A CONCEPTUAL MODEL FOR BINOMIALLY DISTRIBUTED RUNOFF ELEMENTS OF A NATURAL CATCHMENT

A DISSERTATION
submitted in partial fulfilment of
the requirements for the award of the degree
of
MASTER OF ENGINEERING
in
HYDROLOGY

by

B. P. SINGH

UNESCO SPONSORED INTERNATIONAL HYDROLOGY COURSE
UNIVERSITY OF ROORKEE
ROORKEE (INDIA)

NOVEMBER, 1977

A C K N O W L E D G E M E N T

The author expresses his gratitude to Dr.B.S.Mathur, Reader in Hydrology, University of Roorkee for his keen interest guidance and encouragement throughout the course of the present study.

The useful discussions with Dr. Satish Chandra, Professor and Co-Ordinator International Hydrology Course, University of Roorkee are also gratefully acknowledged.

The author is thankful to Sri S.N.Sinha, Executive Engineer Discharge Division, Patna for making necessary data available for this study.

The author is also grateful to the various investigators and research workers in the field of surface Water Hydrology, whose work have been referred in course of the present study.

C E R T I F I C A T E

Certified that the dissertation entitled ' A CONCEPTUAL MODEL FOR BINOMIALLY DISTRIBUTED RUNOFF ELEMENTS OF A NATURAL CATCHMENT', which is being submitted by Er. Bindeshwari Prasad Singh in partial fulfilment of the requirement for award of the degree of Master of Engineering in Hydrology by the University of Roorkee is a record of his own work carried out under my supervision and guidance. To the best of my knowledge the matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

This is further to certify that he has worked for more than nine months from Oct. 1975 and onwards for preparation of this dissertation.

(B. S. Mathur)
Reader in Hydrology
University of Roorkee
Roorkee

ROORKEE

Dated 15.11.1977.

SYNOPSIS

In this study some conceptual models for obtaining direct runoff hydrograph from effective rainfall hyetograph have been reviewed.

A conceptual model based on binomial distribution of runoff elements for obtaining direct runoff hydrograph from effective rainfall has been proposed. The proposed response model is basically a two parameter analytical model. The parameters are (i) Response Sub-area and (ii) travel co-efficient. These parameters are evaluated from one of the recorded events on the catchment. The catchment area is divided into response sub-area which develop their independent hydrographs at the outlet. These are superimposed to get direct runoff hydrograph. Therefore, it has been possible to account for spatial distributions of the rainfall.

The model has been applied on Ajay River catchment located in Chotanagpur plateau of Bihar (India). The catchment area at the outlet (Sikatia) is 992 sq.miles.

The computed hydrographs have been found to be in close agreement with the observed hydrographs.

TABLE OF CONTENTS

CHAPTER - 3

THE PROPOSED RESPONSE MODEL

CHAPTER - 4

APPLICATION OF THE PROPOSED MODEL ON A NATURAL CATCHMENT

CHAPTER - 5

CONCLUSIONS

APPENDICES

...

CHAPTER - 1

THE HYDROLOGICAL SYSTEM

1.0 INTRODUCTION

The transformation of rainfall into runoff is affected by a large number of physiographic and climatic factors obtaining in the natural catchment during the transformation process. To name a few, these are, catchment characteristics, channel characteristics, precipitation, interception evaporation and transpiration etc. It has been noted that the process of formation of the storm runoff is preceded by the process of rainfall abstraction and generation of rainfall excess. It is the rainfall excess which is transported and transformed by the catchment system and finally appears at the outlet as direct runoff.

The transformation of rainfall excess into direct runoff is a complex process. The complexities are mainly due to interdependence and interaction of the numerous physical laws which govern the process. To correlate the hyetograph of rainfall excess to the hydrograph of direct runoff, application of physical laws is impracticable. It is so due to the fact that,

(i) The physical laws governing the process are not clearly established.

(ii) The inter dependence and interaction of the numerous physical laws governing the process is difficult to be ascertained.

(iii) The physiographic and climatic data required for carrying out such a comprehensive analysis will be too much and availability of such a reliable and comprehensive data is rare.

(iv) However if such a comprehensive data is collected, the error in collection may have cumulative effect and the refinement sought by putting in more labour in carrying out complex computations may offset the accuracy.

Therefore direct application of physical laws to correlate the direct runoff with rainfall excess has not been favoured by the hydrologists.

In the recent years with the development of electronic computation machines mathematical experimentation of a hydrologic system has become possible and it can be studied as an engineering system.

1.1 THE CATCHMENT SYSTEM

Like any other engineering system hydrologic system also has the following components -

1. Input function
2. System function
3. Output function or Response

In surface water hydrology where a natural catchment is studied as a hydrologic system the input function is generally the hyetograph, system function is a function represent ing the catchment action on hyetograph. The output function or response is the runoff hydrograph observed at the outlet.

In the Fig. (1.a) a schematic representation of a natural catchment system has been given. In this representation the catchment has been considered as a lumped system, whose components are functions of time only. The concept of lumped system simplifies the solutions to a great extent, which is an obvious advantage. The disadvantage is that the spatial variations of the components cannot be taken into account.

Even then, various research workers have studied the behaviour of the catchment and developed the concept of Unit hydrograph, Linear reservoir, Non-linear reservoir, Linear channel, Time area diagram, Genetic principle etc. These concepts are discussed briefly in the following sub-sections.

1.1.1 Unit Hydrograph Theory

In the year 1932 Sherman postulated the unit Graph theory. The theory states that,

(1) A natural catchment may develop a characteristic unit hydrograph of direct runoff resulting from a unit depth of rainfall excess generated uniformly over the catchment area at a uniform rate within a specified unit duration, which will reflects all the combined physical characteristics of the catchment.

(2) The base or time duration of the hydrograph of direct runoff due to a rainfall excess of unit duration is constant irrespective of the amount of rainfall excess generated uniformly over the catchment area.

(3) The ordinates of direct runoff hydrograph of a common base time are directly proportional to the total amount of direct runoff represented by each hydrograph.

(4) The rainfall excess generated in the anticedent durations do not affect the time distribution of surface runoff due to the rainfall excess generated in the given duration.

The uniformity of rainfall excess over the unit duration and over the catchment area is an essential conditions of the theory. In this theory the catchment has been assumed to be a homogeneous, time invarient linear system. The theory has been illustrated in the fig. (1.b). It implies that when a unit hydrograph of a given effective rainfall duration is available the runoff hydrograph of several durations can be derived.

It also implies that the UH of other durations can also be derived. A general method of derivation of UH of any required duration is the well known S-hydrograph method. It was first suggested by Morgan and Hullinghors(1939).

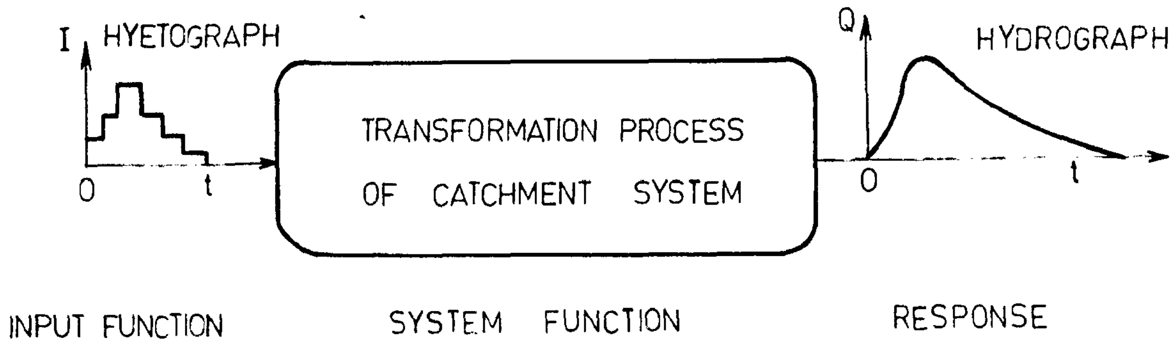


FIG. 1a_CATCHMENT AS AN ENGINEERING SYSTEM

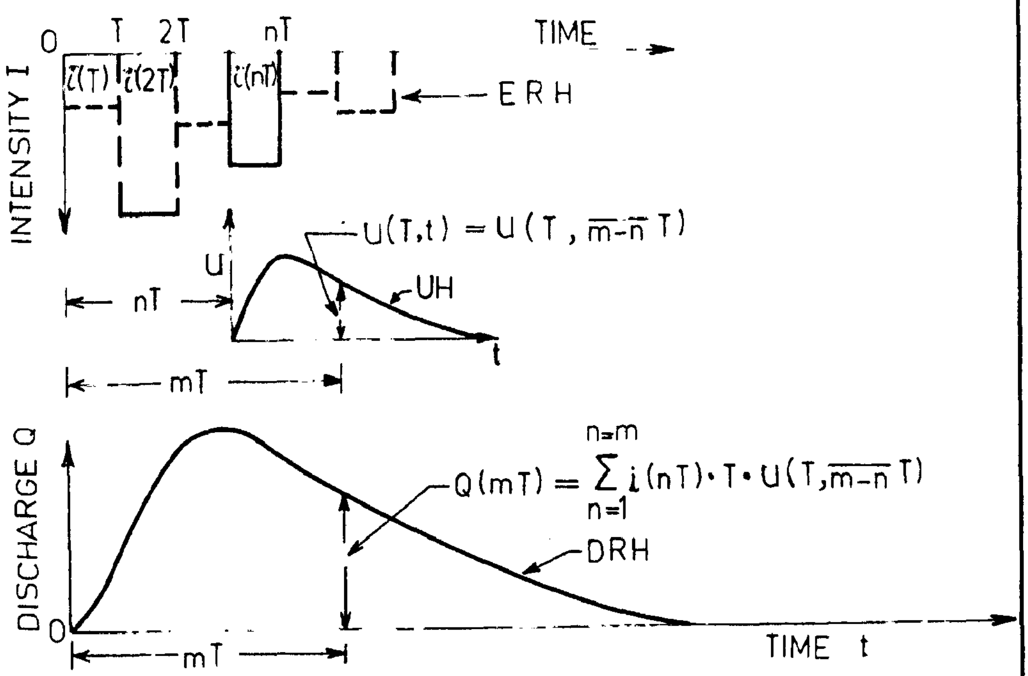


FIG.1b_UNIT HYDROGRAPH THEORY

The theoretical S-hydrograph is a hydrograph produced by a continuous rainfall excess generated for an infinite period at a constant rate say I per unit time. This hydrograph assumes a stretched S-shape and its ordinates ultimately approach the rate of effective rainfall. The S-hydrograph can be constructed graphically by summing up a series of identical UH spaced at intervals equal to the duration of rainfall excess from which it was derived. After the S-hydrograph is constructed the UH of any given duration t_o can be derived by offsetting the position of S-hydrograph by a period equal to the desired duration to. The UH ordinates then are given by

$$U\ (t_o, t) = \frac{S_t - S_{t\ -\ to}}{I\ t_o} \qquad (I.A)$$

Therefore, once a UH for a catchment is established the direct runoff hydrograph for any uniform intensity rainfall excess over unit durations can be computed. If rainfall excess has occured for several unit durations the direct runoff hydrograph can be computed by principle of superposition.

However the assumption of uniformity of precipitation over the unit duration is always likely to be violated. To overcome this difficulty the concept of instantaneous hydrograph was introduced by Clark[2] in the year 1945 for hydrograph analysis. Instantaneous Unit hydrograph is the unit hydrograph of infinitesimally small duration of rainfall excess. In the limit when the infinitesimally small

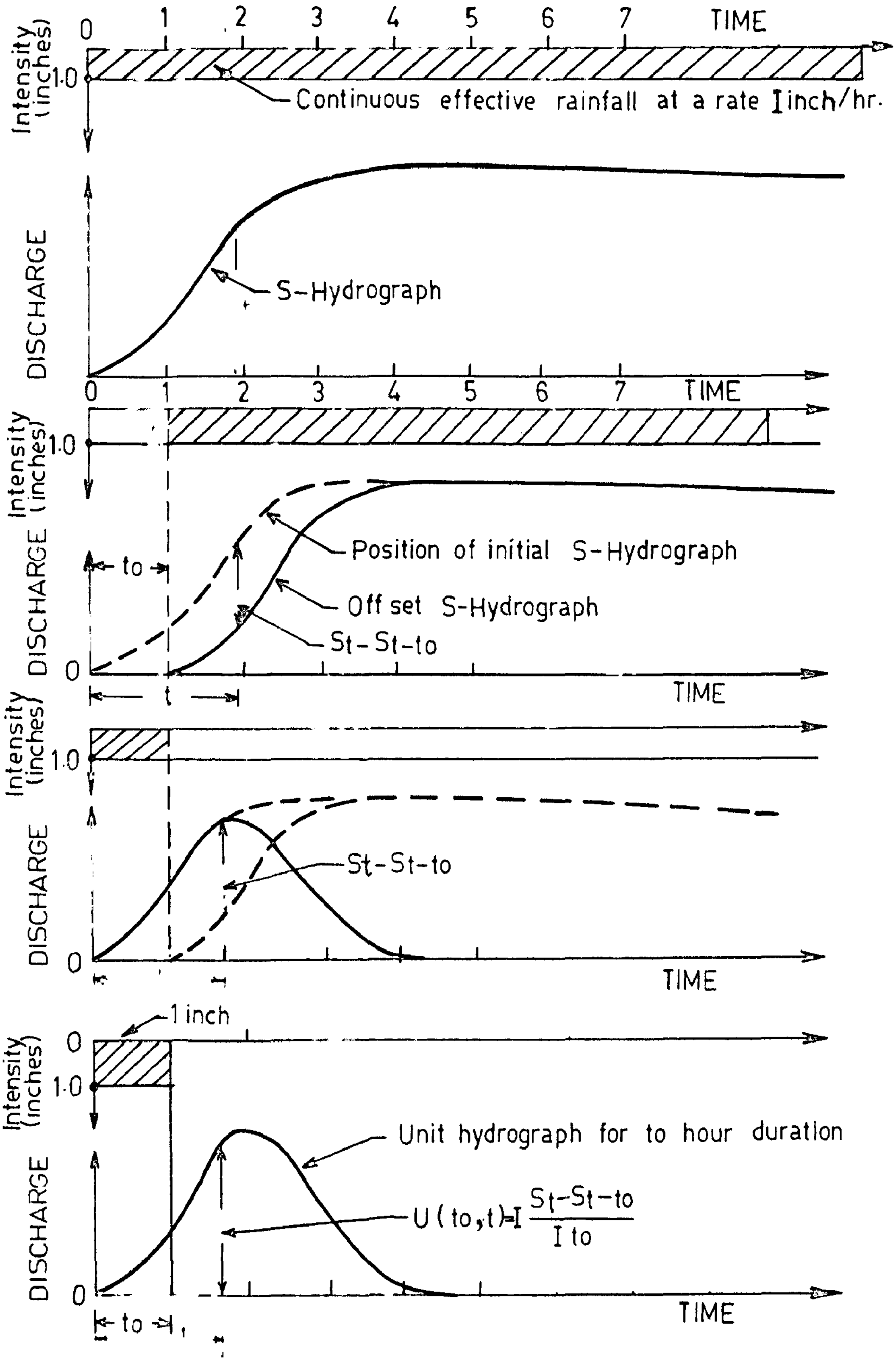


FIG. 1c _ DERIVATION OF UH FROM S-HYDROGRAPH duration of rainfall excess tends to zero, the IUH reduces to be the first derivative of the S-hydrograph whose ordinates have the dimensions of time^{-1}, and whose total area works out to be unity a pure number. This property of the IUH enabled the application of convolution integral(DUHAMEL INTEGRAL) to study the rainfall runoff relationship and representation of the transformation process in mathematical form

$$Q(t) = \int_{0}^{t' \leq t_o} I_{(\tau)} \, U_{(t-\tau)} \, d\tau \qquad (1.B)$$

Where

$I_{(\tau)}$ = Input function

$U_{(t-\tau)}$ = Kernel function (the system function)

$Q_{(t)}$ = Output function

and $t' = t$ when $t \leq t_o$ and $t' = t_o$ when $t > t_o$

The IUH concept has been illustrated in the figure (1.d).

The system function relating to the ~~catching~~ catchment system in the above expression is known as impulse response function also. The properties of this function are,

(i) $0 \leq U(t) \leq A$ (a positive value) when $t > 0$ (1.C)

(ii) $U(t) = 0$, when $t \leq 0$ (1.D)

(iii) $U(t) \rightarrow 0$, when $t \rightarrow \infty$ (1.E)

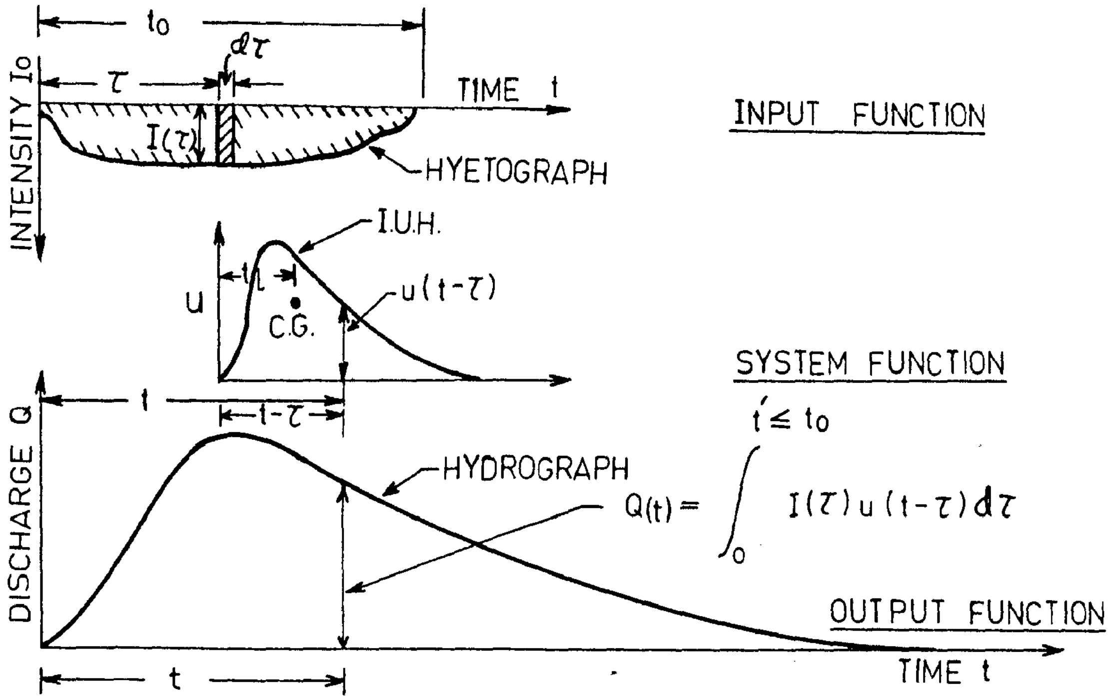


FIG. 1d_INSTANTANEOUS UNIT HYDROGRAPH

(iv) $\int_{0}^{\infty} U(t)\, dt = 1$ (1.F)

(v) $$\frac{\int_{0}^{\infty} U(t)\, .\, t.\, dt}{\int_{0}^{\infty} U(t)\, dt} = t_L, \text{ the Lag time} \quad (1.G)$$

The concept of catchment response to instantaneous unit input provided a powerful tool to compute the surface runoff hydrograph of any distribution of lumped input the rainfall excess.

The Unit hydrograph for a finite unit duration is emporically derived from a recorded isolated event in the catchment. But, emperical derivation of IUH from the recorded events is not possible because no uniform precipitation of infinitesimally small duration over a natural catchment can be thought of. However in practice theoretical concepts are used to derive IUH.

One such method was given by Chow[3] in the year 1962. He used a procedure for approximate determination of IUH from given effective rainfall hyetograph (ERH) and direct runoff hydrograph (DRH). In this method the IUH ordinate at time t is simply equal to the slope of the S-hydrograph at time t. The S-hydrograph can be derived as explained above in this sub-section. As the S-hydrograph derived from the actual data cannot be too exact, the IUH derived by this method is also approximate.

Other methods for determining IUH involve use of conceptual identities like linear reservoir, non-linear reservoir, linear channel and time-area diagram etc. Most popular among them is the concept of treating the catchment system as a cascade of identical linear reservoirs in series proposed by Nash(4) in the year 1957. This concept has been used in derivation of IUH for a natural catchment in this study and is appended at appendix 3.

The various conceptual identities mentioned above are discussed briefly in the following sub-sections -

1.1.2. The Reservoir Concept :

The catchment action on its input precipitation is analogous to the action of a reservoir on its inflow hydrograph. A reservoir also translates and attenuates the inflow hydrograph by regulating its outflowing discharges over certain time period. This analogy suggests that a natural catchment could perhaps be studied by application of reservoir concept. Depending upon the inter-relationship among the inflow (I), storage (S), and outflow (Q), the reservoirs may be classified as linear or non-linear.

1.1.2.1. The Linear Reservoir :

It is a conceptual reservoir in which storage S is directly proportional to the outflow Q -

i.e. $S \propto Q$ (1.H)

or $S = K\,Q$ (1.I)

Where K is the storage constant, which has dimension of time. In representing catchment action it stands for the average delay time imposed on the input rainfall by the catchment.

1.1.2.2. The Non-Linear Reservoir:

It is a conceptual reservoir in which the storage S and outflow Q are not directly proportional. General functional relationship for such a reservoir can be written as

$$S = C \cdot Q^{B} \qquad (1.J)$$

Where C and B are dimensional constants which may be function of input I, Storage S, and outflow Q or their derivatives. For a non-linear reservoir model, C and B are the two characteristic parameters.

1.1.3. The Linear Channel (Concept):

It is a conceptual channel in which the time (T) required to translate a discharge Q of any magnitude through a channel reach of length (x) is always constant. It implies that when an inflow hydrograph is routed through the linear channel its shape and size is not affected. In functional form it may be stated that if $I = f(t)$ is the inflow and T is the translation time for the channel reach of length x then, the outflow $Q_{(t)}$ will be given by

$$Q_{(t)} = f(t-T) \qquad (1.K)$$

1.1.4. The Time Area Concept

The basic concept of the time-area diagram is that a natural catchment can be considered analogous to a linear channel carrying a spatially varied flow. Total area A of the catchment is divided into n sub-areas of individual area ΔA_j ($j = 1, 2, \ldots\ldots, n$), such that $\sum_{j=1}^{n} \Delta A_j = A$, by isochrones of travel time Δt.

It the rainfall excess over the individual inter isochronal areas be I_j ($j = 1, 2, \ldots\ldots n$) then total runoff generated on the jth sub-area shall be $I_j \Delta A_j$.

If T be the time of travel of the runoff from the jth sub-area to the outlet then $T = (j-1) \Delta t$. Let rainfall occur only for the first time interval Δt then the runoff at the outlet due to jth sub-area shall be given by,

$$Q_{(j)} = I_j \; \Delta A_j \; f\,(t-T, \; \Delta t) \qquad (1.L)$$

(By application of linear channel concept).

Dividing both sides by A, it can be written as :

$$\frac{Q(j)}{A} = I_j \frac{\Delta A_j}{A} f\,(t-T, \; \Delta t)$$

Therefore for the whole catchment

$$\sum_{j=1}^{n} \frac{Q(j)}{A} = \sum_{j=1}^{n} I_j \frac{\Delta A_j}{A} \; f\,(t-T, \; \Delta t)$$

Setting $\sum_{j=1}^{n} \frac{Q(j)}{A} = W\,(t)$

$$W(t) = \sum_{j=1}^{n} I_j \frac{\Delta A_j}{A} f(t-T, \Delta t)$$

Taking I_j = Unity for all sub - basins

$$W(t) = \sum_{j=1}^{n} \frac{\Delta A_j}{A} f(t-T, \Delta t) \qquad (1.M)$$

If $\Delta t \to 0$ the above function gives a smooth curve. The ordinates of this curve $W(t)$ are proportional to the sub-area projected on the channel. So, it is called time-area-concentration diagram.

1.1.5. The Genetic Principle

The genetic principle is based on the concepts of isochrones. The isochrones are the lines joining the points of equal time of travel of water particle to the outlet. This explains that the process of flood formation is a result of lag and summation of seperate volumes of water inflowing from some seperate catchments. Mathematically the principle can be expressed as

$$Q(t) = \int_0^t I_{(t)} \frac{dA}{dt} \, dt \qquad (1.N)$$

Where

t = Runoff time lag

$I(t)$= Intensity of rainfall excess at time t.

A = Area of the catchment.

If the rainfall excess over the catchment be observed during Ist time interval only, the runoff at the outlet shall be given by

$$Q_{(1)} = I_{(1)} \times A_1$$

$$Q_{(2)} = I_{(1)} \times A_2 \quad \text{etc.} \qquad (1.P)$$

The catchment areas A_1, A_2 etc. (Fig. 1.e) and (1.f) represent the sub-watersheds limited by adjacent isochrones drawn according to the assumed time interval $\Delta\tau$. Taking $\Delta\tau$ to be the unit in general the genetic formula can be written as

$$Q(t) = \sum_{\tau=1}^{t} I_{(\tau)} \times A_{(t-\tau+1)} \qquad (1.Q)$$

Where

$I_{(\tau)}$ = Average intensity of rainfall excess over the duration $\Delta\tau$.

t = current time

$A_{(t-\tau+1)}$ = Sub watershed area bounded by the $(t-\tau)$ and $(t-\tau+1)$ unit isochrones.

