APPLICATION OF KINEMATIC WAVE THEORY TO SMALL WATERSHEDS

A THESIS

submitted in fulfilment of the requirements for the award of the degree
of
DOCTOR OF PHILOSOPHY
in
HYDROLOGY

By

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OCTOBER, 1989

TO THE MEMORY
OF
MY FATHER
AND
GRANDFATHER

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled "**APPLICATION OF KINEMATIC WAVE THEORY TO SMALL WATERSHEDS**" in fulfilment of the requirement for the award of the Degree of **DOCTOR OF PHILOSOPHY** submitted in the Department of Hydrology of the University is an authentic record of my own work carried out during a period from May, 1986 to October, 1989 under the supervision of Dr. B. S. Mathur.

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

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20/11/90 — Signature of External Examiners

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

I feel priviledged to express my deep sense of gratitude and sincere regards to Dr. B.S. Mathur, Professor, Department of Hydrology, University of Roorkee, Roorkee, India, for his keen interest, invaluable guidance, and constant encouragement throughout the course of the present study.

I wish to express sincere thanks to Dr. D.K. Srivastava, Professor and Head, Dr. Ranvir Singh, Dr. Deepak Kashyap, Dr. D.C. Singhal, Mr. N.K. Goel and Mr. H. Joshi, Readers, Department of Hydrology, University of Roorkee, Roorkee, for their encouragement and co-operation throughout the study.

I express my gratitude to Dr. V.P. Singh, Professor, Department of Civil Engineering, Louisiana State University, U.S.A. for useful discussions on the subject.

I pay a respectful homage to the memory of my father who unfortunately expired before I could complete my research work. On his intensive I started this research work and I would be ever grateful to him for the support which he extended to me.

I would like to express my deep sense of indebtedness to my mother, mother-in-law, brothers, sisters, and to all my relatives, who have given me their moral support and also for their patience through the period of study.

My deepest regards and gratitude to my father-in-law M. Muzaffar Ali (Ex. Member of Parliament) who inspired me to continue the course of research work. The vacuum caused due to the death of my father was very much filled in by him.

I am greatly grateful to my beloved wife Rokshana, daughter Poly for their full co-operation, understanding and also patiently bearing my inattentiveness towards them and allowed me to spend few years away from them.

I am thankful to my colleagues and friends Dr. Hasna Banu, Prof. A.F.M. Abdur Rahman, Mr. M.M. Rahman, Mr. M.A. Islam, Mr. Saiful Islam, Mr. B.C. Basak, Mr. M. Younus Ali and other friends, for their kind co-operation in all respects.

Token but more valuable financial assistance rendered by UNESCO Regional Office, New Delhi, for my research study is gratefully acknowledged. In this connection, I am very much thankful to Dr. Derkatch, Prof. M.F.H. Chowdhary (Ex. V.C.), Mr. L.A. Mandalia, Dr. M.S. Alam and all the officials of UNESCO for their kind co-operation. Thanks are also due to the University of Dhaka, Dhaka, Bangladesh for granting me a long study leave and an initial token study-grant to pursue the research work.

I am thankful to the staff of the Regional Computer Centre and Department of Hydrology, University of Roorkee, Roorkee, for their courteous and prompt assistance all through my work.

Sincere thanks are due to Research Designs and Standards Organisation, Ministry of Railways, Government of India and Central Soil Salinity Research Institute, Karnal (India), for making available the data for this reseasrch work. Without their co-operation this work could never be taken up.

Careful and neat typing of the thesis by Shri Nirmal Kumar, Rashtriya Commercial Institute, Shantinagar, CBRI colony, Roorkee deserves my acknowledgements.

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4.10. 1989

S Y N O P S I S

In the recent past, in many developing tropical countries a good deal of research has been carried out to solve the problems of 'large basins' whereas not much has been done with regard to the hydrologic problems of small watersheds. Small watersheds do play important roles e.g. a village pond is catered by its own small watershed; in hilly watersheds, the generated runoff causes flash flood, resulting into disruptions of communication lines etc. Therefore, it is necessary to look into these aspects of the hydrologic problems with greater attention.

The hydrologic responses of a small watershed basically depends upon the mechanics of surface runoff which is primarily a nonlinear process. In this thesis, the surface hydrologic behaviour of three small natural hilly watersheds (from 82.0 to 1400.0 hectares) and of one agricultural watershed (1073.0 hectares) have been studied in details by the application of 'Kinematic Wave (KW) Theory'.

During the last two or three decades considerable research work has been carried out by various researchers into different aspects of KW theory application to watersheds of varying physiography. A brief summary of these efforts is presented in Chapter II of this thesis. The details of the KW theory i.e. formulation of mathematical equations and few of their possible solutions which happened to be explicit in nature have been discussed in Chapter

III. Out of many possible solutions, the three fully off-centred, first order, numerical finite difference schemes namely, forward-in-time and backward-in-space (termed as Scheme I), backward-in-time and forward-in-space (Scheme II) and backward-in-time and backward-in-space (Scheme III) have been preferred and used in this work. Computations of the models are performed with the help of a digital computer. The computer code has been written in FORTRAN-IV.

Suitable configuration of distributed parameter models have been suggested for applying the KW theory to compute surface runoff (Chapter - III). The data availability on these watersheds is described in Chapter-IV.

The procedural details for the application of KW theory to the four small watersheds have been described in Chapter-V. The physiographic parameters that have been used are namely, the overland slope and roughness; channel slope and its roughness, side slope, bed width and water depth. A systematic sensitivity analysis of these parameters has also been carried out. The results indicated that the effective overland roughness and the overland slope happened to be the most sensitive parameters. During the identification analysis, the values of effective overland roughness parameter have been worked out for different parts of the wet monsoon period (i.e. July to October). The effective overland roughnesses showed an interesting variation with respect to time in case of all the three natural hilly watersheds. Also, suitable conclusions have been drawn with regard to the applicability of the different types of finite difference computational schemes for the two distinct categories of watersheds.

As an extension, application of these principles to ungauged watersheds have also been tried (Chapter-VI). Possible solutions in the form of nomograms and regression models have been suggested. It was interesting to note that the time of concentration (T_c) for the same watershed did not turn out to be a fixed characteristic. Contrary to belief, it was found that T_c varied with rainfall excess intensity and effective overland roughness. Variations in T_c were studied in depths. Regression equations were found for T_c as a function of area, effective overland roughness, overland slope, channel slope and rainfall excess intensity. These concepts were applied to develop a suitable methodology for using these principles for time-area-concentration (TAC) based models. The response of TAC model was found to be encouraging (Chapter-VI). Appropriate discussion of results and conclusions as arrived at in different parts of the thesis have been summarised in Chapter - VII.

Concludingly, it is remarked that the KW theory is a powerful tool in computing the surface hydrologic responses of small watersheds of tropical regions.

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LIST OF SYMBOLS AND ABBEREVIATIONS

SYMBOL	DECRIPTION
A	Area of the watershed; Channel cross-sectional area.
a_1, a_2, a_n	Constant function
A_o	area of overland flow element
$\bar{A}$	Average of the whole representative area
B	Channel width
C	Wave celerity
$\bar{C}$	Average representative wave celerity
D	Hydraulic depth;rainfall excess duration
d	differential operator; rainfall excess depth
f()	function of
F_r	Froude number
F_o	$= f(F_1, F_2, \ldots F_n)$ or $f(F_i)$
g	Acceleration due to gravity
i	Rainfall intensity; Variable grid number along x-direction
i_e, I	Rainfall excess intensity
I(f), I(τ)	Input function
j	Variable grid number along time
K	Constant function; Kinematic wave number
L	Length of the plane
l	Total length of grid line segment (horizontal and vertical)
L_o	length of the overland flow
L_k, L_c	length of the channel
m_o	Kinematic wave parameter of overland flow

m_k	Kinematic wave parameter of channel flow
m	metre
min	minute
N	Effective overland roughness
n	Manning roghness
n_c, n	Manning roughness of channel flow
N_c	Total number of countour intersections
O	Order of
P	Pressure
Q ,Q(t)	Discharge
q	Lateral inflow per unit length
Q_n	Steady uniform discharge
Q_*	Dimensionless discharge
$\bar{q}$	Average inflow per unit length
R_o	Hydraulic radius
R	Hydraulic radius
S_o	Bed slope; Overland slope
S_f	Frictional slope
S	Channel bed slope
s	Sensitivity
T	Width of the free surface of a channel
t	Time instant
t_*	Dimensionless time
u	x-component of mean velocity
V	Average velocity of flow
v	x-component of velocity of the lateral inflow; y-component of velocity
w	Z-component of velocity

