} (y_i - \hat{y}_i)^2 = \sum e_i^2$$

$$= \sum_{i=1}^{n} y_i^2 - n\bar{y}^2 - \sum (\hat{y} - \bar{y})^2$$

= 0.0913

$$R^2 = \frac{\sum_{i=1}^{n} (\hat{y}_i - \bar{y})^2}{\sum_{i=1}^{n} (y_i - \bar{y})^2}$$

= 0.8520

$$s^2 = \frac{(residual\ SSQ)}{(n-2)}$$

= 0.0101

AOV Table

Source	df	SSQ	MS	cal F
Total	11	52.0238		
Mean	1	51.4066		
$R(\beta_1 \backslash \beta_0)$	1	0.5259	0.5259	51.8152
Residual	9	0.0913	0.0101	

And From table in Appendix - L, we get
$F^*(1, 9, 0.95)$ = 5.120
cal F = 51.8152 > $F^*(1, 9, 0.95)$ = 5.120

Since cal F = 51.8152 > $F^*(1, 9, 0.95)$ = 5.120 reject the hypothesis that β_1 = 0 and leave the $\beta_1 X$ term in the linear model.

Since calculated F is more than critical F* at 5% significance level and R2 is nearly one, the linear model :

$$\hat{y} = 1.8162 + 0.7229\ X$$

is satisfactory. The residual S S Q is 0.0913

APPENDIX - I Continued

I.3. Analysis of Variance (AOV) for Umar Sub-basin

No	Event	Runoff Volume (cm) V	Peak Discharge (cumec) Qp	Log V x	Log Qp y	x_i^2	y_i^2	$(x_i-\bar{x})$	$(x_i-\bar{x})^2$	$(y_i-\bar{y})$	$(y_i-\bar{y})^2$	$x_i y_i$
1	23.07.1962	3.91	175.40	0.592	2.244	0.351	5.036	0.092	0.008	-0.084	0.007	1.329
2	05.09.1962	1.91	133.30	0.280	2.125	0.078	4.515	-0.220	0.048	-0.203	0.041	0.595
3	20.07.1964	1.00	103.90	-0.002	2.017	0.000	4.067	-0.502	0.252	-0.312	0.097	-0.004
4	14.08.1964	1.13	130.30	0.051	2.115	0.003	4.473	-0.449	0.201	-0.213	0.046	0.109
5	30.08.1965	8.15	541.50	0.911	2.734	0.830	7.473	0.411	0.169	0.405	0.164	2.490
6	07.09.1965	14.73	544.40	1.168	2.736	1.364	7.485	0.668	0.446	0.408	0.166	3.196
		Total		3.001	13.970	2.626	33.048	0.000	1.125	0.000	0.522	7.714
		Average		0.5001	2.3283							

n = 6

$\bar{y}$ = 2.3283

$\bar{x}$ = 0.5001

$$\beta_1 = \frac{\sum x_i y_i - \bar{x}\sum y_i}{\sum x_i^2 - n\bar{x}^2}$$

= 0.6470

$$\beta_0 = \bar{y} - \beta_1 \bar{x}$$

= 2.0048

Therefore, the least squares linear prediction is

$$\hat{y} = 2.0048 + 0.6470\ X$$

$n\bar{x}^2$ = 1.5006

$n\bar{y}^2$ = 32.5265

$\sum_{i=1}^{n} y_i^2$ = 33.0481

$$\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2 = \beta_1^2 \sum (x_i - \bar{x})^2$$

= 0.4710

$$\sum_{i=1}^{n}(y_i - \hat{y}_i)^2 = \sum e_i^2 = \sum_{i=1}^{n} y_i^2 - n\bar{y}^2 - \sum(\hat{y} - \bar{y})^2$$

= 0.0506

$$R^2 = \frac{\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2}$$

= 0.9030

$$s^2 = \frac{(residual\ SSQ)}{(n-2)}$$

= 0.0126

AOV Table

Source	df	SSQ	MS	cal F
Total	6	33.0481		
Mean	1	32.5265		
$R(\beta_1 \backslash \beta_0)$	1	0.4710	0.4710	37.2527
Residual	4	0.0506	0.0126	

And From table in Appendix - L, we get
F*=(1, 4, 0.95) = 7.710
cal F = 37.2527 > F*(1, 4, 0.95) = 7.710

Since cal F = 37.2527 > F*(1, 4, 0.95) = 7.710 reject the hypothesis that β_1 = 0 and leave the β_1X term in the linear model.

Since calculated F is more than critical F* at 5% significance level and R^2 is nearly one, the linear model :

$$\hat{y} = 2.0048 + 0.6470\ X$$

is satisfactory. The residual S S Q is 0.0506

APPENDIX - I Continued

I.4. Analysis of Variance (AOV) for Kolar Sub-basin

No	Event	Runoff Volume (cm) V	Peak Discharge (cumec) Qp	Log V x	Log Qp y	x_i^2	y_i^2	$(x_i-\bar{x})$	$(x_i-\bar{x})^2$	$(y_i-\bar{y})$	$(y_i-\bar{y})^2$	$x_i y_i$
1	1977	24.01	4871.40	1.380	3.688	1.905	13.599	0.602	0.362	0.440	0.193	5.090
2	1978	7.44	2032.63	0.872	3.308	0.760	10.943	0.093	0.009	0.060	0.004	2.883
3	1979	5.22	1293.00	0.718	3.112	0.515	9.682	-0.061	0.004	-0.136	0.019	2.233
4	1980	4.47	1381.70	0.650	3.140	0.423	9.862	-0.128	0.016	-0.107	0.012	2.042
5	1981	6.54	1968.10	0.816	3.294	0.665	10.851	0.037	0.001	0.046	0.002	2.687
6	1982	1.72	881.35	0.236	2.945	0.055	8.674	-0.543	0.295	-0.303	0.092	0.694
		Total		4.671	19.487	4.324	63.611	0.000	0.687	0.000	0.321	15.629
		Average		0.7785	3.2478							

n = 6

$\bar{y}$ = 3.2478

$\bar{x}$ = 0.7785

$$\beta_1 = \frac{\sum x_i y_i - \bar{x}\sum y_i}{\sum x_i^2 - n\bar{x}^2}$$

= 0.6671

$$\beta_0 = \bar{y} - \beta_1\bar{x}$$

= 2.7285

$n\bar{x}^2$ = 3.6365

$n\bar{y}^2$ = 63.2900

$\sum_{i=1}^{n} y_i^2$ = 63.6109

$$\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2 = \beta_1^2\sum(x_i - \bar{x})^2$$

= 0.3058

$$\sum_{i=1}^{n}(y_i - \hat{y}_i)^2 = \sum e_i^2$$

$$= \sum_{i=1}^{n} y_i^2 - n\bar{y}^2 - \sum(\hat{y} - \bar{y})^2$$

= 0.0151

$$R^2 = \frac{\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2}$$

= 0.9529

$$s^2 = \frac{(residual.SSQ)}{(n-2)}$$

= 0.0038

Therefore, the least squares lineal prediction is

$$\hat{y} = 2.7285 + 0.667\ X$$

AOV Table

Source	df	SSQ	MS	cal F
Total	6	63.6109		
Mean	1	63.2900		
$R(\beta_1 \backslash \beta_0)$	1	0.3058	0.3058	80.9829
Residual	4	0.0151	0.0038	

And From table in Appendix - L, we get
F*=(1, 4, 0.95) = 7.710
cal F = 80.9829 > F*(1, 4, 0.95) = 7.710

Since cal F = 80.8929 > F*(1, 4, 0.95) = 7.710
reject the hypothesis that β_1 = 0 and leave the β_1X term in the linear model.

Since calculated F is more than critical F* at 5% significance level and R^2 is nearly one, the linear model :

$$\hat{y} = 2.7285 + 0.667\ X$$

is satisfactory. The residual S S Q is 0.0151

APPENDIX - I Continued

I.5. Analysis of Variance (AOV) for 3f Sub-zone

No	Event	Runoff Volume (cm) V	Peak Discharge (cumec) Qp	Log V x	Log Qp y	x_i^2	y_i^2	$(x_i-\bar{x})$	$(x_i-\bar{x})^2$	$(y_i-\bar{y})$	$(y_i-\bar{y})^2$	$x_i y_i$
1	1	1.16	543.80	0.065	2.735	0.004	7.483	-0.020	0.000	0.079	0.006	0.177
2	2	1.26	478.30	0.101	2.680	0.010	7.181	0.016	0.000	0.023	0.001	0.269
3	3	0.61	324.50	-0.213	2.511	0.045	6.306	-0.298	0.089	-0.145	0.021	-0.534
4	4	0.88	402.10	-0.053	2.604	0.003	6.783	-0.138	0.019	-0.052	0.003	-0.138
5	5	0.85	247.00	-0.070	2.393	0.005	5.725	-0.155	0.024	-0.264	0.069	-0.167
6	6	4.29	1391.00	0.632	3.143	0.400	9.881	0.547	0.300	0.487	0.237	1.988
7	7	1.59	593.36	0.201	2.773	0.041	7.691	0.116	0.014	0.117	0.014	0.559
8	8	1.55	424.36	0.191	2.628	0.036	6.905	0.106	0.011	-0.028	0.001	0.502
9	9	0.63	255.00	-0.204	2.407	0.042	5.791	-0.289	0.083	-0.250	0.062	-0.490
10	10	0.59	322.70	-0.230	2.509	0.053	6.294	-0.315	0.099	-0.147	0.022	-0.577
11	11	1.52	455.00	0.181	2.658	0.033	7.065	0.096	0.009	0.002	0.000	0.481
12	12	0.64	215.60	-0.194	2.334	0.038	5.446	-0.279	0.078	-0.323	0.104	-0.452
13	13	4.98	1432.00	0.697	3.156	0.486	9.960	0.612	0.375	0.500	0.250	2.200
		Total		1.104	34.531	1.195	92.510	0.000	1.101	0.000	0.790	3.815
		Average		0.0850	2.6562							

n = 13

$\bar{y}$ = 2.6562

$\bar{x}$ = 0.0850

$$\beta_1 = \frac{\sum x_i y_i - \bar{x}\sum y_i}{\sum x_i^2 - n\bar{x}^2}$$

= 0.8007

$$\beta_0 = \bar{y} - \beta_1\bar{x}$$

= 2.5882

Therefore, the least squares lineal prediction is

$$\hat{y} = 2.5882 + 0.8007\,X$$

$n\bar{x}^2$ = 0.0938

$n\bar{y}^2$ = 91.7208

$\sum_{i=1}^{n} y_i^2$ = 92.5104

$$\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2 = \beta_1^2\sum(x_i - \bar{x})^2$$

= 0.7060

$$\sum_{i=1}^{n}(y_i - \hat{y}_i)^2 = \sum e_i^2$$

$$= \sum_{i=1}^{n} y_i^2 - n\bar{y}^2 - \sum(\hat{y} - \bar{y})^2$$

= 0.0836

$$R^2 = \frac{\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2}{\sum_{i=1}^{n}(y_i - \bar{y})^2}$$

= 0.8941

$$s^2 = \frac{(residual\ SSQ)}{(n-2)}$$

= 0.0076

AOV Table

Source	df	SSQ	MS	cal F
Total	13	92.5104		
Mean	1	91.7208		
$R(\beta_1 \backslash \beta_0)$	1	0.7060	0.7060	92.8528
Residual	11	0.0836	0.0076	

And From table in Appendix - L, we get
$F^*(1, 11, 0.95)$ = 4.840
cal F = 92.8528 > $F^*(1, 11, 0.95)$ = 4.840

Since cal F = 92.8454 > $F^*(1, 11, 0.95)$ = 4.840 reject the hypothesis that β_1 = 0 and leave the $\beta_1 X$ term in the linear model.

Since calculated F is more than critical F* at 5% significance level and R^2 is nearly one, the linear model :

$$\hat{y} = 2.5882 + 0.8007\,X$$

is satisfactory. The residual S S Q is 0.0836

APPENDIX - I Continued

I.6. Analysis of Variance (AOV) for Gola Sub-basin

No	Event	Runoff Volume (cm) V	Peak Discharge (cumec) Qp	Log V x	Log Qp y	x_i^2	y_i^2	$(x_i-\bar{x})$	$(x_i-\bar{x})^2$	$(y_i-\bar{y})$	$(y_i-\bar{y})^2$	$x_i y_i$
1	1	8.26	450.43	0.917	2.654	0.841	7.042	0.273	0.074	0.148	0.022	2.434
2	2	7.60	425.61	0.881	2.629	0.776	6.912	0.236	0.056	0.123	0.015	2.316
3	3	6.32	285.67	0.800	2.456	0.641	6.031	0.156	0.024	-0.050	0.002	1.966
4	4	1.99	133.10	0.298	2.124	0.089	4.512	-0.347	0.120	-0.382	0.146	0.633
5	5	1.80	116.59	0.256	2.067	0.066	4.271	-0.388	0.151	-0.439	0.193	0.530
6	6	4.90	531.88	0.691	2.726	0.477	7.430	0.046	0.002	0.220	0.048	1.883
7	7	1.71	245.00	0.232	2.389	0.054	5.708	-0.412	0.170	-0.117	0.014	0.555
8	8	5.13	528.24	0.710	2.723	0.504	7.414	0.065	0.004	0.217	0.047	1.933
9	9	17.62	699.00	1.246	2.844	1.553	8.091	0.601	0.362	0.339	0.115	3.544
10	10	2.60	279.49	0.415	2.446	0.172	5.985	-0.229	0.053	-0.059	0.004	1.016
		Total		6.447	25.058	5.172	63.396	0.000	1.016	0.000	0.605	16.809
		Average		0.6447	2.5058							

n = 10

$\bar{y}$ = 2.5058

$\bar{x}$ = 0.6447

$n\bar{x}^2$ = 4.1562

$n\bar{y}^2$ = 62.7902

$\sum_{i=1}^{n} y_i^2$ = 63.3956

$$\beta_1 = \frac{\sum x_i y_i - \bar{x}\sum y_i}{\sum x_i^2 - n\bar{x}^2}$$

= 0.6438

$$\sum_{i=1}^{n} (\hat{y}_i - \bar{y})^2 = \beta_1^2 \sum (x_i - \bar{x})^2$$

= 0.4211

$$R^2 = \frac{\sum_{i=1}^{n} (\hat{y}_i - \bar{y})^2}{\sum_{i=1}^{n} (y_i - \bar{y})^2}$$

= 0.6956

$$\beta_0 = \bar{y} - \beta_1 \bar{x}$$

= 2.0908

$$\sum_{i=1}^{n} (y_i - \hat{y}_i)^2 = \sum e_i^2 = \sum_{i=1}^{n} y_i^2 - n\bar{y}^2 - \sum (\hat{y} - \bar{y})^2$$

= 0.1843

$$s^2 = \frac{(residual.SSQ)}{(n-2)}$$

= 0.0230

Therefore, the least squares line prediction is

$$\hat{y} = 2.0908 + 0.644\ X$$

AOV Table

Source	df	SSQ	MS	cal F
Total	10	63.3956		
Mean	1	62.7902		
$R(\beta_1 \backslash \beta_0)$	1	0.4211	0.4211	18.2818
Residual	8	0.1843	0.0230	

And From table in Appendix - L, we get
$F^*=(1, 8, 0.95)$ = 5.320
so, cal F = 18.282 > $F^*(1, 8, 0.95)$

Since cal F = 18.2818 > $F^*(1, 8, 0.95)$ = 5.320 reject the hypothesis that $\beta 1$ = 0 and leave the $\beta 1 X$ term in the linear model.