For an instantaneous input I the runoff hydrograph is given by

$$Q_{(t)} = I \times (A_t - A_{t-1}) \qquad (1.R)$$

Where,

A_t = Area of the catchment between 0 and t unit isochrones.

$A_{(t-1)}$ = Area of the catchment between 0 and (t-1) unit isochrones.

Genetic principle does not take into account the transformation of the discharges in the channel section.

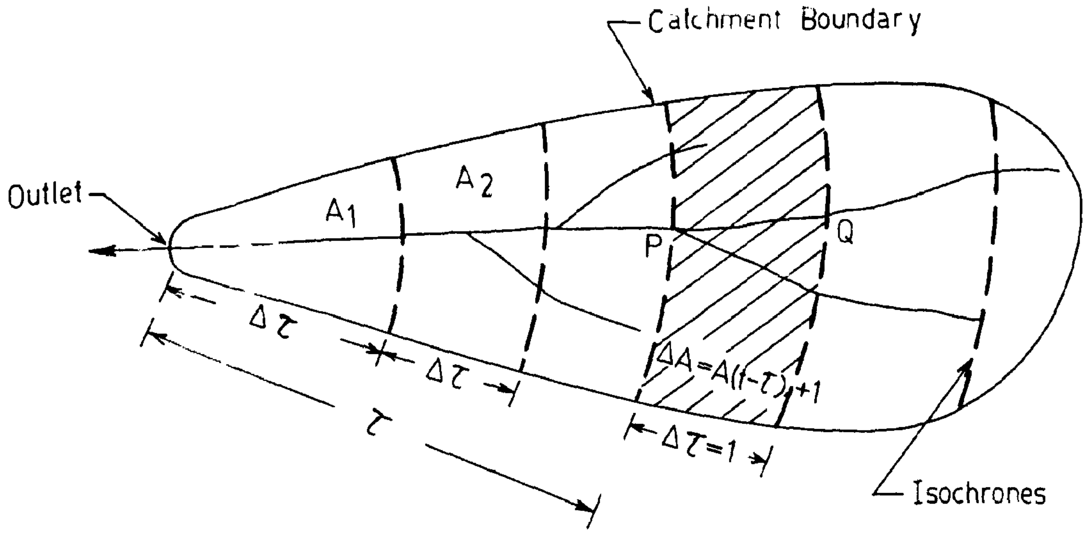


FIG.1e_GENETIC PRINCIPLE OF RUNOFF LOCATION OF ISOCHRONES

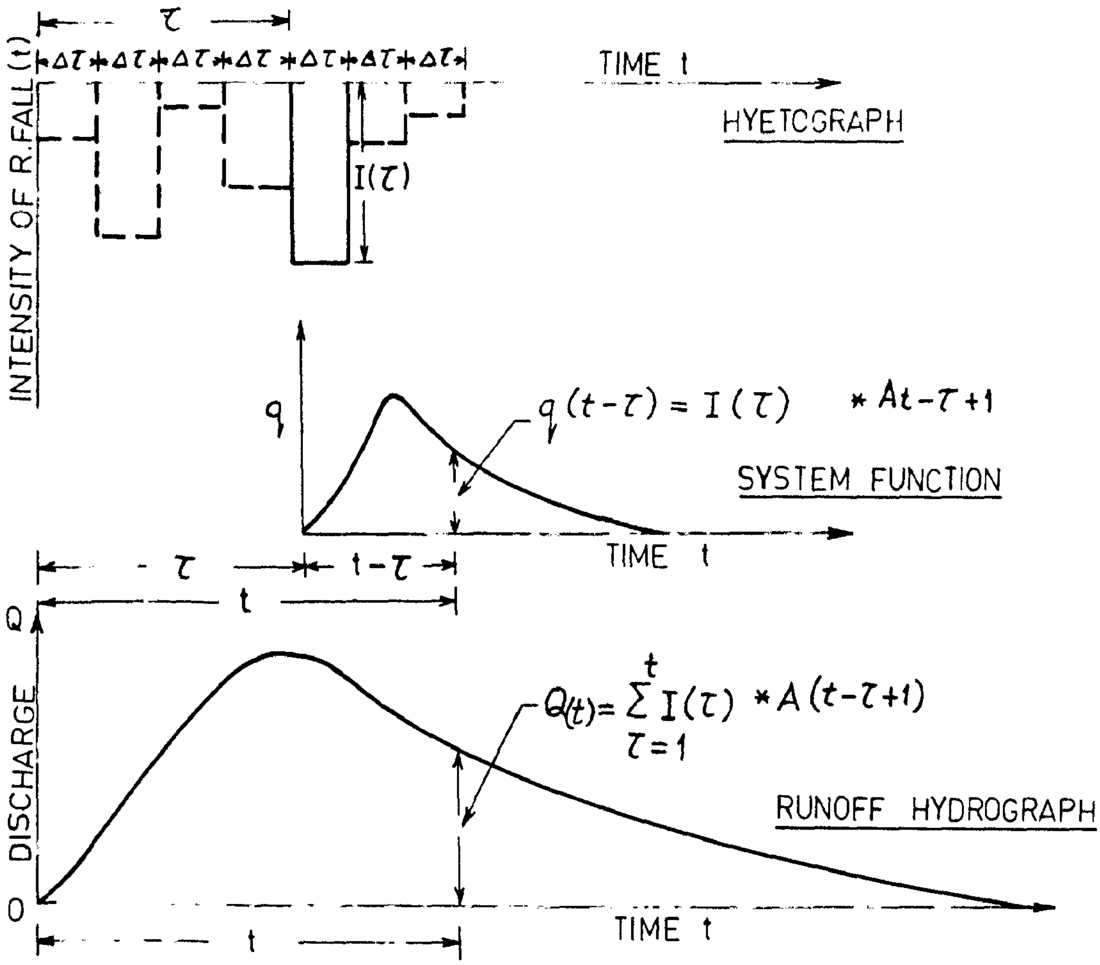


FIG.1f_GENETIC PRINCIPLE OF RUNOFF (The Runoff Hydrograph)

CHAPTER - 2

REVIEW OF SOME CONCEPTUAL MODELS

2.0 INTRODUCTION

For establishing a suitable relationship between the rainfall excess and the runoff, a mathematical model or better known, a conceptual model is considered to be more scientific. The obvious advantage in such an approach is that independent identities of various factors which affect the runoff process are condensed in a few parameters.

Commonly two approaches to this effect viz physical approach and black box approach are followed.

In physical approach actual role of the catchment in affecting the runoff process is investigated. The model is divided to behave as nearly as possible in accordance with the known physical laws. Various components of the model are identified as representing various stages of the process.

In black box approach the physical laws are taken care of by a system function, generally given by mathematical relationships among various parameters. The necessity and adequacy of the parametric values are judged by comparison of model output and output of the actual system. The examples to this approach are, Sherman's[1] UH theory, Bernards[5] method, Clark's[2] method, Nash's[4] cascade of linear reservoirs, Dooge's[6] linear channel model etc.

Based on the nature of the input function considered the conceptual models proposed by various investigators may be grouped in two categories

(1) Lumped models

(2) Distributed models

2.1 LUMPED MODELS

In these models the input function is function of time only. It does not have spatial coordinates. It is considered to be located at a single point in the working space. Some of such models are discussed in the following paragraphs.

O'Kelly[7] proposed that the instantaneous hydrograph for a catchment could be approximated by the discharge from a linear reservoir into which appropriate unit volume of inflow took place in a given time T, the rate of inflow varying with time as the ordinates of an isoceles triangle.

The outflow was assumed to be the IUH. This IUH could be expressed as a mathematical function of time containing two parameter T and K. The values of T and K were chosen by visual comparison of the computed UH with that derived from the observed short duration intense storms.

Clark[2] derived the instantaneous unit hydrograph by routing the time area concentration diagram through a linear reservoir. Time of concentration T and the storage coefficient K were the two parametersof the model T was defined as time interval from the end of excess rainfall to a point on the falling limb of the hydrograph where the ratio of rate of decrease in discharge to total discharge was greatest, and K was a coefficient of linear storage discharge relationship,

$$S_t = K\,Q_{(t)} \qquad (2.A)$$

Where S_t is storage in the reservoir at the instant t and $Q_{(t)}$ the outflow rate at the same instant t. The coefficient K is evaluated as

$$K = \frac{Q}{dQ/dt} \qquad (2.B)$$

Where, Q is the rate of direct surface runoff at the point of inflexion on the falling limb of the hydrograph.

Nash[4] proposed a more general mathematical model for IUH. He assumed that any catchment could be represented by a series of n linear reservoirs each having its storage co-efficient K with the outflow from the first becoming inflow for the 2nd and so so on. The outflow hydrograph from the nth reservoir for an instantaneous unit input in the Ist reservoir was considered to be the instantaneous unit hydrograph for the basin. The IUH ordinates were given by

$$U_{(t)} = \frac{1}{K\,\overline{|(n)}}\left(\frac{t}{K}\right)^{n-1} \cdot e^{-t/K} \qquad (2.C)$$

The parameters of the model n and K were evaluated by method of moment from the observed rainfall excess hyetograph and direct runoff hydrographs, which gave the following relationships

$$M_{DRH1} - M_{ERH1} = n\ K \qquad (2.D)$$

and
$$M_{DRH2} - M_{ERH2} = n\ (n+1)\ K^2 + 2\ nK\ M_{ERH1} \qquad (2.E)$$

Where

M_{ERH1} = Ist moment of the effective rainfall hyetograph about the origin.

M_{ERH2} = 2nd moment of the effective rainfall hyetograph about the origin.

M_{DRH1} = Ist moment of the direct runoff hydrograph about the origin.

M_{DRH2} = 2nd moment of the direct runoff hydrograph about the origin.

The definition of moments adopted here are those prevalent in statistics i.e.

$$M_n = \frac{\int_{-\infty}^{+\infty} y\ x^n\ dx}{\int_{-\infty}^{+\infty} y\ dx} \qquad (2.F)$$

Where

M_n = nth moment

y is ordinate and x abscissa.

2.2 DISTRIBUTED MODELS

In these models the input function involves spatial coordinates as well. In the previous section the model whose input function is lumped have been discussed. In nature the rainfall excess is rarely lumped, it is always distributed. In this section some of the models which are capable of accounting for spatial distribution of rainfall excess are discussed.

Dooge[6] introduced the conceptof linear channel and modified the Nash-model by adding one linear channel before each linear reservoir in the series. He divided the catchment area by means of isochrones and represented each inter isochronal area by a set of linear channel and linear reservoir in the series. The outflow from the linear channel was represented by time area concentration diagram and considered as input to the linear reservoir of the set. The output was obtained by adding the partial curves obtained by routing the time area concentration diagram for the upper most reach of the basin through n-linear reservoirs, next through (n-1) linear reservoirs and so on. A general equation for the IUH of the model for equally spaced identical linear reservoirs was given as

$$U_{(t)} = \frac{1}{T} \int_{0}^{t/K} P(m,n-1) \quad W(\tau') \, dm \qquad (2.6)$$

Where T = Maximum translation time

t = current time measured from time of occurance of the instantaneous rainfall excess.

$P(m,n-1)$ = Poisson's distribution function.

τ = A variable translation time

$m = (t - \tau)/K$

K = Storage coefficient for linear reservoir and translation constant for the linear channel.

$n(\tau)$ = Number of linear reservoirs down stream of τ.

and $W(\tau')$ = Ordinate of the dimensionless time area concentration diagram at time τ'.

As the catchment was divided into several parts by isochrones and input on each part was fed to the model separately, it was possible to account for the spatial distribution of rainfall excess.

Laurenson[8] proposed the concept of distributed input in non linear reservoirs. He divided the catchment by isochrones in several sub-areas. These sub-areas were represented by non-linear reservoirs in series. The inputs were routed through the reservoirs to get the output. This model tried to account for the following features -

(i) Rainfall excess is variable in time and space.

(ii) Storage in the catchment is distributed not lumped.

(iii) Storage discharge relation is nonlinear,

(iv) Different input elements pass through different amount of storages.

The values of the parameters K_1, K_2, K_3.... the storage coefficient of non-linear reservoirs were estimated by observed rainfall runoff records by iteration processes.

Mathur[9] proposed a series model of linear channels to simulate catchment action on distributive nature of input function. In this model the catchment area is divided into subwatersheds according to the scheme,

$$U_{(1,t)} = A_t - A_{t-1} \qquad (2.H)$$

Where $U_{(1,t)}$ represent UH of unit duration, t the current time and A_t the area of the catchment between zero and t unit isochrone.

Each subwatershed is conceptually represented by a linear channel in the series which receives two types of inputs, primary input and secondary input. Primary input is the input received by the linear channel at its upper bound translated down to it from upstream linear channel and secondary input is the distributed input over the subwatershed allotted to the linear channel. It has been assumed that both these inputs are received by the linear channel simultaneously. The concept of primary input maintains the continuity of the runoff hydrograph and the secondary input takes care of the distributive nature of the input rainfall. The assumption of simultaneous translation of both primary and secondary input to the linear channel is valid when the number of linear channel is large and correspondingly the length of linear channel is small. This model has been found capable of accounting for the unevenness of spatial distribution of precipitation over small drainage basins.

Mokliak(10) and Tchekyshkina have proposed a theoretical scheme for determination of discharges from snow melt and rainfall based on general law of runoff formation.

In this scheme catchment area is divided by channel isochrones after maximum velocity of flow in the channel section for a given time interval. The hydrograph ordinates are calculated by specific genetic formula

$$Q_m = \int_o^t h_t \frac{df}{dt} dt \qquad (2.I)$$

Where, t = runoff time lag

h_t = rainfall excess

f = catchment area

Q_m = discharge,

and transformation of discharges in channel section by lag coefficients given by the formula

$$r_t^{(\tau)} = \frac{\lfloor t-1}{\lfloor \tau-1 \ \lfloor t-\tau} r^{\tau} (1-r)^{t-\tau} \qquad (2.J)$$

Where r = constant value of travel coefficient in the isochronal section of the river.

τ = Travel time

t = current time

$r_t^{(\tau)}$ = the lag coefficient for τth sub area at instant t.

Such that,

$$\sum_{t=\tau}^{\infty} r_t^{(\tau)} = 1.00 \qquad (2.K)$$

For ordinary rivers of plains r = 0.7 to 0.8, for rivers having large flood plains r = 0.5 to 0.6 and for marshy and over grown stretches of rivers r = 0.35 to 0.45 have been suggested.

The authors have also suggested that to take into account specific features of formation of low, average and high floods on the river there is a possibility of two or three U.H.designs.

CHAPTER - 3

THE PROPOSED RESPONSE MODEL

3.0 THE PROBLEM

Rainfall excess generated over a natural catchment due to storm precipitation is variable in time and space. The catchment action transforms it into direct runoff which is observed at the outlet. As discussed in section 1.0 the transformation process is very complex in nature. The mechanism of runoff can not be studied by direct application of physical laws.

From review of the efforts made in this direction it appears that the transformation process may be studied by conceptual model representation.

In the following sections an attempt is made to identify the transformation process by a two parameter conceptual model based on certain assumptions.

3.1 THE ASSUMPTIONS

To develope the proposed response model following assumptions have been made -

(1) Natural catchment is a homogeneous time invariant linear system.

(2) The input function i.e. rainfall excess may be taken as lumped over smaller duration of time and over smaller areas.

(3) No regulation of natural flow either exists or is taken into account.

(4) No water is lostfrom the rainfall excess generated in course of its transportation and transformation in river channel or on its flood plains.

(5) The discharges due to the rainfall excess reach the outlet in distributed manner. In doing so, they follow the laws of binomial distribution.

3.2. THE AREA ANALYSIS

For an instantaneous lumped input I to a catchment system, the convolution suggests that,

$$Q_{(t)} = I \times U_{(o,t)} \qquad (3.A)$$

Whoro

$Q_{(t)}$ = Runoff at instant t.

$U_{(o,t)}$ = IUH ordinate at instant, t.

From the genetic principle of runoff it is given by

$$Q_{(t)} = I \times (A_t - A_{t-1}) \qquad (3.B)$$

Where

$Q_{(t)}$ = Runoff at instant t.

A_t = Area of the catchment between 0 and t unit isochrones.

A_{t-1} = Area of tho catohment between 0 and (t-1) unit isochrones.

Thorofore,

$$U_{(o,t)} = A_t - A_{t-1} \qquad (3.C)$$

The equation (3.C) suggests a proportional scheme for the division of catchment area in to sub-areas in proportion to the IUH ordinates. To distinguish these from the isochronal areas it is named as 'Response Sub-area'. And the lines dividing the catchment area have been named as 'Response time Contour' to distinguish then from the isochrones.

The response sub-area may be defined as the area of catchment whose impact is felt at the outlet after the time unit indicated by lower bound of the response time contour enclosing the sub-area.

The response sub-areas involve two quantities (i) the time interval of the response contour enclosing them and (ii) the areal magnitude of the response sub-area. Both the quantities are decided on the basis of the instantaneous unit hydrograph derived for the catchment from observed records. As the assumption is that the catchment is a time invariant linear system, Nash's model for determination of IUH has been adopted. Therefore

$$U_{(t)} = \frac{1}{K\,\Gamma(n)} \left(\frac{t}{K} \right)^{n-1} e^{-t/K} \qquad (3.D)$$

The two parameters n and K of Nash's model are evaluated by the method of moments[4]. Thereafter the IUH ordinates are calculated at suitable time intervals. An infinitesimally small time interval will be the ideal time unit at which the response contours must be drawn. Since the system is assumed to be lumped a finite time interval is adopted. The range in which the adopted time interval $\Delta\tau$ must lie may mathematically be defined as,

$0 \angle \Delta \tau \leq$ Time to peak of IUH

The IUH derived on the basis of Nash's model[4] gives an infinite recession. For the purposes of the proposed model the portion of the recession limb from where the ordinates become sufficiently small compared to peak value, should be curtailed and remaining ordinates adjusted to make the volume unity. These adjusted ordinates of the IUH should be used for computing the area of respective response sub-areas. After the response sub-areas have been decided quantitatively, the next step is to mark them on the catchment area map.

It is assumed that the nature of the response contours will mainly depend on the topographical features of the catchment area. To take them into account the contour map of the catchment is divided into arbitrary square grids. Each grid point is serially numbered and flow time from the grid point to the outlet are computed using any of the available relations (viz Manning's or Chezzys equation etc.). These flow times are assigned to the grid points and then flow time contours at the selected unit time interval $\Delta \tau$ are drawn. The areas bounded by two consecutive flow time contours are planimetered and compared with the required areas on the basis of the IUH ordinates. These may differ. If so, the response contours are drawn on the same map maintaining the nature similar to that of the flow contours so that the areas enclosed between the two consecutive response contours be same as calculated on the basis of IUH ordinates. These response contours are then transferred on a fresh map and used for further analysis in the process of runoff computation.

3.3 THE MECHANICS OF RUNOFF

The genetic principle of runoff which has been used for response sub-area analysis does not take into account the effect of spreading of the water flow due to changes in its flow velocities along the channel. It also neglects the effect of non-uniformity of velocity distribution over stream cross-section. An effort has been made to account for both these factors in the runoff mechanics postulated below -

As shown in figure (3.a) a water particle moving from P to the outlet with the maximum velocity of flow takes the time τ to reach the outlet, and from Q it takes the time $\tau + \Delta\tau$. If the area enclosed between the two response time contours be ΔA which is draining in the reach P Q of the stream and I (τ) be the total rainfall excess generated over the area ΔA at a uniform rate in time $\Delta\tau$, then total run-off volume received by the channel reach P Q is given by

$$R = \Delta A \times I(\tau) \qquad (3.E)$$

It is generally known that the flow velocities are non uniform over the channel section. It also varies along the channel. Under these circumstances total volume R of rainfall excess will not pass the outlet in time $\Delta\tau$, but will take longer time. Before time τ from start of the rain-fall excess over the sub-area ΔA the runoff contribution from this sub-area to the outlet is zero. Let the discharges be

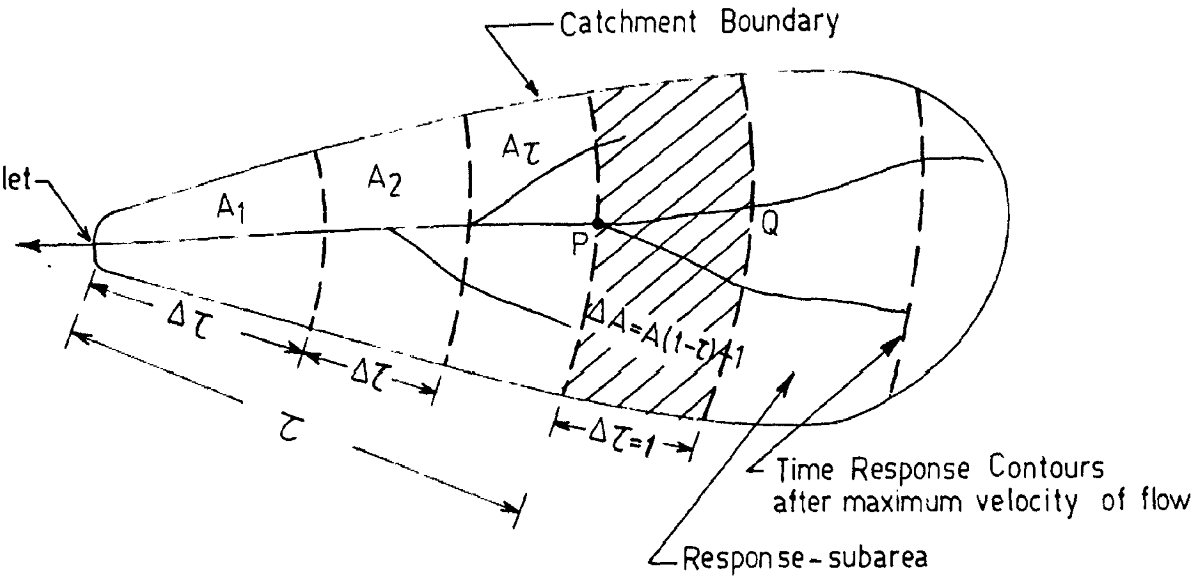


FIG. 3a_RUNOFF MECHANICS LOCATION OF TIME RESPONSE CONTOURS

(i) $R x C_{\tau,\tau}$ in the time interval τ and $\tau + \triangle \tau$

(ii) $R \times C_{\tau,\tau+1}$ in the time interval $\tau + \triangle \tau$ and $\tau+3 \triangle \tau$

(iii) $R x C_{\tau,\tau+2}$ in the time interval $\tau+2\triangle \tau$ and $\tau+3\triangle \tau$ and so on.

Then under the condition of no losses in the river channel or its flood plain

$$R = R\,(C_{\tau,\tau} + C_{\tau,\tau+1} + \ldots\ldots) \qquad (3.F)$$

or
$$\sum_{t=\tau}^{m} C_{\tau,t} = 1 \qquad (3.G)$$

As the coefficients $C_{\tau,t}$ perform the operation of distribution of runoff volume against time it is named as 'the runoff distribution coefficient'.

Study of runoff hydrographs indicate that the tunoff elements from the catchment in different time invervals form a skewed distribution of discharges at the outlet. In the proposed model the discharges at the outlet from any response sub-area ' are calculated by multiplying the total rainfall excess over the sub-area by the 'runoff distribution co-efficients'. Therefore the distribution of these co-efficients over the time must be in agreement with the skewed distribution of the discharges at the outlet.

Various skewed distributions which can take these effects into account are available, namely, the binomial distribution, the Poisson's distribution, Chi-square distribution, the

Gamma distribution etc. In the proposed model the binomial distribution has been adopted for explaining the mechanics of time distribution of runoff. A suitable expression for runoff distribution coefficient has been investigated in the following paras.

The mathematical expression for binomial distribution is $(p+q)^t$, $p+q=1$, p = probability of failures, q = probability of successes, and t = number of trials.