X	Body force along x-direction
x	Distance measured in downstream flow direction
x_o	Distance measured in overland flow
x_c	Distance measured in channel flow
Y	Body force along y-direction
y_o	Mean depth of overland flow
y	Mean depth; component along y direction
y_c	Mean depth of channel flow
z	Component along z-direction
Z	Body force along z-direction; Channel side slope.
ψ	Function
$\psi_1, \psi_2 \cdots \psi_n$	Function
μ	Viscosity
ρ	Mass density
∇	Differential Operator of 1st Order
α_o	Kinematic wave parameter of overland flow
α_k	Kinematic wave parameter of channel flow
Δh	The contour interval
Δx	Space step
Δt	Time step
∂	Partial derivative opereator
θ	The angle between a normal to the contours and the grid
φ	Infiltration index
$\varphi(t)$	Transfer function
∞	Infinity
$\rightarrow$	Tends to

Abbreviations

DRH	Direct Runoff Hydrograph
FEM	Finite Element Method
KW	Kinematic Wave
MSL	Mean Sea Level
PDE	Partial Differential Equation
TAC	Time-Area-Concentration
T_c	Time of Concentration

CHAPTER - I

INTRODUCTION : Objective & Scope

Water is one of the most important natural resources vitally needed for sustenance of life. The use of water is continuously increasing with the increase in population. This increased demand for water has forced the planners, scientists and hydrologists not only to concentrate on major 'basin planning' but also to pay sufficient attention to small watersheds. The watershed planning may help in micro-level water budgeting to meet the localised demands of water for small population concentrations scattered over the entire basin. In India, in majority cases, each village has its own pond (or tank) from which the water is drawn for domestic and other needs. These ponds have their own watersheds from which the rain water is fed to them. Prior to onset of monsoon, after the sustained dry period, most of the ponds either get dried up or are left with insignificant quantities of water. The fine silt or clay which gets accumulated at the bottom of the tank is removed by the local people for plastering of huts, and other general repairs of the houses. Thus, the lost capacity of the pond, due to accumulation of silt, is recouped. Though it may appear quite insignificant, yet the role of small watersheds is very important in most of the developing tropical countries in general and in the Indian subcontinent in particular. The micro-climatic influences onto the small watersheds is of particular significance. Response of a watershed to small, medium, heavy and very heavy rainfall leading to near cloud burst conditions is of importance not only for meeting the water demands but also for the protection of life and property. In hilly and mountainous catchments the situations may get worse. Heavy intensity rains and resulting runoffs may cause flash floods and landslides and thus disrupt the communication lines of vital importance. These disruptions may result into major economic problems as one region gets completely cutoff from the rest of the country.

To find suitable solutions to these problems, it is necessary that the micro-level in-depth studies be carried out for computations of watershed responses to vastly varying meteorologic conditions (i.e. from mild intensity rains to the cloud burst conditions). There are different types of watershed models which have been developed in the past to estimate the peak flows and runoff hydrographs for small watersheds. With the advent of powerful high speed digital computers, it is now possible to study through mathematical models the basic physical processes viz. transformation of rain into the runoff and also the movement of water on the surface as well as in channels. These models rely on mathematical statements to represent the system.

In a small watershed, the generation of runoff can better be studied through the 'overland phase' and 'channel phase'. The conversion of rainfall excess into surface runoff and then the movement of flood flows in the channel is recognised as a nonlinear process. These days, this process is generally modelled through two approaches, viz. the hydrodynamic approaches and the system's approach. For small watersheds, the hydrodynamic approaches have been found more useful for the transformation of rainfall excess into the surface runff (viz. in the overland phase) and the movement of flood flows in the channel (viz. in the channel phase).

Various determistic approaches developed and used by the researchers in the past have been reviewed in this work. It has been found that hydrodynamic approaches have got the specific advantage that these can take care of the geo-physical conditions and are expectedly better suited for the prevailing tropical conditions of South-Asian region. These approaches have popularly been studied through the 'Kinematic Wave (KW) Theory' (Lighthill and Whitham, 1955), and the 'Dynamic Wave theory'. In small watersheds, surface runoffs are dominated by the overland flows and the KW models have been found to give good results. Therefore, for this study the KW models have been used to analyse the problems of small watersheds. The KW models and some of their applications have been discussed in details (Chapter - II).

The kinematic wave models have been preferred because these mathematical models use the simpler form of St. Venant wave models (or dynamic wave models). In KW models local and convective accelerations are neglected and the water surface slope is taken nearly equal to channel bed slope. These models are mathematical as well as deterministic in nature. These can be used to account for lumped, distributed and time-varient aspects. These models are much simpler and can easily be incorporated into a watershed geometry. Further, these models can be coupled with other process models to investigate the effects of land use changes, temporal and spatial variations in rainfall as well as in watershed conditions, and pollutant washoffs. The KW models perform better as these are based on the physics of the runoff process (Stephension and Meadows, 1986). Most significantly, these models do account for the nonlinearity of the process without unduly complicating the solution procedures which otherwise became unmanageable. Further, for both the components i.e. the overland and the channel flow routing, the KW models have been found to be accurate and efficient (Overton and Meadows, 1976). By using KW models, it is possible to find out velocities of flows and thus the corresponding discharges at different stages of the overland as well as channel flows. Further, compared to other mathematical models, the data requirements for the KW models is quite moderate and thus this can be calibrated with limited data.

Keeping in mind the above advantages, KW models have been used in the present work to analyse the problems of small watersheds identified by the Ministry of Railways, Government of India and Central Soil Salinity Research Institute (CSSRI), Karnal, India. Thus, the present work has been undertaken to explore the possibility of application of KW theory to two distinct categories of typical problem ridden watersheds. One category belongs to small hilly natural watersheds and the other of a flat alkaline agricultural watershed under going reclamation.

The three fully off-centred first order explicit finite difference schemes have been employed for the solutions of the KW equations. Details of these schemes have been discussed (Chapter - III).

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

(a) To develop suitable mathematical models based on KW theory, best suited to the typical problems of some small watersheds (i.e. natural high sloped watersheds and agricultural watershed under going reclamation).

(b) To confirm the applicability of finite difference schemes which are most suitable for the two types of watersheds under study.

(c) To carry out the sensitivity analysis for the physiographic parameters of overland as well as channel flow elements and to identify the parameters which are most sensitive. Further, to find out the role of effective overland roughnesses in the context of natural high sloped watersheds.

(d) To explore the effects of inputs on time of concentrations and their influences on 'time-area-concentration' (TAC) based models.

A close look of KW model applications reveals that the KW theory has successfully been applied for the rainfall-runoff relationships of small and medium sized natural and urban watersheds having moderate slopes with more-or-less uniform physiographic conditions. In the context of the watersheds of tropical countries, the physiographic characteristics (viz. the roughness) undergo appreciable changes during the wet monsoon months.

Thus, the present work has been undertaken to see the applicability of the KW theory onto mountainous as well as nearly flat watersheds in tropical conditions. These investigations will, therefore, enable the hydrologists to apply this tool for solving the complex rainfall-runoff relationships of small watersheds.

The subject matter of the present study is arranged in the following seven chapters.

In the current chapter entitled **INTRODUCTION**, the problem is introduced, the objectives and scope of the present work are also outlined.

The Chapter II entitled **REVIEW OF LITERATURE**, deals with terminologies, concepts, previous studies of KW models by the researchers and their applications, the basic equations of KW models and various finite difference methods (which are used in KW equations) are discussed in details.

The Chapter III is named as "**MODEL DEVELOPMENT**". Here KW theory has been discussed. Solution of KW equations through fully off-centred finite difference schemes adopted for this study are described in details.

The Chapter IV entitled **WATERSHEDS UNDER STUDY AND AVAILABILITY OF DATA**, deals the description of the four small watersheds with respect to their locations, areas, soil types, topography etc. for the rainfalls and corresponding runoffs.

The Chapter V on "**MODEL APPLICATION TO GAUGED SMALL WATERSHEDS**" presents the details of application of the KW models to four small gauged natural watersheds. In all thirtyone storm events, whose data were available have been used.