Since calculated F is more than critical F* at 5% significance level and R2 is nearly one, the linear model :

$$\hat{y} = 2.0908 + 0.644\ X$$

is satisfactory. The residual S S Q is 0.1843

APPENDIK – J

REGRESSION ANALYSIS BETWEEN Q_p, T_p and V

J.1. Regression Analysis Between Q_p, T_p and V for Temur Sub-basin

No	Event	Peak Discharge (cumec) Qp	tp (hrs)	Runoff volume (cm) V	Log Qp (X1)	Log Tp (X2)	Log V (X3)	X1 X2	X1 X3	X2 X3	$X2^2$	$X3^2$
1	24.08.1961	430.39	8.00	4.75	2.634	0.903	0.677	2.379	1.782	0.611	0.816	0.458
2	23.07.1962	181.23	10.00	1.08	2.258	1.000	0.033	2.258	0.075	0.033	1.000	0.001
3	05.09.1962	135.92	6.00	0.67	2.133	0.778	-0.174	1.660	-0.371	-0.135	0.606	0.030
4	20.07.1964	214.93	8.00	1.32	2.332	0.903	0.121	2.106	0.281	0.109	0.816	0.015
5	14.08.1964	124.59	9.00	0.81	2.095	0.954	-0.092	2.000	-0.192	-0.087	0.911	0.008
6	30.08.1965	58.16	8.00	0.26	1.765	0.903	-0.585	1.594	-1.032	-0.528	0.816	0.342
7	07.09.1965	59.50	6.00	0.31	1.775	0.778	-0.509	1.381	-0.903	-0.396	0.606	0.259
			55.00		14.992	6.220	-0.528	13.377	-0.359	-0.393	5.568	1.113

For relationship between Qp, Tp and V we use multiple regresion
X1 = b' + b".X2 + m'.X3
where a, b, and c are constants and They can be estimated by least square method, by the following equation

$$\sum X_1 = Nb' + b''\sum X_2 + m'\sum X_3$$
$$\sum X_1X_2 = b'\sum X_2 + b''\sum X_2^2 + m'\sum X_2X_3$$
$$\sum X_1X_3 = b'\sum X_3 + b''\sum X_2X_3 + m'\sum X_3^2$$

From above equation and table we get equation as follow :
1. 14.992 = 7b' + 6.220b" - 0.528m'
2. 13.377 = 6.220b' + 5.568b" - 0.393m'
3. -0.359 = -0.528b' - 0.393b" + 1.113m'

And with Gauss-Seidel Iteration we get :
b' = 2.1769
b" = 0.0212
m' = 0.7176

So. Equation of best fit relationship Qp. Tp and V is :
X1 = 2.1769 + 0.0212X2 + 0.7176X3
or
Log Qp = 2.1769 + 0.0212 Log Tp + 0.7176 Log V

Assumption time to peak(tp) to be same for all the event and taken as average tp =

$$\frac{\sum tp}{N} = \frac{55}{7} = 7.86 hours$$

Then : Log Qp = 2.1769 +0.0212 Log 7.86 + 0.7176 Log V
or
Log Qp = 2.1959 + 0.7176 Log V

Partial Correlation Coefficient :
1 Log Qp and Log Tp = 0.0441
2 Log Qp and Log V = 0.9852
Multiple Correlation Coefficient = 0.9872

APPENDIK – J Continued

J.2. Regression Analysis Between Q_p, T_p and V for Teriya Sub-basin

No	Event	Peak Discharge (cumec) Qp	tp (hrs)	Runoff volume (cm) V	Log Qp (X1)	Log Tp (X2)	Log V (X3)	X1 X2	X1 X3	X2 X3	$X2^2$	$X3^2$
1	1977	161.00	5.00	4.32	2.207	0.699	0.635	1.543	1.402	0.444	0.489	0.404
2	1978	120.00	5.00	1.49	2.079	0.699	0.173	1.453	0.359	0.121	0.489	0.030
3	1979	82.50	5.00	1.79	1.916	0.699	0.253	1.340	0.485	0.177	0.489	0.064
4	1980	114.00	6.00	2.72	2.057	0.778	0.435	1.601	0.895	0.339	0.606	0.189
5	1981	118.00	7.00	2.38	2.072	0.845	0.377	1.751	0.781	0.319	0.714	0.142
6	1982	131.00	6.00	2.92	2.117	0.778	0.465	1.648	0.985	0.362	0.606	0.216
7	1984	89.00	7.00	2.07	1.949	0.845	0.317	1.647	0.618	0.268	0.714	0.100
8	1985	108.00	8.00	2.04	2.033	0.903	0.310	1.836	0.631	0.280	0.816	0.096
9	1986	305.00	5.00	4.40	2.484	0.699	0.643	1.736	1.598	0.450	0.489	0.414
10	1986	575.75	7.00	21.21	2.760	0.845	1.327	2.333	3.662	1.121	0.714	1.760
11	1986	127.00	3.00	2.10	2.104	0.477	0.323	1.004	0.679	0.154	0.228	0.104
			64.00		23.780	8.268	5.258	17.891	12.095	4.034	6.351	3.520

For relationship between Qp, Tp and V we use multiple regresion
X1 = b' + b".X2 + m'.X3
where b', b", and m' are constants and They can be estimated by least square method, by the following equation

$$\sum X_1 = Nb' + b''\sum X_2 + m'\sum X_3$$
$$\sum X_1X_2 = b'\sum X_2 + b''\sum X_2^2 + m'\sum X_2X_3$$
$$\sum X_1X_3 = b'\sum X_3 + b''\sum X_2X_3 + m'\sum X_3^2$$

From above equation and table we get equation as follow :
1. 23.780 = 11b' + 8.268b" +5.258m'
2. 17.891 = 8.268b' + 6.351b" + 4.034m'
3. 12.095 = 5.258b' + 4.034b" + 3.520m'

And with Gauss-Seidel Iteration we get :
b' = 2.0475
b" = -0.3240
m' = 0.7497

So. Equation of best fit relationship Qp. Tp and V is :
X1 = 2.0475 - 0.3240X2 + 0.7497X3
or
Log Qp = 2.0475 - 0.3240 Log Tp + 0.7497 Log V

Assumption time to peak(tp) to be same for all the event and taken as average tp =

$$\frac{\sum tp}{N} = \frac{64}{11} = 5.82 hours$$

Then : Log Qp = 2.0475 - 0.3240 Log 5.82 + 0.7497 Log V
or
Log Qp = 1.7997 + 0.7497 Log V

Partial Correlation Coefficient :
1 Log Qp and Log Tp = -0.3738
2 Log Qp and Log V = 0.9339
Multiple Correlation Coefficient = 0.9342

APPENDIK – J Continued

J.3. Regression Analysis Between Q_p, T_p and V for Umar Sub-basin

No	Event	Peak Discharge (cumec) Qp	tp (hrs)	Runoff volume (cm) V	Log Qp (X1)	Log Tp (X2)	Log V (X3)	X1 X2	X1 X3	X2 X3	$X2^2$	$X3^2$
1	23.07.1962	175.40	23.00	3.91	2.244	1.362	0.592	3.056	1.329	0.806	1.854	0.351
2	05.09.1962	133.30	11.00	1.91	2.125	1.041	0.280	2.213	0.595	0.292	1.084	0.078
3	20.07.1964	103.90	10.00	1.00	2.017	1.000	-0.002	2.017	-0.004	-0.002	1.000	0.000
4	14.08.1964	130.30	4.00	1.13	2.115	0.602	0.051	1.273	0.109	0.031	0.362	0.003
5	30.08.1965	541.50	8.00	8.15	2.734	0.903	0.911	2.469	2.490	0.823	0.816	0.830
6	07.09.1965	544.40	21.00	14.73	2.736	1.322	1.168	3.617	3.196	1.544	1.748	1.364
			77.00		13.970	6.230	3.001	14.645	7.714	3.494	6.865	2.626

For relationship between Qp, Tp and V we use multiple regresion
X1 = b' + b''.X2 + m'.X3
where b', b'', and m' are constants and They can be estimated by least square method, by the following equation

$$\sum X_1 = Nb' + b''\sum X_2 + m'\sum X_3$$

$$\sum X_1X_2 = b'\sum X_2 + b''\sum X_2^2 + m'\sum X_2X_3$$

$$\sum X_1X_3 = b'\sum X_3 + b''\sum X_2X_3 + m'\sum X_3^2$$

From above equation and table we get equation as follow :
1. 13.970 = 6b' + 6.230b'' +3.001m'
2. 14.645 = 6.230b' + 6.865b'' + 3.494m'
3. 7.714 = 3.001b' + 3.494b'' + 2.626m'

And with Gauss-Seidel Iteration we get :
b' = 2.3435
b'' = -0.3887
m' = 0.7765

So. Equation of best fit relationship Qp. Tp and V is :
X1 = 2.3434 - 0.3887X2 + 0.7765X3
or
Log Qp = 2.3435 - 0.3887 Log Tp + 0.7765 Log V

Assumption time to peak(tp) to be same for all the event and taken as average tp =

$$\frac{\sum tp}{N} = \frac{77}{6} = 12.83 hours$$

Then : Log Qp = 2.3435 - 0.3887 Log 12.83 + 0.7765 Log V
or
Log Qp = 1.9127 + 0.7765 Log V

Partial Correlation Coefficient :
1 Log Qp and Log Tp = -0.9153
2 Log Qp and Log V = 0.9913
Multiple Correlation Coefficient = 0.9921

J.4. Regression Analysis Between Q_p, T_p and V for Kolar Sub-basin

No	Event	Peak Discharge (cumec) Qp	tp (hrs)	Runoff volume (cm) V	Log Qp (X1)	Log Tp (X2)	Log V (X3)	X1 X2	X1 X3	X2 X3	$X2^2$	$X3^2$
1	28-8-1983	4871.40	10.00	24.01	3.688	1.000	1.380	3.688	5.090	1.380	1.000	1.905
2	^10-8-1984	2032.63	10.00	7.44	3.308	1.000	0.872	3.308	2.883	0.872	1.000	0.760
3	31-7-1985	1293.00	8.00	5.22	3.112	0.903	0.718	2.810	2.233	0.648	0.816	0.515
4	13-8-1985	1381.70	11.00	4.47	3.140	1.041	0.650	3.270	2.042	0.677	1.084	0.423
5	15-8-1986	1968.10	11.00	6.54	3.294	1.041	0.816	3.430	2.687	0.849	1.084	0.665
6	27-8-1987	881.35	4.00	1.72	2.945	0.602	0.236	1.773	0.694	0.142	0.362	0.055
			54.00		19.487	5.588	4.671	18.280	15.629	4.568	5.347	4.324

For relationship between Qp, Tp and V we use multiple regresion
X1 = b' + b".X2 + m'.X3
where b', b", and m' are constants and They can be estimated by least square method, by the following equation

$$\sum X_1 = Nb' + b''\sum X_2 + m'\sum X_3$$

$$\sum X_1 X_2 = b'\sum X_2 + b''\sum X_2^2 + m'\sum X_2 X_3$$

$$\sum X_1 X_3 = b'\sum X_3 + b''\sum X_2 X_3 + m'\sum X_3^2$$

From above equation and table we get equation as follow :
1. 19.487 = 6b' + 5.588b" + 4.671m'
2. 18.280 = 5.588b' + 5.347b" + 4.568m'
3. 15.629 = 4.671b' + 4.568b" + 4.324m'

And with Gauss-Seidel Iteration we get :
b' = 2.8592
b" = -0.1903
m' = 0.7269

So. Equation of best fit relationship Qp. Tp and V is :
X1 = 2.8592 - 0.1903X2 + 0.7269X3
or
Log Qp = 2.8592 - 0.1903 Log Tp + 0.7269 Log V

Assumption time to peak(tp) to be same for all the event and taken as average tp =

$$\frac{\sum tp}{N} = \frac{54}{6} = 9\ hours$$

Then : Log Qp = 2.8592 - 0.1903 Log 9 + 0.7269 Log V
or
Log Qp = 2.6776 + 0.7269 Log V

Partial Correlation Coefficient :
1 Log Qp and Log Tp = -0.4330
2 Log Qp and Log V = 0.9689
Multiple Correlation Coefficient = 0.9807

APPENDIK – J Continued

J.5. Regression Analysis Between Q_p, T_p and V for 3f Sub-Zone

No	Event	Peak Discharge (cumec) Qp	tp (hrs)	Runoff volume (cm) V	Log Qp (X1)	Log Tp (X2)	Log V (X3)	X1 X2	X1 X3	X2 X3	$X2^2$	$X3^2$
1	1	543.80	6.00	1.16	2.735	0.778	0.065	2.129	0.177	0.050	0.606	0.004
2	2	478.30	5.00	1.26	2.680	0.699	0.101	1.873	0.269	0.070	0.489	0.010
3	3	324.50	5.00	0.61	2.511	0.699	-0.213	1.755	-0.534	-0.149	0.489	0.045
4	4	402.10	4.00	0.88	2.604	0.602	-0.053	1.568	-0.138	-0.032	0.362	0.003
5	5	247.00	8.00	0.85	2.393	0.903	-0.070	2.161	-0.167	-0.063	0.816	0.005
6	6	1391.00	4.00	4.29	3.143	0.602	0.632	1.892	1.988	0.381	0.362	0.400
7	7	593.36	10.00	1.59	2.773	1.000	0.201	2.773	0.559	0.201	1.000	0.041
8	8	424.36	10.00	1.55	2.628	1.000	0.191	2.628	0.502	0.191	1.000	0.036
9	9	255.00	11.00	0.63	2.407	1.041	-0.204	2.506	-0.490	-0.212	1.084	0.042
10	10	322.70	8.00	0.59	2.509	0.903	-0.230	2.266	-0.577	-0.208	0.816	0.053
11	11	455.00	8.00	1.52	2.658	0.903	0.181	2.400	0.481	0.163	0.816	0.033
12	12	215.60	10.00	0.64	2.334	1.000	-0.194	2.334	-0.452	-0.194	1.000	0.038
13	13	1432.00	7.00	4.98	3.156	0.845	0.697	2.667	2.200	0.589	0.714	0.486
		Total	96.00		34.531	10.976	1.104	28.952	3.815	0.789	9.553	1.195

For relationship between Qp, Tp and V we use multiple regresion
X1 = b' + b''.X2 + m'.X3
where b', b'', and m' are constants and They can be estimated by least square method, by the following equation

$$\sum X_1 = Nb' + b''\sum X_2 + m'\sum X_3$$
$$\sum X_1X_2 = b'\sum X_2 + b''\sum X_2^2 + m'\sum X_2X_3$$
$$\sum X_1X_3 = b'\sum X_3 + b''\sum X_2X_3 + m'\sum X_3^2$$

From above equation and table we get equation as follow :
1. 34.531 = 13b' + 10.976b'' +1.104m'
2. 28.952 = 10.976b' + 9.553b'' +0.789m'
3. 3.815 = 1.104b' + 0.789b'' + 1.195m'

And with Gauss-Seidel Iteration we get :
b' = 2.8701
b'' = -0.3296
m' = 0.7585

So. Equation of best fit relationship Qp. Tp and V is :
X1 = 2.8701 - 0.3296 X2 + 0.7585 X3
or
Log Qp = 2.8701 - 0.3296 Log Tp + 0.7585 Log V

Assumption time to peak(tp) to be same for all the event and taken as average tp =

$$\frac{\sum tp}{N} = \frac{96}{13} = 7.385 hours$$

Then : Log Qp = 2.8701 - 0.3296 Log 7.385 + 0.7585 Log V
or
Log Qp = 2.5839 + 0.7585 Log V

Partial Correlation Coefficient :
1 Log Qp and Log Tp = -0.5823
2 Log Qp and Log V = 0.9563
Multiple Correlation Coefficient = 0.9644