The expansion of the above binomial expression is given by

$$(p+q)^t = \binom{t}{o} p^t q^o + \binom{t}{1} p^{t-1} q + \binom{t}{2} p^{t-2} q^2 + \dots + \binom{t}{\tau} p^{t-\tau} q^{\tau} + \dots + \binom{t}{t} p^o q^t \quad (3.H)$$

The τth term of this expansion is given by

$$T_{\tau} = \frac{(t)\,!}{(\tau-1)!\,(t-\tau+1)!}\, p^{t-\tau+1}\, q^{\tau-1} \quad (3.I)$$

In surface water hydrology if an input is applied to a catchment in a chosen unit duration of time, however small its responses from the catchment last for several time intervals say t. Then the time of concentration is given by (t-1). Therefore replacing t by (t-1) in the above expression,

$$(p+q)^{t-1} = \binom{t-1}{o} p^{t-1} q^o + \binom{t-1}{1} p^{t-2} q + \dots + \binom{t-1}{\tau} p^{t-1-\tau} q^{\tau} + \dots + \binom{t-1}{t-1} p^o q^{t-1} \quad (3.J)$$

The τth term of the above expansion is given by

$$T_\tau = \binom{t-1}{\tau-1} p^{t-\tau} q^{\tau-1}$$

$$= \frac{(t-1)!}{(\tau-1)!\,(t-\tau)!} q^{\tau-1} p^{t-\tau} \qquad (3.K)$$

In the above expression q stands for probability of successes and p for probability of failures, such that, $p + q = 1.$ or $p = (1 - q)$

Bringing in the concept of probability of successes of the rainfall excess generated over the response sub-area to travel down the slopes and reach the channel section in the Ist time interval and then transformation of those discharges in various reaches of the channel section and probability of their reaching the outlet in different time intervals, the analysis is advanced further. Assuming C_τ to be the probability of successes of the rainfall excess generated over the τth response sub-area, and therefore setting $q = C_\tau$ and $p = (1 - C_\tau)$ the expression for T_τ can be written as

$$T_\tau = \frac{(t-1)!}{(\tau-1)!\,(t-\tau)!} C_\tau^{\tau-1} (1-C_\tau)^{t-\tau} \qquad (3.L)$$

As the probability C_τ is associated with the travel of rainfall excess to join the channel and become runoff, it has been named as 'the travel coefficient'.

The equation (3.L) was examined for its applicability for finding out 'the runoff distribution coefficients defined after eqn. (3.F) and (3.G) for computation of discharges from the τth response sub-area at different instants t. For $\tau = 1$ and $t = 1,2,\ldots$ etc. the above expression (eqn.(3.L)) gave the values of runoff distribution coefficients as follows

$T_1 = C_{1,1}$ = Runoff distribution coefficient for the first sub-area for first time interval

$$= \frac{(1-1)!}{(1-1)!\,(1-1)!}\, C_\tau^{(1-1)}\,(1-C_\tau)^{(1-1)}$$

$$= 1$$

$T_2 = C_{1,2}$ = Runoff distribution coefficient for the Ist sub-area for 2nd time interval

$$= \frac{(2-1)!}{(1-1)!\,(2-1)!}\, C_\tau^{\,1-1}\,(1-C_\tau)^{2-1}$$

$$= (1 - C_\tau)$$

Similarly

$$C_{1,3} = (1 - C_\tau)^2$$

$$C_{1,4} = (1 - C_\tau)^3, \text{ and so on}$$

From above,

$$\sum_{t=\tau}^{\infty} C_{\tau,t} > 1$$

But, the condition for the runoff distribution coefficients is that

$$\sum_{t=\tau}^{\infty} C_{\tau,t} = 1 \qquad (3.M)$$

So, the equation (3.L) needs modification to suit the requirement of equation (3.M). The equation (3.L) was therefore modified by multiplying the R.H.S. by C_τ, which gave

$$C_{\tau,t} = \frac{(t-1)!}{(\tau-1)!\,(t-\tau)!} C_\tau^{\tau} (1 - C_\tau)^{t-\tau} \qquad (3.N)$$

From equation (3.N) the values of the runoff distribution coefficients for Ist sub-area are

$$C_{1,1} = C_\tau$$

$$C_{1,2} = C_\tau (1-C_\tau)$$

$$C_{1,3} = C_\tau (1-C_\tau)^2 \qquad \text{etc.}$$

Such that

$$\sum_{t=1}^{\infty} C_{1,t} = 1 \qquad (3.P)$$

It may be noted that the above expression (eqn.(3.N)) is defined only for $t \geq \tau$, which is in agreement with the concept of travel time for the τth sub-area whose impact shall be felt at the outlet at time $t \geq \tau$ i.e. the upper bound of the enclosing response time contour of the response sub-area. In the proposed model the equation (3.N) has been adopted for calculation of 'the runoff distribution coefficients' which are given in terms of 'the travel co-efficients C_τ'.

The assumption of homogeneity of the catchment system enables to take C_τ constant for each and every response sub-area of the catchment.

Therefore 'the runoff distribution coefficient $C_{\tau,t}$ in terms of 'the travel coefficient' C_τ, 'the response sub-area number' τ, and 'the current time' t, is given by

$$C_{\tau,t} = \frac{(t-1)!}{(\tau-1)!\,(t-\tau)!}\; C_\tau^{\tau}\,(1-C_\tau)^{t-\tau} \qquad (3.N)$$

3.4 THE PARAMETERS OF THE MODEL

From the sections 3.2 and 3.3 it is evident that in the proposed model 'The Response Sub-area' and 'The Travel Coefficient' are the two basic parameters.

The method of quantitative evaluation and the demarcation of the response sub-area on the catchment area map have been explained in detail in section 3.2. Next step is estimation of the value of 'The travel coefficient'.

The selection of a suitable value of the 'travel coefficient' C_τ is a matter of trial and error, But a rough idea can be had from the following relationship.

$$1 > C_\tau \geq \frac{\text{Peak of UH of } \Delta\tau \text{ duration}}{\text{Peak of IUH}}$$

Where $\Delta\tau$ is the adopted time interval for drawing the response time contours.

The value of C_τ so derived may be adopted for computing the hydrograph of the rainfall runoff event recorded over the catchment. The observed and computed hydrographs are compared and if necessary value of C_τ adjusted till a reasonable match is obtained.

The value of C_τ which gives the best fit is adopted asparametric value of the model. The assumption of homogeneity enables adoption of constant value of C_τ for all response sub-areas of the catchment. Therefore, once the value of travel co-efficient C_τ is established, runoff distribution coefficients $C_{\tau,t}$ for all response sub-areas may be calculated by the relationship given in the equation (3.N).

3.5 THE STEPS IN APPLICATION OF THE PROPOSED MODEL TO A NATURAL CATCHMENT

As discussed in section 3.3 the proposed model has two basic parameters, viz. the response sub-area and travel co-efficient. The method of evaluation of these parameters has been discussed in detail in the -sections 3.2... and 3.4... The procedure of application of the proposed model to a natural catchment can thus be summarised as follows -

(i) An isolated, short duration, intense, wide-spread, uniform storm is selected from the observed records of the catchment. The runoff hydrograph observed at the outlet due to the selected storm is identified and isolated from the continuous records.

(ii) Baseflow is separated from the runoff hydrograph assuming a suitable distribution and storm runoff hydrograph is found out. Average depth of storm runoff is calculated over the entire catchment area.

(iii) From the observed rainfall records, the hyetograph of rainfall excess is drawn. For this prupose a suitable value of ϕ - index is adopted. A reasonably accurate value of ϕ - index can be arrived at from the analysis of storm rainfall and storm runoff depth assuming constant value of ϕ - index.

(iv) The IUH of the catchment is derived. The Nash's model(4) is proposed for the same. The value of the two parameters n and k of the Nash's model are evaluated by method of moments and IUH ordinates are computed by the relationship of equation (3.P).

$$U_{(t)} \frac{1}{k|\overline{n}} \left(\frac{t}{k} \right)^{n-1} e^{-t/k} \qquad (3.P)$$

The procedure has been outlined in appendix 3.

(v) Neglecting the recession limb of the IUH from the point where the ordinates become sufficiently small in comparison to peak value, the remaining ordinates are adjusted to make the area of the IUH unity.

(vi) If further smaller time interval are not possible, the time to peak of the IUH is identified and adopted as unit interval $\triangle \tau$ for drawing response contours. A unit graph for duration $\triangle \tau$ is also derived from the IUH.

(vii) The catchment area is divided in accordance with the proposed area distributive scheme, following the nature of flow contours as discussed in 3.2. The sub-areas so arrived at are the response-sub-areas. Thus the IUH ordinate at $\Delta\tau$ from the origin represents the Ist response sub-area, the ordinate at $2\Delta\tau$ from the origin represents 2nd response sub-area and so on.

(viii) Average rainfall excess over individual response sub-areas at time steps $\angle\tau$ during the storm are found out, such that the average depth over entire catchment equals to the observed storm runoff depth found out in (ii) above. This may require slight adjustment of the value of ϕ - Index adopted in(iii) above.

(ix) Trial value of the travel coefficient C_τ is chosen as per conditions laid down in section (3.4), and values of the runoff distribution coefficients $C_{\tau,t}$ for each response sub-area is calculated from the relationship given by equation (3.N).

(x) Runoff hydrograph for the catchment at the outlet is computed as per scheme given in Table 1.

(xi) The computed hydrograph is compared with the observed hydrograph and if suitable match is obtained the value of C_τ is adopted as parametric value of the model.

(xii) If not, the value of C_τ is suitably adjusted to improve the result and steps (ix) to (xi) are repeated until a reasonably well match is obtained.

SCHEME FOR COMPUTATION OF FLOOD HYDROGRAPH

esponse ub-area No. τ	Area of Resps. Sub-area	Rainfall excess (I) according to time interval: Time interval	Rainfall excess	Runoff generated AxI	Elements of Hydrograph ordinates according to time interval: 1	2	3	4	5	6
1	A	$C_{\tau,t}$			$C_{1,1}$	$C_{1,2}$	$C_{1,3}$	$C_{1,4}$		
		1	I_{a1}	R_{a1}	$C_{1,1} \cdot R_{a,1}$	$C_{1,2}R_{a,1}$	$C_{1,3}R_{a,1}$	$C_{1,4}R_{a1}$		
		2	I_{a2}	R_{a2}		$C_{1,1}R_{a2}$	$C_{1,2}R_{a2}$	$C_{1,3}R_{a2}$	$C_{1,4}R_{a2}$	
		3	I_{a3}	R_{a3}			$C_{1,1}R_{a3}$	$C_{1,2}R_{a3}$	$C_{1,3}R_{a3}$	$C_{1,4}R_{a3}$
2	B	$C_{\tau,t}$				C_{22}	C_{23}	C_{24}	C_{25}	
		1	I_{b1}	R_{b1}		$C_{22} \cdot R_{b1}$	$C_{23}R_{b1}$	$C_{24}R_{b1}$	$C_{25}R_{b1}$	
		2	I_{b2}	R_{b2}			$C_{22}R_{b2}$	$C_{23}R_{b2}$	$C_{24}R_{b2}$	$C_{25}R_{b2}$
		3	I_{b3}	R_{b3}				$C_{22}R_{b3}$	$C_{23}R_{b3}$	$C_{24}R_{b3}$
3	D	$C_{\tau,t}$					C_{33}	C_{34}	C_{35}	C_{36}
		1	I_{d1}	R_{d1}			$C_{33}R_{d1}$	$C_{34}R_{d1}$	$C_{35}R_{d1}$	$C_{36}R_{d1}$
		2	I_{d2}	R_{d2}				$C_{33}R_{d2}$	$C_{34}R_{d2}$	$C_{35}R_{d2}$
		3	I_{d3}	R_{d3}					$C_{33}R_{d3}$	$C_{34}R_{d3}$
			Total		Q_1	Q_2	Q_3	Q_4	Q_5	Q_6

OTE (1) $R_a = AxI_a$, $R_b = BxI_b$, $R_d = DxI_d$ etc. (2) Q_1, Q_2, Q_3 etc. are the final computed discharges at time intervals 1,2,3 etc. at the outlet.

Having established the parameters of the model it can be used to compute runoff hydrograph for any distribution of rainfall excess over the catchment as per scheme of Table 1.

The detailed application of the model on a natural catchment namely Ajay river basin in Bihar (India) is given in the Chapter 4.

CHAPTER-4

APPLICATION OF THE PROPOSED MODEL ON A NATURAL CATCHMENT

4.0. INTRODUCTION

The proposed response model has been applied to Ajay river catchment.

The Ajay river catchment is located in Chhotanagpur platan of the state of Bihar, (India).

The river Ajay originates from the hills on the border of Munger, Southal pargana and Hazaribagh districts at an elevation of about 1050 ft. above mean sea level. The Pathro and the Jainti are its main tributaries. The gauge site is located at Sikatia where the average bed level of the river is 530 ft.

The geographical situation of the catchment is between latitude 24° 5' to 24° 36' N and longitude 86°15' to 87° o' E. General orientation of the drainages in the catchment is from North-west to South-east. Total catchment area at Sikatia is 992 sq.miles.

The catchment area is spotted with low and scattered hills. In the plains there are scattered out crops of rocks punctured at places with pits exposing mica and white clay deposits. A considerable area of the catchment is covered by gneiss and schists. Soil cover is usually thin. The lands are hideously cut with gullies and ravines. The area shows either exposure of the rock itself diping towards north or immediate product of its decomposition. The forest in the catchment is scattered except in some portions of Hazaribagh district on some hillocks and some isolated patches in plains. These forests consist of fire wood plants, sal, Bamboos and thorny bushes.

Valleys and plains of the catchment are cultivated. The main crops are paddy in valleys and maiz in plains.

4.1. AVAILABLE DATA

Storm precipitation in the catchment and runoff data at Sikatia are collected by the Irrigation Department of Government of Bihar. Daily rainfall data at nine rain gauging stations in the catchment and daily average discharge data at Sikatia are available for analysis. The nine rain gauge stations are located at (a) Sarath (b) Dhobna (c) Maheshmunda (d) Madhupur (e) Sarwan (f) Daranga (g) Deoghar (h) Bichkhorwa and (k) Kiajori. These rain gauges are more or less uniformly distributed over the catchment. The rainfall was measured in inches and discharges in cubic feet per second. The rainfall and runoff data which were used for fitting and testing the model are given in appendix-2.

4.2 EVALUATION OF MODEL PARAMETER

As discussed in section 3.3, model parameters to be evaluated are (i) The Response Sub-area and (ii) The Travel Coefficient. For evaluation of response sub-area the first step is to derive instantaneous unit hydrograph of the catchment at the outlet. The derivation of IUH has been done in appendix 3. The resultant IUH is given in Fig.(4.a).

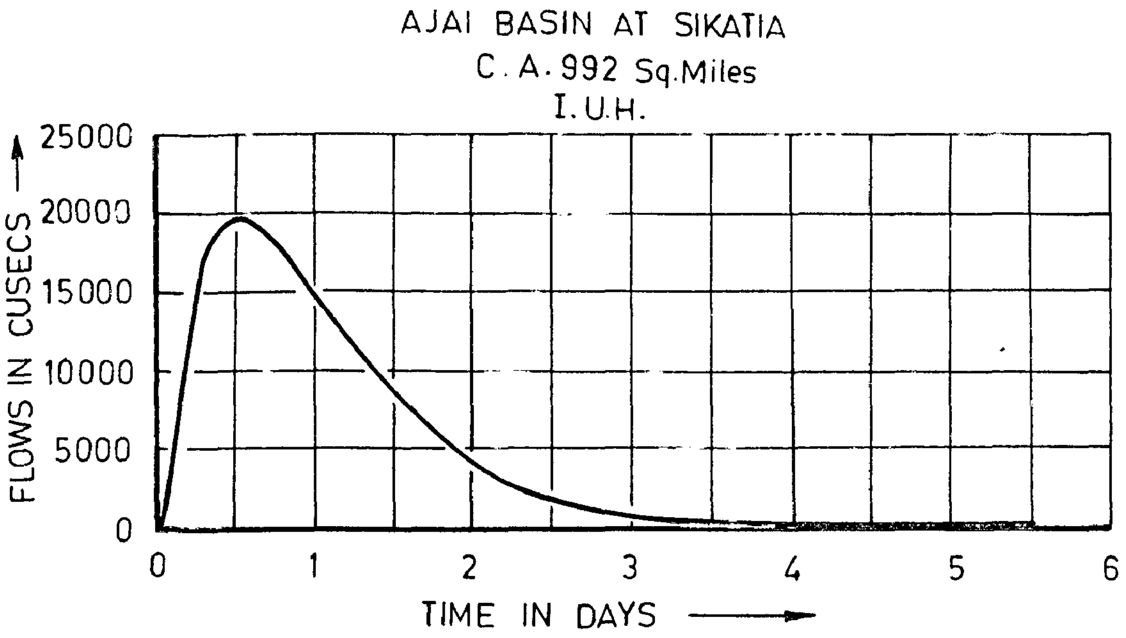


FIG.4a_INSTANTANEOUS UNIT HYDROGRAPH

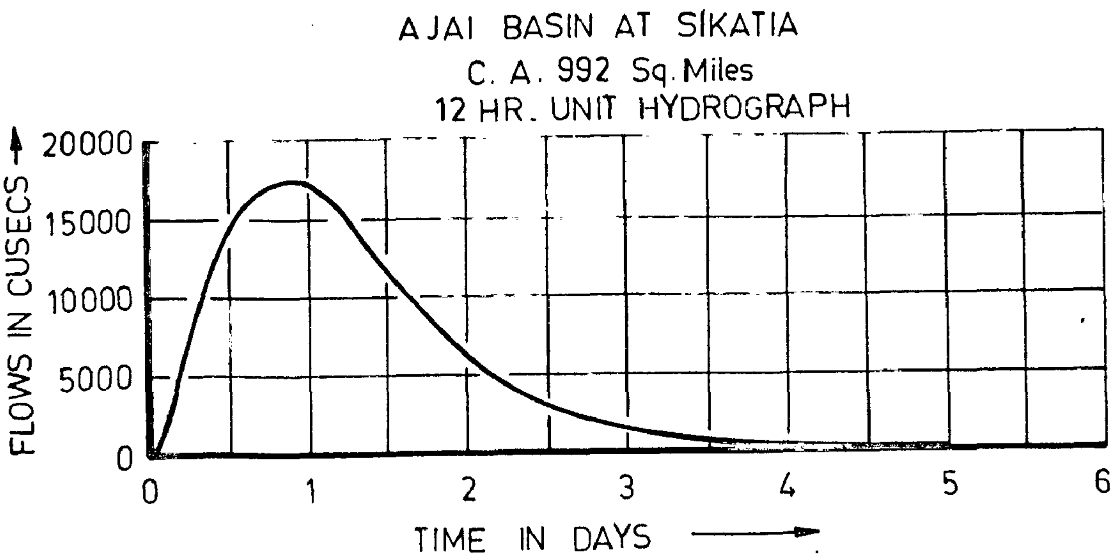


FIG. 4b _12 HR. UNIT HYDROGRAPH

A unit hydrograph for the duration equal to the time to the peak of the IUH has also been derived from the IUH Fig.(4.b).

According to the hypothesis postulated in section 3.2. response sub-area are proportional to the IUH ordinates. So, the response sub-areas have been computed in Table 2. The time interval adopted is 1/2 day the time to peak of the IUH.

TABLE 2

Time in days	IUH Ordinates in cusecs	Percentage of total	Area in sq.miles	Response sub-area in acres	Response Sub-area No.
0.5	19795	39.460	391.50	2,50,560	1
1.0	14777	29.472	292.37	1,87,117	2
1.5	8305	16.571	164.28	1,05,139	3
2.0	4124	8.250	81.88	52,403	4
2.5	1620	3.250	32.25	20,640	5
3.0	866	1.727	17.12	10,957	6
3.5	381	0.765	7.60	4,864	7
4.0	159	0.318	3.15	2,016	8
4.5	66	0.131	1.30	832	9
5.0	28	0.056	0.55	352	10
5.5	0	-	-	-	

The first trial value of the travel coefficient was approximated by the ratio of peak value of UH and peak value of IUH. Trial and error to give a good fit to the observed hydrograph was carried out and travel coefficient value was established at 0.97. Then applying the formula for runoff distribution coefficient ($C_{\tau, t}$) given in section 3.3.... The $C_{\tau, t}$ values for each response watershed were calculated (Appendix-4).

4.3 DIVISION OF THE CATCHMENT AREA IN RESPONSE SUB-AREAS

Following the procedure described under section 3.2. the area was divided as follows -

Total concentration time as obtained from UH of 1/2 day duration was 5.5 days. The elevation of the remotest point of the catchment is 1050 ft and that at the outlet is 530 ft. The length of the channel as measured on the plan works out to 56 miles or 2,95,680 ft. So average slope is 1 in 568.6.

Mannings formula for velocity of flow in open channel is

$$v = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad (4.A)$$

But the value of R and n are not known. So an assumption was made that the quantity $\frac{1.486}{n} \times R^{2/3}$ is constant. Putting this quantity equal to α the equation for v may be written as

$$v = \alpha \cdot S^{1/2} \qquad (4.B)$$

From the above para the length of the channel and time of concentration are known so velocity

$$v = \frac{\text{Length of channel}}{\text{Time of concentration}}$$

$$= \frac{295680}{5.5 \times 24 \times 3600} \text{ ft/sec.}$$

$$= 0.622 \text{ ft/sec.}$$

Similarly $S = \dfrac{\text{Fall from remotest point to outlet}}{\text{Length of the channel}}$

$$= \frac{520}{295680} = \frac{1}{568.6}$$

$\therefore$ From the above equation

$$\alpha = \frac{V}{\sqrt{S}} = \frac{0.622}{\sqrt{\frac{1}{568.6}}} = 14.732.$$

Now value of α is known.

The area was divided in 3 miles grid and at each grid point the elevation was estimated from the contours. Connecting that point to the outlet through the natural drainage time of travel to the outlet was calculated from the formula $T = \dfrac{L}{\alpha\sqrt{S}}$ (4.C)

Where, T = Time of travel in seconds

L = distance of the grid point from the outlet measured along the drainage in ft.

S = Average slope of the drainage.

α = 14.732 calculated above.

The travel time so calculated was assigned to each and every grid point. Then the equal time of travel was interpolated. These lines are referred henceforth as flow contours. The interval of the flow contours was taken equal to the time chosen for devision of the catchment into response sub-area. The inter flow-contour areas were planimetered and compared with the response sub-area computed in table-2. The two areas did not tally. So the flow contours were adjusted on the same map maintaining their nature so that the inter flow contour areas tally with the response sub-area. The adjusted flow contours were 'the response time contours'.

The response time contours for Ajay river catchment are given in the map at figure (4.c).

The Thiessen polygons were drawn on the same map for the nine rain gauge stations.

Respective areas according to response sub-areas and Thiessen polygon were measured and are given in table-3.

4.4. IDENTIFICATION OF RAINFALL EXCESS

From the available records following five storms were selected to examine the working of the model.

(i) Storm of 13-17 Oct. 1971

(ii) Storm of 3-7 June 1971

(iii) Storm of 9-13 July 1971

(iv) Storm of 28 Sept. to 2 Oct. 1972

(v) Storm of 11-13 Oct. 1973

Rainfall and runoff data for these storms are given in Appendix-2. Runoff hydrograph for full monsoon season of 1971 was plotted and master recession curve was derived Fig.(4-d) and Fig.(4-e).