The Chapter VI on **EXTENSION OF PROPOSED CONCEPTS TO UNGAUGED WATERSHED**, is devoted for extension of the proposed concepts to ungauged watersheds. The concept of variable time of concentration (T_c) has been investigated and its application is made through time-area-concentration (TAC) based model.

Finally, the Chapter VII is devoted to "**CONCLUSIONS**". In this chapter a summary of results is also presented along with the conclusions and suggestions for future development and extension of this work.

CHAPTER - II

REVIEW OF LITERATURE

2.1 INTRODUCTION

The previous chapter, a brief description of the statement of the problem, its scope and the objectives were presented. In this chapter, the concepts, terminologies, kinematic waves vs. dynamic waves, basic equations of kinematic wave models and their applications, different types of solution techniques which are used in the kinematic wave equations, review of some kinematic hydrologic model developments will be introduced.

2.2 THE HYDROLOGIC SYSTEM

"The word hydrology is derived from the Greek words hydro, meaning water, and logos, meaning science. In this broad sense hydrology is concerned with all water on the earth, its occurrence, distribution and circulation, its physical and chemical properties, its effects on the environment and on life of all forms" (Raudkivi, 1979).

The hydrologic cycle is a simple link to the general circulation of water from the oceans or seas to the atmosphere, to the ground, and back to the oceans or seas. It can be summarized graphically as shown in figure 2.1.

A hydrologic system "is a set of physical, chemical and/or biological processes acting upon an input variable or variables, to convert it (them) into an output variable (or variables)"

(Clarke, 1973). In the natural process, generally the input function is the hyetograph and the transfer (or system) function is a function of representative catchment action on the hyetograph. The response or output function is the runoff hydrograph which are observed at the outlet as shown in figure 2.2.

According to Dooge (1973), hydrologic problems are treated through system approach. He had identified the problems under two main categories. These are : (i) hydrologic analysis, and (ii) hydrologic synthesis. The hydrologic analysis is again classified as three types, identification, forecasting and detection. The various categories of hydrologic problems so formed are given in Table 2.1.

TABLE 2.1 : Identification of Surface Hydrologic Problems (After Dooge, 1973).

Type of Problem in Surface Hydrology		Input (Hyetograph) I(t)	System Function Ø(t)	Response (DRH) Q(t)
A. Surface Hydrologic Analysis Problems	(1) Identification Problem	✓	?	✓
	(2) Forecasting Problem	✓	✓	?
	(3) Detection Problem	?	✓	✓
B. System Synthesis (Simulation)		✓	?	✓

It is clear (Figure 2.2) that the system function (or transfer function) describes the nature of the system and the physical laws governing its operation.

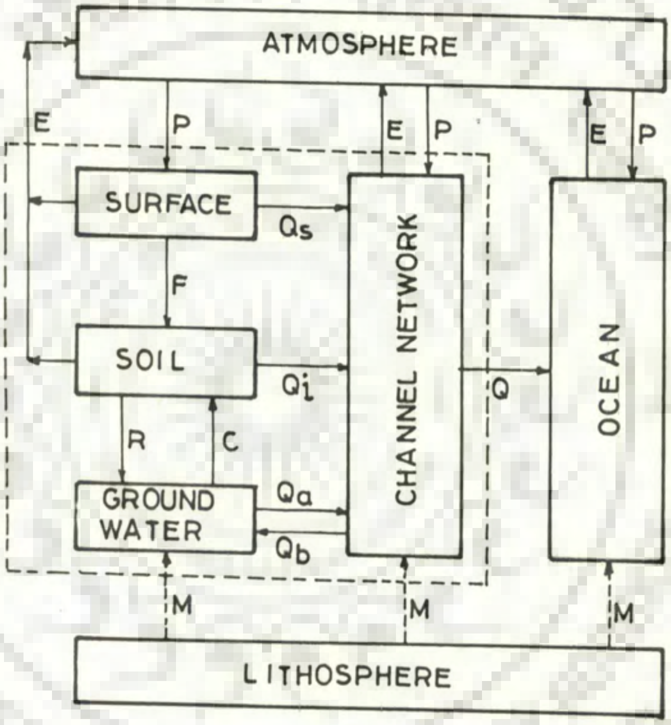


FIG. 2.1 THE HYDROLOGIC CYCLE IN SYSTEMS NOTATIONS (AFTER HALL, 1984 (MODIFIED FROM DOOGE, 1973))

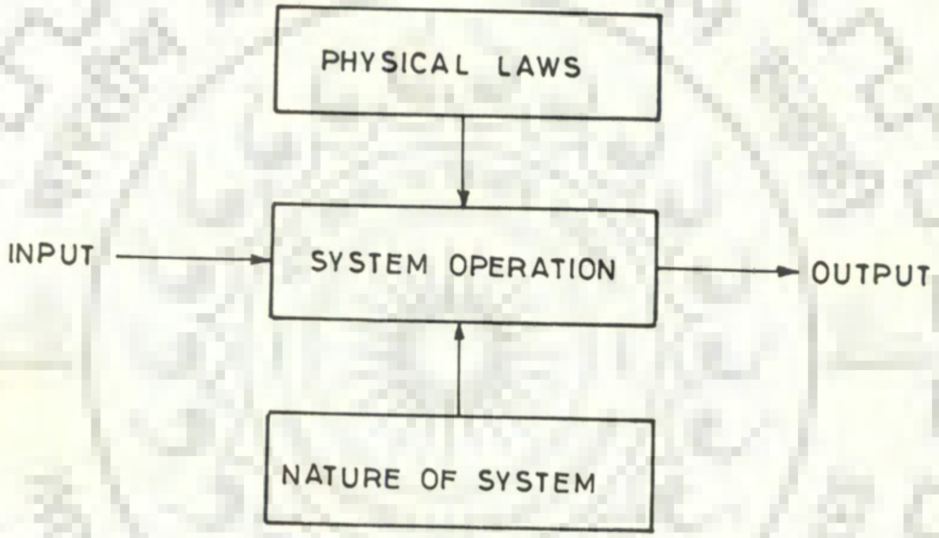


FIG. 2.2 THE CONCEPT OF SYSTEM OPERATION
(AFTER DOOGE 1973)

2.3 NATURE OF HYDROLOGIC SYSTEM

The nature of the hydrologic system is completely defined, through one property each, mentioned in the following three sets of possible system behaviours.

(1) Linear or nonlinear

(2) Lumped or distributed

(3) Time-invariant or time-variant

A brief description of these three characteristics is given in the following subsections.

2.3.1 The Linear and Nonlinear Systems

A hydrologic system is said to be linear, if a step input fed to the system produces an output which is directly proportional to the input. A linear system may be represented mathematically by a linear equation. The principle of superposition applies to it may be defined mathematically as under :

$$f(\psi_1)_t + f(\psi_2)_t + \ldots\ldots + f(\psi_n)_t = f(\psi_1 + \psi_2 + \ldots\ldots + \psi_n)_t \quad \ldots \quad (2.1)$$

Where ψ is any response function and t is the time of its applicability.

The principle of homogeneity which is a particular case of principle of superposition, thus may be described as below

$$Kf(\psi)_t = f(K\psi)_t \quad \ldots \quad \ldots \quad (2.2)$$

where K is a constant function.

A nonlinear system is represented by a nonlinear function. The extent of nonlinearity depends upon the system itself.

The hydrologic system may be defined by a general type of differential equation and can be written as follows :

$$f(\psi) = a_n \frac{d^n\psi}{dt^n} + a_{n-1} \frac{d^{n-1}\psi}{dt^{n-1}} + \ldots + a_1 \frac{d\psi}{dt} + a_o\psi \quad \ldots \qquad (2.3)$$

The system would be nonlinear if any of these coefficients a_o, a_1, a_2, ... a_n etc. are the function of ψ or the function ψ carries an exponent other than unity.

2.3.2 The Lumped and Distributed Systems

The hydrologic system is said to be lumped if its input functions or parameters do not vary with respect to spatial coordinates (Figure 2.3(a)). For the lumped systems average conditions or values of input and parameters are applicable. Thus, lumped systems are represented by ordinary differential equations.