APPENDIK – J Continued

J.6. Regression Analysis Between Q_p, T_p and V for Gola Sub-basin

No	Event	Peak Discharge (cumec) Qp	tp (hrs)	Runoff volume (cm) V	Log Qp (X1)	Log Tp (X2)	Log V (X3)	X1 X2	X1 X3	X2 X3	$X2^2$	$X3^2$
1	1977	450.43	14.00	8.26	2.654	1.146	0.917	3.041	2.434	1.051	1.314	0.841
2	1977	425.61	14.00	7.60	2.629	1.146	0.881	3.013	2.316	1.010	1.314	0.776
3	1978	285.67	14.00	6.32	2.456	1.146	0.800	2.815	1.966	0.917	1.314	0.641
4	1979	133.10	18.00	1.99	2.124	1.255	0.298	2.666	0.633	0.374	1.576	0.089
5	1980	116.59	32.00	1.80	2.067	1.505	0.256	3.111	0.530	0.386	2.265	0.066
6	1982	531.88	6.00	4.90	2.726	0.778	0.691	2.121	1.883	0.537	0.606	0.477
7	1984	245.00	3.00	1.71	2.389	0.477	0.232	1.140	0.555	0.111	0.228	0.054
8	1995	528.24	8.00	5.13	2.723	0.903	0.710	2.459	1.933	0.641	0.816	0.504
9	1986	699.00	18.00	17.62	2.844	1.255	1.246	3.571	3.544	1.564	1.576	1.553
10	1986	279.49	6.00	2.60	2.446	0.778	0.415	1.904	1.016	0.323	0.606	0.172
			133.00		25.058	10.391	6.447	25.841	16.809	6.915	11.612	5.172

For relationship between Qp, Tp and V we use multiple regresion

X1 = b' + b".X2 + m'.X3

where b', b", and m' are constants and They can be estimated by least square method, by the following equation

$$\sum X_1 = Nb' + b''\sum X_2 + m'\sum X_3$$

$$\sum X_1 X_2 = b'\sum X_2 + b''\sum X_2^2 + m'\sum X_2 X_3$$

$$\sum X_1 X_3 = b'\sum X_3 + b''\sum X_2 X_3 + m'\sum X_3^2$$

From above equation and table we get equation as follow :

1. 25.058 = 10b' + 10.391b" + 6.447m'
2. 25.841 = 10.391b' + 11.612b" + 6.915m'
3. 16.809 = 6.447b' + 6.915b" + 5.172m'

And with Gauss-Seidel Iteration we get :

b' = 2.4846

b" = -0.4368

m' = 0.7369

So. Equation of best fit relationship Qp. Tp and V is :

X1 = 2.4846 - 0.4368 X2 + 0.7369 X3

or

Log Qp = 2.4846 - 0.4368 Log Tp + 0.7369 Log V

Assumption time to peak(tp) to be same for all the event and taken as average tp =

$$\frac{\sum tp}{N} = \frac{133}{10} = 13.30\,hours$$

Then : Log Qp = 2.4846 - 0.4368 Log 13.30 + 0.7369 Log V

or

Log Qp = 1.9937 + 0.7369 Log V

Partial Correlation Coefficient :

1 Log Qp and Log Tp = -0.8902

2 Log Qp and Log V = 0.9651

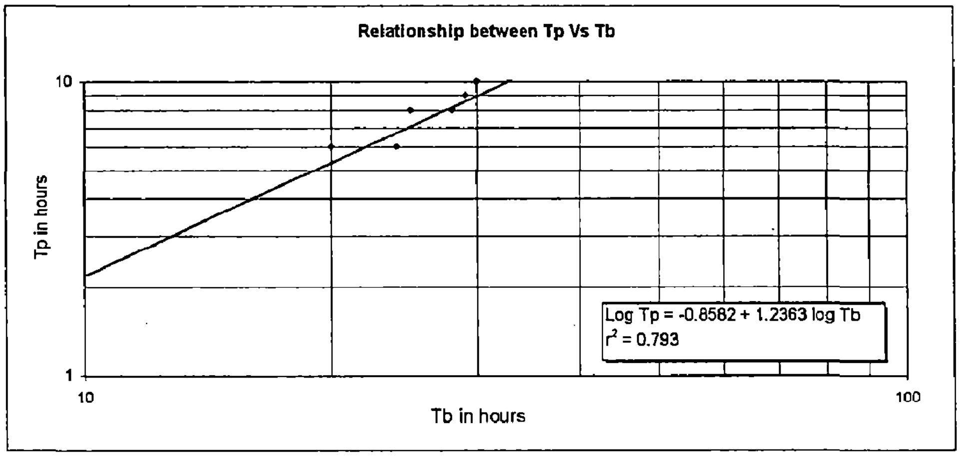
Multiple Correlation Coefficient = 0.9679

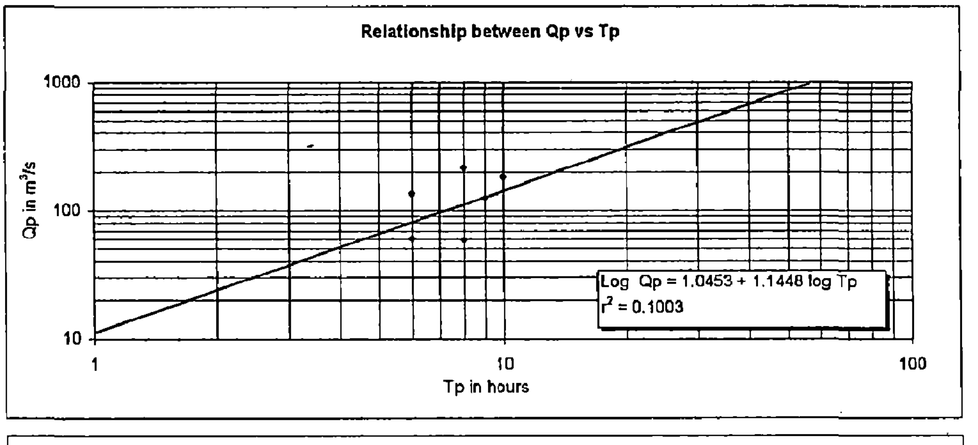
APPENDIX - K

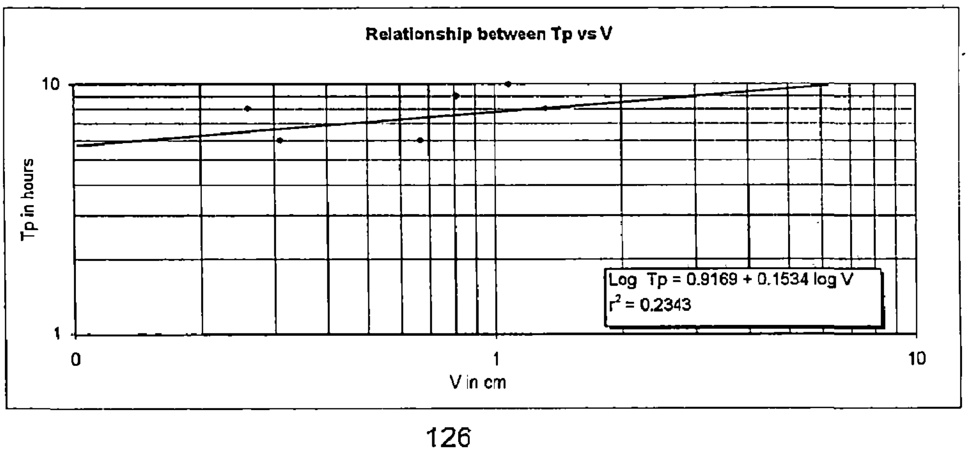
DATA AND REGRESSION ANALYSIS BETWEEN Tp & Tb, Qp & Tp AND Tp & V

K.1. Data and Regression analysis between Tp & Tb, Qp & Tp and Tp & V for Temur Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	tp (hrs)	Tb (hrs)
1	23.07.1962	518.67	56.62	0.3027	181.23	1.08	10.00	30.00
2	05.09.1962	518.67	56.62	0.3027	135.92	0.67	6.00	24.00
3	20.07.1964	518.67	56.62	0.3027	214.93	1.32	8.00	28.00
4	14.08.1964	518.67	56.62	0.3027	124.59	0.81	9.00	29.00
5	30.08.1965	518.67	56.62	0.3027	58.16	0.26	8.00	25.00
6	07.09.1965	518.67	56.62	0.3027	59.50	0.31	6.00	20.00
7	24.08.1961	518.67	56.52	0.3027	430.39	4.75	8.00	-
					Average		7.83	26.00



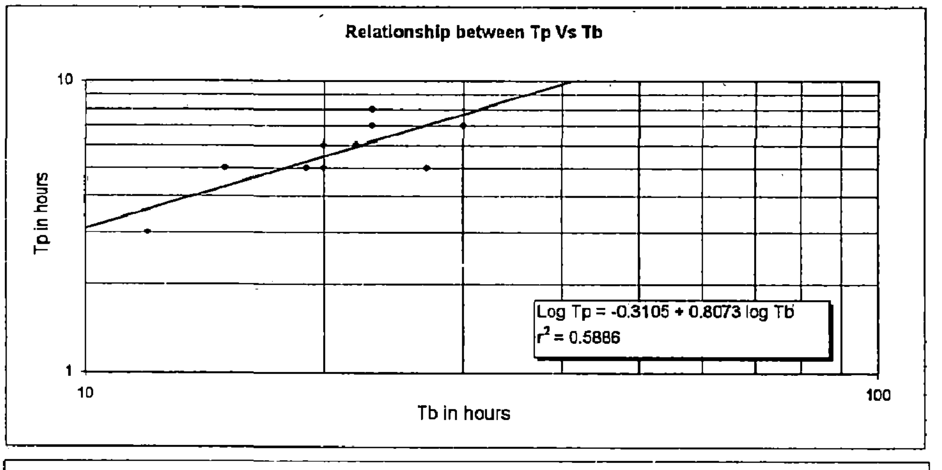


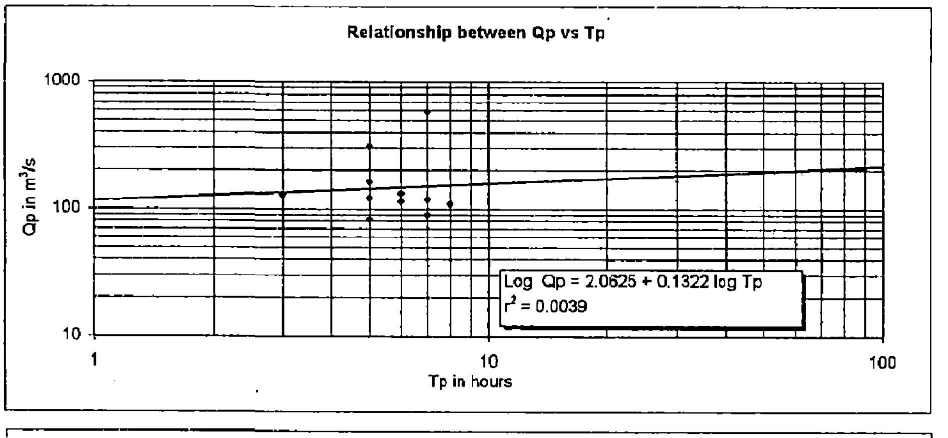


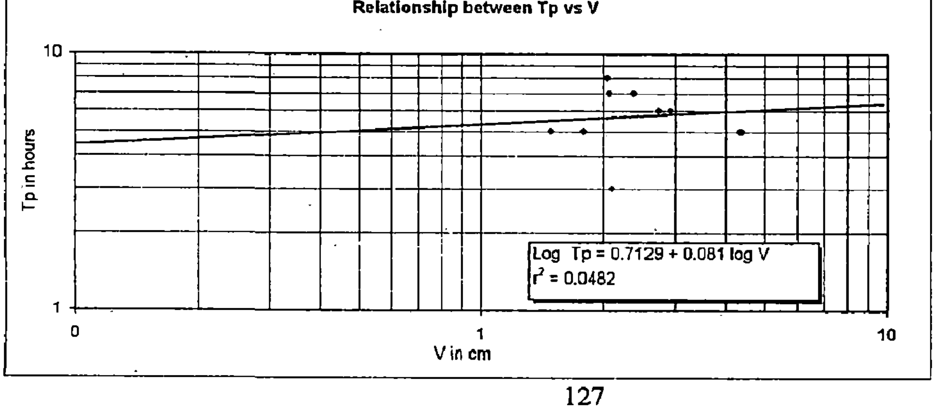
APPENDIX - K Continued

K.2. Data and Regression analysis between Tp & Tb, Qp & Tp and Tp & V for Teriya Sub-basin

No	Hydrograph	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	tp (hrs)	Tb (hrs)
1	1977	114.22	35.42	0.321	161.00	4.32	5.00	27.00
2	1978	114.22	35.42	0.321	120.00	1.49	5.00	15.00
3	1979	114.22	35.42	0.321	82.50	1.79	5.00	20.00
4	1980	114.22	35.42	0.321	114.00	2.72	6.00	22.00
5	1981	114.22	35.42	0.321	118.00	2.38	7.00	23.00
6	1982	114.22	35.42	0.321	131.00	2.92	6.00	20.00
7	1984	114.22	35.42	0.321	89.00	2.07	7.00	23.00
8	1985	114.22	35.42	0.321	108.00	2.04	8.00	23.00
9	1986	114.22	35.42	0.321	305.00	4.40	5.00	19.00
10	1986	114.22	35.42	0.321	575.75	21.21	7.00	30.00
11	1986	114.22	35.42	0.321	127.00	2.10	3.00	12.00
						Average	5.82	21.27