FIG.4c — THE CATCHMENT OF RIVER AJAI
BIHAR (INDIA)
SCALE 1 INCH=4 MILES

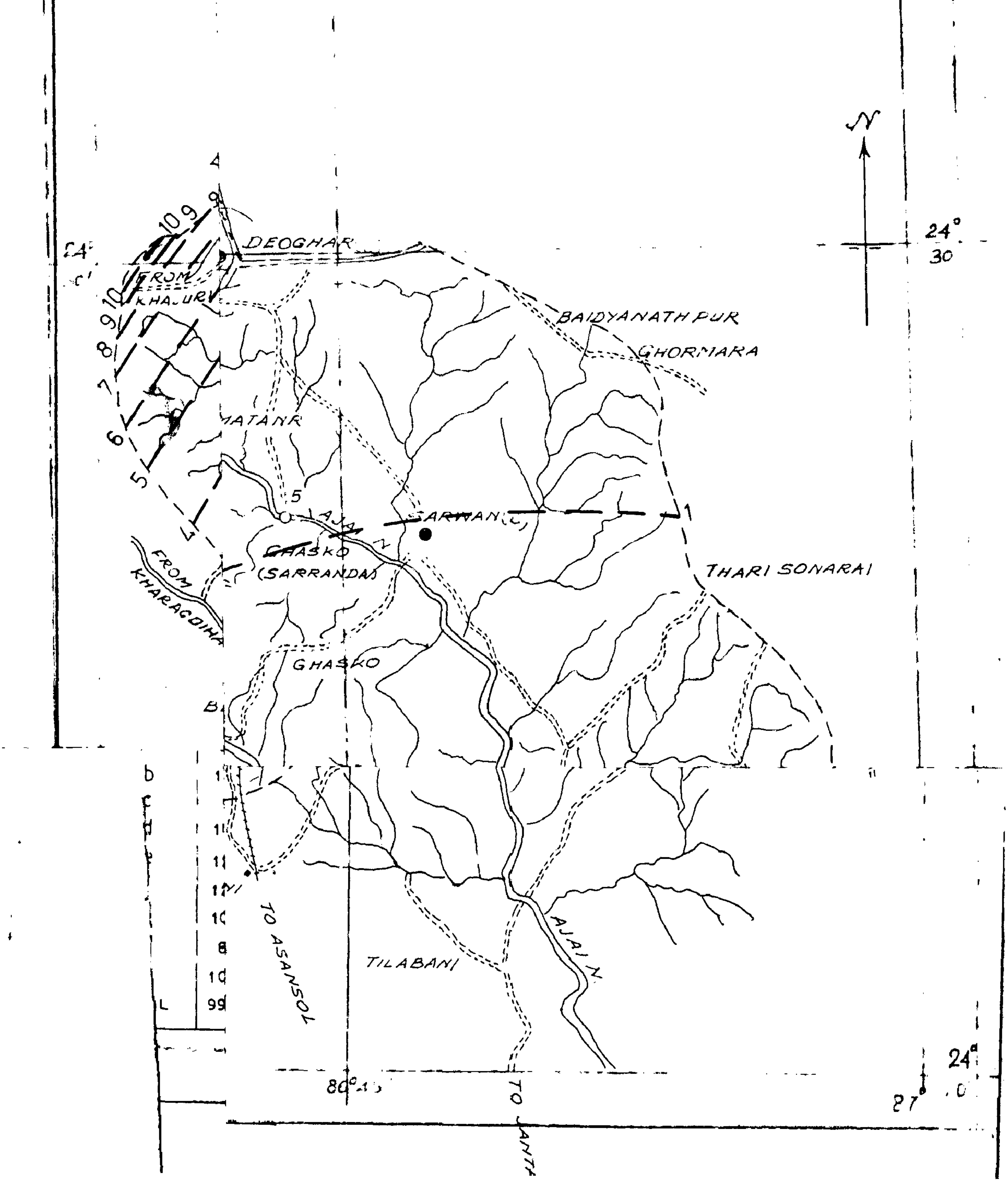
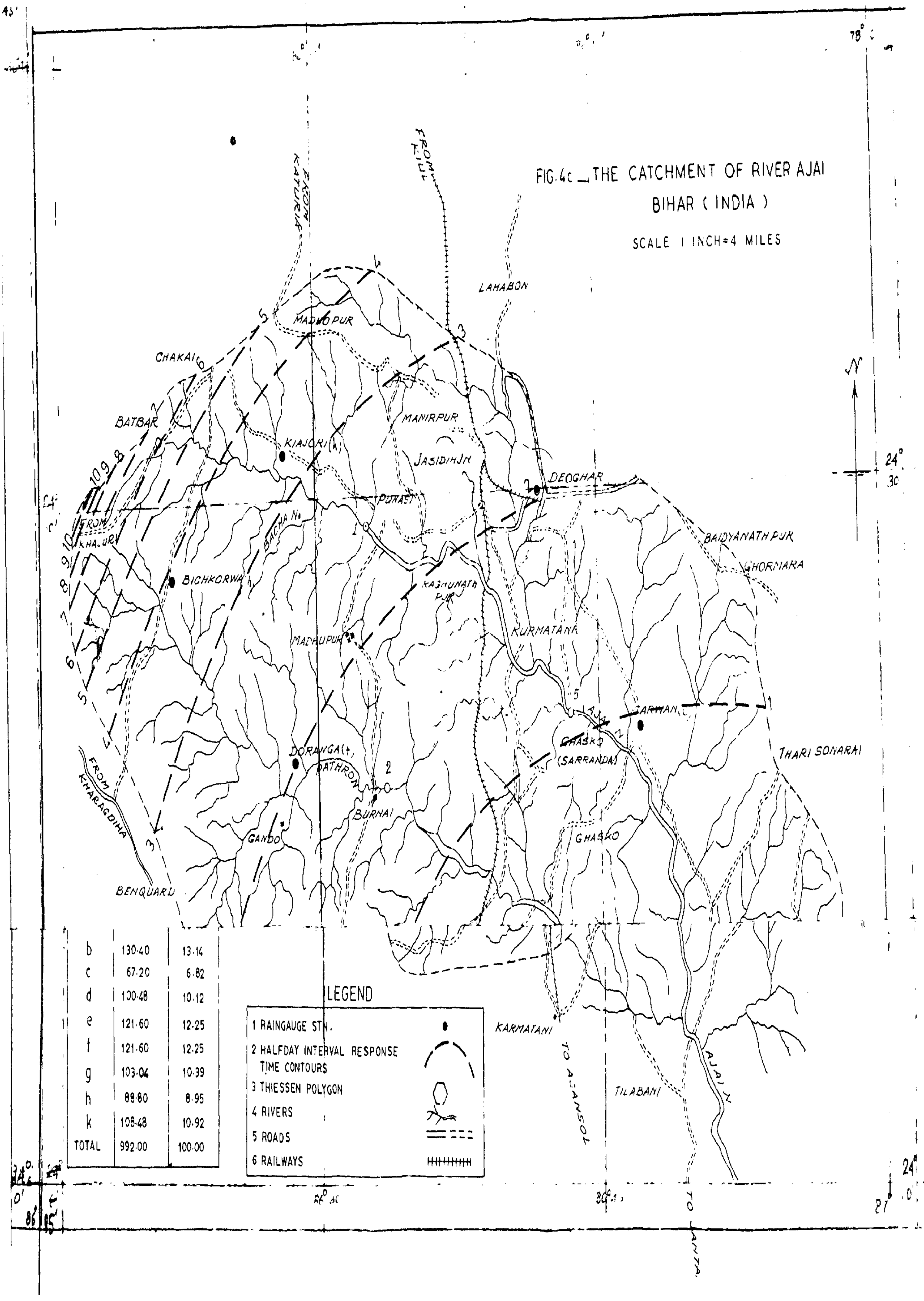


FIG. 4c _ THE CATCHMENT OF RIVER AJAI
BIHAR (INDIA)
SCALE 1 INCH=4 MILES



b	130·40	13·14
c	67·20	6·82
d	100·48	10·12
e	121·60	12·25
f	121·60	12·25
g	103·04	10·39
h	88·80	8·95
k	108·48	10·92
TOTAL	992·00	100·00

AJAY BASIN (1971 FLOOD)

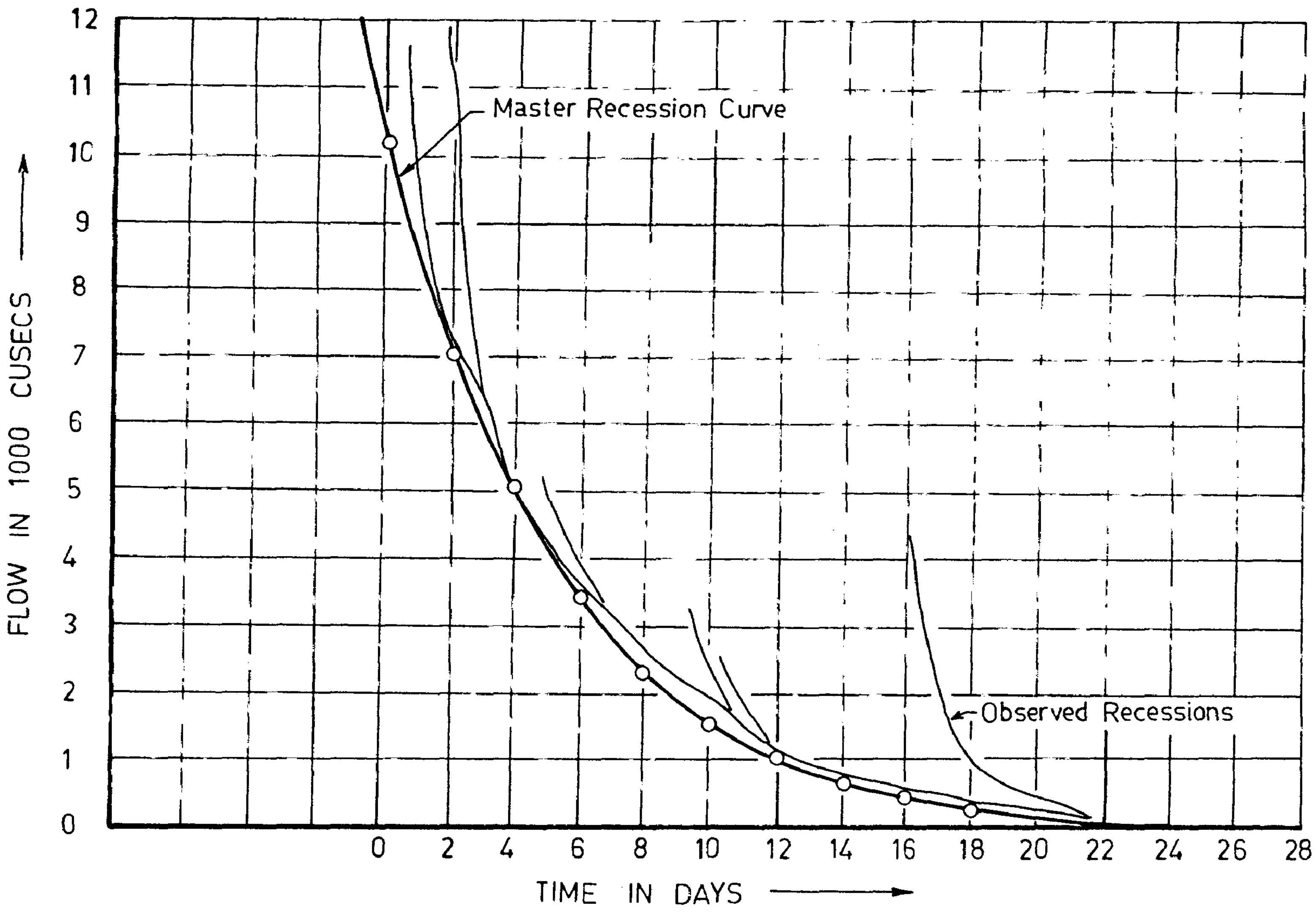


FIG. 4d — MASTER RECESSION CURVE

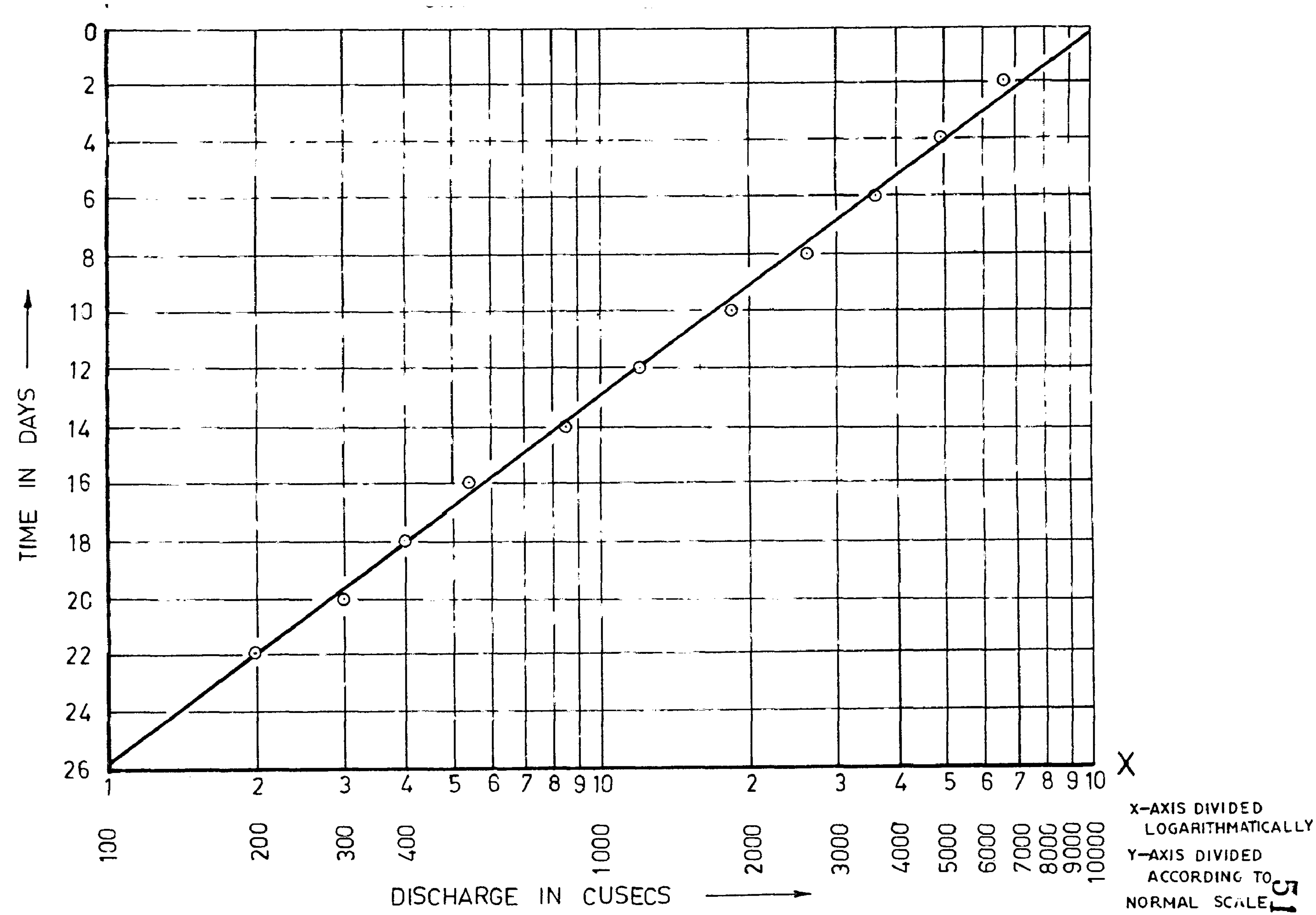


FIG. 4e— GROUND WATER RECESSION CURVE (1971 FLOODS)

The master recession curved was used to locating tho point of recession on the falling limb of the absorved hydrographs resulted from the storms listed above. The point of recession and the point of rise in the risings limb were joined by straight line for the purposes of separation of base flow. After separating base flow depth of direct runoff volume ovor the catchment was calculated. Substracting this value from the gross weighted average depth of rainfall over the catchmont the abstractions were estimated. Projecting the direct runoff depth over the gross weighted rainfall assuming constant rate of abstractions the value of ϕ-index was calculated.

Then weighted average rainfall on individual response sub-areas was calculated by the same Thiessen method. Substracting the value of ϕ-index from these values, rainfall excess over each response sub-area was calculated separately.

These rainfall excesses were averaged over the whole basin and compared with the direct runoff depth computed earlier. If these differed ϕ-index value was adjusted so that the rainfall excesses over the individual sub-basins when averaged over the whole catchment became equal to the direct runoff depth.

These rainfall excesses were taken as the input to the model. The values of the rainfall excess calculated in accordance with the above procedure are given from Table-5 to 9.

TABLE NO.3

AREAS OF INDIVIDUAL RESPONSE SUB-AREA ACCORDING TO THIESSEN POLYGON

(Time Interval 1/2 Day)

Response Sub-Area No.1			Response Sub-Area No.2			Response Sub-Area No.		
Poly-gon No.	Area Sq. Miles	Weight-age %	Poly-gon No.	Area Sq. Miles	Weight-age %	Poly-gon No.	Area Sq. Miles	Weight-age %
a	150.40	38.42	b	7.32	2.50	c	16.29	9.90
b	123.03	31.42	d	38.28	13.10	f	50.57	30.80
d	62.20	15.90	o	65.78	22.50	g	42.19	25.70
e	55.82	14.26	c	50.91	17.40	h	10.00	6.09
			f	71.03	24.30	k	45.23	27.51
			g	59.05	20.20			
Total	391.60	100.00		292.37	100.00		164.28	100.00

Response Sub-Area No.4			Response Sub-Area No.5			Response Sub-Area No.		
Poly-gon No.	Area Sq. Miles	Weight-age %	Poly-gon No.	Area Sq. Miles	Weight-age %	Poly-gon No.	Area Sq. Miles	Weight-age %
g	1.40	1.71	h	17.36	53.85	h	3.88	22.65
h	36.40	44.50	k	14.89	46.15	k	13.24	77.35
k	44.08	53.79						
Total	81.88	100.00		32.25	100.00		17.12	100.00

Response Sub-Area No.7			Response Sub-Area No.8			Response Sub-Area No.		
Poly-gon No.	Area Sq. Miles	Weight-age %	Poly-gon No.	Area Sq. Miles	Weight-age %	Poly-gon No.	Area Sq.Miles	Weight-age %
h	7.60	100.00%	h	3.15	100%	h	1.30	100.00%
Total	7.60	100		3.15	100		1.30	100

Response Sub-Area No.10		
h	0.55	100%
Total	0.55	100

TABLE NO.4

AREA OF INDIVIDUAL RESPONSE SUB-AREA

ACCORDING TO THIESSEN POLYGON

(Time Interval One day)

Response Sub-Area 1			Response Sub-Area 2			Response Sub-Area No.3		
Stn.	Area sq.miles	%	Stn.	Area sq.miles	%	Stn.	Area sq.miles	%
a	150.40	22.0	e	16.29	6.61	h	21.24	43.00
b	130.40	19.10	f	50.57	20.55	k	28.13	57.00
c	50.91	7.45	g	43.59	17.70			
d	100.48	14.70	h	46.40	18.81			
e	121.50	17.70	k	39.31	36.33			
f	71.03	10.40						
g	59.06	8.65						
Total	683.87	100.00		246.16	100.00		49.37	100.00

Response Subarea No.4			Response Sub-area No.5		
Stn.	Area sq.miles	%	Stn.	Area sq.miles	%
h	10.75	100.00	h	1.85	10.00
Total	10.75				

TABLE-5

STORM FROM 15 OCT. TO 17 OCT.1971

(All rainfall excess figures are in inches)

Response sub-area No.	Rainfall excess according to date		
	15.10.71	16.10.71	17.10.71
1	-	1.1875	-
2	0.3002	0.9686	-
3	0.5153	0.7763	-
4	0.1488	0.4435	-
5	0.0388	0.5983	-
6	0.3163	0.0358	
7-10	-	1.4283	-

TABLE-6

STORM OF 3rd JUNE TO 7TH JUNE 1971

(All rainfall excess figures are in inches)

Response Sub-area No.	Rainfall excess according to date				
	3.6.71	4.6.71	5.6.71	6.6.71	7.6.71
1	-	1.9007	0.2457	0.4925	0.0340
2	0.1996	1.4803	0.2707	0.8406	0.0557
3	1.1635	1.0833	0.1901	1.1738	0.2631
4	1.0266	0.2075	-	1.9139	0.3170
5	0.8179	0.2341	-	1.9829	0.2578
6	1.5929	0.0321	-	1.7504	0.4565
7-10	-	0.5349	-	2.3749	-

TABLE-7

STORM OF 9TH TO 13TH JULY 1971

(All rainfall excess figures in inches)

Response sub-area No.	Rainfall excess according to date 9.7.71	10.7.71	11.7.71	12.7.71	13.7.71
1	0.1609	0.3854	0.0677	-	0.0232
2	0.1834	-	1.6565	0.2470	0.6883
3	-	-	1.8904	0.5994	0.9990
4	-	-	0.9348	1.2525	0.4051
5	-	-	0.7409	1.2589	0.3599
6	-	-	1.5059	1.3174	0.3659
7-10	-	-	-	1.1709	0.3509

TABLE-8

STORM OF 28TH SEPT. TO 2 OCT!1971

(All rainfall excess figures in inches)

Response sub-area No.	Rainfall excess according to date 28.9.71	29.9.71	30.9.71	1.10.71	2.10.71
1	0.0664	0.4367	2.4593	0.5630	0.0517
2	0.0344	0.3198	1.5003	0.9191	-
3	-	0.3394	0.9940	0.7881	0.0727
4	-	0.0407	0.7060	0.1923	0.1413
5	-	0.0045	0.7643	0.1015	0.0878
6	-	0.1480	0.5080	0.3100	0.2870
7-10	-	-	1.1420	-	-

TABLE-9

STORM OF 11TH to 13TH OCT.1973

(All rainfall figures in inches)

Response sub-area No.	Rainfall excess according to date 11.10.73	12.10.73	13.10.73
1	0.4896	2.1744	-
2	0.5334	2.2913	0.5117
3	0.5640	1.8719	0.6088
4	0.9721	1.4392	-
5	1.1056	1.6696	-
6	0.2696	1.2506	-
7-10	2.1181	2.3881	

Detailed calculation of the rainfall excesses are appended vide Appendix-5.

4.5. RUNOFF COMPUTATIONS

In the previous section input rainfall excess have been calculated. The model parameters response sub-area and the travel coefficient have already been established in section 4.2.

The runoff distribution coefficients were calculated vide appendix-4. Using the rainfall excess given from table No.5 to table No.9 as input function, runoff computation were carried out as per scheme of table no.1. The table of computations are given in appendix-6.

4.6. COMPARISON OF OBSERVED AND COMPUTED RUNOFF HYDROGRAPHS

The runoff hydrograph of the five storms as observed were plotted at suitable scale on normal graph paper. On the same graph paper the computed runoff vide appendix-5 were also plotted. These are given from figure (4.f) to (4.j).

It may be seen that observed and computed hydrographs of the five storms as presented through fig.(4.f) to fig.(4.j) match reasonably well. In almost all the computed hydrographs the runoff at the beginning of the storm is more and at the end of the storm it is less than the observed one. This may be attributed to the assumption of constant value of ϕ-index which remains more in the beginning and goes on reducing as the storm duration advances.