The system is defined as distributed if the input or the transfer function and other parameters do vary with the spatial coordinates (Figure 2.3(b)). Such systems are mathematically represented by the partial differential equations. The theoretical solution of such systems (a differential equation) thus requires complete knowledge of the boundary conditions.

2.3.3 Time-Invariant and Time-Variant Systems

A time-variant system is one in which the input-output

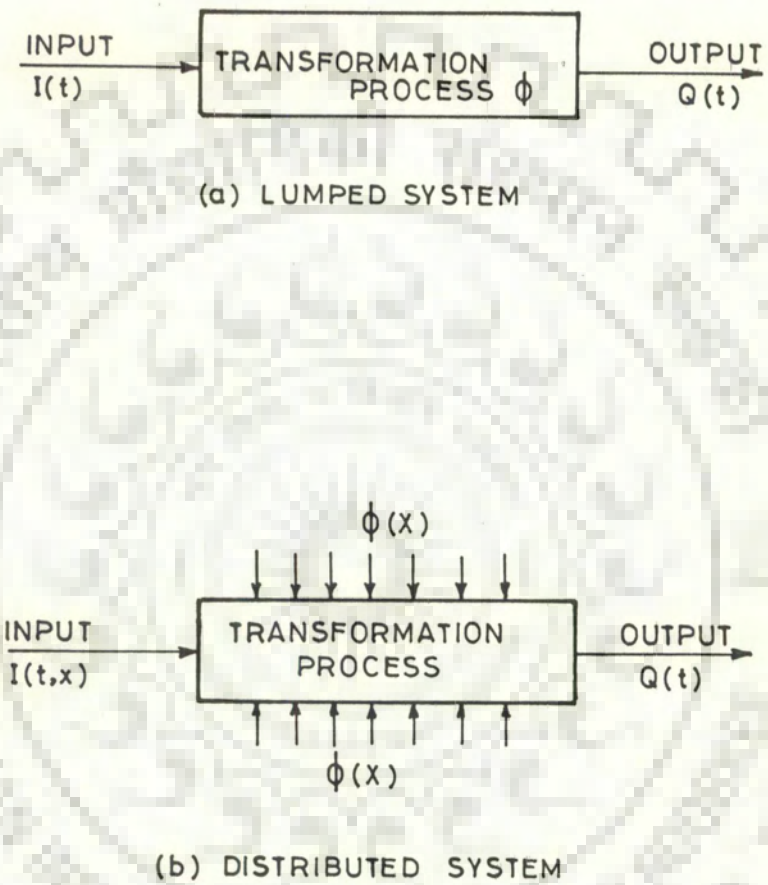


FIG. 2.3 THE LUMPED AND DISTRIBUTED SYSTEMS

relationship is not dependent upon the time at which the input is applied to the system. The concept of time-invarianting makes the analysis simpler.

A time-variant system is one in which the input-output relationship is a dependent function of time. It may be concluded that a lumped, linear and time-invariant system is easiest to work with. But in surface hydrology, the hydrologic system, happens to be distributed, nonlinear and time-variant in its behaviour. This behaviour of the system is the most complex one and quite difficult not only to formulate mathematically but also to solve. Therefore, a compromise has to be reached so that the complicated natural hydrologic system may be solved satisfactorily by making suitable assumptions with respect to the aspects stated above.

2.4 PRESENT TRENDS IN HYDROLOGIC STUDIES

In a broad sense, the hydrologic studies can be described through the following two approaches :

(i) deterministic approaches, and (ii) stochastic approaches. If all variables and parameters are considered free from random variations the approaches are referred to as deterministic approaches (Clarke, 1973). On the other hand, if random variables are taken care of, in terms of their distributions as well as their probabilities of occurrences, the approaches are defined as stochastic. Since the present study deals with the deterministic aspects, therefore, in the following section emphasis is given on the basic concepts and other relating approaches dealing with deterministic aspect only.

2.4.1 Conceptual Models and Mathematical Models

"A model is a simplified representation of a complex system. It simulates some but not all the characteristics of the system" (Singh, 1988).

Conceptual models rely heavily on theory to interpret phenomena rather than to represent the physical process. These models have been evolved in surface hydrology which simulate the catchment behaviour through conceptual element e.g. linear reservoirs, nonlinear reservoirs, linear channels and also through their combinations. Some of the conceptual models which are found more useful for application to small watersheds are :

Clark Model (1945); Nash Models (1957, 1960); Singh Model (1962); Dooge Model (1959); Mathur Model (1972); Pedersen et al. Model (1980) etc.

Most of the conceptual models are directly or indirectly related to the theories of 'Unit Hydrograph' and 'Instantaneous Unit Hydrograph'.

"A mathematical model is a simplified representation of a complex system in which the behaviour of the system is represented by a set of equations, perhaps together with logical statements, expressing relations between variables and parameters" (Clarke, 1973).

Complex watershed problems may be analysed by using a suitable mathematical technique known as "Mathematical model". The model adopted may be much simpler than the actual system.

Such a mathematical model rely on mathematical statements to represent the system. These models are generally more useful in hydrologic studies when compared with physical models. The relationship between watershed response and its parameters can be studied suitably with the help of mathematical models. It is quite difficult to draw a clear line between the conceptual and mathematical models. It has rightly been stated that "any mathematical model formulated to represent a process or phenomenon will be conceptual to some extent and the reliability of the model will be based upon the extent to which it can be or has been verified" (Overton and Meadows, 1976).

A mathematical model is generally developed through a four step process. These steps are : (i) an examination of the physial problem, (2) replacement of the physical problem by an equivalent mathematical problem, (3) solution of the mathematical problem with the accepted techniques of mathematics, and (4) interpreting the mathematical results in terms of the physical problem (Freeze, 1978; Haberman, 1977). Dynamic models and their simplified forms of kinematic models are the examples of this category. Since the present study is devoted to kinematic wave models, therefore, the literature relating to the development of kinematic wave model and its application to the hydrologic problem are reviewed in more details in the following section.

2.5 KINEMATIC WAVES Vs. DYNAMIC WAVES

Kinematics is defined as the study of motion without the influences of mass and force, whereas, dynamics is defined as the study of motion, in which these influences (mass and force) are included. Initially both kinds of waves phenomena are present in flood waves i.e. the dynamic waves as well as the kinematic waves (KW). But certain characteristics of a watershed can make kinematic waves a dominant characteristic of that flood event (DeVries and MacArthur, 1979).

Generally dynamic waves characterise the movement of long waves in shallow water, like a large flood wave in a wide river when inertial and pressure forces are important. However, if the inertia and pressure forces are not so important, kinematic waves govern the flows. Therefore, in case of KW, the force in the direction of the channel axis due to the weight of the fluid flowing down owing to gravity, is approximately balanced by the resistive forces of channel bed friction. The friction force is representatively taken care of by the Manning's roughness equation in most cases. In such cases, kinematic flows do not change significantly and the flow remains approximately uniform along the channel. To an observer on the bank of the river, no visible surface wave will be noticeable and the passage of the flood wave will be seen as an apparently uniform rise and fall on the water surface elevation over a relatively long period of time (Figure 2.4). So, kinematic flows are also sometimes called as uniform unsteady flows (DeVries et al. 1979).

Dynamic waves normally have much higher velocities and attenuate quickly than kinematic waves. Therefore, flood wave travel can better be defined through KW phenomenon rather than dynamic wave concept.