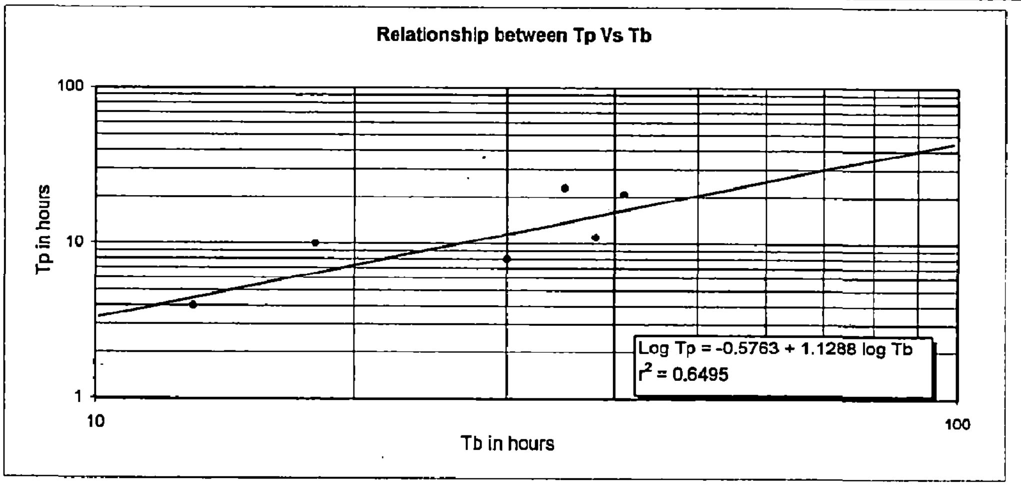


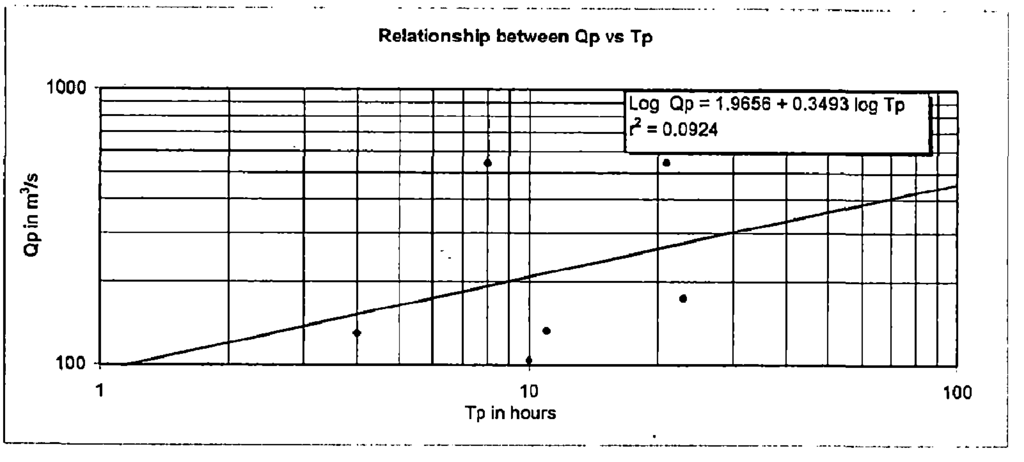


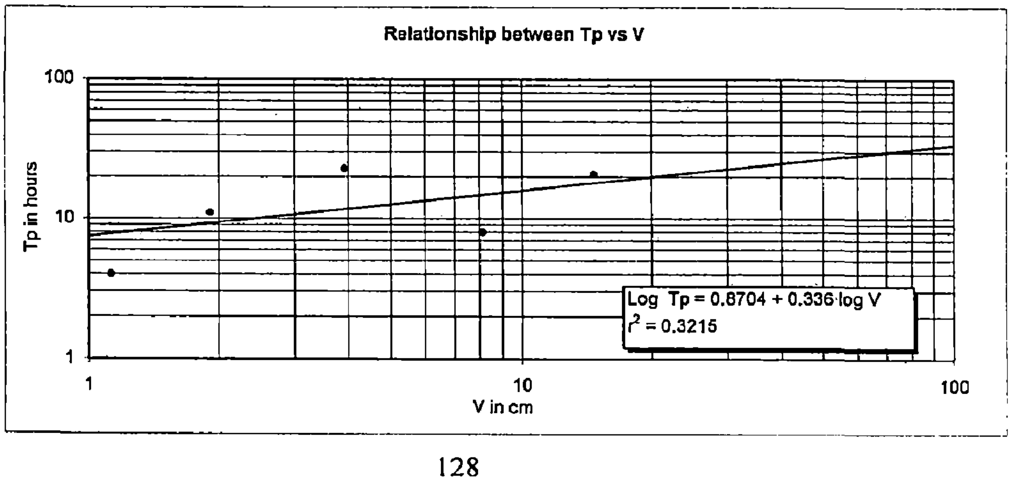
APPENDIX - K Continued

K.3. Data and Regression analysis between Tp & Tb, Qp & Tp and Tp & V for Umar Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	tp (hrs)	Tb (hrs)
1	23.07.1962	223.77	33.6	0.306	175.40	3.91	23.00	35.00
2	05.09.1962	223.77	33.6	0.306	133.30	1.91	11.00	38.00
3	20.07.1964	223.77	33.6	0.306	103.90	1.00	10.00	18.00
4	14.08.1964	223.77	33.6	0.306	130.30	1.13	4.00	13.00
5	30.08.1965	223.77	33.6	0.306	541.50	8.15	8.00	30.00
6	07.09.1965	223.77	33.6	0.306	544.40	14.73	21.00	41.00
						Average	12.83	29.17



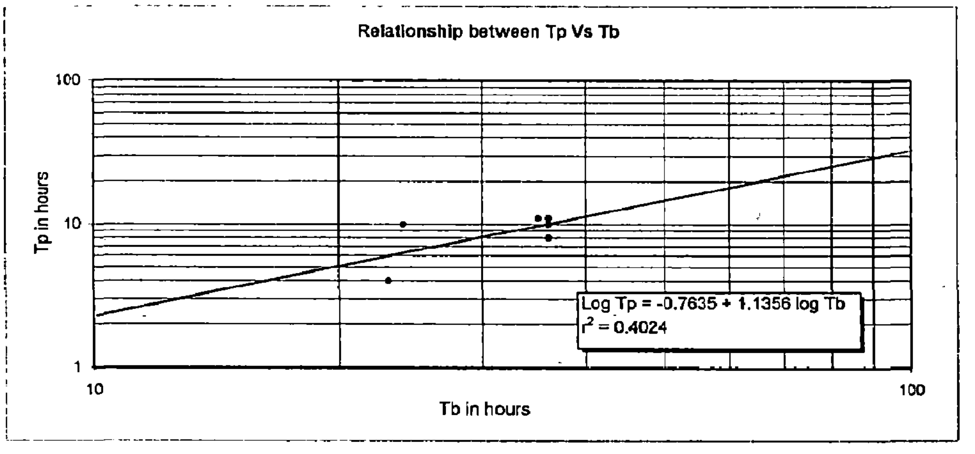


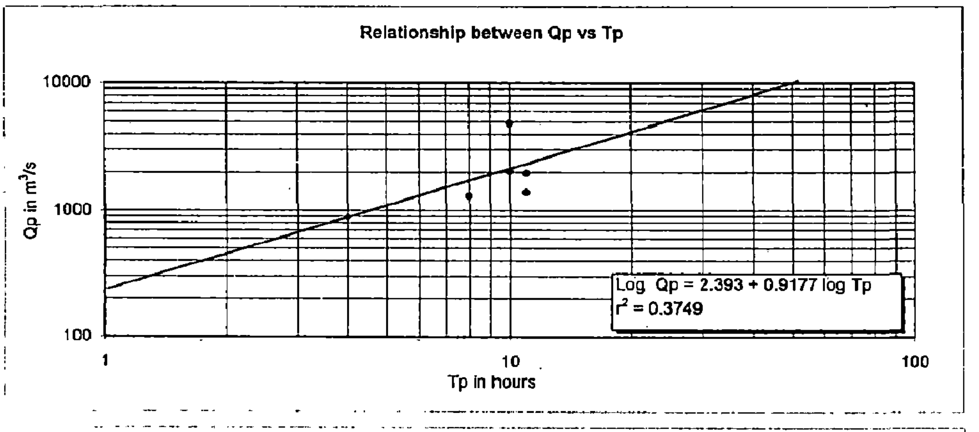


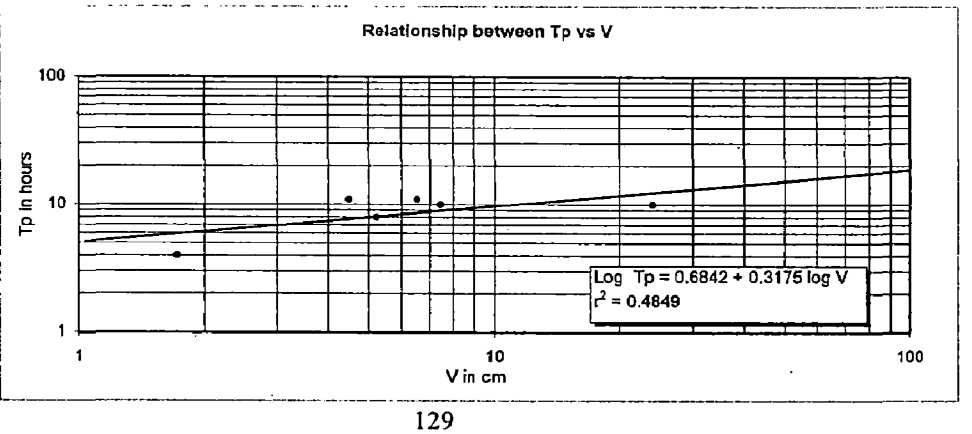
APPENDIX - K Continued

K.2. Data and Regression analysis between Tp & Tb, Qp & Tp and Tp & V for Kolar Sub-basin

No	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	tp (hrs)	Tb (hrs)
1	28-8-1983	903.88	75.34	0.53	4871.40	24.01	10	36
2	^10-8-1984	903.88	75.34	0.53	2032.63	7.44	10	24
3	31-7-1985	903.88	75.34	0.53	1293.00	5.22	8	36
4	13-8-1985	903.88	75.34	0.53	1381.70	4.47	11	35
5	15-8-1986	903.88	75.34	0.53	1968.10	6.54	11	36
6	27-8-1987	903.88	75.34	0.53	881.35	1.72	4	23
						Average	9.000	31.667



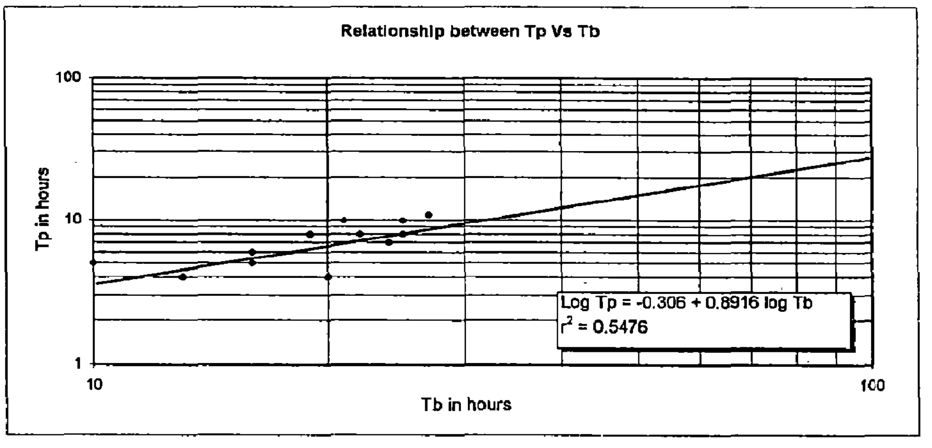


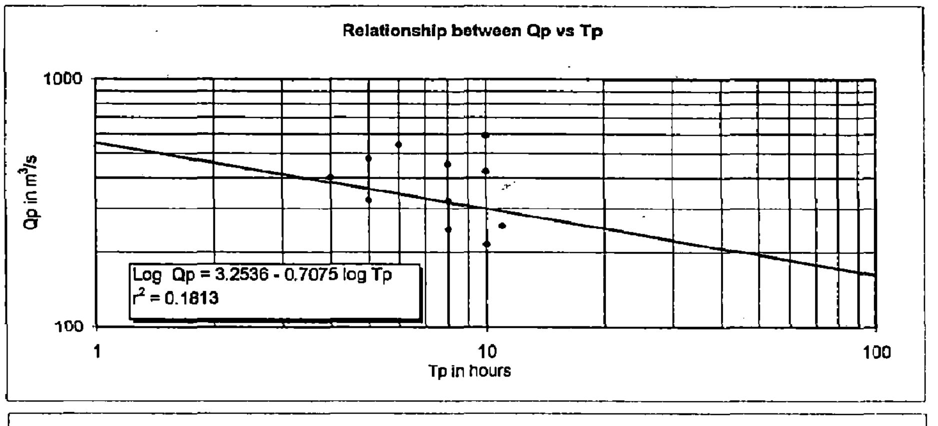


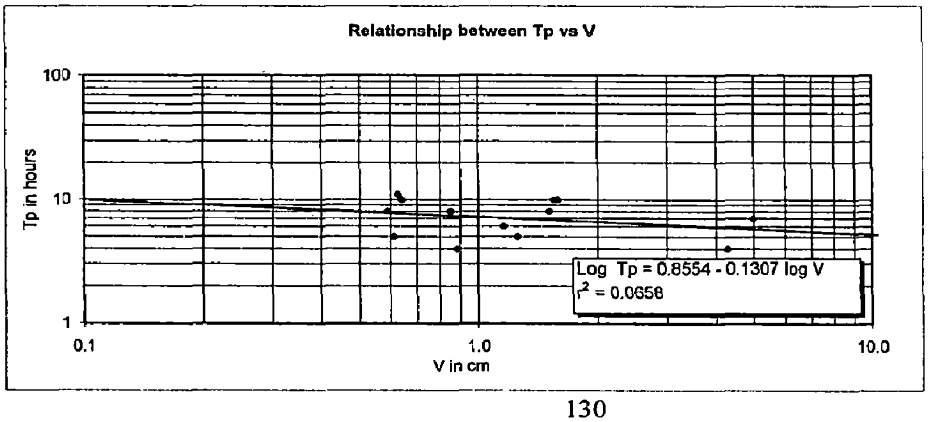
APPENDIX - K Continued

K.5. Data and Regression analysis between Tp & Tb, Qp & Tp and Tp & V for 3f Sub-Zone

No.	Event	Catchment Area (km2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	tp (hrs)	Tb (hrs)
1	1	823.62	61.08	0.124	543.80	1.16	6.00	16.00
2	2	823.62	61.08	0.124	478.30	1.26	5.00	16.00
3	3	823.62	61.08	0.124	324.50	0.61	5.00	10.00
4	4	823.62	61.08	0.124	402.10	0.88	4.00	13.00
5	5	823.62	61.08	0.124	247.00	0.85	8.00	19.00
6	6	823.62	61.08	0.124	1391.00	4.29	4.00	20.00
7	7	823.62	61.08	0.124	593.36	1.59	10.00	25.00
8	8	823.62	61.08	0.124	424.36	1.55	10.00	21.00
9	9	823.62	61.08	0.124	255.00	0.63	11.00	27.00
10	10	823.62	61.08	0.124	322.70	0.59	8.00	22.00
11	11	823.62	61.08	0.124	455.00	1.52	8.00	25.00
12	12	823.62	61.08	0.124	215.60	0.64	10.00	25.00
13	13	823.62	61.08	0.124	1432.00	4.98	7.00	24.00
						Average	7.38	20.23



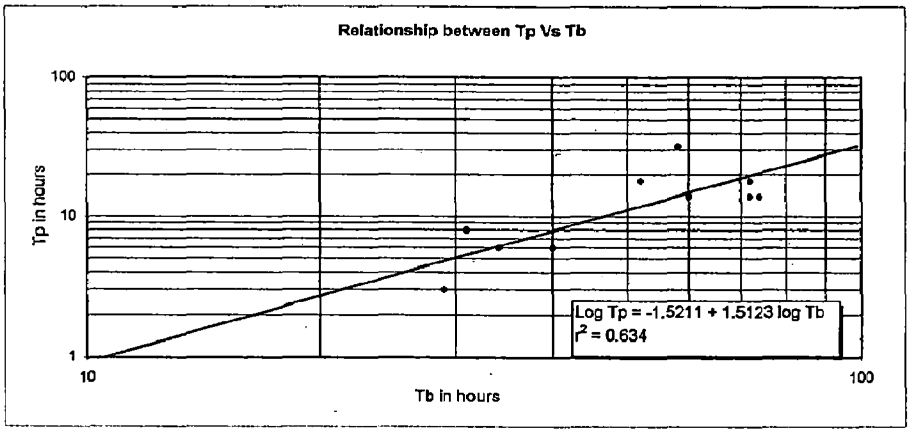


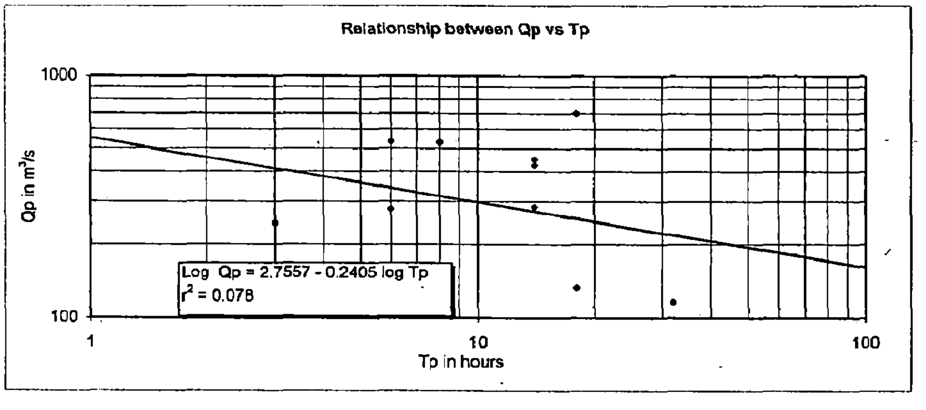


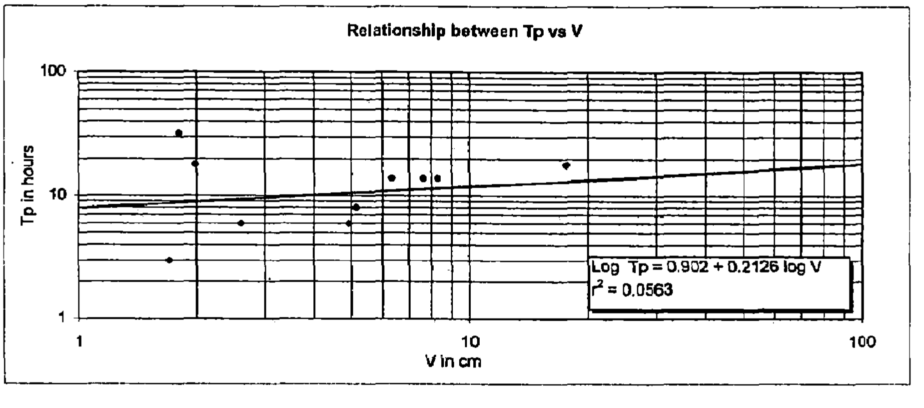
APPENDIX - K Continued

K.5. Data and Regression analysis between Tp & Tb, Qp & Tp and Tp & V for Gola Sub-basin