AJAY BASIN AT SIKATIA (BIHAR)
C. A. 992 SQ MILES

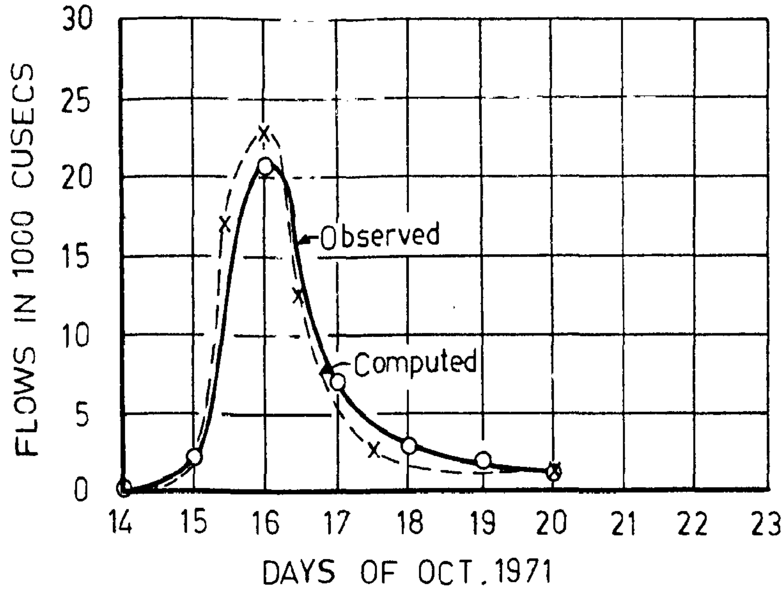


FIG.4f_RUNOFF HYDROGRAPH OF STORM DATED 13-17 OCT. 71

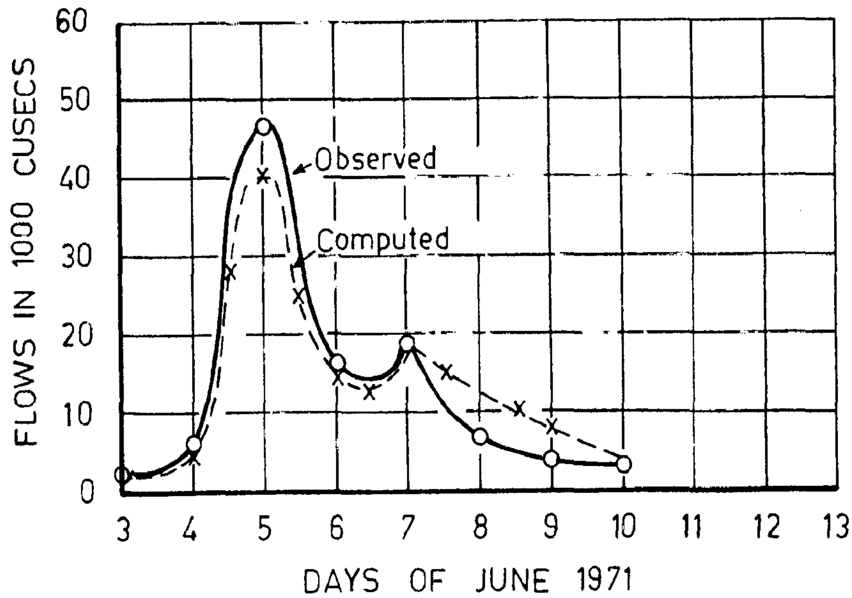


FIG.4g_RUNOFF HYDROGRAPH OF STORM DATED 3-7 JUNE 71

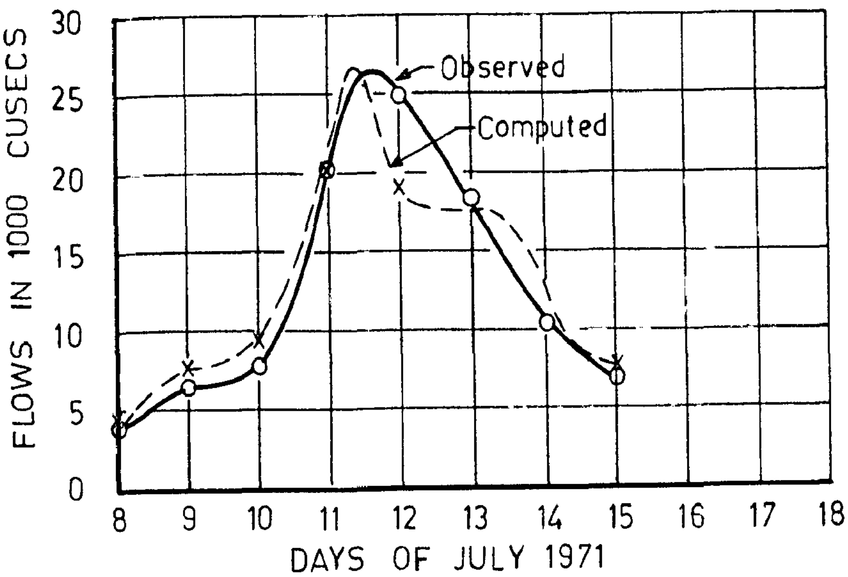


FIG. 4h_RUNOFF HYDROGRAPH OF STORM DATED 9-13 JULY 71

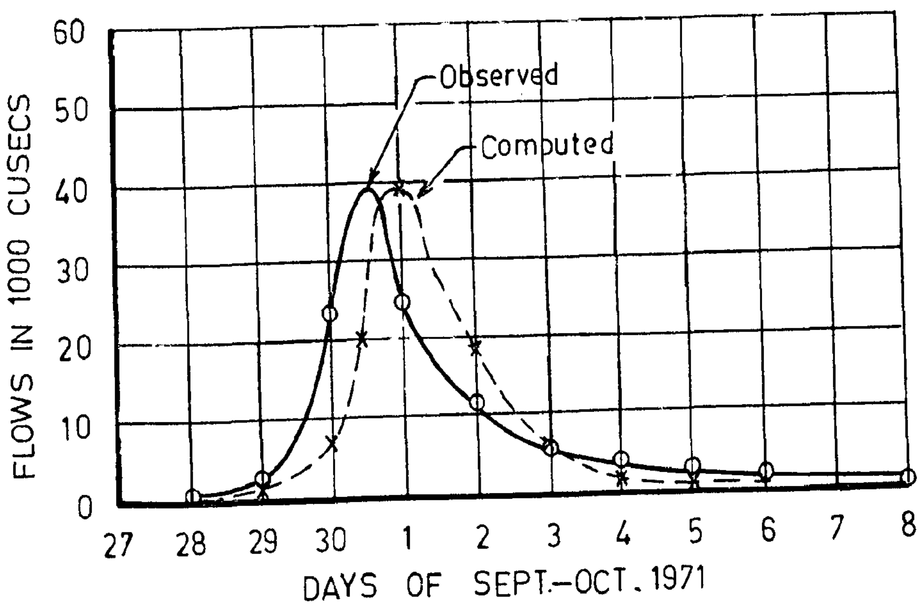


FIG. 4i_RUNOFF HYDROGRAPH OF STORM DATED 28 SEPT – 2 OCT 1971

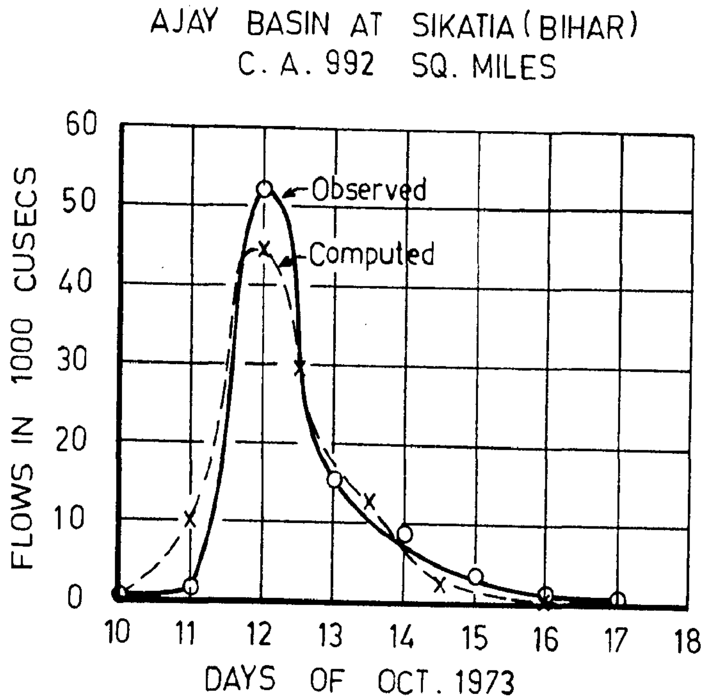


FIG. 4j_RUNOFF HYDROGRAPH OF STORM DATED 11-18 OCT.1973

CHAPTER-5

CONCLUSIONS

5.0 SUMMARY OF CONCLUSIONS

(1) In this study a simple conceptual model has been developed which is capable of taking into account the spatial distribution of rainfall excess to some extent.

(2) To represent the catchment action in terms of the two parameters viz. ' the response sub-area' and 'travel coefficient', a procedure has been developed utilising one of the observed events on the catchment.

(3) The proposed response model has been applied on a natural catchment viz. Ajay river catchment located in Chhotanagpur platau area of Bihar (C.A.992 sq. miles). The computed hydrographs are in good agreement with the observed hydrographs.

(4) As the response sub-areas develop their separate hydrographs at the outlet, the effects due to any change in land use which may affect the runoff from the particular subarea can be identified.

5.1. PROPOSED MODEL VERSUS EXISTING MODELS

In the existing conceptual models the transformation of rainfall excess in to the runoff is explained through conceptual identities, like, linear channel, linear and non-linear reservoir and time-area concentration diagrams etc. In the present model non of these identities have been used. In the proposed model the basic runoff mechanics has been explained considering the contributions of the response sub-areas to be binomially distributed in time (Section 3.3)

A new concept of response sub-area has been introduced which are different from the inter-isochronal areas. The isochronal areas are supposed to contribute to the outlet between the time limit indicated by upper and lower time units of the adjacent isochrones whereas, the response subareas are assumed to start contributing to the outlet from the time indicated by the lower bound value of the response contours and continues as per binomial distribution suggested in the runoff scheme for infinite time intervals.

The model though uses basic runoff mechanics from first principle, it is a practical tool. Computations can be computerised but at the same time these may be carried out with the help of ordinary computation machines.

The proposed model is basically a two parameter analytical model and the parameters can be evaluated easily from a recorded event on the catchment.

Since the catchment area is divided into response sub-areas it is possible to account for spatial distribution of rainfall excess.

5.2. SCOPE OF FURTHER INVESTIGATION

The proposed model has been tested for a basin of very high drainage density of area around 1000 sq.miles. The model may be applied to larger basins provided sufficiently uniform rainfall is recorded at least once to establish the parameters.

The assumption that no control section exists in the catchment, puts a serious limitation on the applicability of the model to multi river basins. Some methodology can be investigated to combine the individual hydrographs computed for individual rivers to predict the resultant hydrograph at the outlet.

Runoff distribution coefficients are calculated by the formula proposed in section (3.3) which is based on binomial law of distribution.

Other skewed distribution viz. Poissons distribution, Chi-square distribution, F-distribution etc. with suitable approximation may also be worth trying to compare the basic responses and so the sub-area responses.

APPENDICES

APPENDIX-1

GLOSSARY OF SYMBOLS

A, B, D	Area of response watersheds of the catchment
C_τ	Travel coefficient
τ	Response sub-area number
I_j	Input rainfall excess
$C_{\tau,t}$	Runoff distribution coefficient
t	current time
n	No. of linear reservoir in Nash model for IUH
K	Storage constant for linear reservoir
Q(t)	Computed runoff discharges at time t
S	Storage function
α	Constant for approximating flow contours
T	Total translation time
IUH	Instantaneous unit hydrograph
WARF	Weighted average rainfall
EWARF	Effective weighted average rainfall
UH	Unit Hydrograph
u(o,t), U(t)	Ordinate of IUH at time t.
$u(\Delta\tau, t)$	Ordinate of UH of duration $\Delta\tau$ at the instant t.
DRO	Direct Runoff

O.R.O.	Observed runoff
B.F.	Base flow
R.F.	Rainfall
R.R.F.	Recorded rainfall
M_{ERH1}	First moment of the effective rainfall hyetograph about origin
M_{DRH1}	First moment of the direct runoff hydrograph about the origin
R.O.	Runoff generated over the response sub-area
A_c	Area of the response sub-area in acres.

APPENDIX-2

RAINFALL RUNOFF DATA OF AJAY CATCHMENT AT SIKATIA

TABLE-10

Date	Recorded rainfall according to stations in inches									Recorded
	Sarath	Dhobna	Maheshmunda	Maheshpur	Sarwan	Daranga	Deoghar	Bijkhorwa	Krajori	Average daily discharge cu
	a	b	c	d	e	f	g	h	k	
1	2	3	4	5	6	7	8	9	10	11
June 1, 1971	0.16	-	-	-	-	-	-	-	-	2040
2	-	0.21	0.60	-	-	0.55	0.39	-	-	2256
3	-	-	1.11	1.45	-	0.30	0.04	-	2.40	2602
4	2.35	2.50	1.20	1.46	1.71	1.95	2.10	0.80	0.15	5359
5	0.36	0.67	0.93	0.91	0.12	0.50	0.45	-	0.34	46900
6	0.79	0.50	0.91	0.61	1.40	1.30	1.11	2.64	1.83	16223
7	0.21	0.45	0.12	0.34	0.16	0.45	0.49	0.25	0.36	18884
8	0.09	0.39	0.13	-	-	0.30	0.61	0.17	0.40	6920
9	0.62	0.01	0.11	-	0.13	0.05	0.06	0.14	0.22	3928
10	-	-	0.45	-	-	-	0.04	-	0.13	3155
11	0.64	-	0.05	-	-	-	-	-	-	2800
12	-	-	-	-	-	-	-	-	-	2288

Contd.

Table 10 contd.

1	2	3	4	5	6	7	8	9	10	11
July 8, 1971	-	-	-	0.13	-	-	-	-	-	4049
9	-	1.75	2.69	-	-	0.25	-	-	0.15	6513
10	0.32	1.46	-	1.26	-	-	0.12	-	-	5350
11	0.43	0.56	6.75	0.53	0.22	2.80	0.29	-	2.45	20308
12	0.29	0.37	2.45	0.51	-	0.30	0.30	1.56	1.75	24758
13	0.65	0.15	0.63	0.50	0.25	0.76	3.26	0.74	0.76	17233
14	0.22	0.42	0.11	0.67	0.51	0.45	1.30	0.20	0.18	10412
15	-	2.01	-	2.86	0.37	0.45	-	-	-	6995
Sept.27, 1971	-	-	-	-	0.45	-	-	-	-	1271
28	-	0.20	-	1.33	-	0.26	-	-	-	598
29	0.38	0.95	0.08	1.26	-	1.28	0.07	[illegible]	0.46	2054
30	3.94	1.75	1.07	1.33	2.76	1.45	1.64	1.35	0.53	23276
1	-	1.75	1.00	-	1.55	0.37	2.33	-	0.67	24531
2	0.49	0.25	0.33	-	0.16	0.13	0.28	-	0.64	11540
3	0.12	0.35	0.42	-	-	0.26	0.21	-	0.19	5678
4	-	0.07	0.24	-	-	-	0.06	-	0.32	3835
5	-	-	0.08	-	0.20	-	0.16	0.60	-	2770

Contd.

Table 10 Contd.

1	2	3	4	5	6	7	8	9	10	11
6	-	-	-	-	-	-	-	-	-	1862
7	-	-	-	-	-	-	-	-	-	1232
8	0.26	-	-	-	-	-	-	-	0.33	960
9	0.61	-	0.28	-	-	-	-	-	-	244
10	-	-	-	-	-	-	-	-	-	1665
11	-	-	-	-	-	-	-	-	-	903
Oct.13, 1971	0.46	-	-	-	-	-	-	-	-	386
14	-	-	-	-	-	-	-	-	-	360
15	-	-	-	1.27	-	0.06	2.43	-	0.89	1968
16	2.42	1.35	1.71	-	1.44	2.13	0.83	1.80	-	20920
17	-	0.05	-	-	-	-	-	0.05	-	6925
18	-	-	-	-	-	-	-	-	-	2723
19	-	-	-	-	-	-	-	-	-	1917
20	-	-	-	-	-	-	-	-	-	1130
21	-	-	-	-	-	-	-	-	-	952
22	0.05	-	-	-	-	-	-	-	-	773
23	0.28	0.30	-	-	-	-	-	-	-	736

Contd.

Table 10 contd.

1	2	3	4	5	6	7	8	9	10	11
Oct.10 1973	-	-	-	-	-	-	-	-	-	909
11	0.80	1.00	0.13	-	0.98	2.22	-	2.39	-	1020
12	2.90	2.95	5.66	0.20	2.62	3.54	0.12	2.66	1.09	52250
13	0.05	0.50	0.33	-	0.21	-	3.30	-	-	14700
14	0.20	0.10	-	-	-	-	0.20	0.47	-	8648
15	-	-	0.15	0.01	-	-	0.07	-	-	3800
16	-	-	-	-	-	-	-	-	-	1194
17	-	-	-	-	-	-	-	-	-	547
18	-	-	-	-	-	-	-	-	-	265
19	-	-	-	-	-	-	-	-	-	180
20	-	-	-	-	-	-	-	-	-	130
21	-	-	-	-	-	-	-	-	-	100

APPENDIX-3

DERIVATION OF I.U.H. AND 12 HOUR U.H.FOR AJAY RIVER CATCHMENT AT SIKATIA

For derivation of I.U.H. Nash model was used. The model parameters n and k were evaluated by method of moments as explained in section 2.1. through equation numbers (2.C) to(2.F).

A short duration storm which occurred on the catchment more or less uniform in space was selected such a storm was of 15-16 Oct. 1971.

After separating base flow from the runoff hydrograph and computing the weighted effective rainfall by Thiessen method. Following data were available for deriving an I.U.H.

Time interval in days	ERH in cusecs	DRH in cusecs
0		0
1	4755	1480
2	25978	20307
3		6180
4		1850
5		916
6		0

By using Nash's method of moment values of n and k in the equation (2.C) have been evaluated below (Refer fig.5).

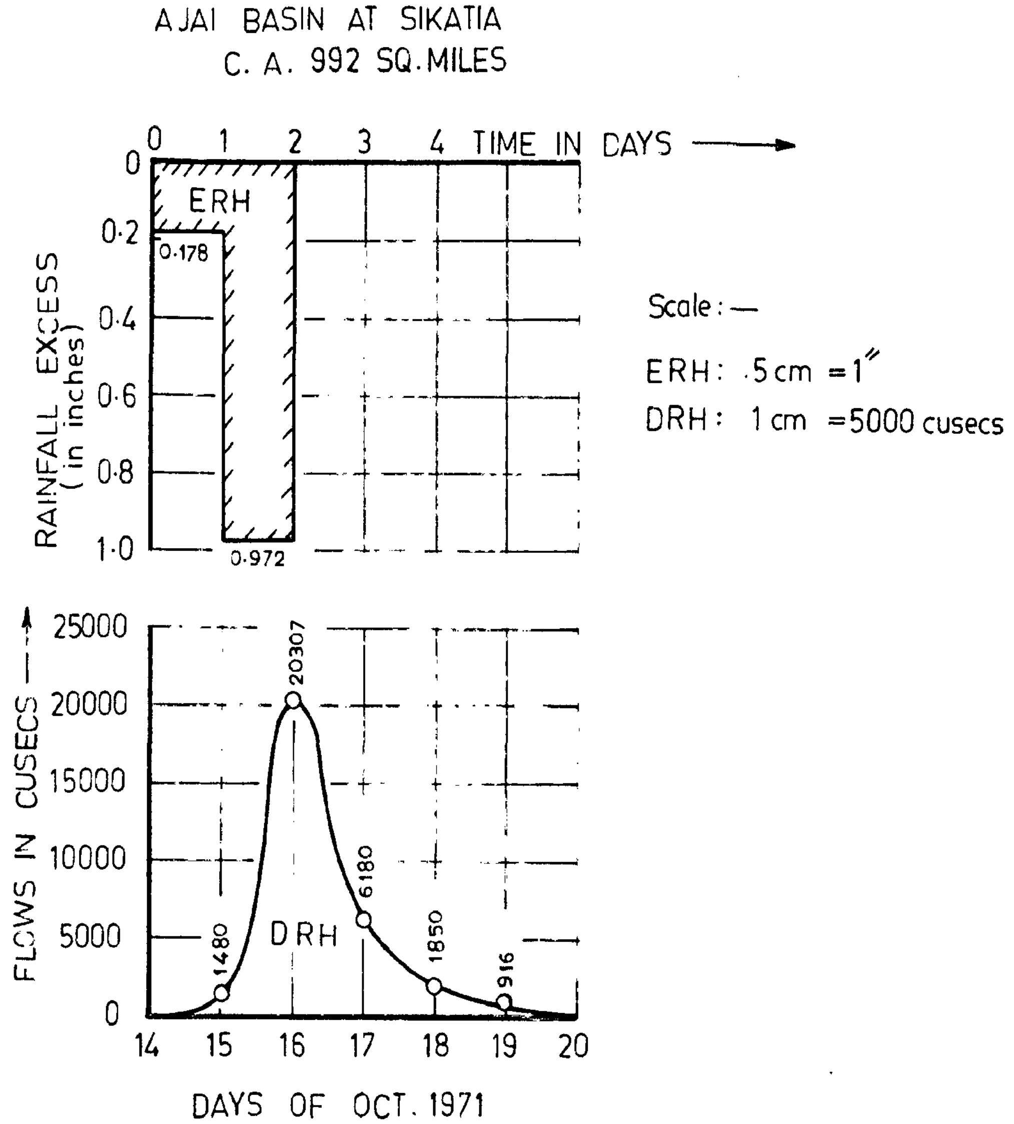


FIG. 5 — ERH AND DRH OF STORM DATED 15–16 OCT. 1971

$$M_{ERH2} = \frac{4755x1x0.5 + 25978 \text{ x } 1 \text{ x } 1.5}{(4755 + 25978)}$$

$= 1.3453$ days

$$M_{ERH2} = \frac{2378 \text{ x}0.5 + 38967 \text{ x } 1.5}{30733}$$

$= 1.9406 \text{ days}^2$

$$M_{DRH1} = \frac{1480 \text{ x } 1 + 20307 \text{ x } 2 + 1680x3+1850x4+916x5}{1480+ 20307+6180+1850+916}$$

$= 2.3627$ days

$$M_{DRH2} = \frac{1480x\ 1+ 40614 \text{ x } 2+ 18540 \text{ x}3+ 7400x4x4580x3}{30733}$$

$= \quad 6.2092 \text{ day}^2$

In Nash's model values of n and k are given by

$$M_{DRH1} - M_{ERH1} = nk \qquad (I)$$

and,

$$M_{DRH2}-M_{ERH2} = n(n+1)K^2+2nk \quad M_{ERH1} \qquad (II)$$

From (I) we have

$2.3627 - 1.3453 = nk$

or $nk = 1.0154$

$$\text{or } k = \frac{1.0154}{n} \qquad (III)$$

and from (II) we have

$$6.2092-1.9406 = n(1-n)\ \frac{(1.0154)^2}{n^2}$$

$$+ 2x\ 1.0154 \times 1.3453$$

$$\text{or } 4.2686 = \frac{1+n}{n}\ (1.03144) + 2.7320$$

$$\text{or } 1.5366\ n = 1.03144 + 1.03144n$$

$$\text{or } 0.50516\ n = 1.03144$$

$$\text{or} \quad n = \frac{1.03144}{0.50516} = 2.0418$$

Say 2

$$\therefore \quad k = \frac{1.0154}{n} = \frac{1.0154}{2}$$

$$= 0.5077 \text{ days}$$

Putting the values of n and k obtained above in Nash model for IUH. We get,

$$U9t) = \frac{1}{k\sqrt{n}}\ \left(\frac{t}{k}\right)^{n-1}\ e^{-t/k}$$

$$= \frac{1}{0.5077 \times \sqrt{2}}\ \left(\frac{t}{0.5077}\right)^{2-1}\ e^{-t/0.5077}$$

$$= \frac{1}{0.5077 \times 0.5077}\ t.e^{-1.97t}$$

$$= 3.88 \times t \times e^{-1.97\ t}$$

IUH has been calculated on the next page in the Table No.11.

Volume of 1 inch runoff over the catchment uniformly supplied in 24 hours.

$$= \frac{992 \times 5280 \times 5280}{12 \times 24 \times 3600} \text{ Cusecs}$$

$$= \quad 26{,}674 \text{ cusecs}$$

TABLE-11

Days t	1.97 t = α	$e^{-α}$	IUH=U(t) = $3.88xtxe^{-α}$	IUH in terms of discharges 26674 x $U_{(t)}$
0	0	0	0	0
0.25	0.4925	0.61110	0.59200	15,800
0.50	0.9850	0.37345	0.72400	19,300
0.75	1.4775	0.22821	0.66450	17,720
1.00	1.9700	0.13946	0.54100	14,410
1.25	2.4625	0.08523	0.41300	11,020
1.50	2.9550	0.05208	0.30350	8,090
1.75	3.4475	0.03185	0.21650	5,775
2.00	3.9400	0.01945	0.15080	4,025
2.25	4.4325	0.011724	0.10230	2,735
2.50	4.9250	0.006108	0.05928	1.581
2.75	5.4175	0.004400	0.04695	1,252
3.00	5.9100	0.002714	0.03155	842
3.25	6.4025	0.001672	0.02110	562
3.50	6.8950	0.001025	0.01392	371
3.75	7.3875	0.000625	0.00909	243
4.00	7.8800	0.000375	0.00582	155
4.25	8.3725	0.000228	0.00376	100
4.50	8.8650	0.000140	0.002445	65
4.75	9.3575	0.000084	0.001548	41
5.00	9.8500	0.0000525	0.001018	27
5.25	10.3425	$4.5x10^{-5}x.795$	0.000729	19
5.50	10.8350	$4.5x10^{-5}x.434$	0.000509	14
5.75	11.3275	$4.5x10^{-5}x.265$	0.0000266	7
6.00	11.8200	$4.5x10^{-5}x.162$	0.0001698	5
			Total	1,04,260

In theoretical sense the discharges from the catchment will continue upto infinite time. But for practical purposes looking to the IUH ordinates we may curtail it at 5th day and take the ordinate at 5.5th day as zero. How to make the volume of the IUH equal to unity we may adjust the ordinates before 5th day. This work is accomplished in table-12.

TABLE-12

Time days	U(t)	Adjusted U(t)	IUH discharges Q(t)	Adjusted IUH discharges Q(t)
1	2	3	4	5
0	0	0	0	
0.25	0.592000	0.606410	15,800	161.59
0.50	0.724000	0.741630	19,300	19795
0.75	0.664500	0.68068	17,720	18137
1.00	0.541000	0.554190	14410	14777
1.25	0.413000	0.423070	11020	11300
1.50	0.303500	0.310880	8090	8305
1.75	0.216500	0.221770	5775	5906
2.00	0.150800	0.154470	4025	4124
2.25	0.102300	0.104888	2735	2795
2.50	0.059280	0.060728	1581	1620
2.75	0.046950	0.048070	1252	1283
3.00	0.031550	0.032423	8421	866
3.25	0.021100	0.021600	562	576
3.50	0.013920	0.014253	371	381

Contd.

Table 12 Contd.

1	2	3	4	5
3.75	0.009090	0.009296	243	246
4.00	0.005820	0.005958	155	159
4.25	0.003760	0.003843	100	102
4.50	0.002445	0.002495	65	66
4.75	0.001548	0.001588	41	42
5.00	0.001018	0.001038	27	28
5.25	0.000729	0.000729	19	19
5.50	0.000509	0	14	0
	Total =	4000000		

Response sub-area

As explained in section (3.4) the next step is to select the time interval to be taken as unit and divide the catchment into several sub-areas according to their response time.

The IUH peak reaches at 1/2 day i.e. 12 hours so the time interval should be less than or equal to 1/2 day. The areas proportional to the IUH ordinate at 1/2 day 1 day 1.5 day 2 day etc. have been calculated in Table-13.

TABLE-13

Time days	IUH ordinate cusecs	Percentage of total	Area in sq.miles	Area in acres	Sub-area nos.
0.0	0	0			
0.5	19795	39.460	391.50	250560	1
1.0	14777	29.472	292.37	187117	2
1.5	8305	16.571	164.28	105139	3
2.0	4124	8.250	81.88	52403	4
2.5	1620	3.250	32.25	20640	5
3.0	866	1.727	17.12	10957	6
3.5	381	0.765	7.60	4864	7
4.0	159	0.318	3.15	2016	8
4.5	66	0.131	1.30	832	9
5.0	28	0.056	0.55	352	10
5.5	0	0			
Total	50121	100.00	992.00		

TABLE-14

1/2 DAY UH FROM IUH

Time days	IUH cusecs	12 Hrs. or 1/2 day UH cusecs
0.25	16159	8080
0.50	19795	9898
0.75	18137	17148
1.00	14777	17286
1.25	11300	14718
1.50	8305	11541
1.75	5906	8603
2.00	4124	6230
2.25	2795	4350
2.75	1283	2039
3.00	866	1243
3.25	576	930
3.50	381	624
3.75	246	411
4.00	159	270
4.25	102	174
4.50	66	112
4.75	42	72
5.00	28	47
5.25	19	30
5.50	0	14
5.75	0	9
6.00	0	0

IUH and 1/2 day UH computed above are plotted as Fig.(4.a) and (4.b) respectively.