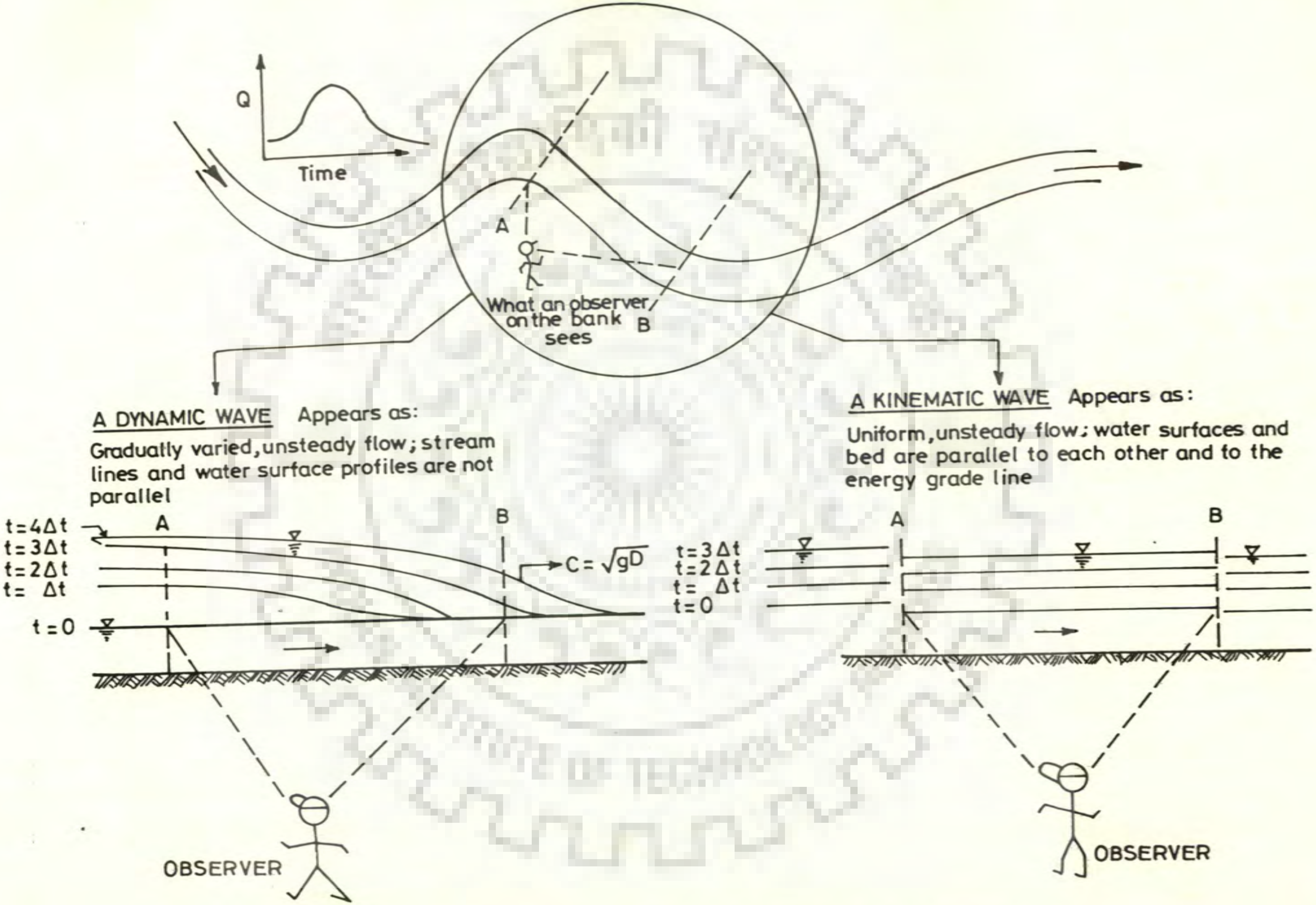


FIG. 2.4 VISUALIZATION OF DYNAMIC AND KINEMATIC WAVES
(AFTER DeVRIES AND MacARTHUR, 1979)

In a shallow open channel, the speed of small gravity waves (or wave speed) is often called as wave celerity and is denoted by C. The value of C is equal to $\sqrt{gD}$ (i.e. $C = \sqrt{gD}$) where D is the hydraulic depth and equals to A/T; A being the cross-sectional area and T is the width of the free surface of a channel (Chow, 1959). In the case of overland flow, the hydraulic depth D is represented by mean depth y_o.

Dominance of kinematic or dynamic wave is to be ascertained through the Froude's number. The Froude number (F_r) is defined as the ratio of the fluid speed (the mean cross-sectional velocity) to the wave celerity. Therefore, it also represents the ratio of inertial forces to gravity forces (i.e. $F_r = V/\sqrt{gD}$), where V is the mean velocity.

From the analysis, Lighthill and Whitham (1955) found that when the average velocity of flow V is greater than twice the speed of the wave relative to the water, depth of flow will continue to increase and a surge or bore will be developed. It is further shown that when Froude number is 2, kinematic waves dominate over dynamic waves. However found, for $F_r < 2$ the dynamic waves are damped.

Generally, the kinematic wave approximation would be good when $F_r > 1$ (supercritical) because the waves could not move upstream (since $V > \sqrt{gy_o}$ or $\sqrt{gD}$). Therefore, kinematic waves ultimately prevail the flow characteristics occurring for overland flows and small watershed channel flow when $F_r \leq 2$.

Thus KW theory is one of the most important methods for estimating stormwater runoff rates and their volumes. Though it is a simplified form of dynamic wave approach, yet it has proved itself to be an efficient method for simulating stormwater runoffs from small watersheds (Overton and Meadows, 1976).

Formulation of KW equations, assumptions involved in this theory, its applicability alongwith their solution techniques, all are discussed in the following sections.

2.6 KINEMATIC WAVE EQUATIONS FROM ST. VENANT EQUATIONS

The equation of continuity and the equation of momentum for gradually varied, unsteady, one dimensional, incompersssible flows were developed by Franch mathematician De Saint Venant in 1871 (Rovey et al., 1977). These two equations are quasi-linear, hyperbolic, partial differential equations. In its one dimensional form, these equations describe the changes in stream flow in the vertical and in the longitudinal directions.

The St. Venant equations characterising the dynamic flow can be written as :

$$\text{Continuity :} \quad \frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q + (i - \varnothing) \quad \ldots \quad (2.4)$$

$$\text{Momentum :} \quad \frac{\partial u}{\partial t} + u \frac{\partial u}{\partial x} + g \frac{\partial y_o}{\partial x} = g(S_o - S_f) - q \frac{u-v}{A} \quad (2.5)$$

where :

A = Cross-sectional area of flow;

Q = Discharge of the channel;

g = Acceleration due to gravity;

y_o = Mean depth;

x = Distance measured in downstream flow direction;

t = Time (seconds);

q = Lateral inflow rate;

u = X-component of mean velocity;

i = Rainfall intensity;

$\emptyset$ = infiltration rate;

S_o = Average bottom slope;

S_f = Frictional slope defined by the Manning's equation

and v = X-component of velocity for lateral inflow (assumed that v is negligible to the total momentum balance for channel routing and equal to zero).

In the continuity equations (2.4) the terms are respectively known as :

$\frac{\partial A}{\partial t}$ = Rate of rise term;

$\frac{\partial Q}{\partial x}$ = Storage term;

q = Lateral inflow rate and

$(i-\emptyset)$ = Intensity of excess rainfall.

In the momentum equation (2.5) various terms are successively known as :

$\frac{\partial u}{\partial t}$ = Acceleration term;

$\frac{\partial u}{\partial x}$ = Velocity head term;

$\frac{\partial y_o}{\partial x}$ = Depth taper term;

S_o-S_f = Bed slope minus friction term and

$q \frac{(u-v)}{A}$ = Lateral inflow term (This term is very small with respect to other terms and therefore neglected).

The equation (2.5) may be rewritten in the following form for a ready reference to the various types of wave models that are recognized.

Term	I		II		III		IV	
Equation of motion	$\frac{1}{g}\frac{\partial u}{\partial t}$	+	$\frac{u}{g}\frac{\partial u}{\partial x}$	+	$\frac{\partial y_o}{\partial x}$	+	$(S_f - S_o)$	$= 0$ ... (2.6)
	Local acceleration		Convective acceleration		Depth slope		(Friction slope - Bed slope)	

Wave model and terms used to describe it are :

(1) Kinematic wave only term IV = 0

(2) Diffusion wave III + IV = 0

(3) Steady dynamic wave II+III+IV=0

(4) Dynamic wave I+II+III + IV=0 ... (2.7)

(5) Gravity wave I + II + III = 0

and other terms are neglected.

By making suitable assumptions the KW equations can be deduced from the dynamic form of equations (Equations 2.4 and 2.5) given by St. Venant and these can be applied to channel flows.

2.6.1 General Assumptions and Kinematic Wave Equations

In order to apply the St. Venant equations to channel flows, the following general assumptions are needed.

(1) Flow is primarily one dimensional. It is considered in the longitudinal direction of flow. Therefore, the depth and the velocity variations are considered only in the longitudinal direction. Flow characteristics in the direction perpendicular to the direction of flow are, therefore, neglected.

(2) The velocity distribution is considered constant and the water surface is horizontal across any section perpendicular to the longitudinal flow axis.