No	Hydrograph	Catchment Area (km^2) A	River Length (km) L	River bed slope (%) S	Peak Discharge (cumec) Qp	Runoff volume (cm) V	tp (hrs)	Tb (hrs)
1	1977	450	23.5	1.4	450.43	8.26	14.00	74.00
2	1977	450	23.5	1.4	425.61	7.60	14.00	72.00
3	1978	450	23.5	1.4	285.67	6.32	14.00	60.00
4	1979	450	23.5	1.4	133.10	1.99	18.00	52.00
5	1980	450	23.5	1.4	116.59	1.80	32.00	58.00
6	1982	450	23.5	1.4	531.88	4.90	6.00	34.00
7	1984	450	23.5	1.4	245.00	1.71	3.00	29.00
8	1995	450	23.5	1.4	528.24	5.13	8.00	31.00
9	1986	450	23.5	1.4	699.00	17.62	18.00	72.00
10	1986	450	23.5	1.4	279.49	2.60	6.00	40.00
						Average	13.30	52.20







APPENDIX - L

CONFIDENCE LIMIT 95 %

L.1. Confidence Limit 95% for Temur Sub-basin

No	Event	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Log Qp (Y)	Log V (X)	$(Yi-\bar{Y})$	$(Xi-\bar{X})$	$(Yi-\bar{Y})^2$	$(Xi-\bar{X})^2$	$(Yi-\bar{Y})(Xi-\bar{X})$
1	24.08.1961	430.39	4.75	2.63	0.68	0.49	0.75	0.24	0.57	0.37
2	23.07.1962	181.23	1.08	2.26	0.03	0.12	0.11	0.01	0.01	0.01
3	05.09.1962	135.92	0.67	2.13	-0.17	-0.01	-0.10	0.00	0.01	0.00
4	20.07.1964	214.93	1.32	2.33	0.12	0.19	0.20	0.04	0.04	0.04
5	14.08.1964	124.59	0.81	2.10	-0.09	-0.05	-0.02	0.00	0.00	0.00
6	30.08.1965	58.16	0.26	1.76	-0.59	-0.38	-0.51	0.14	0.26	0.19
7	07.09.1965	59.5	0.31	1.77	-0.51	-0.37	-0.43	0.13	0.19	0.16
		Total		14.99	-0.53	0.00	0.00	0.57	1.07	0.77
		Average		2.14	-0.08					

Analysis of Confidence limit 95 % (Soewarno, 1995) :

a. Coefficient Correlation

$$R=\frac{\sum_{i=1}^{n}(Xi-\bar{X})(Yi-\bar{Y})}{\left[\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}\left\{\sum_{i=1}^{n}(Yi-\bar{Y})^2\right\}\right]^{1/2}}$$

R = 0.9871

R^2 = 0.9744

b. Standard deviation

$$\sigma x=\left[\frac{\sum_{i=1}^{n}(Xi-\bar{X})^2}{(n-1)}\right]^{1/2} = 0.4229$$

$$\sigma y=\left[\frac{\sum_{i=1}^{n}(Yi-\bar{Y})^2}{(n-1)}\right]^{1/2} = 0.3086$$

c. Standard Error

$$Sey=\sigma_y(1-R^2)^{1/2} = 0.0493$$

$$Sex=\sigma x(1-R^2)^{1/2} = 0.06762$$

d. Confidence limit of m (Regression Coefficient)

Regression equation : y = 2.1961 + 0.7202 X

So, b = 2.1961

m = 0.7202

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

Coefficient Regression Deviation

$$Sa=\frac{Sey}{\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}^{1/2}} = 0.0476$$

Value of t-test

$$t=\frac{m-A}{Sa} = 15.12294$$

tα = 2.228 less than 15.12294

so null Hypothesis is pushed away and alternative hypothesis OK

So that the confidence limit 95 % is

m-tα (Sa) < m < m + tα (Sa)

0.6141 < m < 0.826304

so is OK

e. Confidence Limit 95 % Coefficient Corelation

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

$$t=\frac{R(n-2)^{1/2}}{(1-R^2)^{1/2}}$$

t = 13.81

tα = 2.228 less than 13.81

so null Hypothesis is pushed away and alternative hypothesis is OK

so that confidence limit 95 % of Coefficient Correlation is O.K.

APPENDIX - L Continued

L.2. Confidence Limit 95% for Teriya Sub-basin

No	Event	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Log Qp (Y)	Log V (X)	$(Yi-\bar{Y})$	$(Xi-\bar{X})$	$(Yi-\bar{Y})^2$	$(Xi-\bar{X})^2$	$(Yi-\bar{Y})(Xi-\bar{X})$
1	1977	161.00	4.32	2.21	0.64	0.05	0.16	0.00	0.02	0.01
2	1978	120.00	1.49	2.08	0.17	-0.08	-0.31	0.01	0.09	0.03
3	1979	82.50	1.79	1.92	0.25	-0.25	-0.22	0.06	0.05	0.06
4	1980	114.00	2.72	2.06	0.44	-0.10	-0.04	0.01	0.00	0.00
5	1981	118.00	2.38	2.07	0.38	-0.09	-0.10	0.01	0.01	0.01
6	1982	131.00	2.92	2.12	0.47	-0.04	-0.01	0.00	0.00	0.00
7	1984	89.00	2.07	1.95	0.32	-0.21	-0.16	0.05	0.03	0.03
8	1985	108.00	2.04	2.03	0.31	-0.13	-0.17	0.02	0.03	0.02
9	1986	305.00	4.40	2.48	0.64	0.32	0.17	0.10	0.03	0.05
10	1986	575.75	21.21	2.76	1.33	0.60	0.85	0.36	0.72	0.51
11	1986	127.00	2.10	2.10	0.32	-0.06	-0.16	0.00	0.02	0.01
		Total		23.78	5.26	0.00	0.00	0.62	1.01	0.73
		Average		2.16	0.48					

Analysis of Confidence limit 95 % (Soewarno, 1995) :

a. Coefficient Correlation

$$R=\frac{\sum_{i=1}^{n}(Xi-\bar{X})(Yi-\bar{Y})}{\left[\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}\left\{\sum_{i=1}^{n}(Yi-\bar{Y})^2\right\}\right]^{1/2}}$$

R = 0.9230

R^2 = 0.8520

b. Standard deviation

$$\sigma x=\left[\frac{\sum_{i=1}^{n}(Xi-\bar{X})^2}{(n-1)}\right]^{1/2} = 0.3172$$

$$\sigma y=\left[\frac{\sum_{i=1}^{n}(Yi-\bar{Y})^2}{(n-1)}\right]^{1/2} = 0.2484$$

c. Standard Error

$$Sey=\sigma_y(1-R^2)^{1/2} = 0.0956$$

$$Sex=\sigma x(1-R^2)^{1/2} = 0.12203$$

d. Confidence limit of m (Regression Coefficient)

Regression equation : y = 1.8162 + 0.7229 X

So, b = 1.8162

m = 0.7229

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

Coefficient Regression Deviation

$$Sa=\frac{Sey}{\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}^{1/2}} = 0.0953$$

Value of t-test

$$t=\frac{m-A}{Sa} = 7.587304$$

tα = 2.228 less than 7.587304

so null Hypothesis is pushed away and alternative hypothesis OK

So that the confidence limit 95 % is

m-tα (Sa) < m < m + tα (Sa)

0.51062 < m < 0.935178

so is OK

e. Confidence Limit 95 % Coefficient Corelation

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

$$t=\frac{R(n-2)^{1/2}}{(1-R^2)^{1/2}}$$

t = 7.20

tα = 2.228 less than 7.20

so null Hypothesis is pushed away and alternative hypothesis OK

so that confidence limit 95 % of Coefficient Correlation is O.K.

APPENDIX - L Continued

L.3. Confidence Limit 95% for Umar Sub-basin

No	Event	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Log Qp (Y)	Log V (X)	$(Yi-\overline{Y})$	$(Xi-\overline{X})$	$(Yi-\overline{Y})^2$	$(Xi-\overline{X})^2$	$(Yi-\overline{Y})(Xi-\overline{X})$
1	23.07.1962	175.40	3.91	2.24	0.59	-0.08	0.09	0.01	0.01	-0.01
2	05.09.1962	133.30	1.91	2.12	0.28	-0.20	-0.22	0.04	0.05	0.04
3	20.07.1964	103.90	1.00	2.02	0.00	-0.31	-0.50	0.10	0.25	0.16
4	14.08.1964	130.30	1.13	2.11	0.05	-0.21	-0.45	0.05	0.20	0.10
5	30.08.1965	541.50	8.15	2.73	0.91	0.41	0.41	0.16	0.17	0.17
6	07.09.1965	544.40	14.73	2.74	1.17	0.41	0.67	0.17	0.45	0.27
		Total		13.97	3.00	0.00	0.00	0.52	1.13	0.73
		Average		2.33	0.50					

Analysis of Confidence limit 95 % (Soewarno, 1995) :

a. Coefficient Correlation

$$R=\frac{\sum_{i=1}^{n}(Xi-\overline{X})(Yi-\overline{Y})}{\left[\left\{\sum_{i=1}^{n}(Xi-\overline{X})^2\right\}\left\{\sum_{i=1}^{n}(Yi-\overline{Y})^2\right\}\right]^{1/2}}$$

R = 0.9503

R^2 = 0.9030

b. Standard deviation

$$\sigma x=\left[\frac{\sum_{i=1}^{n}(Xi-\overline{X})^2}{(n-1)}\right]^{1/2} = 0.4744$$

$$\sigma y=\left[\frac{\sum_{i=1}^{n}(Yi-\overline{Y})^2}{(n-1)}\right]^{1/2} = 0.3230$$

c. Standard Error

$$Sey=\sigma_y(1-R^2)^{1/2} = 0.1006$$

$$Sex=\sigma x(1-R^2)^{1/2} = 0.14773$$

d. Confidence limit of m (Regression Coefficient)

Regression equation : y = 2.005 + 0.647 X

So, b = 2.005

m = 0.647

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

Coefficient Regression Deviation

$$Sa=\frac{Sey}{\left\{\sum_{i=1}^{n}(Xi-\overline{X})^2\right\}^{1/2}} = 0.0948$$

Value of t-test

$$t=\frac{m-A}{Sa} = 6.824346$$

tα = 2.228 less than 6.824346

so null Hypothesis is pushed away and alternative hypothesis OK

So that the confidence limit 95 % is

m-tα (Sa) < m < m + tα (Sa)

0.43577 < m < 0.858231

so is OK

e. Confidence Limit 95 % Coefficient Corelation

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

$$t=\frac{R(n-2)^{1/2}}{(1-R^2)^{1/2}}$$

t = 6.10

tα = 2.228 less than 6.10

so null Hypothesis is pushed away and alternative hypothesis OK

so that confidence limit 95 % of Coefficient Correlation is O.K.

APPENDIX - L Continued

L.4. Confidence Limit 95% for Kolar Sub-basin

No	Event	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Log Qp (Y)	Log V (X)	$(Yi-\bar{Y})$	$(Xi-\bar{X})$	$(Yi-\bar{Y})^2$	$(Xi-\bar{X})^2$	$(Yi-\bar{Y})(Xi-\bar{X})$
1	28-8-1983	4871.40	24.01	3.69	1.38	0.44	0.60	0.19	0.36	0.26
2	^10-8-1984	2032.63	7.44	3.31	0.87	0.06	0.09	0.00	0.01	0.01
3	31-7-1985	1293.00	5.22	3.11	0.72	-0.14	-0.06	0.02	0.00	0.01
4	13-8-1985	1381.70	4.47	3.14	0.65	-0.11	-0.13	0.01	0.02	0.01
5	15-8-1986	1968.10	6.54	3.29	0.82	0.05	0.04	0.00	0.00	0.00
6	27-8-1987	881.35	1.72	2.95	0.24	-0.30	-0.54	0.09	0.29	0.16
		Total		19.49	4.67	0.00	0.00	0.32	0.69	0.46
		Average		3.25	0.78					

Analysis of Confidence limit 95 % (Soewarno, 1995) :

a. Coefficient Correlation

$$R=\frac{\sum_{i=1}^{n}(Xi-\bar{X})(Yi-\bar{Y})}{\left[\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}\left\{\sum_{i=1}^{n}(Yi-\bar{Y})^2\right\}\right]^{1/2}}$$

R = 0.9762
R^2 = 0.9529

b. Standard deviation

$$\sigma x=\left[\frac{\sum_{i=1}^{n}(Xi-\bar{X})^2}{(n-1)}\right]^{1/2} = 0.3707$$

$$\sigma y=\left[\frac{\sum_{i=1}^{n}(Yi-\bar{Y})^2}{(n-1)}\right]^{1/2} = 0.2533$$

c. Standard Error

$$Sey=\sigma_y(1-R^2)^{1/2} = 0.0550$$

$$Sex=\sigma x(1-R^2)^{1/2} = 0.08043$$

d. Confidence limit of m (Regression Coefficient)

Regression equation : y = 2.7285 + 0.6671 X
So, b = 2.7285
m = 0.6671

Hypothesis
- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

Coefficient Regression Deviation

$$Sa=\frac{Sey}{\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}^{1/2}} = 0.0663$$

Value of t-test

$$t=\frac{m-A}{Sa} = 10.06175$$

tα = 2.228 less than 10.06175
so null Hypothesis is pushed away and alternative hypothesis OK

So that the confidence limit 95 % is
m-tα (Sa) < m < m + tα (Sa)
0.51938 < m < 0.814818
so is OK

e. Confidence Limit 95 % Coefficient Corelation

Hypothesis
- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

$$t=\frac{R(n-2)^{1/2}}{(1-R^2)^{1/2}}$$

t = 9.00
tα = 2.228 less than 9.00
so null Hypothesis is pushed away and alternative hypothesis OK
so that confidence limit 95 % of Coefficient Correlation is O.K.