Time base of the IUH is 5.5 day. Therefore the time of concentration for the basin = 5.5 days.

APPENDIX - 4

EVALUATION OF TRAVEL COEFFICIENT AND RUNOFF DISTRIBUTION COEFFICIENT

The value of travel coefficient C_τ was calculated by trial and error. First trial value was taken as

$$C_\tau = \frac{\text{Peak of UH of } \Delta\tau \text{ duration}}{\text{Peak of IUH}}$$

Where $\Delta\tau$ is the time to peak of the IUH.

Progressively C_τ value was changed to improve the output response computed on the basis of trial C_τ value. In the present case $C_\tau = 0.97$ gave a better match and so it was adopted. The values of runoff distribution coefficient C_τ t are given in Table 15.

AVEL COEFFICIENTS C_τ = 0.97

Respo sub-a No. τ	to time interval t						
		1	12	13	14	15	16
1							
2							
3							
4							
5							
6	001						
7	018	0.0001					
8	254	0.0025	0.0002				
9	052	0.0308	0.0034	0.0003			
10	374	0.2212	0.0365	0.0033	0.0004	0.0001	
11		0.7152	0.2360	0.0425	0.0055	0.0006	0.0001

APPENDIX - 5

IDENTIFICATION OF RAINFALL EXCESS FOR FIVE STORMS IN AJAY BASIN AT SIKATIA (BIHAR)

CALCULATION FOR W.A.R.F. OVER CATCHMENT - STORM DATE 15-17 OCT. 1971

Raingauge Stations	Weightage %	15.10.71		16.10.71		17.10.71	
		R.R.F. inch	WARF inch	R.R.F. inch	WARF inch	R.R.F. inch	WARF inch
a	15.16	-	-	2.42	0.3665	-	-
b	13.14	-	-	1.35	0.1773	0.05	0.0066
c	6.82	-	-	1.71	0.1107	-	-
d	10.12	1.27	0.1285	-	-	-	-
e	12.25	-	-	1.44	0.1764	-	-
f	12.25	0.06	0.0735	2.13	0.2610	-	-
g	10.39	2.43	0.2520	0.83	0.0861	-	-
h	8.95	-	-	1.80	0.1611	0.05	0.0045
k	10.92	0.89	0.0972	-	-	-	-
Total			0.5512		1.3451		0.0111

TABLE - 15

VALUES OF RUNOFF DISTRIBUTION COEFFICIENTS FOR DIFFERENT RESPONSE SUB-AREAS FOR TRAVEL COEFFICIENTS $C_\tau = 0.97$

$$C_{\tau,t} = \frac{(t-1)!}{(\tau-1)!\,(t-\tau)!}\,(C_\tau)\,\tau\,(1-C_\tau)^{t-\tau},\ t \geq \tau$$

Response sub-area No. τ	Travel Coeff. C_τ	Values of runoff distribution coefficients $C_{\tau,t}$ according to time interval t															
		t=1	2	3	4	5	6	7	8	9	10	1	12	13	14	15	16
1	0.97	0.9700	0.0291	0.0009													
2	0.97	-	0.9409	0.0565	0.0025	0.0001											
3	0.97	-	-	0.9127	0.0321	0.0049	0.0003										
4	0.97	-	-	-	0.8851	0.1062	0.0080	0.0005									
5	0.97	-	-	-	-	0.8588	0.1288	0.0116	0.0008								
6	0.97	-	-	-	-	-	0.8329	0.1499	0.0157	0.0009	0.0001						
7	0.97	-	-	-	-	-	-	0.8080	0.1697	0.0204	0.0018	0.0001					
8	0.97	-	-	-	-	-	-	-	0.7835	0.1880	0.0254	0.0025	0.0002				
9	0.97	-	-	-	-	-	-	-	-	0.7600	0.2052	0.0308	0.0034	0.0003			
10	0.97	-	-	-	-	-	-	-	-	-	0.7374	0.2212	0.0365	0.0033	0.0004	0.0001	
11	0.97	-	-	-	-	-	-	-	-	-	-	0.7152	0.2360	0.0425	0.0055	0.0006	0.0001

CALCULATION OF OBSERVED R.O. DEPTH

Date	O.R.O. Cusecs	B.F. Cusecs	D.R.O. Cusecs
14.10.71	360	360	0
15.10.71	1968	488	1480
16.10.71	20923	616	20307
17.10.71	6925	745	6180
18.10.71	2723	873	1850
19.10.71	1917	1001	916
20.10.71	1130	1130	0
Total			30733

Computation for ϕ Index

$$\text{O.R.O. Depth} = \frac{30733 \times 24 \times 3600 \times 12}{992 \times 5280 \times 5280}$$

$$= \frac{30733}{26674}''$$

$$= 1.15 \text{ inches}$$

Gross W.A.R.F. = 0.5512 + 1.3451 + 0.0111 = 1.9074"

$\therefore$ Total loss (1.9074" - 1.15")

= 0.7574 inch

$$\therefore \text{Av. loss} = \frac{0.7574}{3}'' \text{/day}$$

= 0.37315"

Assuming constant value of ϕ - Index it works out to be 0.3732"/day.

W.A.R.F. EXCESS OVER THE BASIN

Date	WARFF
15.10.71	0.1780"
16.10.71	0.9720"
17.10.71	-
Total	1.15"

CALCULATION OF WARF ON INDIVIDUAL SUB BASINS - DATE-WISE(R.F.IN INCHES)

STORM OF 15-17 OCT. 1971

Sub-Basin No.	R.G. Stn.	Weigh-tage %	15.10.71		16.10.71		17.10.71	
			RRF	WARF	RRF	WARF	RRF	WARB
1	a	38.42	-	-	2.42	0.9300	-	-
	b	31.42	-	-	1.35	0.4240	0.05	0.0157
	d	15.90	1.27	0.2020	-	-	-	-
	e	14.26	-	-	1.44	0.2052	-	-
Total				0.2020		1.5592		0.0157
2	b	2.50	-		1.35	0.0338	0.05	0.0013
	d	13.10	1.27	0.1663	-	-	-	-
	e	22.50	-	-	1.44	0.3240	-	-
	c	17.40	-	-	1.71	0.2975	-	-
	f	24.30	0.06	0.0146	2.13	0.5175	-	-
	g	20.20	2.43	0.4910	0.83	0.1675	-	-
Total				0.6719		1.3403		0.0013
3	c	9.90	-	-	1.71	0.1693	-	-
	f	30.80	0.06	0.0185	2.13	0.6560	-	-
	g	25.70	2.43	0.6240	0.83	0.2130	-	-
	h	6.09	-	-	1.80	0.1097	0.05	0.0030
	k	27.51	0.89	0.2445	-	-	-	-
Total				0.8370		1.1480		0.0030

contd.../

1	2	3	4	5	6	7	8	9
4	g	1.71	2.43	0.0415	0.83	0.0142	-	-
	h	44.50	-	-	1.80	0.8010	0.05	0.0225
	k	53.79	0.89	0.4790	-	-	-	-
Total				0.5205		0.9700		0.0270
5	h	53.85	-	-	1.80	0.9700	0.05	0.0270
	k	46.15	0.89	0.4105	-	-	-	-
Total				0.4105		0.9700		0.0270
6	h	22.65	-	-	1.80	0.4075	0.05	0.0113
	k	77.35	0.89	0.6880	-	-	-	-
Total				0.6880		0.4075		0.0113
7-10	h	100.00	-	-	1.80	1.8000	0.05	0.0500
Total						1.8000		0.0500

Sub-Basin No.	Weigh-tage ø	W.A.R.F. inches 15/10	16/10	17/10	Net W.A.R.F. with ø = 0.3732 15	16	17	Net W.A.R.F. in for entire catchment 15	16	17
1	39.460	0.2020	1.5592	0.0167	-	1.1860	-	-	0.4680	-
2	29.472	0.6719	1.3403	0.0013	0.2987	0.9671	-	0.0880	0.2850	-
3	16.571	0.8870	1.1480	0.0030	0.5138	0.7748	-	0.0850	0.1282	-
4	8.250	0.5205	0.8152	0.0225	0.1473	0.4420	-	0.0122	0.0365	-
5	3.250	0.4105	0.9700	0.0270	0.0373	0.5968	-	0.0012	0.0194	-
6	1.727	0.6880	0.4075	0.0113	0.3148	0.0343	-	0.0054	0.0006	-
7-10	1.270	-	1.2000	0.0500	-	1.4268	-	-	0.0181	-
							Total	0.1918	0.9558	

Sub-Basin No.	Weigh-tage ø	Net W.A.R.F. with ø = 0.3717					
1	39.460	-	1.1875	-	-	0.4685	-
2	29.472	0.3002	0.9686	-	0.0885	0.2855	-
3	16.571	0.5153	0.7763	-	0.0854	0.1285	-
4	8.250	0.1488	0.4435	-	0.0123	0.0366	-
5	3.250	0.0388	0.5983	-	0.0013	0.0194	-
6	1.727	0.3163	0.0358	-	0.0055	0.0006	-
7-10	1.270	-	1.4283	-	-	0.0182	-
					0.1930	0.9573	
						0.1930	
					Total	1.1503 inch	

Total of Net WARF over the catchment = 0.1918

0.9558

Total = 1.1476

R.O. depth = 1.1500

$\therefore$ Deficit rain. = 0.0024

ϕ - Index must be reduced by an amount x to make up this deficit in net rainfall. For the value of x, we get

$$x\left[0.39460 + 0.01270 + 2(0.29472 + 0.16571 + 0.08250 + 0.03250 + 0.01727)\right] = 0.0024$$

$$\text{or} \quad x\left[0.40730 + 1.1854\right] = 0.0024$$

$$\text{or} \quad x = \frac{0.0024}{1.5927} = 0.001504 \quad \text{Say} \quad 0.0015''$$

$$\therefore \phi \text{ revised} = 0.3732 - 0.0015 = 0.3717''/\text{day}$$

AJAY BASIN AT SIKATIA (BIHAR)

CALCULATION FOR WARF OVER THE CATCHMENT STORM DATE 3 - 7 JUNE 1971 (All rainfall figures in inches)

Stn.	Weight %	3.6.71 R.R.F.	3.6.71 WARF	4.6.71 RRF	4.6.71 WARF	5.6.71 RRF	5.6.71 WARF	6.6.71 RRF	6.6.71 WARF	7.6.71 RRF	7.6.71 WARF
a	15.16	-	-	2.35	0.3560	0.36	0.0545	0.79	0.1197	0.21	0.0318
b	13.14	-	-	2.50	0.3285	0.67	0.0880	0.50	0.0657	0.45	0.0591
c	6.82	1.11	0.0750	1.20	0.0819	0.93	0.0635	0.91	0.0621	0.12	0.0081
d	10.12	1.45	0.1468	1.46	0.1478	0.91	0.0911	0.61	0.0617	0.34	0.0344
e	12.25	-	-	1.71	0.2095	0.12	0.0147	1.40	0.1715	0.16	0.0196
f	12.25	0.30	0.0367	1.95	0.2390	0.50	0.0612	1.30	0.1591	0.45	0.0551
g	10.38	0.04	0.0041	2.10	0.2180	0.45	0.0468	1.11	0.1152	0.49	0.0509
h	8.95	-	-	0.80	0.0716	-	-	2.64	0.2365	0.25	0.0224
k	10.92	2.40	0.2620	0.15	0.0164	0.34	0.0371	1.83	0.2000	0.86	0.0939
Total			0.5246		1.6687		0.4569		1.1915		0.3753

CALCULATION FOR O.R.O. DEPTH

Date	O.R.O. Cusecs	B.F. Cusecs	D.R.O. Cusecs
3.6.71	2602	2602	0
4.6.71	5359	2681	2678
5.6.71	46900	2760	44140
6.6.71	16223	2839	13334
7.6.71	18834	2997	15936
8.6.71	6920	2997	3923
9.6.71	3928	3076	852
10.6.71	3155	3155	0
		Total	80943

Gross W.A.R.F. over the Catchment

3.6.71	0.5246
4.6.71	1.6687
5.6.71	0.4559
6.6.71	1.1915
7.6.71	0.3763
Total	4.2170"

R.O.Depth = $\frac{80943 \times 24 \times 3600 \times 12}{992 \times 5280 \times 5280}$

$= \frac{80943}{26674} = 3.032"$

Loss of rain = (4.217 - 3.032)" = 1.185

$\therefore$ Loss rate = $\frac{1.185}{5}$ = 0.237"/day

Non of the WARF are less than 0.237"/day

$\therefore$ ϕ - Index = 0.237"/day

Dates	3.6.71	4.6.71	5.6.71	6.6.71	7.6.71
R.F. Excess	0.2876	1.4317	0.2199	0.9545	0.1383"

CALCULATION OF WARF OVER INDIVIDUAL SUB BASINS

STORM DATE 3.6.71 to 7.6.71

Sub-Basin No.	R.G. Stn.	Weightage in %	R.R.F.	3.6.71 WARF	4.6.71 RRF	4.6.71 WARF	5.6.71 RRF	5.6.71 WARF	6.6.71 RRF	6.6.71 WARF	7.6.71 RRF	7.6.71 WARF
1	a	33.42	-	-	2.35	0.9040	0.36	0.1383	0.79	0.3038	0.21	0.0307
	b	31.42	-	-	2.50	0.7860	0.67	0.2108	0.50	0.1573	0.45	0.1416
	d	15.90	1.45	0.2305	1.46	0.2320	0.91	0.1446	0.61	0.0970	0.34	0.0540
	e	14.26	-	-	1.71	0.2433	0.12	0.0171	1.40	0.1995	0.16	0.0223
Total				0.2305		2.1658		0.5108		0.7576		0.2991
	b	2.50	-	-	2.50	0.0625	0.67	0.0167	0.50	0.0125	0.45	0.0112
	d	13.10	1.45	0.1900	1.46	0.1911	0.91	0.1192	0.61	0.0799	0.34	0.0445
	e	25.50	-	-	1.71	0.3845	0.12	0.0260	1.40	0.3150	0.16	0.0330
	c	17.40	1.11	0.1930	1.20	0.2088	0.93	0.1616	0.91	0.1581	0.12	0.0209
	f	24.30	0.30	0.0729	1.95	0.4740	0.50	0.1215	1.30	0.3160	0.45	0.1092
	g	20.20	0.04	0.0088	2.10	0.4245	0.45	0.0909	1.11	0.2242	0.49	0.0990
Total				0.4647		1.7454		0.5358		1.1057		0.3208

1	2	3	4	5	6	7	8	9	10	11	12	13
3	c	9.90	1.11	0.1099	1.20	0.1189	0.93	0.0921	0.91	0.0900	0.12	0.0119
	f	30.80	0.30	0.0924	1.95	0.6005	0.50	0.1540	1.30	0.4005	0.45	0.1387
	g	25.70	0.04	0.0103	2.10	0.5390	0.45	0.1156	1.11	0.2850	0.49	0.1259
	h	6.09	-	-	0.80	0.0487	-	-	2.64	0.1609	0.25	0.0152
	k	27.51	2.40	0.7560	0.15	0.0413	0.34	0.0935	1.83	0.5025	0.86	0.2365
Total				1.4286		1.3484		0.4552		1.4389		0.5282
4	g	1.71	0.04	0.0007	2.10	0.0359	0.45	0.0077	1.11	0.0190	0.49	0.0084
	h	44.50	-	-	0.80	0.3560	-	-	2.64	1.1750	0.25	0.1112
	k	53.79	2.40	1.2910	0.15	0.0807	0.34	0.1830	1.83	0.9850	0.86	0.4625
Total				1.2917		0.4726		0.1907		2.1790		0.5821
5	h	53.85	-	-	0.80	0.4315	-	-	2.64	1.4220	0.25	0.1347
	k	46.15	2.40	1.0830	0.15	0.0677	0.34	0.1536	1.83	0.8260	0.86	0.3882
Total				1.0830		0.4992		0.1536		2.2480		0.5229

1	2	3	4	5	6	7	8	9	10	11	12	13
6	h	22.65	-	-	0.80	0.1812	-	-	2.64	0.5980	0.25	0.0566
	k	77.35	2.40	1.8580	0.15	0.1160	0.34	0.2632	1.83	1.4175	0.86	0.6650
Total				1.8580		0.2972		0.2632		2.0155		0.7216
7-10	h	100.00	-	-	0.80	0.8000	-	-	2.64	2.6400	0.25	0.2500
Total						0.8000				2.6400		0.2500

CALCULATION OF ERF ON INDIVIDUAL SUB-BASINS OF THE CATCHMENT

STORM DATE 3 - 7 JUNE 1971

(All rainfall figures in inches)

Sub-Basin No.	Weigh-tage %	WARF on individual sub-basins according to date					EWARF on individual sub-basins, ϕ=0.237"/d				
		3	4	5	6	7	3	4	5	6	7
1	2	3	4	5	6	7	8	9	10	11	12
1	39.460	0.2305	2.1658	0.5108	0.7576	0.2991	-	1.9288	0.2738	0.5206	0.0621
2	29.472	0.4647	1.7454	0.5358	1.1057	0.3208	0.2277	1.5084	0.2988	0.8687	0.0838
3	16.571	1.4286	1.3484	0.4552	1.4389	0.5282	1.1916	1.1114	0.2182	1.2019	0.2912
4	8.250	1.2917	0.4726	0.1907	2.1790	0.5821	1.0547	0.2356	-	1.9420	0.3451
5	3.250	1.0830	0.4992	0.1536	2.2480	0.5229	0.8460	0.2622	-	2.0110	0.2859
6	1.727	1.8580	0.2972	0.2632	2.0155	0.7216	1.6210	0.0602	0.0262	1.7785	0.4846
7-10	1.270	-	0.8000	-	2.6400	0.2500	-	0.5630	-	2.4030	0.0130

		EWARF for entire basin					EWARF on individual sub-basins ϕ = 0.2651"/day				
1	39.460	-	0.7600	0.1080	0.2055	0.0245	-	1.9007	0.2457	0.4925	0.0340
2	29.472	0.0670	0.4445	0.0880	0.2560	0.0247	0.1996	1.4803	0.2707	0.8406	0.0557
3	16.571	0.1975	0.1842	0.0362	0.2001	0.0483	1.1635	1.0833	0.1901	1.1738	0.2631
4	8.250	0.0870	0.0194	-	0.1602	0.0285	1.0266	0.2075	-	1.9139	0.3170
5.	3.250	0.0275	0.0085	-	0.0653	0.0093	0.8179	0.2341	-	1.9829	0.2578
6	1.727	0.0280	0.0014	0.0005	0.0308	0.0084	1.5929	0.0321	-	1.7504	0.4565
7	1.270	-	0.0072	-	0.0306	0.0002	-	0.5349	-	2.3749	
		0.4070	1.4252	0.2327	0.9485	0.1439					

Total EWARF = 0.4070

1.4252

0.2327

0.9485

0.1439

3.1573

O.R.O. depth = 3.0320

Surplus depth = 0.1253

$\therefore$ ϕ = Index must be increased by an amount x given by

$$x \left[4 \times (0.39460 + 0.03250 + 0.03250) + 5x(0.2947 + 0.16571 + 0.01727) + 3 \times 0.01270 \right] = 0.1253''$$

or $x \left[4 \times 0.50960 + 5 \times 0.47770 + 0.03810 \right] = 0.1253''$

or $(2.03840 + 2.38850 + 0.03810)\, x = 0.1253''$

or $4.46500\, x = 0.1253$

or $x = \dfrac{0.1253}{4.465} = 0.0281$

But EWARF on individual basins show that 0.0262''

40.0130'' are less than x.

$\therefore$ Revised value of $x = \dfrac{0.1253-(0.0005+0.0002)}{4.46500-(0.01727+0.01270)}$

$$= \frac{0.1246}{4.43503} = 0.0281''$$

$\therefore$ Revised value of ϕ index = 0.2370 + 0.0281

= 0.2651'' / day

AJAY BASIN AT SIKATIA (BIHAR)

CALCULATION FOR W.A.R.F. OVER THE CATCHMENT - STORM DATE 9-13 JULY 1971

(All rainfall Fig. in inches)

Stn.	Weight %	9.7.71		10.7.71		11.7.71		12.7.71		13.7.71		14.7.71	
		R.R.F.	W.A.R.F.	R.R.F.	WARF	R.R.F.	WARF	R.R.F.	WARF	R.R.F.	WARF	R.R.F.	WARF
a	15.16	-	-	0.30	0.0455	0.43	0.0651	0.29	0.0439	0.65	0.0985	0.22	0.033
b	13.14	1.75	0.2300	1.46	0.1920	0.56	0.0736	0.37	0.0486	0.15	0.0197	0.42	0.055
c	6.82	2.69	0.1836	-	-	6.75	0.4610	2.45	0.1670	0.63	0.0430	0.11	0.007
d	10.12	-	-	1.26	0.1275	0.53	0.0536	0.51	0.0516	0.50	0.0506	0.67	0.067
e	12.25	-	-	-	-	0.22	0.0269	-	-	0.25	0.0306	0.51	0.062
f	12.25	0.25	0.0306	-	-	2.80	0.3430	0.30	0.0367	0.76	0.0930	0.45	0.055
g	10.39	-	-	0.12	0.2124	0.29	0.0301	0.30	0.3115	3.26	0.3385	1.30	0.135
h	8.95	-	-	-	-	-	-	1.56	0.1386	0.74	0.0663	0.20	0.0179
k	10.92	0.15	0.0164	-	-	2.45	0.2675	1.75	0.1910	0.76	0.0830	0.18	0.0196
			0.4606		0.3774		1.3208		0.8899		0.8232		0.453

CALCULATION FOR D.R.O. DEPTH

Date	O.R.O. Cusecs	B.F. Cusecs	D.R.O. Cusecs
8.7.71	4049	4049	0
9.7.71	6513	4470	2043
10.7.71	7535	4871	2644
11.7.71	20308	5312	14996
12.7.71	24758	5733	19025
13.10.71	17233	6154	11079
14.10.71	10412	6575	3837
15.10.71	6995	6995	0
		Total	53623

O.R.O.depth

$$= \frac{53623 \times 24 \times 3600 \times 12}{992 \times 5280 \times 5280}$$

$$= \frac{53623}{26674} = 2.010''$$

Taking the rainfall upto inflexion point which comes on 13.7.71 only effective in producing R.O. We get gross rainfall

$$= 0.4606 + 0.3774 + 1.3208 + 0.8899 + 0.8232$$

$$= 3.8719$$

$\therefore$ Lost rain $= (3.8719 - 2.010)'' = 1.8619$

$\therefore$ Average loss $= \frac{1.8619}{5} = 0.3724$

Non of W.A.R.F. are less than 0.3724"/day

$\therefore$ ϕ Index $= 0.3724''$/day

GROSS W.A.R.F. EXCESS OVER THE BASIN

Date	Gross W.A.R.F. inch	ϕ Index inch	R.F. Excess inch
9.7.71	0.4606	0.3724	0.0882
10.7.71	0.3774	0.3724	0.0050
11.7.71	1.3208	0.3724	0.9484
12.7.71	0.8899	0.3724	0.5175
13.7.71	0.8232	0.3724	0.4508
		Total	2.0099
			= 2.01"

CALCULATION FOR WARF OVER INDIVIDUAL SUB-BASINS - STORM DATE 9-13 JULY 1971

(All rainfall figures in inches)

Sub-Basin No.	R.G. Stn.	Weigh-tage %	9.7.71		10.7.71		11.7.71		12.7.71		13.7.71	
			R.R.F.	W.A.R.F.	R.R.F.	W.A.R.F.	R.R.F.	W.A.R.F.	R.R.F.	WARF	R.R.F.	W.A.R.F.
1	2	3	4	5	6	7	8	9	10	11	12	13
1	a	38.42	-	-	0.30	0.1153	0.43	0.1652	0.29	0.1114	0.65	0.2500
	b	31.42	1.75	0.5500	1.46	0.4590	0.56	0.1760	0.37	0.1163	0.15	0.0472
	d	15.90	-	-	1.26	0.2002	0.53	0.0842	0.51	0.0811	0.50	0.0795
	e	14.26	-	-	-	-	0.22	0.0314	-	-	0.25	0.0356
Total				0.5500		0.7745		0.4568		0.3098		0.4123
2	b	2.50	1.75	0.0438	1.46	0.0362	0.56	0.0140	0.37	0.0093	0.15	0.0037
	d	13.10	-	-	1.26	0.1650	0.53	0.0695	0.51	0.0668	0.50	0.0655
	e	22.60	x2x68	-	-	-	0.22	0.0495	-	-	0.25	0.0562
	c	17.40	2.69	0.4680	-	-	6.75	1.1740	2.45	0.4265	0.63	0.1095
	f	24.30	0.25	0.0607	-	-	2.80	0.6800	0.30	0.0729	0.76	0.1845
	g	20.20	-	-	0.12	0.0243	0.29	0.0536	0.30	0.0606	3.26	0.6580
				0.5725		0.2255		2.0456		0.6361		1.0774

contd.../

1	2	3	4	5	6	7	8	9	10	11	12	13
3	c	9.90	2.69	0.2665	-	-	6.75	0.6690	2.45	0.2425	0.63	0.0624
	f	30.80	0.25	0.0770	-	-	2.80	0.8620	0.30	0.0924	0.76	0.2341
	g	25.70	-	-	0.12	0.0303	0.29	0.0745	0.30	0.0771	3.26	0.8375
	h	6.09	-	-	-	-	-	-	1.56	0.0950	0.74	0.0451
	k	27.51	0.15	0.0413	-	-	2.45	0.6740	1.75	0.4815	0.76	0.2090
Total				0.3848		0.0308		2.2795		0.9885		1.3881
4	g	1.71	-	-	0.12	0.0020	0.29	0.0049	0.30	0.0051	3.26	0.0557
	h	44.50	-	-	-	-	-	-	1.56	0.6945	0.74	0.3295
	k	53.79	0.15	0.0413	-	-	2.45	0.6740	1.75	0.4815	0.76	0.2090
Total				0.0807		0.0020		1.3239		1.6416		0.7942
5	h	53.85	-	-	-	-	-	-	1.56	0.8400	0.74	0.3980
	k	46.15	0.15	0.0802	-	-	2.45	1.1300	1.75	0.8080	0.76	0.3510
Total				0.0692				1.1300		1.6480		0.7490
6	h	22.65	-	-	-	-	-	-	1.56	0.3535	0.74	0.1675
	k	77.35	0.15	0.1200	-	-	2.45	1.8950	1.75	1.3530	0.76	0.5875
Total				0.1200				1.8950		1.7065		0.7550
7-10	h	100.00	-	-	-	-	-	-	1.56	1.5600	0.74	0.7400
Total										1.5600		0.7400