(3) The water surface profile varies gradually with hydrostatic pressure prevailing at all points in the flow such that all vertical accelerations within the water column can be neglected.

(4) Resistance to flow can be approximated by steady flow formulae such as the Manning's equation.

(5) Momentum transferred to the fluid flow from lateral inflows is negligible.

(6) The longitudinal axis of the flow channel can be approximated by a straight line, therefore, no lateral secondary circulations occur.

(7) The channel boundaries may be treated as fixed non-eroding and non-aggrading, and

(8) The flow is incompressible and homogeneous in nature.

In most cases of overland and channel flows, the terms pertaining to the changes in momentum and acceleration can be neglected. In fact, this amounts to assuming that the energy line is parallel to the bed of channel. That is $S_f = S_o$. Thus the resulting simplified (in Equation 2.5) equations are the 'Kinematic Equations' and these characterise the unsteady uniform flow conditions. The equations for friction gradient can be written in the form as the area of flow A, and discharge Q, as:

$$Q = \alpha A^m \quad \ldots \qquad (2.8)$$

where α and m are kinematic wave routing parameters which are directly related to the watershed and flow characteristics.

Equations (2.4) and (2.8) together are known as the kinematic wave equations.

2.7 APPLICABILITY OF KINEMATIC WAVE APPROXIMATIONS

The applicability of kinematic wave approximations under different conditions is to be discussed in this section. By using Henderson's (1966) approach the momentum equation (2.5) can be normalized for a steady uniform discharge Q_n. The normalized form of the equation may be written as :

$$Q = Q_n \left[1 - \frac{1}{S_o}\left(\frac{\partial y_o}{\partial x} + \frac{u}{g}\frac{\partial u}{\partial x} + \frac{1}{g}\frac{\partial u}{\partial t} + \frac{qu}{gA}\right)\right]^{1/2} \qquad (2.9)$$

If the sum of the terms to the right of the minus sign is much less than one (i.e. inertia, local inflow, and pressure terms are relatively small compared to S_o), then :

$$Q \approx Q_n \quad \ldots \quad (2.10)$$

This suggests that unsteady flows may be approximated by a uniform flow formula e.g. Manning's equation which is generally used for turbulent flows, can also be used in this case.

In the kinematic wave approximation, the discharge (or flow rate) is expressed as a unique function of depth of flow. This suggests that the normal flows of the nature may be described by a unique depth-discharge relationship (Overton and Meadows, 1976; DeVries et al., 1979; Stephension et al., 1986) which provides a simple tool for calculating flows from stormwater runoffs. Ponce et al. (1978) proposed that most of the overland flow problems can be modelled through kinematic flows. For this purpose the use of a linear stability analysis of the St. Venant equations to examine the applicability of the kinematic and diffusion models in open channel flows was suggested.

The characteristics of rising hydrographs for a large variety of flow conditions have been studied by Woolhiser and Liggett (1967). They found that the dynamic components in equation (2.5) will be damped enough to be neglected if dimensionless kinematic flow number (K) is greater than 10. The dimensionless kinematic flow number (K) is defined as

$$K = \frac{S_o L}{y_o F_r^2} \quad \ldots \quad (2.11)$$

$$\text{and} \quad F_r = V / \sqrt{g y_o} \quad \ldots \quad (2.12)$$

where V is the mean flow velocity (= Q/A), F_r is Froude's number and L is the length.

The parameter K is now frequently used to test the applicability of kinematic wave equations to the overland flow modelling. The kinematic wave approximation gives very good results if $K > 20$, reasonable if $K > 10$, and poor if $K < 10$. It was also concluded that the kinematic wave approximation may be used instead of the full St. Venant (dynamic wave) equations if $K > 20$ and $F_r \geq 0.5$. Overton and Meadows (1976) recommended that kinematic wave approximation may be used only for $K > 10$, regardless of the Froude number value. Morris and Woolhiser (1980) re-evaluated and concluded that for the low values of Froude number the kinematic wave approximation can be used if $F_r^2 K \geq 5$. This conclusion is compatible with the condition $K > 20$ for $F_r \geq 0.5$ given by Woolhiser and Liggett (1967). Later on, Vieira (1983) made a general conclusion that the kinematic wave solution is a good approximation for St. Venant equation if $K > 50$ regardless the Froude number value.

Results published by Woolhiser and Ligget (1967) are reproduced in figure 2.5. Here t_* is dimensionless time (= tv/L) and the dimensionless discharge Q_* equals qL/Vg. It is also reported that the maximum error in the outflow hydrograph for $K = 10$ is of the order of 10 per cent and it decreases rapidly as K increases. The kinematic wave solution and the general solution are found to coincide as $K \to \infty$.

Concludingly, it may be remarked that for surface hydrologic estimates the kinematic wave approximations can be used with advantage for $K > 10$ (DeVries et al. 1979). Some of the solution techniques for solving the kinematic wave equations are discussed in the following sections.

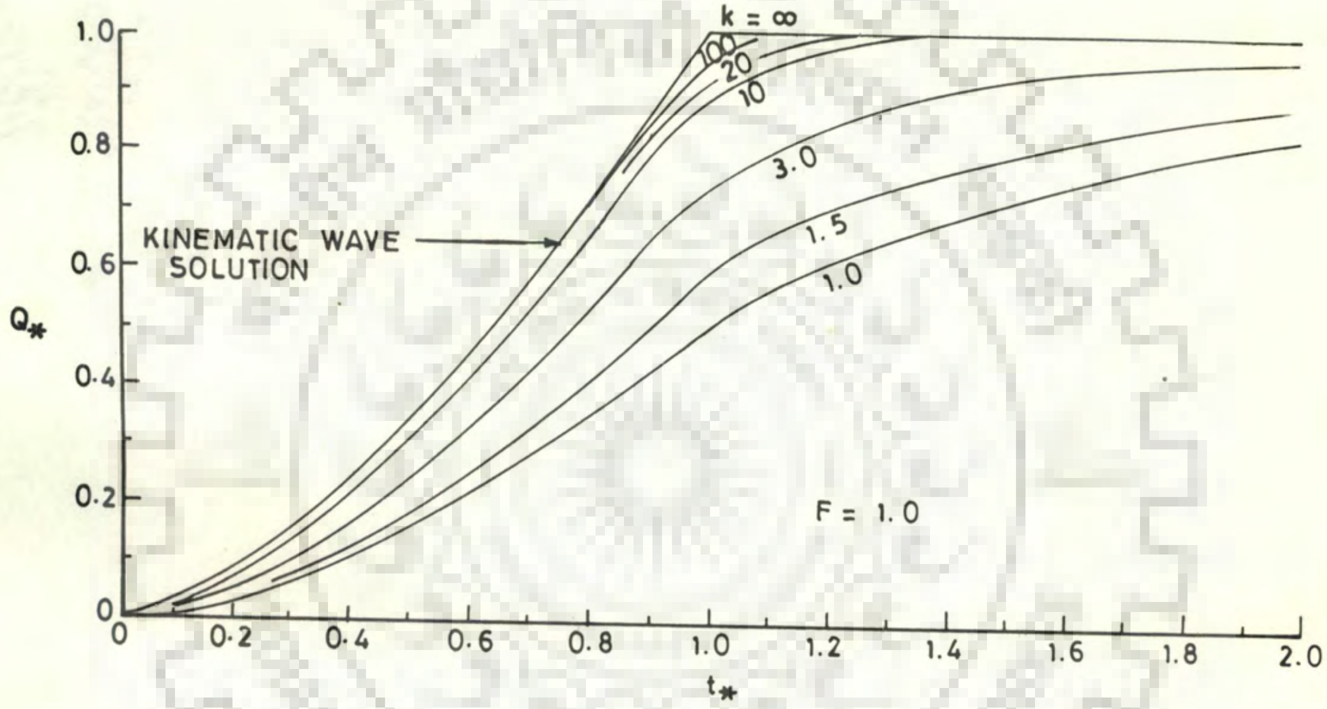


FIG. 2. 5 THE RISING HYDROGRAPH - VARIATION WITH k, FOR F = 1
(AFTER LIGGETT AND WOOLHISER, 1967)

2.8 MATHEMATICAL TECHNIQUES FOR THE SOLUTION OF KINEMATIC WAVE EQUATIONS

In general, many techniques are now available for solving the kinematic wave equations. These may be classified as :

(1) Analytical solution techniques (simplified form), and

(2) Numerical solution techniques.