APPENDIX - L Continued

L.5. Confidence Limit 95% for 3f Sub-zone

No	Event	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Log Qp (Y)	Log V (X)	$(Yi-\bar{Y})$	$(Xi-\bar{X})$	$(Yi-\bar{Y})^2$	$(Xi-\bar{X})^2$	$(Yi-\bar{Y})(Xi-\bar{X})$
1	1	543.80	1.16	2.74	0.06	0.08	-0.02	0.01	0.00	0.00
2	2	478.30	1.26	2.68	0.10	0.02	0.02	0.00	0.00	0.00
3	3	324.50	0.61	2.51	-0.21	-0.14	-0.30	0.02	0.09	0.04
4	4	402.10	0.88	2.60	-0.05	-0.05	-0.14	0.00	0.02	0.01
5	5	247.00	0.85	2.39	-0.07	-0.26	-0.15	0.07	0.02	0.04
6	6	1391.00	4.29	3.14	0.63	0.49	0.55	0.24	0.30	0.27
7	7	593.36	1.59	2.77	0.20	0.12	0.12	0.01	0.01	0.01
8	8	424.36	1.55	2.63	0.19	-0.03	0.11	0.00	0.01	0.00
9	9	255.00	0.63	2.41	-0.20	-0.25	-0.29	0.06	0.08	0.07
10	10	322.70	0.59	2.51	-0.23	-0.15	-0.32	0.02	0.10	0.05
11	11	455.00	1.52	2.66	0.18	0.00	0.10	0.00	0.01	0.00
12	12	215.60	0.64	2.33	-0.19	-0.32	-0.28	0.10	0.08	0.09
13	13	1432.00	4.98	3.16	0.70	0.50	0.61	0.25	0.37	0.31
		Total		34.53	1.10	0.00	0.00	0.79	1.10	0.88
		Average		2.66	0.08					

Analysis of Confidence limit 95 % (Soewarno, 1995) :

a. Coefficient Correlation

$$R=\frac{\sum_{i=1}^{n}(Xi-\bar{X})(Yi-\bar{Y})}{\left[\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}\left\{\sum_{i=1}^{n}(Yi-\bar{Y})^2\right\}\right]^{1/2}}$$

R = 0.9456

R^2 = 0.8941

b. Standard deviation

$$\sigma x=\left[\frac{\sum_{i=1}^{n}(Xi-\bar{X})^2}{(n-1)}\right]^{1/2} = 0.3029$$

$$\sigma y=\left[\frac{\sum_{i=1}^{n}(Yi-\bar{Y})^2}{(n-1)}\right]^{1/2} = 0.2565$$

c. Standard Error

$$Sey=\sigma_y(1-R^2)^{1/2} = 0.0835$$

$$Sex=\sigma x(1-R^2)^{1/2} = 0.09859$$

d. Confidence limit of m (Regression Coefficient)

Regression equation : y = 2.5882 + 0.8007 X

So, b = 2.5882

m = 0.8007

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

Coefficient Regression Deviation

$$Sa=\frac{Sey}{\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}^{1/2}} = 0.0796$$

Value of t-test

$$t=\frac{m-A}{Sa} = 10.06409$$

tα = 2.228 less than 10.06409

so null Hypothesis is pushed away and alternative hypothesis OK

So that the confidence limit 95 % is

m-tα (Sa) < m < m + tα (Sa)

0.62344 < m < 0.97796

so is OK

e. Confidence Limit 95 % Coefficient Corelation

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

$$t=\frac{R(n-2)^{1/2}}{(1-R^2)^{1/2}}$$

t = 9.64

tα = 2.228 less than 9.64

so null Hypothesis is pushed away and alternative hypothesis OK

so that confidence limit 95 % of Coefficient Correlation is O.K.

APPENDIX - L Continued

L.6. Confidence Limit 95% for Gola Sub-basin

No	Event	Peak Discharge (cumec) Qp	Runoff volume (cm) V	Log Qp (Y)	Log V (X)	$(Yi-\bar{Y})$	$(Xi-\bar{X})$	$(Yi-\bar{Y})^2$	$(Xi-\bar{X})^2$	$(Yi-\bar{Y})(Xi-\bar{X})$
1	1977	450.43	8.26	2.65	0.92	0.15	0.27	0.02	0.07	0.04
2	1977	425.61	7.60	2.63	0.88	0.12	0.24	0.02	0.06	0.03
3	1978	285.67	6.32	2.46	0.80	-0.05	0.16	0.00	0.02	-0.01
4	1979	133.10	1.99	2.12	0.30	-0.38	-0.35	0.15	0.12	0.13
5	1980	116.59	1.80	2.07	0.26	-0.44	-0.39	0.19	0.15	0.17
6	1982	531.88	4.90	2.73	0.69	0.22	0.05	0.05	0.00	0.01
7	1984	245.00	1.71	2.39	0.23	-0.12	-0.41	0.01	0.17	0.05
8	1995	528.24	5.13	2.72	0.71	0.22	0.07	0.05	0.00	0.01
9	1986	699.00	17.62	2.84	1.25	0.34	0.60	0.11	0.36	0.20
10	1986	279.49	2.60	2.45	0.42	-0.06	-0.23	0.00	0.05	0.01
		Total		25.06	6.45	0.00	0.00	0.61	1.02	0.65
		Average		2.51	0.64					

Analysis of Confidence limit 95 % (Soewarno, 1995) :

a. Coefficient Correlation

$$R = \frac{\sum_{i=1}^{n}(Xi-\bar{X})(Yi-\bar{Y})}{\left[\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}\left\{\sum_{i=1}^{n}(Yi-\bar{Y})^2\right\}\right]^{1/2}}$$

R = 0.8340

R^2 = 0.6956

b. Standard deviation

$$\sigma x = \left[\frac{\sum_{i=1}^{n}(Xi-\bar{X})^2}{(n-1)}\right]^{1/2} = 0.3360$$

$$\sigma y = \left[\frac{\sum_{i=1}^{n}(Yi-\bar{Y})^2}{(n-1)}\right]^{1/2} = 0.2594$$

c. Standard Error

$$Sey = \sigma_y(1-R^2)^{1/2} = 0.1431$$

$$Sex = \sigma x(1-R^2)^{1/2} = 0.18537$$

d. Confidence limit of m (Regression Coefficient)

Regression equation : y = 2.0908 + 0.6438 X

So, b = 2.0908

m = 0.6438

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

Coefficient Regression Deviation

$$Sa = \frac{Sey}{\left\{\sum_{i=1}^{n}(Xi-\bar{X})^2\right\}^{1/2}} = 0.1420$$

Value of t-test

$$t = \frac{m - A}{Sa} = 4.535202$$

tα = 2.228 less than 4.535202

so null Hypothesis is pushed away and alternative hypothesis OK

So that the confidence limit 95 % is

m-tα (Sa) < m < m + tα (Sa)

0.32752 < m < 0.960078

so is OK

e. Confidence Limit 95 % Coefficient Corelation

Hypothesis

- Null Hypothesis $\rho = 0$
- Alternative hypothesis $\rho \neq 0$

$$t = \frac{R(n-2)^{1/2}}{(1-R^2)^{1/2}}$$

t = 4.28

tα = 2.228 less than 4.28

so null Hypothesis is pushed away and alternative hypothesis OK

so that confidence limit 95 % of Coefficient Correlation is O.K.

APPENDIX - M

Runoff curve numbers for selected agricultural, suburban, and urban land uses (anticedent moisture condition II, Ia = 0.2S)

Land Use Description	Hydrologic Soil Group			
	A	B	C	D
Cultivated land[1] : without conservation treatment	72	81	88	91
with conservation treatment	62	71	78	81
Pasture or range land : poor condition	68	79	86	89
good condition	39	61	74	80
Meadow : good condition	30	58	71	78
Wood or forest land : thin stand, poor cover, no mulch	45	66	77	83
good cover[2]	25	55	70	77
Open Spaces, lawns, parks, golf courses, cemeteries, etc				
good condition : grass cover on 75% or more of the area	39	61	74	80
fair condition : grass cover on 50% to 75% of the area	49	69	79	84
Comercial and business areas (85% impervious)	89	92	94	95
Industrial districts (72% impervious)	81	88	91	93
Residential[3] :				
Average lot size — Average % impervious[4]				
1/8 acre or less — 65	77	85	90	92
1/4 acre — 38	61	75	83	87
1/3 acre — 30	57	72	81	86
1/2 acre — 25	54	70	80	85
1 acre — 20	51	68	79	84
Paved parking lots, roofs, driveways, etc[5]	98	98	98	98
Street and roads :				
Paved with curbs and storm sewers[5]	98	98	98	98
gravel	76	85	89	91
dirt	72	82	87	89

[1]For a more detailed description of agricultural land use curve numbers, refer to Soil Concervation Service, 1973

[2]Good cover is protected from grazing and litter and brush cover soil.

[3]Curve number are computed assuming the runoff from the house and driveway is directed towards the streea with a minimum of roof water directed to lawns where additional infiltration could occur.

[4]The remaining pervious area (lawn) are considered to be in good pasture condition for these curve numbers.

[5]In some warmer climates of the country a curve number of 95 may be used

APPENDIX - N.1

CALCULATION OF PMF AT T = 50 YEARS FOR TEMUR SUB-BASIN BY UH METHOD

Refer to Flood Estimation Refort for Upper Narmada & Tapi Sub-basin (Sub-zone 3c) by CWC, (1983) we get
Total Depth of Rainfall = 13.11 cm
Base flow = 0.05 cumec x Catchment Area (518.67 sq km)
= 25.93 cumec
Loss rate = 0.3 cm/hr

Calculation of Incremental Rainfall for given Pmp 13.11 cm

Time (hrs)	% rainfall	Cumulatif Rainfall	Incremental Rainfall	Losses (0.3 cm/hr)	Excess Rainfall
0.00	0.00	0.00	0.00	0.00	0.00
1.00	44.40	5.82	5.82	0.30	5.52
2.00	57.78	7.57	1.75	0.30	1.45
3.00	70.00	9.18	1.60	0.30	1.30
4.00	80.00	10.49	1.31	0.30	1.01
5.00	86.70	11.37	0.88	0.30	0.58
6.00	95.56	12.53	1.16	0.30	0.86
7.00	100.00	13.11	0.58	0.30	0.28
			Total of Direct Runoff Depth (cm)		11.01

Calculation of Flood HYDROGRAPH (50 - year) of Temur Sub-basin up to bridge no. 249 of Upper Narmada Basin

Time (hrs)	Ordinate of 1 hrs UH	Due to ERH 0.58 cm m3/s	Due to ERH 1.01 cm m3/s	Due to ERH 1.45 cm m3/s	Due to ERH 5.52 cm m3/s	Due to ERH 1.30 cm m3/s	Due to ERH 0.86 cm m3/s	Due to ERH 0.28 cm m3/s	Base Flow m3/s	Flood Hydrograph m3/s
1	2	3	4	5	6	7	8	9	10	11= (3+..+10)
0	0.00	0.00							25.93	25.93
1	10.00	5.78	0.00						25.93	31.72
2	30.00	17.35	10.11	0.00					25.93	53.40
3	70.00	40.49	30.33	14.54	0.00				25.93	111.29
4	130.00	75.19	70.77	43.62	55.21	0.00			25.93	270.72
5	200.00	115.67	131.43	101.79	165.63	13.02	0.00		25.93	553.47
6	244.66	141.50	202.20	189.04	386.46	39.06	8.62	0.00	25.93	992.81
7	206.00	119.14	247.35	290.82	717.71	91.14	25.85	2.82	25.93	1520.77
8	160.00	92.54	208.27	355.76	1104.17	169.27	60.31	8.46	25.93	2024.71
9	122.00	70.56	161.76	299.55	1350.73	260.41	112.00	19.75	25.93	**2300.69**
10	90.00	52.05	123.34	232.66	1137.29	318.56	172.31	36.67	25.93	2098.82
11	65.00	37.59	90.99	177.40	883.33	268.22	210.79	56.42	25.93	1750.68
12	45.00	26.03	65.72	130.87	673.54	208.33	177.48	69.01	25.93	1376.91
13	30.00	17.35	45.50	94.52	496.88	158.85	137.85	58.11	25.93	1034.98
14	22.00	12.72	30.33	65.44	358.85	117.18	105.11	45.13	25.93	760.70
15	12.00	6.94	22.24	43.62	248.44	84.63	77.54	34.41	25.93	543.76
16	5.00	2.89	12.13	31.99	165.63	58.59	56.00	25.39	25.93	378.55
17	0.00	0.00	5.06	17.45	121.46	39.06	38.77	18.34	25.93	266.06
18			0.00	7.27	66.25	28.64	25.85	12.69	25.93	166.64
19				0.00	27.60	15.62	18.95	8.46	25.93	96.58
20					0.00	6.51	10.34	6.21	25.93	48.99
21						0.00	4.31	3.39	25.93	33.63
22							0.00	1.41	25.93	27.34
23								0.00	25.93	25.93

COMPARISON OF PMF BY PEAK DISCHARGE DISTRIBUTION AND UH METHOD
FOR TEMUR SUB BASIN OF UPPER NARMADA BASIN
AT RETURN PERIODE T = 50 YEAR

No	Time Peak Tp (hrs)	Runoff Volume V (cm)	PMF	
			UH Method (m^3/s)	Peak Disch. Distribution (m^3/s)
1	9.00	11.01	2300.69	909.82

APPENDIX - N.2.

CALCULATION OF PMF AT T = 50 YEARS
FOR UMAR SUB-BASIN
BY UH METHOD

From the Flood estimation report for Upper Narmada & Tapi Sub-basin by CWC, (1983)
we get,
Total Depth of Rainfall = 14.03 cm
Base flow = 0.05 cumec x Catchment Area (223.77 sq. km)
= 11.1885 cumec
Loss rate = 0.3 cm/hr

Calculation of Incremental Rainfall for given Pmp 15.6 cm

Time (hrs)	% rainfall	Cumulatif Rainfall	Incremental Rainfall	Losses (0.3 cm/hr)	Excess Rainfall
0.00	0.00	0.00	0.00	0.00	0.00
1.00	55.55	7.79	7.79	0.30	7.49
2.00	72.22	10.13	2.34	0.30	2.04
3.00	84.44	11.85	1.71	0.30	1.41
4.00	93.33	13.09	1.25	0.30	0.95
5.00	100.00	14.03	0.94	0.30	0.64
			Total of Direct Runoff Depth (cm)		12.53

Calculation of Flood HYDROGRAPH AT T =50 - year

Time (hrs)	Ordinate of 1 hrs UH	Due to ERH 0.95 cm m^3/s	Due to ERH 2.04 cm m^3/s	Due to ERH 7.49 cm m^3/s	Due to ERH 1.41 cm m^3/s	Due to ERH 0.64 cm m^3/s	Base Flow m^3/s	Flood Hydrograph m^3/s
1	2	3	4	5	6	7	10	11= (3+...+10)
0	0.00	0.00					0.00	0.00
1	5.00	4.74	0.00				11.19	15.92
2	20.00	18.95	10.19	0.00			11.19	40.33
3	50.00	47.36	40.78	37.47	0.00		11.19	136.80
4	102.00	96.62	101.94	149.87	7.07	0.00	11.19	366.70
5	125.18	118.58	207.96	374.68	28.29	3.18	11.19	743.88
6	106.00	100.41	255.22	764.35	70.72	12.72	11.19	1214.61
7	76.00	71.99	216.11	938.06	144.28	31.79	11.19	1413.42
8	50.00	47.36	154.95	794.33	177.06	64.85	11.19	1249.74
9	33.00	31.26	101.94	569.52	149.93	79.59	11.19	943.43
10	23.00	21.79	67.28	374.68	107.50	67.39	11.19	649.83
11	16.00	15.16	46.89	247.29	70.72	48.32	11.19	439.57
12	10.00	9.47	32.62	172.35	46.68	31.79	11.19	304.10
13	5.00	4.74	20.39	119.90	32.53	20.98	11.19	209.73
14	2.00	1.89	10.19	74.94	22.63	14.62	11.19	135.47
15	0.00	0.00	4.08	37.47	14.14	10.17	11.19	77.05
16			0.00	14.99	7.07	6.36	11.19	39.61
17				0.00	2.83	3.18	11.19	17.20
18					0.00	1.27	11.19	12.46
19						0.00	11.19	11.19

COMPARISON PMF BY PEAK DISCHARGE DISTRIBUTION AND UH METHOD
FOR UMAR SUB BASIN (UPPER NARMADA BASIN)
AT RETURN PERIODE T = 50 YEARS

No	Time to Peak Tp (hrs)	Runoff Volume V (cm)	PMF UH Method (m^3/s)	PMF Peak Disch. distribution (m^3/s)
1	7.00	12.53	1413.42	530.44

APPENDIX - N.3.