CALCULATION OF ERF ON INDIVIDUAL SUB BASINS OF AJAY CATCHMENT (STORM DATE 9-13 JULY 1971)

Sub-Basins No.	Weigh-tage %	WARF Inches Basin wise 9.7.71	10.7.71	11.7.71	12.7.71	13.7.71	ERF with ø = 0.3724"/day 9.7.71	10.7.71	11.7.71	12.7.71	13.7.71
1	2	3	4	5	6	7	8	9	10	11	12
1	39.460	0.5500	0.7745	0.4568	0.3098	0.4123	0.1776	0.4021	0.0844	-	0.0330
2	29.472	0.5725	0.2255	2.0456	0.6361	1.0774	0.2001	-	1.6732	0.2637	0.7050
3	16.571	0.3848	0.0308	2.2795	0.9385	1.3381	0.0124	-	1.9071	0.6161	1.0157
4	8.250	0.0807	0.0020	1.3239	1.6416	0.7942	-	-	0.9515	1.2692	0.4218
5	3.250	0.0692	-	1.1300	1.6480	0.7490	-	-	0.7576	1.2766	0.3763
6	1.727	0.1200	-	1.8950	1.7065	0.7550	-	-	1.5226	1.3341	0.3326
7-10	1.270	-	-	-	1.5600	0.7400	-	-	-	1.1876	0.3676
		WARF for whole catchment				ERF with ø = 0.3891					
1	39.460	0.0701	0.3589	0.0333	-	0.0158	0.1609	0.3854	0.0677	-	0.0232
2	29.472	0.0590	-	0.4930	0.0776	0.2080	0.1834	-	1.6565	0.2470	0.6883
3	16.571	0.0021	-	0.3160	0.1021	0.1683	-	-	1.8904	0.5994	0.9930
4	8.250	-	-	0.0785	0.1047	0.0348	-	-	0.9348	1.2525	0.4061
5	3.250	-	-	0.0246	0.0415	0.0122	-	-	0.7409	1.2589	0.3599
6	1.727	-	-	0.0264	0.0231	0.0066	-	-	1.5059	1.3174	0.3659
7-10	1.270	-	-	-	0.0151	0.0047	-	-	-	1.1709	0.3509
		0.1312	0.1589	0.9688	0.3641	0.4504					

$\therefore$ Total ERF depth over the basin

$=$ 0.1312
0.1589
0.9688
0.3641
0.4504
2.0734

O.R.O. depth $=$ 2.0100

Surplus 0.0634

$\therefore$ ϕ-Index should be increased by an amount x given by the following relation to absorb the surplus depth.

$$\left[4x(0.39460+0.29472+0.16571)+3x(0.08250+0.03250+0.01727)+2x0.01270\right] x = 0.0634$$

or, $(4x0.85503+3x0.13227+0.02540)\ x = 0.0634$

or, $'3.42012+0.39681+0.02540)\ x = 0.0634''$

or, $3.84233\ x = 0.0634$

or $x = 0.0165''/\text{day}$

$\therefore$ ϕ revised $= 0.3724+0.0165'' = 0.3889$

But rainfall on 9.7.71 in 3rd sub-basin becomes 0.3889".

$\therefore$ Now for x' (revised value of x) we get

$(3.84233 - 0.1657)\ x = (0.0634 - 0.0021)$

or $(3.67662)\ x = (0.0613)$

or $x = \frac{0.0613}{3.67662} = 0.0167''$

$\therefore$ ϕ 2nd revised $= (0.3724\ \ 0.0167'')/\text{day}$

$= 0.3891$

AJAY BASIN AT SIKATIA (BIHAR)
CALCULATION FOR WARF OVER THE CATCHMENT - STORM DATE 28.9.71 TO 2.10.71 (All rainfall figures are in inches)

Stn.	Weight percen-tage	28.9.71		29.9.71		30.9.71		1.10.71		2.10.71	
		RRF	WARF	RRF	WARF	RRF	WARF	RRF	WARF	RRF	WARF
a	15.16	-	-	0.38	0.0575	3.94	0.5970	-	-	0.49	0.0720
b	13.14	0.20	0.0263	0.95	0.1249	1.75	0.2300	1.75	0.2300	0.25	0.0328
c	6.82	-	-	0.08	0.0054	1.07	0.0730	1.00	0.0682	0.33	0.0225
d	10.12	1.33	0.1346	1.26	0.1276	1.33	0.1346	-	-	-	-
e	12.25	-	-	-	-	2.76	0.3380	1.55	0.1900	0.16	0.0196
f	12.25	0.26	0.0318	1.28	0.1568	1.45	0.1775	0.37	0.0453	-	-
g	10.39	-	-	0.07	0.0072	1.64	0.1702	2.33	0.2420	0.28	0.0291
h	8.95	-	-	-	-	1.35	0.1209	-	-	-	-
k	10.92	-	-	0.46	0.0502	0.53	0.0579	0.67	0.0731	0.64	0.0699
Total			0.1927		0.5296		1.8991		0.8686		0.2459

CALCULATION FOR O.R.O. DEPTH

Date	O.R.O. Cumecs	B.F. Cusecs	D.R.O. Cusecs
28.9.71	598	598	0
29.9.71	2054	566	1488
30.9.71	23276	534	22742
1.10.71	24531	502	24029
2.10.71	11540	470	11070
3.10.71	5678	437	5241
4.10.71	3835	405	3430
5.10.71	2770	373	2397
6.10.71	1862	341	1521
7.10.71	1232	309	923
8.10.71	960	277	683
9.10.71	244	244	0
Total			73524

$$\text{DRO Depth} = \frac{12 \times 73524 \times 24 \times 3600}{992 \times 5280 \times 5280}$$

$$= 2.76''$$

Gross weighted average rainfall

$$= 0.1927$$
$$0.5296$$
$$0.8991$$
$$0.8686$$
$$\underline{0.2459}$$
$$3.7359$$

$\therefore$ Rain lost $= (3.7359 - 2.7600)$

$$= 0.9759$$

$\therefore$ Average loss rate $= \dfrac{0.9759''}{5}$ /day

$$= 0.1952''/\text{day}$$

$$\therefore \phi\text{-Index} = \frac{0.9759 - 0.1927}{4} = \frac{0.7832}{4} = 0.1958''/\text{day}$$

$\therefore$ R.F. excesses are

29.9.71	0.3338
30.9.71	1.7033
1.10.71	0.6728
2.10.71	0.0501
Total	2.7600

AJAY CATCHMENT AT SIKATIA CALCULATION FOR WARF OVER INDIVIDUAL SUB BASINS STORM DATE 28.9.71 TO 2.10.1971

(All rainfall figures are in inches)

Sub basin No.	Stn.	Weightage percentage	28.9.71 RRF	28.9.71 WARF	29.9.71 RRF	29.9.71 WARF	30.9.71 RRF	30.9.71 WARF	1.10.71 RRF	1.10.71 WARF	2.10.71 RRF	2.10.71 WARF
1	2	3	4	5	6	7	8	9	10	11	12	13
1	a	38.42	-	-	0.38	0.1460	3.94	1.5120	-	-	0.49	0.1882
	b	31.42	0.20	0.0629	0.95	0.2985	1.75	0.5500	1.75	0.5500	0.25	0.0785
	d	15.90	1.33	0.2115	1.26	0.2002	1.33	0.2115	-	-	-	-
	e	14.26	-	-	-	-	2.76	0.3938	1.55	0.2210	0.16	0.0228
Total				0.2744		0.6447		2.6673		0.7710		0.2895
2	b	2.50	0.20	0.0050	0.95	0.0238	1.75	0.0437	1.75	0.0437	0.25	0.0063
	d	13.10	1.33	0.1742	1.26	0.1650	1.33	0.1742	-	-	-	-
	e	22.50	-	-	-	-	2.76	0.6210	1.55	0.3485	0.16	0.0360
	c	17.40	-	-	0.08	0.0139	1.07	0.1862	1.00	0.1740	0.33	0.0575
	f	24.30	0.26	0.0632	1.28	0.3110	1.45	0.3522	0.37	0.0899	-	-
	g	20.20	-	-	0.07	0.0141	1.64	0.3310	2.33	0.4710	0.28	0.0565
Total				0.2424		0.5278		1.7083		1.1271		0.1563

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13
3	c	9.90	-	-	0.08	0.0089	1.07	0.1060	1.00	0.0990	0.33	0.0327
	f	30.80	0.26	0.0801	1.28	0.3940	1.45	0.4465	0.37	0.1140	-	-
	g	25.70	-	-	0.07	0.0180	1.64	0.4215	2.33	0.5990	0.28	0.0720
	h	6.09	-	-	-	-	1.35	0.0822	-	-	-	-
	k	27.51	-	-	0.46	0.1265	0.53	0.1458	0.67	0.1841	0.64	0.1760
Total				0.0801		0.5474		1.2020		0.9961		0.2807
4	g	1.71	-	-	0.07	0.0012	1.64	0.0280	2.33	0.0398	0.28	0.0048
	h	44.50	-	-	-	-	1.35	0.6010	-	-	-	-
	k	53.79	-	-	0.46	0.2475	0.53	0.2850	0.67	0.3605	0.64	0.3445
Total						0.2487		0.9140		0.4003		0.3493
5	h	53.85	-	-	-	-	1.35	0.7275	-	-	-	-
	k	46.15	-	-	0.46	0.2125	0.53	0.2448	0.67	0.3095	0.64	0.2958
Total						0.2125		0.9723		0.3095		0.2958

1	2	3	4	5	6	7	8	9	10	11	12	13
6	h	22.65	-	-	-	-	1.35	0.3060	-	-	-	-
	k	77.35	-	-	0.46	0.3560	0.53	0.4100	0.67	0.5180	0.64	0.4950
Total						0.3560		0.7160		0.5180		0.4950
7-10	h	100.00	-	-	-	-	1.35	1.3500	-	-	-	-
Total								1.3500				

$\phi = 0.1958$

Sub basin No.	Weightage percentage	WARF on individual basins in inches					EWARF on individual sub basins in inches				
		28	29	30	1	2	28	29	30	1	2
1	39.460	0.2744	0.6447	2.6673	0.7710	0.2895	0.0786	0.4489	2.4715	0.5752	0.0937
2	29.472	0.2424	0.5278	1.7083	1.1271	0.1563	0.0466	0.3320	1.5125	0.9313	-
3	16.571	0.0801	0.5474	1.2020	0.9961	0.2807	-	0.3516	1.0062	0.8003	0.0349
4	8.250	-	0.2487	0.9140	0.4003	0.3493	-	0.0529	0.7182	0.2045	0.1535
5	3.250	-	0.2125	0.9723	0.3095	0.2958	-	0.0167	0.7765	0.1137	0.1000
6	1.727	-	0.3560	0.7160	0.5180	0.4950	-	0.1602	0.5202	0.3222	0.2992
7-10	1.270	-	-	1.3500	-	-	-	-	1.1542	-	-

		EWARF for whole catchment $\phi = 0.1958''$ /day					EWARF on individual sub basins $\phi = 0.2030''$ /day				
1	39.460	0.0310	0.1770	0.9750	0.2270	0.0370	0.0664	0.4367	2.4593	0.5630	0.0517
2	29.472	0.0137	0.0978	0.4455	0.2745	-	0.0344	0.3198	1.5003	0.9191	-
3	16.571	-	0.0582	0.1665	0.1326	0.0141	-	0.3394	0.9940	0.7881	0.0727
4	8.250	-	0.0044	0.0593	0.0169	0.0127	-	0.0407	0.7060	0.1923	0.1413
5	3.250	-	0.005	0.0252	0.0037	0.0033	-	0.0045	0.7643	0.1015	0.0878
6	1.727	-	0.0028	0.0090	0.0056	0.0052	-	0.1480	0.5080	0.3100	0.2870
7-10	1.270	-	-	0.0147	-	-	-	-	1.1420	-	-
		0.0447	0.3407	1.6952	0.6603	0.0723					

.'. Total ERF depth over the catchment

0.0447
0.3407
1.6952
0.6603
0.0723

2.8132"

O.R.O. depth = 2.7600"

Surplus depth = 0.0532"

Now, value of ϕ must be increased by an amount x given by the following relation in order to absorb the surplus depth of 0.0582"

$$x \left[5 \times 0.39460 + 4 \times (0.29472 + 0.16571 + 0.08250 + 0.03250 + 0.01727) + 1 \times 0.01270 \right] = 0.0532''$$

$$\text{or } x \left[1.97300 + 4(0.59270) + 0.01270 \right] = 0.0532''$$

$$\text{or } x \left[1.97300 + 2.37080 + 0.01270 \right] = 0.0532''$$

$$\text{or } x \left[4.35650 \right] x = 0.0532$$

$$\therefore \quad x = \frac{0.0532}{4.3565} = 0.0122''/\text{day}$$

$$\therefore \quad \phi \text{ revised} = (0.1958 + 0.0122)''/\text{day}$$

$$= 0.2080''/\text{day}$$

AJAY BASIN AT SIKATIA (BIHAR) CALCULATION FOR WARF OVER THE CATCHMENT STORM DATE 11-13 OCT. 1973

(Rainfall figures in inches)

R.G. Stn.	Weightage percentage	11.10.73 RRF	11.10.73 WARF	12.10.73 RRF	12.10.73 WARF	13.10.73 RRF	13.10.73 WARF
a	15.16	0.80	0.1212	2.90	0.4395	0.05	0.0076
b	13.14	1.00	0.1314	2.95	0.3878	0.50	0.0657
c	6.82	0.13	0.0089	5.66	0.3860	0.33	0.0225
d	10.12	-	-	0.20	0.0202	-	-
e	12.25	0.98	0.1200	2.62	0.3210	0.21	0.0257
f	12.25	2.22	0.2695	3.54	0.4330	-	-
g	10.39	-	-	0.12	0.0125	3.30	0.3425
h	8.95	2.39	0.2140	2.66	0.2380	-	-
k	10.92	-	-	1.09	0.1190	-	-
			0.8650		2.3570		0.4640

CALCULATION FOR R.O. DEPTH

Date	ORO cusecs	BF cusecs	DRO cusecs
10.10.73	909	909	0
11.10.73	1020	857	163
12.10.73	52250	805	51445
13.10.73	14700	753	13947
14.10.73	8648	701	7947
15.10.73	3800	649	3151
16.10.73	1194	598	596
17.10.73	547	547	0
			77249

Computation for ϕ index

$$\text{R.O. depth} = \frac{77249 \times 24 \times 12 \times 3600}{992 \times 5280 \times 5280}$$

$$= 2.90''$$

WARF over the catchment

$$= 0.8650$$
$$2.3570$$
$$\underline{0.4640}$$
$$\text{Total} = 3.6860$$

$\therefore$ Lost rain $= (3.6860-2.90)'' = 0.7860$

$\therefore$ Rate of loss $= \frac{0.7860}{3} = 0.2620''$/day

Non of the WARF is less than 0.2620"

$\therefore$ ϕ index $=$ 0.262" /day

Effective WARF over the catchment

11.10.73	– 0.603
12.10.73	– 2.095
13.10.73	– 0.202
	2.900

CALCULATION FOR WARF OVER INDIVIDUAL SUB BASINS OF THE CATCHMENT
(R.F. IN INCHES) STORM DATE 11-13 OCT. 1973

Sub basin No.	R.G. Stn.	Weightage percentage	11.10.73		12.10.73		13.10.73	
			RRF	WARF	RRF	WARF	RRF	WARF
1	2	3	4	5	6	7	8	9
1	a	38.42	0.80	0.3075	2.90	1.1140	0.05	0.0192
	b	31.42	1.00	0.3142	2.95	0.9270	0.50	0.1570
	d	15.90	-	-	0.20	0.0318	-	-
	c	14.26	0.98	0.1398	2.62	0.3735	0.21	0.0299
Total				0.7615		2.4463		0.2061
2	b	2.50	1.00	0.0250	2.95	0.0738	0.50	0.0125
	d	13.10	-	-	0.20	0.0262	-	-
	e	22.50	0.98	0.2202	2.62	0.5890	0.21	0.0472
	c	17.40	0.13	0.0226	5.66	0.9850	0.33	0.0574
	f	24.30	2.22	0.5375	3.54	0.8650	-	-
	g	20.20	-	-	0.12	0.0242	3.30	0.6665
Total				0.8053		2.5632		0.7836

Contd.

1	2	3	4	5	6	7	8	9
3	c	9.90	0.13	0.0129	5.66	0.5610	0.33	0.0327
	f	30.80	2.22	0.6775	3.54	1.0900	-	-
	g	25.70	-	-	0.12	0.0308	3.30	0.8480
	h	6.09	2.39	0.1455	2.66	0.1620	-	-
	k	27.51	-	-	1.09	0.3000	-	-
Total				0.8359		2.1438		0.8807
4	g	1.71	-	-	0.12	0.0021	3.30	0.0564
	h	44.50	2.39	1.0640	2.66	1.1830	-	-
	k	53.79	-	-	1.09	0.5860	-	-
Total				1.0640		1.7111		0.0564
5	h	53.85	2.39	1.2875	2.66	1.4320	-	-
	k	46.15	-	-	1.09	0.5025	-	-
Total				1.2875		1.9355		-

Contd.

1	2	3	4	5	6	7	8	9
6	h	22.65	2.39	0.5415	2.66	0.6025	-	-
	k	77.35	-	-	1.09	0.9200	-	-
Total				0.5415		1.5225		
7-10	h	100.00	2.39	2.3900	2.66	2.6600	-	-
Total				2.3900		2.6600		

CALCULATION FOR ERF ON INDIVIDUAL SUB BASINS OF AJAY CATCHMENT AT SIKATIA
STORM DATE 11-13 OCT. 1973

(All rainfall figures in inches)

Sub basin No.	Weightage percentage	GROSS WARF INCH			NET ARF IN ϕ = 0.2620		
		11.10.73	12.10.73	13.10.73	11.10.73	12.10.73	13.10.73
1	39.460	0.7615	2.4463	0.2061	0.4995	2.1843	-
2	29.472	0.8053	2.5632	0.7836	0.5433	2.3012	0.5216
3	16.571	0.8359	2.1438	0.8807	0.5739	1.8818	0.6187
4	8.250	1.0640	1.7111	0.0564	0.8020	1.4491	-
5	3.250	1.2875	1.9355	-	1.0255	1.6735	-
6	1.727	0.5415	1.5225	-	0.2795	1.2605	-
7-10	1.270	2.3900	2.6600	-	2.1280	2.3980	-
		WARF FOR ENTIRE BASIN			NET ARF WITH ϕ = 0.2719"/day		
1	39.460	0.1970	0.8630	-	0.4896	2.1744	-
2	29.472	0.1630	0.6780	0.1537	0.5334	2.2913	0.5117
3	16.571	0.0949	0.3115	0.1024	0.5640	1.8719	0.6088
4	8.250	0.0662	0.1195	-	0.7921	1.4392	-
5	3.250	0.0329	0.0537	-	1.0156	1.6636	-
6	1.727	0.0048	0.0218	-	0.2696	1.2506	-
7-10	1.270	0.0270	0.0304	-	2.1181	2.3881	-
		0.5858	2.0779	0.2561			

$$
\begin{array}{lr}
\text{WARF excess} & = 0.5858 \\
 & 2.0779 \\
 & 0.2561 \\
\hline
 & 2.9198 \\
\text{R.O. Depth} & = 2.9000 \\
\hline
\therefore \text{Surplus rain} & = 0.0198''
\end{array}
$$

The value of ϕ index must be increased by an amount x to absorb the surplus rain. For the value of x

$$x \left[2(0.39460 + 0.08250 + 0.03250 + 0.0172 + 0.01270) + 3(0.29472 + 0.16571) \right] = 0.0198$$

$$\text{or } x \left[2 \times 0.53957 + 3 \times 0.46043 \right] = 0.0198$$

$$\text{or } x\,(1.07914 + 1.38129) = 0.0198$$

$$\text{or } 2.46043\,x = 0.0198$$

$$\text{or } x = 0.0099''$$

$$\therefore \phi \text{ revised} = (0.2620 + 0.0099)''\text{/day}$$

$$= 0.2719''\text{/day}$$

APPENDIX - 6

COMPUTATION OF FLOOD HYDROGRAPHS FOR FIVE STORMS

COMPUTATION OF FLOOD HYDROGRAPH OF STORM DATED 15-17 OCT. 1971
TIME INTERVAL 12 HOURS (ALL FLOWS IN CUSECS)

R.O. = 0.044x4cx I

Sub-basin No.	Area in Acres AC	ERF according to time interval in inch		R.O. generated cusecs	ELEMENTS OF HYDROGRAPH ORDINATES ACCORDING TO TIME INTERVAL																	
		Interval	ERF(I)		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	250560				0.9700	0.0291	0.0009															
		1	-																			
		151 2	-																			
		3	0.5938	12490			12110	364	11													
		161 4	0.5938	12490				12110	364	11												
2	187117					0.9409	0.0568	0.0025	0.0001													
		1	0.1501	2360		2220	131	6														
		2	0.1501	2360			2220	133	6													
		3	0.4843	7600			7150	430		19	1											
		4	0.4843	7600				7150		430	19	1										
3	105139						0.9127	.0821	0.0049	0.0003												
		1	0.2577	2278			2080	187	11		1											
		2	0.2577	2278				2080	187	11	1											
		3	0.3882	3435					3135	282	17	1										
		4	0.3882	3435						3135	282	17	1									
4	52403							.8851	0.1062	0.0080	0.0005											
		1	0.0744	327				289	35	3												
		2	0.0744	327					289	35	3	8										
		3	0.2218	976						863	103	8										
		4	0.2218	976							863	103	8									
5	20640								0.8588	0.1288	0.0116	0.0008										
		1	0.0194	34					29	4												
		2	0.0194	34						29	4											
		3	0.2882	499							428	64	6									
		4	0.2882	499								428	64	6								

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
6	10957									0.8329	0.1499	0.0157	0.0009	0.0001								
		1	0.1582	145						21	22	2										
		2	0.1582	145							121	22	2									
		3	0.0179	16								13	2									
		4	0.0179	16									13	2								
7	4864										0.8080	0.1697	0.0204	0.0018	0.0001							
		1	-	-																		
		2	-	-																		
		3	0.7142	280									226	47	6	1						
		4	0.7142	280										226	47	6	1					
8	2016											0.7835	0.1880	0.0254	0.0025	0.0002						
		1	-	-																		
		2	-	-																		
		3	0.7142	116										91	22	3						
		4	0.7142	116											91	22	3					
9	832												0.7600	0.2052	0.0308	0.0034	0.0003					
		1	-	-																		
		2	-	-																		
		3	0.7142	48											37	10	1					
		4	0.7142	48												37	10	1				
10	352													0.7374	0.2212	0.0366	0.0033	0.004	0.0001			
		1	-	-																		
		2	-	-																		
		3	0.7142	20												15	4	1				
		4	0.7142	20													15	4	1			
Total					0	2220	16543	22019	11648	4944	1864	659	322	372	203	24	34	6	1			
Base Flow				360	424	488	562	616	680	745	809	873	937	1001	1065	1130						
Grand Total				360	424	2708	17095	22935	12328	5689	2673	1532	1259	1373	1268	1224						
Date				14		15		16		17		18		19	20							

COMPUTATION OF FLOOD HYDROGRAPH STORM DATE 3-7 JUNE 1971

R.O. = 0.084 x Ac x I

TIME INTERVAL 12 HOURS (ALL FLOWS IN CUSECS)