A brief discussion of these techniques is presented in the following sections.

2.8.1 Analytical Solution Techniques

Analytical methods of solving partial differential equations are usually restricted to linear cases with simple geometric and boundary conditions. By these methods the solution of St. Venant equations and Kinematic wave equations have been obtained only for some simplified cases. Graphical solutions are in vogue since a long time. During the last two decades, the development of modern digital computers has opened new horizons and has given rise to the sophisticated numerical techniques.

A number of researchers have developed exact as well as approximate analytical solutions for the kinematic flow approximations to compare the runoff from planes of different types and forms (Wooding, 1965(a); Parlange et al., 1981; Rose et al., 1983; Campbell et al., 1984; Moore, 1985; Moore et al., 1987).

However, numerical techniques are more rational when compared to the exact and approximate analytical solutions.

The techniques enable researchers to incorporate more parameters if accuracy of solution so demands.

2.8.2 Numerical Solution Techniques

Availability of powerful digital computers has enabled the researchers to seek the solutions of complicated partial differential equations (linear or nonlinear) for different boundary conditions which may be applicable. Methodology used is termed as "Numerical Methods". These are the algorithms that use only arithmatic operations and also certain logical operations such as algebraic comparison.

Thus the formulation of a well posed numerical model requires both the derivation of a set of difference equations based on the mathematical model and a demonstration of an adequacy of these difference equations. The latter involves an analysis of the consistency, convergency and stability of the numerical model (Noye, 1982) which are discussed in the current section.

The definition of consistency, convergency and stability are presented here as under :

Consistency :

The difference equations and auxiliary conditions of the numerical model must be consistent with the differential equations and initial and boundary conditions of the mathematical model. The numerical model is said to be consistent, if the truncation error, that is, the discrepancy (difference) between the finite difference approximations and the continuous derivatives, tends to zero, as the grid spacings get smaller and smaller. The term truncation error refers to the error which is introduced in approximating the solution of a mathematical problem by a numerical method.

Convergence :

The solution to the finite difference equation must be convergent to the solution of the differential equation as the grid spacings get smaller and smaller or tend to attain zero values. That is, the difference between the exact solutions of the numerical and mathematical models should vanish as the grid spacings tends to zero. In defining convergence a term 'discretization error' is often used. The discretization error refers to the departure of the finite difference approximation from the solution of 'Partial Differential Equation' (PDE) at any grid point. There are two important concepts closely associated with the convergence of a particular finite difference procedure, namely, consistency and stability. Consistency is a necessary condition for convergence.

Stability :

This is the third important feature to which a finite difference method of solving a PDE must satisfy. In a repetitive solution step by step the stability refers to the growth (or decay) of errors produced in the finite difference solution by the errors of the previous steps.

Round-off error :

Computers can solve only a finite number of digits to represent each number at each step of a calculation. Hence round-off errors are incorporated. The computation is stable if the growth of these errors are within reasonable limits or controlled.

A numerical model with consistent equations, convergent solutions and stable error propagation forms a computationally stable scheme and gives results which are quite close to the exact solution.

In principle, the method of finite differences can be applied on to nonlinear mathematical models, but consistency, stability and convergence are more difficult to prove (Noye, 1982).

There are a large number of numerical techniques for solving St. Venant and kinematic wave equations. Each one of these has its own specific advantages in terms of convergence, stability, consistency accuracy and efficiency. These techniques can be classified as under :

(1) Characteristics methods

(2) Finite differences methods

  (a) Explicit methods

  (b) Implicit methods

(3) Finite element methods

A brief description of these methods is given in the following sections.

2.8.2.1 Characteristics methods

In the method of characteristics the two simultaneous partial differential equations (e.g. continuity and momentum) are replaced by four ordinary differential equations. This approach was known to hydrologists for quite sometime. The graphical solutions of method of characteristics were in use much before computer era began. Now, the characteristics equations are solved numerically through digital computers. In this method the flow characteristics through a set of 'Characteristics Curves' and the solution is sought along another 'Characteristic Curves'.

The characteristics method is considered to be a standard with which the other methods are compared or judged. This method is stable and accurate. Accuracy is checked through conservation of mass at different times and comparing the solutions with different point spacing (Liggett et al., 1967).

Recently more elaborate methods of characteristics have been developed. Four-point method of characteristics has been developed by Abbott and Verway (1970) which is a powerful tool.

2.8.2.2 Finite difference methods

The finite difference methods can be used to solve the complicated mathematical models which may consist of partial differential equations. The solutions may use either an explicit or an implicit finite difference schemes. Both the schemes, have been used in solving the stream flow routing equations. These schemes may have their own limitations with respect to the stability for the allowable size of the grid spacing. Some of the basic features of the two schemes are described in the followng sections.

2.8. 2.2.1 Explicit finite difference methods

The term explicit refers to those difference methods that advance the solution point by point from one time line to the next. This method utilizes initial values and upstream boundary information and seeks the solution for the remaining grid points one at a time. This way, the solution proceed is sought for all the points until the entire solution domain is solved. To ensure stability, the step sizes Δt(time) and Δx (space) must satisfy the Courant condition

$$\Delta t \leq \frac{\Delta x}{|V \pm C|} \quad \text{throughout the computation space.}$$

This puts a limitation on the size of the time step Δt. The methodology continues to be as popular in the present day as it used to be in the past.

Explicit finite difference schemes are widely used for the solution of the one-dimensional St. Venant equation as well as for solving the kinematic wave equations. Some of the popular schemes used under this technique are named below

(a) Unstable method
(b) Diffusing method
(c) Leap-frog method
(d) Lax Wendroff method

and (e) Fully off-centered schemes

Explicit finite difference schemes are simple to apply. By using a fixed regular grid and it is easier to follow the variation of the flow properties along the overland as well as channel flows. These methods are found to be accurate as well as economical when used properly. Kibler and Woolhiser (1970) found that for the study of overland flows explicit finite difference scheme with second order accuracy, happen to be the most satisfactory one. Applications some of the above mentioned schemes was reviewed by Liggett and Woolhiser (1967). Constantinides (1982) employed various schemes for solving the one dimensional KW equations in order to suggest the suitability under the prevailing conditions. He found that backward-central explicit finite difference scheme yielded extremely accurate results. Singh (1979) reported that a single step, second order, explicit Lax-Wendroff scheme was more convenient and efficient. He also established the superiority for solving KW equations of this scheme over the other explicit schemes in general.

Fully off-centered schemes for solving the kinematic wave equations were also developed and used (Li et al., 1975; Huang, 1978; DeVries et al., 1979; HEC-1, 1981 and 1985; Ponce, 1986). These gave very satisfactory results. The detailed study of stability and convergence of these schemes have been reported by Ponce et al. (1979). These schemes are easy to understand, efficient and easy to apply to the rural as well as urban hydrologic problems.

2.8.2.2.2 Implicit finite difference methods

Implicit finite differene methods utilize initial value and the upper and the lower boundary information for giving a solution to the unknown grid points at the next time level simultaneously. In this method, the resulting algebraic equations are generally non-linear and the dependent variables occur implicitly.

In general, implicit finite difference methods are unconditionally stable. This implies computational stability for any size of the increments in time Δt and space Δx.

The main advantage of this methodology is that the ratio of space to the time interval, $\Delta x/\Delta t$ is not governed by any stability criteria. However, the size of Δt and Δx are still governed by the accuracy desired. The use of very large values of Δx and Δt have led to poor results because of involvement of unacceptably large truncation errors.

Among the various implicit schemes reported in literature, the four point scheme proposed by Strelkoff (1970) and Amien et al., (1970, 1975) is the most popular. Also a six point implicit scheme was suggested by Liggett and Woolhiser (1967) and Abbott et al. (1967).

Explicit and implicit schemes have been used by Wylie (1970) for unsteady free-surface flow computations. Brakensick (1967a) applied three types of finite difference schemes for solving the kinematic wave equations describing the flood wave travel. It was concluded that a four point implicit scheme having centre on the two upper points, gave the most satisfactory results.

2.8.3 Finite Element Method (FEM)

The finite element method is a recent approach being applied to solve the partial differential equations that govern hydraulic processes. In the finite element method, the system is divided into a number of elements and the partial differential equations describing the phenomenon are integrated at the nodal points of the elements.