CALCULATION OF PMF AT T = 50 YEAR
FOR 3f SUB-ZONE
BY UH METHOD

Refer to Flood Estimation Report for Lower Godavari Sub-Zone 3f (Revised) by CWC, (1995), we get :

Depth of Rainfall = 14.61 cm
Base flow = 0.05 cumec x Catchment Area (823.62 sq km)
= 41.2 cumec
Loss rate = 0.2 cm/hr

Calculation of Incremental Rainfall for given rainfall 14.74 cm

Time (hrs)	% rainfall	Cumulatif Rainfall	Incremental Rainfall	Losses (0.2 cm/hr)	Excess Rainfall (cm)
1.00	38.00	5.55	5.16	0.20	4.96
2.00	57.00	8.33	2.78	0.20	2.58
3.00	69.00	10.08	1.75	0.20	1.55
4.00	77.00	11.25	1.17	0.20	0.97
5.00	84.00	12.27	1.02	0.20	0.82
6.00	90.00	13.15	0.88	0.20	0.68
7.00	93.00	13.59	0.44	0.20	0.24
8.00	96.00	14.03	0.44	0.20	0.24
9.00	98.00	14.32	0.29	0.20	0.09
10.00	100.00	14.61	0.29	0.20	0.09
			Direct Runoff Depth (cm)		12.22

Calculation of Flood Hydrograph of 3f Sub-zone up to bridge no. 807 of Lower Godavari Basin at T = 50 Years

Time (hrs)	Ordinate of 1 hrs UH	Due to ERH 0.09 cm m³/s	Due to ERH 0.24 cm m³/s	Due to ERH 0.68 cm m³/s	Due to ERH 0.97 cm m³/s	Due to ERH 2.58 cm m³/s	Due to ERH 4.96 cm m³/s	Due to ERH 1.55 cm m³/s	Due to ERH 0.82 cm m³/s	Due to ERH 0.24 cm m³/s	Due to ERH 0.09cm m³/s	Total D.S.R.O m³/s	Base Flow m³/s	Flood Hydrograph m³/s
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
0	0.00	0.00										0.00	41.20	41.20
1	4.00	0.37	0.00									0.37	41.20	41.57
2	8.00	0.74	0.95	0.00								1.69	41.20	42.89
3	25.00	2.30	1.91	2.71	0.00							6.92	41.20	48.12
4	45.00	4.15	5.96	5.41	3.88	0.00						19.39	41.20	60.59
5	80.00	7.38	10.72	16.92	7.75	10.30	0.00					53.07	41.20	94.27
6	130.00	11.99	19.06	30.45	24.22	20.61	19.84	0.00				126.16	41.20	167.36
7	180.00	16.60	30.98	54.13	43.60	64.40	39.68	6.21	0.00			255.59	41.20	296.79
8	240.00	22.13	42.89	87.96	77.50	115.92	124.00	12.43	3.29	0.00		486.12	41.20	527.32
9	271.62	25.04	57.19	121.79	125.94	206.07	223.20	38.83	6.58	0.95	0.00	805.60	41.20	846.80
10	245.00	22.59	64.73	162.38	174.38	334.87	396.80	69.89	20.57	1.91	0.37	1248.49	41.20	1289.69
11	210.00	19.36	58.38	183.78	232.51	463.66	644.80	124.26	37.02	5.96	0.74	1770.47	41.20	1811.67
12	175.00	16.13	50.04	165.77	263.15	618.22	892.80	201.92	65.82	10.72	2.31	2286.87	41.20	2328.07
13	140.00	12.91	41.70	142.09	237.36	699.67	1190.40	279.58	106.95	19.06	4.15	2733.86	41.20	2775.06
14	105.00	9.68	33.36	118.41	203.45	631.10	1347.24	372.77	148.09	30.98	7.38	2902.44	41.20	2943.64
15	82.00	7.56	25.02	94.72	169.54	540.94	1215.20	421.88	197.45	42.89	11.99	2727.19	41.20	2768.39
16	64.00	5.90	19.54	71.04	135.63	450.78	1041.60	380.53	223.46	57.19	16.60	2402.28	41.20	2443.48
17	50.00	4.61	15.25	55.48	101.72	360.63	868.00	326.17	201.56	64.73	22.13	2020.28	41.20	2061.48
18	42.00	3.87	11.92	43.30	79.44	270.47	694.40	271.61	172.77	58.38	25.04	1631.40	41.20	1672.60
19	34.00	3.13	10.01	33.83	62.00	211.22	520.80	217.45	143.97	50.04	22.59	1275.05	41.20	1316.25
20	30.00	2.77	8.10	28.42	48.44	164.86	406.72	163.09	115.19	41.70	19.36	998.63	41.20	1039.83
21	26.00	2.40	7.15	23.00	40.69	128.80	317.44	127.36	88.38	33.36	16.14	782.72	41.20	823.92
22	22.00	2.03	6.20	20.30	32.94	108.19	248.00	99.40	67.48	25.02	12.91	622.44	41.20	663.64
23	19.00	1.75	5.24	17.59	29.06	87.58	208.32	77.66	52.65	19.54	9.68	509.09	41.20	550.29
24	16.00	1.48	4.53	14.89	25.19	77.28	168.64	65.23	41.13	15.25	7.56	421.17	41.20	462.37
25	14.00	1.29	3.81	12.86	21.31	66.97	148.80	52.81	34.55	11.91	5.90	360.22	41.20	401.42
26	11.00	1.01	3.34	10.83	18.41	56.67	128.96	46.60	27.97	10.01	4.61	308.40	41.20	349.60
27	9.00	0.83	2.62	9.47	15.50	48.94	109.12	40.38	24.68	8.10	3.87	263.53	41.20	304.73
28	6.00	0.55	2.14	7.44	13.56	41.21	94.24	34.17	21.39	7.15	3.13	225.00	41.20	266.20
29	4.00	0.37	1.43	6.09	10.66	36.06	79.36	29.51	18.10	6.20	2.77	190.54	41.20	231.74
30	1.00	0.09	0.95	4.06	8.72	28.33	69.44	24.85	15.63	5.24	2.40	159.72	41.20	200.92
31	0.00	0.00	0.24	2.71	5.81	23.18	54.56	21.74	13.16	4.53	2.03	127.96	41.20	169.16
32	0.00	0.00	0.00	0.68	3.88	15.46	44.64	17.09	11.52	3.81	1.75	98.81	41.20	140.01
33			0.00	0.00	0.97	10.30	29.76	13.98	9.05	3.34	1.48	68.87	41.20	110.07
34				0.00	0.00	2.58	19.84	9.32	7.40	2.62	1.29	43.05	41.20	84.25
35					0.00	0.00	4.96	6.21	4.94	2.14	1.01	19.27	41.20	60.47
36						0.00	0.00	1.55	3.29	1.43	0.83	7.10	41.20	48.30
37							0.00	0.00	0.82	0.95	0.55	2.33	41.20	43.53
38								0.00	0.00	0.24	0.37	0.61	41.20	41.81
39									0.00	0.00	0.09	0.09	41.20	41.29
40										0.00	0.00	0.00	41.20	41.20
41											0.00	0.00	41.20	41.20

COMPARISON PMF BY PEAK DISCHARGE DISTRIBUTION AND UH METHOD
FOR SUB-ZONE 3F OF GODAVARI BASIN
AT RETURN PERIODE T = 50 YEARS

No	Time to Peak Tp (hrs)	Runoff Volume V (cm)	PMF UH Method (m³/s)	PMF Peak Disch. Distribution (m³/s)
1	14.00	12.22	2943.64	2909.38

APPENDIK – O

PROCEDURE OF REGRESSION AND ANALYSIS OF VARIANCE

1. Linear Regression by Least Square Method

To develop the family of Standardized Peak Discharge Distribution given in section 2.5 and section 2.6 linear regressions by Least Square Method is proposed. Procedure of regression and analysis of variance are briefly discussed in following paragraphs.

The variable X and Y are often referred to as independent and dependent variables respectively, although these roles can be interchanged. The criterion for the best-fit line in the method of least square is that the sum of all squares of deviations of observed points from the fitted function is minimized to produce least square.

Let us assume the least square line approximating the set of points (x_1,y_1), (x_2,y_2),......,(x_N,y_N) has equation :

$$Y = a + bx \qquad (1)$$

The error in the estimate of Y at point (x_i,y_i) is e_i so that :

$$\sum_{i=1}^{N} e_i = \sum_{i=1}^{N}(y_i - \hat{y}) = \sum_{i=1}^{N}(y_i - a - bx_i) \qquad (2a)$$

Since the error e_i can be both positive and negative, to avoid the negative sign we consider the square of the error e_i and denote it by Z such that

$$Z=\sum_{i=1}^{N}(y_i - a - bx_i)^2 \qquad (2b)$$

The criterion to get the equation of the best fit line is to minimize the error Z so that

$$\frac{\partial Z}{\partial a} = \frac{\partial}{\partial a}\sum_{i=1}^{N}(y_i - a - bx_i)^2 = 0 \qquad (3)$$

$$\frac{\partial Z}{\partial b} = \frac{\partial}{\partial b}\sum_{i=1}^{N}(y_i - a - bx_i)^2 = 0 \qquad (4)$$

Solving Eqs. (3) and (4) we get

$$a = \frac{\sum yi}{N} - \frac{b\sum xi}{N} \qquad (5)$$

$$b = \frac{\sum x_i y_i - \sum x_i \sum y_i / N}{\sum x_i^2 - (\sum x_i)^2 / N} \qquad (6)$$

with correlation coefficient (r) = $\frac{N\sum x_i y_i - \sum x_i \sum y_i}{\sqrt{\{N\sum x_i^2 - (\sum x_i)^2\}\{N\sum y_i^2 - (\sum y_i)^2\}}}$ (7)

2. Multiple Regression

Frequently we come across cases in hydrology when we are interested in estimating one dependent variable from more than one independent variable, like relation between peak discharge (Q_p), time to peak (T_p) and runoff volume (V).

Relationship between a dependent variable X_1 and the independent variables X_2, X_3, ...X_n, and is called a linear multiple regression equation of X_1 on X_2, X_3,......X_n, with two independent variables, the equation is:

$$X_1 = b' + b''X_2 + m'X_3 \qquad (8)$$

where b', b" and m' are constants.

The coefficients b', b" and m' are estimated by least square method, by solving the following equations (Mutreja, 1986):

$$\begin{aligned} \sum X_1 &= b'N + b''\sum X_2 + m'\sum X_3 \\ \sum X_1X_2 &= b'\sum X_2 + b''\sum X_2^2 + m'\sum X_2X_3 \\ \sum X_1X_3 &= b'\sum X_3 + b''\sum X_2X_3 + m'\sum X_3^2 \end{aligned} \qquad (9)$$

3. Partial Correlation

It is often important to measure the correlation between a dependent variable and one particular independent variable when all the other variables involved are kept constant so as to determine the portion of variance of the dependent variable explained by this particular independent variable.

The partial correlation coefficient measures the association of the dependent variable X_1 with any given independent variable X_i. Thus, they measure the variation of X_1, which is explained only and only by the given X_i. A simple method of estimating the partial coefficient r_{1-i} involves the determination of:

1. The multiple correlation coefficient R_1 between X_1 and all the independent variable
2. The multiple correlation coefficient R_{1-i} between X_1 and all the independent variable except the chosen X_i.

The partial correlation coefficient r_1-i is then given by (Mutreja, 1986):

$$r^2_{1-i} = 1 - \frac{1-R^2_1}{1-R^2_{1-i}} \tag{10}$$

where : R is multiple correlation coefficient

$$R = \sqrt{1-\frac{S_e^{\,2}}{S_d^{\,2}}} \tag{11}$$

S_e is standard error of estimate of X_1 on X_2 and X_3 and S_d is standard deviation of variable X_1.

$$Se = \sqrt{\frac{\sum(X_1-\hat{X}_1)^2}{N}} \tag{12a}$$

and

$$Sd = \sqrt{\frac{\sum\ (X_1-\overline{X}_1)^2}{N}} \tag{12b}$$

Since R^2_{1-i} is smaller than R^2_1, $r_{1-i} < 1.0$. The greater the difference between R_1 and R_{1-i}, the smaller the ratio on the right side of the equation and consequently greater is the value of r_{1-i}. It is clear from Eq. (11) that considerable computations are involved in the case of many variable if all partial correlation coefficients are to be determined.

4. Analysis of Variance

The total variation in the responses of sums of squares (total SSQ) is the sum of SSQ due to the mean $\bar{y}$ plus the SSQ about regression plus the SSQ due to regression is represented by:

$$\sum_{i=1}^{n} y_i^2 = n\bar{y}^2 + \sum_{i=1}^{n}(y_i - \hat{y}_i)^2 + \sum_{i=1}^{n}(y_i - \bar{y})^2 \tag{13}$$

This breakdown of the variation in the data is generally displayed in a table called the analysis of variance (AOV) table (see Table 1).

And mathematical model in the linear term represented by

$$Y = \beta_0 + \beta_1 X + \in \qquad (14)$$

Table. 1 AOV Table

Source of variation	df	SSQ	Mean square (MS)	Calculated R^2 and F
Total	n	$\sum_{i=1}^{n} y_i^2$		$R^2 = \frac{\sum_{i=1}^{n}(\hat{y}_i - \bar{y})}{\sum_{i=1}^{n}(y_i - \bar{y})^2}$
Mean	1	$n\bar{y}^2$		
Due to regression $R(\beta_1 \backslash \beta_0)$	1	$\sum_{i=1}^{n}(\hat{y}_i - \bar{y})^2 = R(\beta 1 \backslash \beta 0)SSQ$	$rms = R(\beta_1 \backslash \beta_0)SSQ/1$	cal F = $\frac{rms}{s^2}$
About regression (residual)	n-2	$\sum_{i=1}^{n}(y_i - \hat{y}_i)^2 (subtraction)$	$s^2 = \frac{(residual.SSQ)}{(n-2)}$	

The symbols in Table 1 are defined as

$R(\beta_1 \backslash \beta_0)$ = reduction in the total SSQ due to regression after adjusting for the constant term β_0. It is measure of just the effect of the linear term≠ $\beta_1 X$ being in the model.

Residual = an error term; a measure of how far the predicted values miss the observed values. It is the variation in the responses that is not accounted for by the mean and the linear term (in this case). The SSQ for this quantity can be obtained by subtraction.

rms = regression means square

s^2 = error mean square

cal F = calculated F value to be compared with a corresponding critical tabulated value

Under the assumption that

$$\in_i \sim NID(0, \sigma^2)$$

we can use the quantities in the AOV table to test the hypothesis

$$H_0 : \beta_1 = 0$$

At the 100(1-α) percent confidence level. Specifically, if

cal F > $F^*(\nu_1,\nu_2,1-\alpha) = F^*(1,n-2,1-\alpha)$

we reject the hypothesis and conclude that the term $\beta_1 X$ should be in the model, otherwise we conclude that there is no evidence that $\beta_1 X$ should be in the model. The quantity $F^*(1, n-2, 1-\alpha)$ is the critical point of an F distribution, and is obtained from a table in Appendix-P. Specifically, an F distribution has the form

$$f(w;\nu_1\nu_2) = \frac{\left(\frac{\nu_1+\nu_2-2}{2}\right)!\left(\frac{\nu_1}{\nu_2}\right)^{\nu_1/2}\left[w^{(\frac{\nu_1}{2}-1}\right]}{\left(\frac{\nu_1-2}{2}\right)!\left(\frac{\nu_2-2}{2}\right)!\left(1+\frac{\nu_1 w}{\nu_2}\right)^{(\nu_1+\nu_2)/2}} \qquad W > 0 \qquad (15)$$

where ν_1 and ν_2 are integer parameters called the *numerator* and *denominator degrees of freedom*, respectively.