Sub-basin No.	Area acres Ac	ERF according time Interval	ERF(I) inch	R.O. generated cusecs	ELEMENTS OF HYDROGRAPH ORDINATES ACCORDING TO TIME INTERVAL 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
1	250560		-	-	0.9700	0.0291	0.0009																	
		1	-	-																				
		2	-	-																				
		3	0.9504	20,000			19400	482	18															
		4	0.9504	20,000				19400	482	18														
		5	0.1229	2585					2508	75	2													
		6	0.1229	2585						2508	75	2												
		7	0.2463	5185							5025	151	5											
		8	0.2463	5185								5025	151	5										
		9	0.0170	358									348	10										
		10	0.0170	358									242	348	10									
2	187117					0.9409	0.0566	0.0025	0.0001															
		1	0.0998	1570		1478	89	4																
		2	0.0998	1570			1478	89	4															
		3	0.7402	11640				10950	658	39	1													
		4	0.7402	11640					10950	658	39	1												
		5	0.1354	2130						2005	120	5												
		6	0.1354	2130							2005	120	5											
		7	0.4203	6610								6210	373	16	1									
		8	0.4203	6610									6210	373	16	1								
		9	0.0279	439										413	25	1								
		10	0.0279	439											413	25	1							
3							0.9127	0.0821	0.0049	0.0003														
		1	0.5818	5140			4690	422	25	2														
		2	0.5818	5140				4690	422	25	2													
		3	0.5417	4785					4366	392	23	1												
		4	0.5417	4785						4366	392	23	1											
		5	0.0951	840							767	69	4											
		6	0.0951	840								767	69	4										
		7	0.5869	5185									4730	426	25	1								
		8	0.5869	5185										4730	426	25	1							
		9	0.1316	1162											1061	95	6							
		10	0.1316	1162												1061	95	6						

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
4	52403							0.8861	0.1062	0.0080	0.0005													
		1	0.5133	2258				1998	240	18	1													
		2	0.5133	2258					1998	240	18	1												
		3	0.1038	456						492	59	4												
		4	0.1038	456							492	59	4											
		5	-	-																				
		6	-	-																				
		7	0.9570	4210										3725	447	34	2							
		8	0.9570	4210											3725	447	34	2						
		9	0.1585	697												616	74	6						
		10	0.1585	697													616	74	6					
5	20640								0.8588	0.1288	0.0116	0.0008												
		1	0.4090	908					780	117	11	1												
		2	0.4090	908						780	117	11	1											
		3	0.1171	260							223	33	3											
		4	0.1171	260								223	33	3										
		5	-	-																				
		6	-	-																				
		7	0.9915	2200											1888	283	25	2						
		8	0.9915	2200												1888	283	25	2					
		9	0.1289	286													246	37	3					
		10	0.1289	286														246	37	3				
		Total				1478	25657	38035	22450	11734	9372	12706	11937	10053	8037	4477	1383	398	48	3				
		Sub-basin 6-10								610	720	134	115	145	78	705	1196	943	571	195				
		Base flow		2602	2642	2681	2721	2760	2800	2839	2879	2918	2958	2997	3037	3076	3116	3155						
		Grand total		2602	2642	4159	28378	40795	25250	15183	12971	15758	15010	13195	11152	8258	5595	4496	9					
		Dates of June 1971		3		4		5		6		7		8		9		10						

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
6	10957									0.8329	0.1499	0.0157	0.0009	0.0001										
		1	0.7965	733						610	110	12	1											
		2	0.7965	733							610	110	12	1										
		3	0.0161	15								12	2											
		4	0.0161	15									12	2										
		5	-	-																				
		6	-	-																				
		7	0.8752	805												670	121	13	1					
		8	0.8752	805													670	121	13	1				
		9	0.2283	210														175	31	3				
		10	0.2283	210															175	31	3			
7	4864										0.8080	0.1697	0.0204	0.0018	0.0001									
		1	-	-																				
		2	-	-																				
		3	0.2675	109										88	19	2								
		4	0.2675	109											88	19 2	2							
		5	-	-																				
		6	-	-																				
		7	1.1875	485													392	82	10	1				
		8	1.1875	485														392	82	10	1			
		9	-	-																				
		10	-	-																				
8	2016											0.7835	).1880	0.0264	0.0025	0.0002								
		1	-	-																				
		2	-	-																				
		3	0.2675	45											35	8	1							
		4	0.2675	45												35	8	1						
		5	-	-																				
		6	-	-																				
		7	1.1875	201														158	38	6				
		8	1.1875	201															158	38	6			
		9	-	-																				
		10	-	-																				

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
9	832												0.7600	0.2062	0.0308	0.0034	0.0003							
		1	-	-																				
		2	-	-																				
		3	0.2675	19											14	4								
		4	0.2675	19												14	4							
		5	-	-																				
		6	-	-																				
		7	1.1875	83														63	63	17	3			
		8	1.1875	83																63	17	3		
		9	-	-																				
		10	-	-																				
10	352													0.7374	0.2214	0.0365	0.0033	0.0004						
		1	-	-																				
		2	-	-																				
		3	0.2675	8												6	2							
		4	0.2675	8													6	2						
		5	-	-																				
		6	-	-																				
		7	1.1875	35																26	8	1		
		8	1.1875	35																	26	8	1	
		9	-	-																				
		10	-	-																				
										610	720	134	145	145	78	705	1196	943	571	195	63	12	1	

1

R.O. = 0.084 x Ac x I

COMPUTATION OF FLOOD HYDROGRAPH STORM DATE 9-13 JULY 1971

TIME UNIT 12 HOURS (ALL FLOWS IN CUSECS)

Sub-basin No.	Area Acres Ac	ERF according to time interval (Inches) Interval	ERF(I)	R.O. generated cusecs	ELEMENTS OF HYDROGRAPH ORDINATES IN CUSECS ACCORDING TO TIME INTERVAL 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	250560																					
					0.9700	0.0291	0.0009															
		1	0.0805	1695	1642	49	2															
	9	2	0.0805	1695		1642	49	2														
		3	0.1927	4055			3940	118	4													
	10	4	0.1927	4055				3940	118	4												
		5	0.0333	703					683	20	1											
	11	6	0.0339	703						683	20	1										
		7	-	-																		
	12	8	-	-																		
		9	0.0116	244									236	7								
	13	10	0.0116	244										236	7							
2	187117																					
						0.9409	0.0566	0.0025	0.0001													
		1	0.0917	1442		1358	81	4														
		2	0.0917	1443			1358	84	4													
		3	-	-																		
		4	-	-																		
		5	0.8283	13020						12250	735	33	1									
		6	0.8283	13020							12250	735	33	1								
		7	0.1235	1942								1828	110	5								
		8	0.1235	1942									[illegible]	306	6 14							
		9	0.3442	5420										5100	306	14	1					
		10	0.3442	5420											5100	306	14	1				
3	105139																					
							0.9127	0.0821	0.0049	0.0003												
		1	-	-																		
		2	-	-																		
		3	-	-																		
		4	-	-																		
		5	0.9452	8350							7626	686	41	3								
		6	0.9452	8350								7626	686	41	3							
		7	0.2887	2555									2330	210	13	1						
		8	0.2887	2555										2330	210	13	1					
		9	0.4995	4410											4026	362	22	1				
		10	0.4995	4410												4026	362	22	1			

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
4	52403							0.8861	0.1062	0.0082	0.0005											
		1	-	-																		
		2	-	-																		
		3	-	-																		
		4	-	-								1820	218	16	1							
		5	0.4674	2058									1820	218	16	1	1					
		6	0.4674	2058										2440	293	22	1					
		7	0.6263	2760											2440	293	22	1				
		8	0.6263	2760												790	95	7				
		9	0.2026	892													790	95	7			
		10	0.2026	892																		
5	20640								0.8688	0.1288	0.0116	0.0008										
		1	-	-																		
		2	-	-																		
		3	-	-																		
		4	-	-									552	73	7	1						
		5	0.3705	643										552	73	7	1					
		6	0.3705	643											936	140	13	1				
		7	0.6295	1090												936	140	13	1			
		8	0.6295	1090													268	40	4			
		9	0.1799	312														268	40	4		
		10	0.1799	312																		

Total		1642	3049	5430	4165	809	12957	20631	12727	7864	11302	13435	6711	1630	449	53	4			
Sub-basin 6-10											580	684	620	801	553	407	202	83	27	4
Base flow	4049	4259	4470	4680	4871	5081	5312	5522	5733	5943	6154	6364	6575	6785	6995					
Grand total	4049	5901	7519	10110	9016	5890	18269	26153	18460	13797	18036	20483	13906	8216	7997					
Date	8		9	10			11		12		13		14		15					

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
6	10957									0.8329	0.1499	0.0157	0.0009	0.0001										
		1	-	-																				
		2	-	-																				
		3	-	-																				
		4	-	-																				
		5	0.7560	696										580	104	11	1							
		6	0.7560	696											580	104	11	1						
		7	0.6587	606												505	91	10	1					
		8	0.6587	606													140	25	3					
		9	0.1830	168																				
		10	0.1830	168														140	25	3				
7	4864										0.8080	0.1697	0.0204	0.0018	0.0001									
		1	-	-																				
		2	-	-																				
		3	-	-																				
		4	-	-																				
		5	-	-																				
		6	-	-																				
		7	0.5855	239													193	40	5					
		8	0.5855	239														193	40	5				
		9	0.1755	72															58	12	1			
		10	0.1755	72																58	12	1		
8	2016											0.7835	0.1880	0.0254	0.0025	0.0002								
		1	-	-																				
		2	-	-																				
		3	-	-																				
		4	-	-																				
		5	-	-																				
		6	-	-																				
		7	0.5855	99														78	19	3				
		8	0.5855	99															78	19	3			
		9	0.1755	30																24	6			
		10	0.1755	30																	24	6		

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25
9	832																							
													0.7600	0.2052	0.0308	0.0034	0.0003							
		1	-	-																				
		2	-	-																				
		3	-	-																				
		4	-	-																				
		5	-	-																				
		6	-	-																				
		7	0.5855	41																				
		8	0.5855	41																				
		9	0.1755	12																				
		10	0.1755	12																				
																			31	8	1			
																				31	8	1		
																					9	2		
																						9	2	
10	362																							
														0.7374	0.2212	0.0366	0.0033	0.0004	0.0001					
		1	-	-																				
		2	-	-																				
		3	-	-																				
		4	-	-																				
		5	-	-																				
		6	-	-																				
		7	0.5855	17																				
		8	0.5855	17																				
		9	0.1755	5																				
		10	0.1755	5																				
																				13	3	1		
																					13	3	1	
																						4	1	
														580	684	620	801	553	407	202	83	27	4	

COMPUTATION OF FLOOD HYDROGRAPH STORM DATE 28.9.71 TO 2.10.1971

~~STORM RATE~~

R.O. = 0.084 x Ac x I

TIME INTERVAL 12 HOURS(ALL FLOWS IN CUSECS)

Sub-basin No.	Area acres Ac		ERF according to interval inches: Interval	ERF(I)	R.O. generated cusecs	ELEMENTS OF HYDROGRAPH ORDINATES ACCORDING TO TIME INTERVAL: 1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
1	2		3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
						0.9700	0.0291	0.0009																
1	250560		1	0.0332	700	680	20																	
		28	2	0.0332	700		680	20																
			3	0.2184	4600			4460	134	4														
		29	4	0.2184	4600				4460	134	4													
			5	1.2297	25900					25120	753	23												
		30	6	1.2297	25900						25120	753	23											
			7	0.2815	5925							5750	172	5										
		1	8	0.2815	5925								5750	172	5									
			9	0.0259	545									529	16									
		2	10	0.0259	545										529	16								
2	187117						0.9409	0.0565	0.0025	(.0001														
			1	0.0142	224		211	13																
			2	0.0142	224			211	13															
			3	0.1599	2520				2370	142	6													
			4	0.1599	2520					2370	142	6												
			5	0.7502	11800						11100	666	30	1										
			6	0.7502	11800							11100	666	30	1									
			7	0.4596	7225								6795	408	18	1								
			8	0.4596	7225									6795	408	18	1							
			9	-	-																			
			10	-	-																			
3	101539							0.9127	0.0821	0.0049	0.0003													
			1	-	-																			
			2	-	-																			
			3	0.1697	1500						1369	123	7											
			4	0.1697	1500							1369	123	7										
			5	0.4970	4390								4005	360	22	1								
			6	0.4970	4390									4066	366	22	1							
			7	0.3941	3480										3180	286	17	1						
			8	0.3941	3480											3180	286	17	1					
			9	0.0364	322												294	26	2					
			10	0.0364	322													294	26	2				

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
4	52403							0.8851	0.1062	0.0080	0.0005												
		1	-	-																			
		2	-	-																			
		3	0.0204	89						79	9	1											
		4	0.0204	89							79	9	1										
		5	0.3530	1553								1376	165	12	1								
		6	0.3530	1553									1375	155	12	1							
		7	0.0962	423										374	45	3							
		8	0.0962	423											374	45	3						
		9	0.0707	311												272	33	2					
		10	0.0707	311													272	33	2				
5	20640								0.8588	0.1288	(.0116	0.0008											
		1	-	-																			
		2	-	-																			
		3	0.0023	4							3												
		4	0.0023	4								3											
		5	0.3822	663									570	85	8	1							
		6	0.3822	663										570	85	8	1						
		7	0.0508	88											76	11	1						
		8	0.0508	88												76	11	1					
		9	0.0439	76													65	10	1				
		10	0.0439	76														65	10	1			
Total					680	911	4704	6977	29139	38696	22514	19196	13613	5672	1234	766	415	113	13	1			
Sub-basin 6-10												57	67	206	420	459	308	208	160	27	2		
Base flow				536	582	566	560	534	518	502	486	470	453	437	421	405	389	373	357				
Grand total				598	1262	1477	5264	7611	29657	39198	23010	19723	14133	6315	2076	1630	1112	694	530				
				28		29		30		1		2		3		4		5					

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
6	10957									0.8329	0.1499	0.0157	0.0009	0.0001									
		1	-	-																			
		2	-	-																			
		3	0.0740	68								57	10	1									
		4	0.0740	68									57	10	1								
		5	0.2540	234										195	35	4							
		6	0.2540	234											195	35	4						
		7	0.1660	143												119	21	2					
		8	0.1660	143													119	21	2				
		9	0.1435	132														110	20	2			
		10	0.1435	132															110	20	2		
7	4864										0.8080	0.1697	0.0204	0.0018	0.0001								
		1	-	-																			
		2	-	-																			
		3	-	-																			
		4	-	-																			
		5	0.5710	234											189	40	5						
		6	0.5710	234												189	40	5					
		7	-	-																			
		8	-	-																			
		9	-	-																			
		10	-	-																			
8	2016											0.7835	0.1880	0.0264	0.0025	0.0002							
		1	-	-																			
		2	-	-																			
		3	-	-																			
		4	-	-																			
		5	0.5710	92												72	17	2					
		6	0.5710	92													72	17	2				
		7	-	-																			
		8	-	-																			
		9	-	-																			
		10	-	-																			

Contd.

RO = 0.042 x Ac x I

Sub-basin No.	Area in Acres Ac	ERF as per time interval Date	ERF (I) inch	R.O. generated Cusecs	29	30	1	2	3	4	5	6	
1	437677				0.9700	0.0291	0.0009						
		1	0.0528	971	748	22	1						
		2	0.3965	7300		7080	212	7					
		3	2.0480	37700			36,550	1097	34				
		4	0.7150	13150				12750	~~1097~~382	12			
		5	0.0296	545					528	16	1		
2	157542					0.9409	0.0565	0.0025	0.0001				
		1											
		2	0.2400	1589			1495	90	4				
		3	0.8990	5950				5600	336	15	1		
		4	0.5920	3920					3690	223	10		
		5	0.0959	634						596	26	2	
3	31597						0.9127	0.0821	0.0049	0.0003			
		1											
		2	0.0442	59				54	5				
		3	0.6680	885					808	72	4		
		4	0.1748	232						212	19	1	
		5	0.1586	210							192	17	1
4	6880							0.8851	0.1062	0.0080	0.0005		
		1											
		2											
		3	1.1420	330						292	35	3	
		4											
		5											
5	1184								0.8588	0.1288	0.0116	0.0008	
		1											
		2											
		3	1.1420	57							49	7	1
		4											
		5											
	Total				748	7102	38258	19598	5787	1438	337	30	2
	B.F.			598	566	534	502	470	437	405	373	341	
	G.T.			598	1314	7636	38760	20068	6224	1843	710	371	
	Dates			29	29								

ERF ON INDIVIDUAL SUD BASINS FOR 1 DAY TIME LAG STORM 28.9.71 TO 2.10.71

Sub-Basin No.	28	29	30	1	2
1 and 2	$\frac{0.0262 + 0.0102}{0.68932}$ $= \frac{0.0364}{0.68932}$ $= 0.0528$	$\frac{0.1722 + 0.4420}{0.68932}$ $= \frac{0.2662}{0.68932}$ $= 0.3965$	$\frac{0.9690 + 0.4420}{0.68932}$ $= \frac{1.4130}{0.68932}$ $= 2.0480$	$\frac{0.2222 + 0.2705}{0.68932}$ $= \frac{0.4927}{0.68932}$ $= 0.7150$	$\frac{0.0204 + 0}{0.68932}$ $= \frac{0.0204}{0.68932}$ $= 0.0296$
3 and 4		$\frac{0.0563 + 0.0033}{0.24821}$ $= \frac{0.0596}{0.24821}$ $= 0.2400$	$\frac{0.1648 + 0.0583}{0.24821}$ $= \frac{0.2231}{0.24821}$ $= 0.8990$	$\frac{0.1310 + 0.0159}{0.24821}$ $= \frac{0.1469}{0.24821}$ $= 0.5920$	$\frac{0.0121 + 0.0117}{0.24821}$ $= \frac{0.0238}{0.24821}$ $= 0.0959$
5 and 6		$\frac{0.0001 + 0.0026}{0.04977}$ $= \frac{0.0027}{0.04977}$ $= 0.0442$	$\frac{0.0245 + 0.0088}{0.04977}$ $= \frac{0.0333}{0.04977}$ $= 0.6680$	$\frac{0.0033 + 0.0054}{0.04977}$ $= \frac{0.0087}{0.04977}$ $= 0.1748$	$\frac{0.0029+0.0050}{0.04977}$ $= \frac{0.0079}{0.04977}$ $= 0.1586$
7 and 8 and 9 and 10			1.1420		

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
9	832	1	-	-									0.7600	0.2062	0.0308	0.0034	0.0003						
		2	-	-																			
		3	-	-																			
		4	-	-																			
		5	0.5710	40													30	8	1				
		6	0.5710	40														30	8	1			
		7	-	-																			
		8	-	-																			
		9	-	-																			
		10	-	-																			
10	352													0.7374	0.2212	0.0365	0.0033	0.004	0.0001				
		1	-	-																			
		2	-	-																			
		3	-	-																			
		4	-	-																			
		5	0.5710	17														13	4				
		6	0.5710	17															13	4			
		7	-	-																			
		8	-	-																			
		9	-	-																			
		10	-	-																			
Total 6-10												57	67	206	420	459	308	208	160	27	2		

COMPUTATION OF FLOOD HYDROGRAPH TORM DATE 1. OCTOBER 1973
TIME INTERVAL 12 HOURS (ALL FLOWS IN CUS

R.O. = 0.084 x Ac x I

Sub-basin No.	Area acre Ac	ERF according to time interval		R.O. generated cusecs	ELEMENTS OF HYDROGRAPH ORDINATES IN CUSECS ACCORDING TO TIME INTERVAL																	
		Interval	ERF(I) inch		11		12		13		14		15		16		17					
					1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23
1	250560				0.9700	0.0291	0.0009															
		1	0.2448	5155	5000	150	5															
	11	2	0.2448	5155		5000	150	5														
		3	1.0872	22900			22200	665	21													
	12	4	1.0872	22900				22200	665	21												
		5	-	-																		
	13	6	-	-																		
2	187117					0.9409	0.0565	0.0025	0.0001													
		1	0.2667	4200		3950	237	11														
		2	0.2667	4200			3950	237	11													
		3	1.1467	18010				16960	1019	45	2											
		4	1.1467	18010					16960	1019	45	2										
		5	0.2559	4020						3880	227	10										
		6	0.2559	4020							3880	227	10									
3	105139						0.9127	0.0821	0.0049	0.0003												
		1	0.2820	2495			2280	204	12	1												
		2	0.2820	2495				2280	204	12	1											
		3	0.9360	8200					7485	673	40	2										
		4	0.9360	8200						7485	673	40	2									
		5	0.3044	2695							2460	221	13	1								
		6	0.3044	2695								2460	221	13	1							
4	62403							0.8851	0.1062	0.0080	0.0005											
		1	0.3961	1743				1542	185	14	1											
		2	0.3961	1743					1542	185	14	1										
		3	0.7196	3165						2800	336	25	2									
		4	0.7196	3165							2800	336	25	2								
		5	-	-																		
		6	-	-																		

Contd.

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
5	20640								0.8583	0.1288	0.0116	0.0008											
		1	0.5078	880					756	113	10	1											
		2	0.5078	880						756	113	10	1										
		3	0.8318	1440							1236	185	17	1									
		4	0.8318	1440								1236	135	17	1								
		5	-	-																			
		6	-	-																			
6	10957									0.8329	0.1499	0.0157	0.0009	0.0001									
		1	0.1348	124						103	19	2	2										
		2	0.1348	124							103	19	2										
		3	0.6253	576								480	86	9	1								
		4	0.6253	576									480	86	9	1							
		5	-	-																			
		6	-	-																			
7	4864										0.8080	0.1697	0.0204	0.0018	0.0001								
		1	1.0591	432							349	73	9	1									
		2	1.0591	432								349	73	9	1								
		3	1.1941	488									395	83	10	1							
		4	1.1941	488										395	83	10	1						
		5	-	-																			
		6	-	-																			
8	2016											0.7835	0.1880	0.0254	0.0025	0.0002							
		1	1.0591	179								140	34	5									
		2	1.0591	179									140	34	5								
		3	1.1941	202										158	38	5	1						
		4	1.1941	202											158	38	5	1					
		5	-	-																			
		6	-	-																			
Total					5000	9100	28822	44104	28860	17107	12309	5799	1695	814	299	55	7	1					
Sub-basin 9-10													56	94	111	27	55	12					
Base flow				909	935	857	883	805	831	753	779	701	727	649	675	598	624	547					
Grand total				909	5935	9957	29705	44909	29691	17860	13088	6500	2478	1557	1085	750	686	560					
Date				10		11		12		13		14		15		16		17					

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
9	832												0.7600	0.2058	0.0308	0.0034	1.003						
		1	1.0591	74									56	15	2	2							
		2	1.0591	74										56	15	2							
		3	1.1941	84											64	17	3						
		4	1.1941	84												64	17	3					
		5	-	-																			
		6	-	-																			
10	352													0.7374	0.2212	0.0365	0.0033	0.0004					
		1	1.0591	31										23	7	1	1						
		2	1.0591	31											23	7	1						
		3	1.1941	35												26	8	1					
		4	1.1941	35													26	8	1				
		5	-	-																			
		6	-	-																			
													56	94	111	97	55	12	1				

R E F E R E N C E S

1. Sherman, L.K.

   " Stream flow from rainfall by unit graph methods" , Engineering News Record, Vol.108, pp.501-505, 1932.

2. Clark, C.O.

   " Storage and Unit hydrograph", Trans. ASCE, Vol.110 pp. 1419-1446, 1945.

3. Chow, V.T.

   " Hand Book of Applied Hydrology" McGraw-Hill, New York, 1964.

4. Nash, J.E.

   " A Unit Hydrograph study with particular reference to British catchments", Min.Proc. Inst.Civ.Engrs. No.17, 1960.

5. Bernard, M.M.

   " An approach to determine stream flow", Trans.ASCE Vol. 100, pp.349-360, 1965.

6. Dooge, J.C.I.

   " A general theory of unit hydrograph" J.of Geoph. Res. Vol.64, No.2 pp.241-256, 1959.

7. O' Kelly, J.J.

   " The employment of unit hydrograph to determine the flows of Irish arterial drainage channels", Proc. Inst. Civ. Engrs. Pt. III Vol.4, p.365, 1955.

8. Laurenson, E.M.

   " A catchment storage model for runoff routing" J.Hydrol., 2, pp.141-163, 1964.

9. Mathur, B.S.

" Runoff hydrograph for uneven spatial distribution of rainfall", Ph.D. Thesis, I.I.T.Delhi, 1972.

10. Mokliak, V.I. and T.A.TChokushkina

" Calculation of flood discharges by means of unit hydrograph" IASH Symposia on Flood and their computations, Leningrad, 1967 (Vol.2).

11. Sharma, S.S.

" Study of infiltration characteristics of south Bihar river catchments" M.E.(W.R.D.) Dissertation, U.O.R., 1969.

12. Wilson, E.M.

" Engineering Hydrology" The Macmillan Press Ltd. London and Bolsingstoke, 1969.

13. Chebotarev, N.P.

" Theory of stream runoff" Translated from Russian into English under Israel Program of Scientific translation. Jerusalem , 1966.

14. Diskin, M.H.

" Transfer function for the analysis of rainfall runoff relations". IASH Symposia on flood and their computations, Leningrad Volume II, 1967.

15. Johnston, D. and Cross, W.P.

" Elements of applied hydrology" , 1949.

16. Linsley, R.K.Jr., Kohler, M.A. and Paulhus, J.L.H.

" Applied Hydrology" , 1949.