In general the critical points F^* (ν_1, ν_2, 1-α) is the value of w such that

$$\int_0^{F^*(\nu_1,\nu_2,1-\alpha)} f(w;\nu_1,\nu_2)dw = 1-\alpha$$

The value of F^* (ν_1,ν_2, 1-a) are available in literature for numerous values of ν_1 and ν_2 with specific values of α

The question now is, "Why can we reject the hypothesis that $\beta_1 = 0$ in the model

$$Y = \beta_0 + \beta_1 X + \in$$

at the 100 (1-α) percent level just because

$$cal\, F > F^*(1,n-2,1-\alpha)$$

where *cal* F is from the AOV in Table 1

The question can be resolved that the SSQ, mean squares, and the calculated *F* quantities in the AOV table are sample values that are calculated from the given data $\{x_i,y_i\}$. The given Y_i's,

$$Y_i = \beta_0 + \beta_1 x_i + \in_i$$

are random variables so every function of them is also a random variable with a certain distribution. Specifically, under the assumptions (Gillet, 1976):

$$\in_i \sim NID(0,\sigma^2) \qquad \text{for } i = 1,2, \ldots, n \qquad (16)$$

$$\beta_1 = 0$$

in the model

$$Y_i = \beta_0 + \beta_1 x_i + \in_i \qquad \text{for } i = 1,2, \ldots, n \qquad (17)$$

it can be shown that

$$\frac{RMS}{\sigma^2} = \frac{\sum_{i=1}^{n}(\hat{Y}_i - \overline{Y})^2}{\sigma^2} \sim x^2(1) \qquad (18)$$

$$\frac{(n-2)S^2}{\sigma^2} = \frac{(n-2)\sum_{i=1}^{n}(Y_i - \hat{Y}_i)}{(n-2)\sigma^2} = \frac{\sum_{i=1}^{n}(Y_i - \hat{Y}_i)}{\sigma^2} \sim x^2(n-2) \qquad (19)$$

and the random variables RMS/σ^2 and $[(n-2)S^2]\sigma^2$ are independent. In addition, the ratio

$$W = \frac{\left(\frac{RMS}{\sigma^2}\right)/1}{\left(\frac{(n-2)S^2}{\sigma^2}\right)/(n-2)} = \frac{\left(\frac{RMS}{\sigma^2}\right)}{\left(\frac{S^2}{\sigma^2}\right)} = \frac{RMS}{S^2} \qquad (20)$$

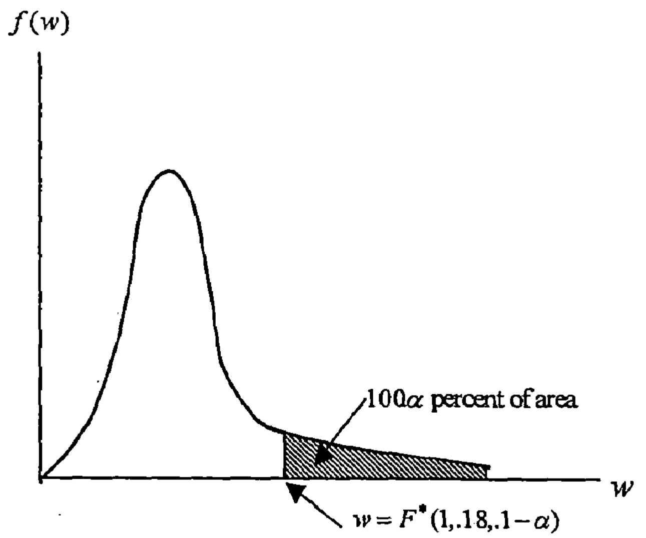


Figure 1 : F Distribution with $\nu_1 = 1$, $\nu_2 = 18$.

has an F distribution with 1 and n-2 numerator and denominator degrees of freedom, respectively. That is,

$$W = \frac{RMS}{S^2} \sim F(1, n-2) \qquad (21)$$

Thus under assumptions *(16)* and *(17)*, the calculated F value in Table 1 is just a value from W in Equation *(21)*.

If we assume Equation *(16)* holds, and if cal F in Table 1 is greater than $F^*(1, n-2, 1-\alpha)$, we reject the hypothesis that cal F came from W, whose distribution is denoted as $f(w;1, n-2)$, and run a 100α percent risk of rejecting a true hypothesis. For example, if cal $F = 5.62$, then

$$calF = 5.62 > F^*(1,18,0.95) = 4.41$$

so we reject the hypothesis that cal F came from $f(w;1,18)$ and run a 5 percent risk of being wrong. In other words, cal F could be a value of the random variable W with density $f(w;1.18)$ and be greater than 4.41 percent of the time. Figure 2 illustrates this concept.

When we reject the hypothesis that cal F is a value of W whose density is $f(w:1,n-2)$, we are actually saying that assumption *(17)* does not hold, that is $\beta_1 \neq 0$. Thus, to test the hypothesis

$$H_0 : \beta_1 = 0$$

is the model

$$Y = \beta_0 + \beta_1 X + \in$$

at the $100(1-\alpha)$ percent significance level we collect a set of data $\{x_i, y_i\}$. with $i= 1, 2, \ldots, n$, and calculate the AOV according to Table 1. If

$$cal\,F > F^*(1, n-2, 1-\alpha)$$

we reject the hypothesis that $\beta_1 = 0$ and conclude that the term $\beta_1 X$ is significant and should be in the model. Otherwise, we assume $\beta_1 = 0$ and do not put the term $\beta_1 X$ in the model. If assumption in Eq. *(17)* does not hold, then the above test is not valid and should not be used. The only problem that arises when we use the test under the false assumption is that we may leave out terms that should be in the model.

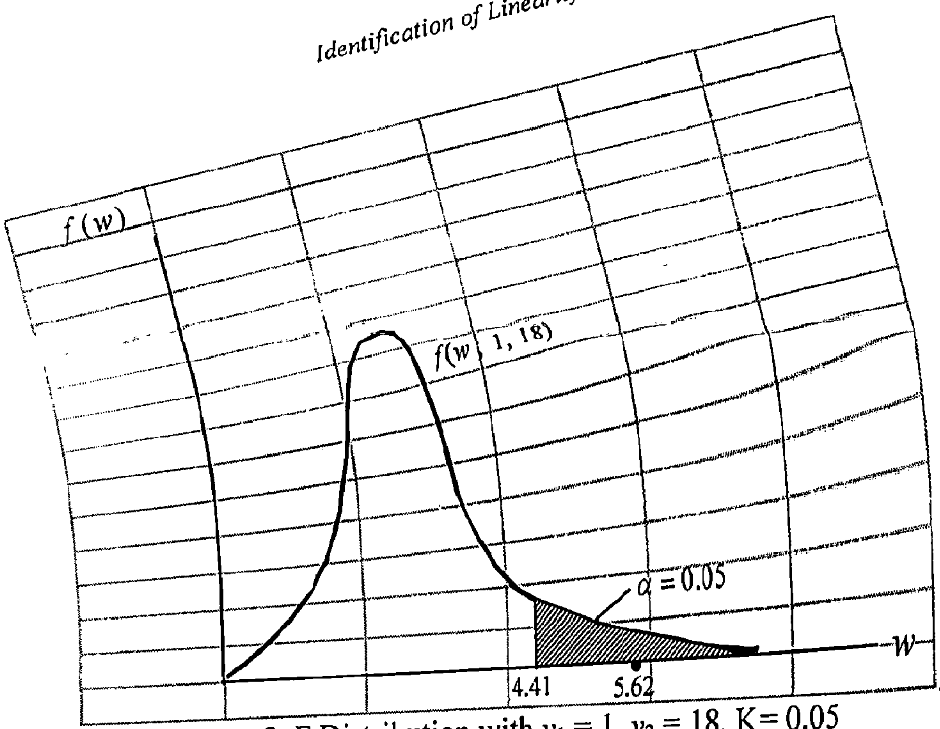


Figure 2. F Distribution with $\nu_1 = 1$, $\nu_2 = 18$, K= 0.05

If the only errors in ϵ_i with $i = 1, 2, \ldots, n$ are truly random errors, then the assumption

$$\epsilon_i \sim NID(0, \sigma^2)$$

is usually valid by the central limit theorem. However, if ϵ_i contains variation caused by the need for more independent variables or higher-order power of X in the model, the quantity

$$s^2 = \frac{\sum_{i=1}^{n}(y_i - \hat{y}_i)^2}{n-2}$$

contains more than just random errors and in fact also contains the variation due to not having enough independent variables and / or higher-order power of X in the model. Consequently, there may be a need for the linear term $\beta_1 X$ in the model, but the extra variation picked up in s^2 will cause the quantity

$$\frac{rms}{s^2}$$

to be smaller than it would be if s^2 only contained variation due to random errors. In this case, we may possibly conclude that $\beta_1 = 0$ when in reality it is not. One way to determine if s^2 contains more than just random errors is to examine a quantity called the *square of the multiple correlation coefficient.*

APPENDIX - P

Critical Value for F Test with a = 0.05

v_2 \ v_1	1	2	3	4	5	6	7	8	9	10	15	20	30	40	60	∞
1	161	200	216	225	230	234	237	239	241	242	248	250	251	252	271	254
2	18.5	19.2	19.2	19.2	19.3	19.3	19.4	19.4	19.4	19.4	19.4	19.4	19.5	19.5	19.5	19.5
3	10.1	9.55	9.28	9.12	9.01	8.94	8.89	8.85	8.81	8.79	8.70	8.66	8.62	8.59	8.57	8.53
4	7.71	6.94	6.59	6.39	6.26	6.16	6.09	6.04	6.00	5.96	5.86	5.80	5.75	5.72	5.69	5.63
5	6.61	5.79	5.41	5.19	5.05	4.95	4.88	4.82	4.77	4.74	4.62	4.56	4.50	4.46	4.43	4.37
6	5.99	5.14	4.78	4.53	4.39	4.28	4.21	4.15	4.10	4.06	3.94	3.87	3.81	3.77	3.74	3.67
7	5.59	4.74	4.35	4.12	3.97	3.87	3.79	3.73	3.68	3.64	3.51	3.44	3.38	3.34	3.30	3.23
8	5.32	4.46	4.07	3.84	3.69	3.58	3.50	3.44	3.39	3.35	3.22	3.15	3.08	3.04	3.01	2.93
9	5.12	4.26	3.86	3.63	3.48	3.37	3.29	3.23	3.18	3.14	3.01	2.94	2.86	2.83	2.79	2.71
10	4.96	4.10	3.71	3.45	3.33	3.22	3.14	3.07	3.02	2.98	2.85	2.77	2.70	2.66	2.62	2.54
11	4.84	3.98	3.59	3.36	3.20	3.09	3.01	2.95	2.90	2.85	2.72	2.65	2.57	2.53	2.49	2.40
12	4.75	3.89	3.49	3.26	3.11	3.00	2.91	2.85	2.80	2.75	2.62	2.54	2.47	2.43	2.38	2.30
13	4.67	3.81	3.41	3.18	3.03	2.92	2.83	2.77	2.71	2.67	2.53	2.46	2.38	2.34	2.30	2.21
14	4.60	3.74	3.34	3.11	2.96	2.85	2.76	2.70	2.65	2.60	2.46	2.39	2.31	2.27	2.22	2.13
15	4.54	3.68	3.29	3.06	2.90	2.79	2.71	2.64	2.59	2.54	2.40	2.33	2.25	2.20	2.16	2.07
16	4.49	3.63	3.24	3.01	2.85	2.74	2.66	2.59	2.50	2.49	2.35	2.28	2.19	2.15	2.11	2.01
17	4.45	3.59	3.20	2.96	2.81	2.70	2.61	2.55	2.49	2.45	2.31	2.23	2.15	2.10	2.06	1.96
18	4.41	3.55	3.16	2.93	2.77	2.66	2.58	2.51	2.46	2.41	2.27	2.19	2.11	2.06	2.02	1.92
19	4.38	3.52	3.13	2.90	2.74	2.63	2.54	2.48	1.42	2.38	2.23	2.16	2.07	2.03	2.98	2.88
20	4.35	3.49	3.10	2.87	2.71	2.60	2.51	2.45	2.39	2.35	2.20	2.12	2.04	1.99	1.95	1.84
21	4.32	3.47	3.07	2.84	2.68	2.57	2.49	2.42	2.37	2.32	2.18	2.10	2.01	1.96	1.92	1.81
22	4.30	3.44	3.05	2.82	2.66	2.55	2.46	2.40	2.34	2.30	2.15	2.07	1.98	1.94	1.89	1.78
23	4.28	3.42	3.03	2.80	2.64	2.53	2.44	2.37	2.32	2.27	2.13	2.05	1.96	1.91	1.86	1.76
24	4.26	3.40	3.01	2.78	2.62	2.51	2.42	2.36	2.30	2.25	2.11	2.03	1.94	1.89	1.84	1.73
25	4.24	3.39	2.99	2.76	2.60	2.49	2.40	2.34	2.28	2.24	2.09	2.01	1.97	1.87	1.82	1.71
30	4.17	3.32	2.92	2.69	2.53	2.42	2.33	2.27	2.21	2.16	2.01	1.93	1.84	1.79	1.74	1.62
40	4.08	3.23	2.84	2.61	2.45	2.34	2.25	2.18	2.12	2.08	1.92	1.84	1.74	1.69	1.64	1.51
60	4.00	3.15	2.76	2.53	2.37	2.25	2.17	2.10	2.04	1.99	1.84	1.75	1.65	1.59	1.53	1.39
120	3.92	3.07	2.68	2.45	2.29	2.18	2.09	2.02	1.96	1.91	1.75	1.66	1.55	1.50	1.43	1.25
∞	3.84	3.00	2.60	2.37	2.21	2.10	2.01	1.94	1.88	1.83	1.67	1.57	1.46	1.39	1.32	1.00

This table give the critical value for an F test for various numerator degrees of freedom v_1 and denominator degrees of freedom v_2. The significant level is $\alpha = 0.05$

For v1 = 3, v2 = 9, $\int_{3.86}^{\infty} F(3, 9; 0.05)\, dF = 0.05$

$$W = \frac{RMS}{S^2} \sim F(1, n-2) \qquad (21)$$

Thus under assumptions *(16)* and *(17)*, the calculated F value in Table 1 is just a value from W in Equation *(21)*.

If we assume Equation *(16)* holds, and if cal F in Table 1 is greater than F^* *(1,n-2,1-α)*, we reject the hypothesis that cal F came from W, whose distribution is denoted as $f(w;1, n\text{-}2)$, and run a 100α percent risk of rejecting a true hypothesis. For example, if cal $F = 5.62$, then

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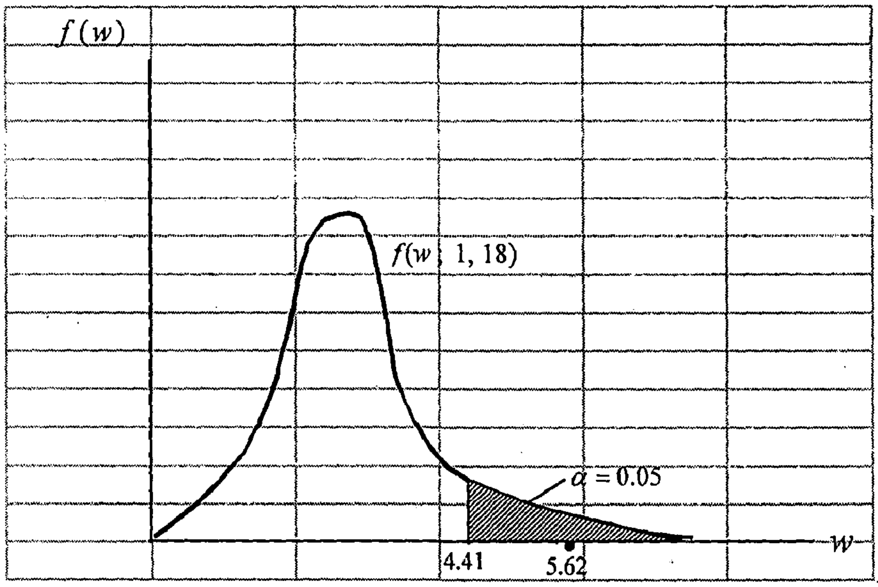

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