For solving the St. Venant and Kinematic wave equations only a few researchers have used the FEM. The finite element methodology is quite regorous and tidious method. Cooley et al. (1976), King (1977), and Stephension et al. (1986) has used this technique for flood routing in channels and natural streams.

Various researchers have use the technology of the KW theory for surface hydrologic computations of watersheds. A brief literature review of the various models developed on this line is presented in the following section .

2.9 REVIEW OF KINEMATIC HYDROLOGIC MODEL DEVELOPMENTS

In the previous sections the KW theory and the various techniques for solving the KW equations alongwith their implications (i.e. stability, convergence etc.) have been discussed. In order to review the research carried out on these lines, a detailed survey of literature is presented in this section.

Very good bibliography of work on flood routing with references upto 1960 is available (Yevjevich, 1960). The researches having their direct relevance to prsent work will be reviweed in this section. Various investigators have studied the one dimensional gradually varied unsteady flows. The solutions so obtained reveal that the proposed simplifications of complete momentum equation and the continuity equation are quite accurate. Keulegan (1945) is one of the pioneer researchers who applied sound physical principles to overland flows. He derived the continuity and momentum equations and introduced several approximations to the dynamic equations known St. Venant equations by neglecting various terms of minor significance. Thus he arrived at a set of simplified equations which are now termed as the 'Kinematic Wave Equations'. These equations are quite adequate for 'Overland flow' and 'Channel flow routing' of small watersheds.

An in-depth study of continuity and momentum equations through KW theory was taken by Lighthill and Whitham (1955). They considered propagation of flood waves in rivers as mainly Kinematic (a balance of bed slope and friction slope). They developed kinematic wave equations for overland flows through general treatment of kinematic wave theory and showed that, when the inflow is constant (or a function of distance only) an explicit solution can be obtained. They suggested that the solution has been found by numerical integration along the characteristics when the lateral inflow is a function of time and distance. They also investigated kinematic shock waves. They found that for $F_r \leq 2$, the dynamic components decay exponentially and the kinematic waves ultimately dominate the flows.

In 1955, Iwagaki worked on the problems of steep rivers by using the methods of characteristics. He used the kinematic assumptions implicitly in his analysis of unsteady flows in steep channels. Henderson (1963) described the conditions under which flows in a prismatic channel without lateral components could be classified as kinematic. He developed the criteria for kinematic flows. It was stated that the partial derivative terms appearing in the equation of momentum must be neglected in comparison to bed slope (i.e. the flow must be essentially uniform). Henderson and Wooding (1964) had applied the kinematic wave techniques to the flow over a sloping plane.

Wooding (1965(a), 1965(b), 1966) applied kinematic wave theory to the overland and channel components of the runoff. He considered a hypothetical V-shaped watershed formed by two rectangular planes (i.e. open book system) discharging into a straight channel formed by them in between. He concluded that kinematic wave theory is applicable to gradually varied unsteady flows if the Froude number is less than 2. This way the runoff mechanics was studied through the kinematic wave theory to get the response of the elementary watershed system. By taking nearly the same geometrical representations and based on kinematic wave theory, other researchers also made their valuable contributions (Harley et al., 1970; Smith and Woolhiser, 1971a, 1971b; Lane et al., 1975; and Rovey et al., 1977).

Morgali and Linsley (1965) used the finite difference methods for solving the shallow water equations as applicable to compute runoffs from natural watersheds. Schaake (1965) used numerical methods for computing runoffs from urban watersheds; whereas Brakensick (1966) and Brakensick et al. (1966) also used the same approach for computing runoffs from the rural watersheds. Chen and Hansen (1966) used the dimensionless forms of the continuity and momentum equations and tested the flow profile characteristics and discharge-depth relationships with distance and time as parameters. A generalization of the catchment stream model has also been found by Eagleson (1967).

Woolhiser and Liggett (1967) have reported in details about the applicability of kinematic wave models under different conditions. They have analysed the rising hydrograph (unsteady and one dimensional flow over a plane) of overland flow on a single plane. They used both the shallow water equations and kinematic approximations in their dimensionless forms. They also examined the accuracy of the kinematic wave approximation and have found it to be very good if the dimensionless parameter K (known as Kinematic number) for planes is greater than 20; reasonabe if greater than 10, and poor if its value is below 10. Their results showed that for K=10, about 10 per cent error would be introduced by neglecting the dynamic terms from the solution. The error decreases if K increases and coincides with the general solution when $K \rightarrow \infty$ (Figure 2.5). Later on, Morris and Woolhiser (1980) and Woolhiser (1981) have investigated in more details the applicability of kinematic wave equations under different conditions. They showed that the additional criterion $S_o L/y_o > 5$ is also required along with other criteria (where S_o is the bed slope and L is the length). Kibler (1968) had also used kinematic wave equations for overland flow computations and discussed the optimization techniques.

Brakensick (1967a) first introduced the concept of the kinematic cascade models. He transformed an upland watershed into a cascade of planes discharging into a single channel. Later on kinematic cascade models have been used by other researchers too (Kibler and Woolhiser, 1970; Lane and Woolhiser, 1977; Croley et al., 1981; Jayaseelan, 1984). Kinematic cascade models continue to remain popular for the computation of overland flows and channel flow routings of small watersheds.

Brakensick (1966, 1967b) has used kinematic flood routing methods and has described the properties of the kinematic techniques in the context of hydraulic and hydrologic flood routings. Overton and Brakensick (1970) have also applied this approximation for a V-shaped configuration. They calculated a lag time based on catchment dimensions, roughness, and rainfall rates etc. The relationship between lag time and rainfall rate agreed with observed data for several storm events on experimental catchments. Their sensitivity analysis showed the solution more sensitive to errors in rainfall rather than to errors physiographic parameter estimations.

Schaake (1970), applied the kinematic wave models by splitting the watershed into a number of segments over which the model parameters were assumed to be uniform. Based upon geometric characteristics and assumed type of flow for a segment, he presented a technique to compute the kinematic parameters. By applying the kinematic equations Morgali (1970) had also examined the experimental laminar and turbulent overland flow hydrographs. More detailed studies of kinematic wave equations and their applications to different watersheds have been reported by Mahmood and Yevjevich (1975); Stephension (1981), Stephension et al. (1986); Overton and Meadows (1976); Raudkivi (1979);

Eagleson (1970), Haan et al. (1982); Kirkby (1978).

For water routing, use of a nonlinear kinematic wave approximation has also been reported by some researchers (Singh, 1974; Li et al., 1975; Singh et al., 1976; Huang, 1978; Ponce, 1986).

Woolhiser (1969) had applied kinematic flow equations on an inverted cone-shaped surface having a specified degree of convergence at the apex. Subsequently, this direction of research was further explored in more depths and details. Worth mentioning are the efforts of Singh and his co-researchers (Singh, 1975a, 1975b, 1975c, 1975d, 1975e, 1975f, 1976a, 1976b, 1976c, 1976d, 1976e, 1976f, 1977, 1978 and 1979; Sherman and Singh, 1976a, 1976b; Shelburne and Singh 1976, Singh and Shelburne 1977) and diverging section (Singh and Agiralioglu 1981a and 1981b).

Kinematic wave models have also been applied to simulate runoffs from agricultural watersheds (Woolhiser et al., 1970; Langford and Turner, 1973). It was further expanded to irrigation systems as well. Chen (1966) applied it to solve the hydrologic problems of irrigated lands comprising of porous beds. This work was critically discussed by Woolhiser (1970). Later on, this technique was used for solving problems of surface irrigation system by other researchers (Cunge and Woolhiser, 1975; Sherman and Singh, 1978, 1982; Singh and Sherman 1983; Singh and Ram 1983).

Many researchers worked for the kinematic wave theory applications to watershed hydrology (Ponce et al., 1978; Singh, 1978; Brutsaert, 1968; Morris et al., 1980; Borah et al., 1980 and 1982; Beven, 1979, 1982 and 1985; Weinmann et al., 1979; Cundy et al., 1985; Vieira, 1983; Blandford et al., 1983; Singh and Agiralioglu, 1981; Woolhiser, 1982; Field, 1982; Fread, 1982). Allent (1984) worked on kinematic wave theory and compared his findings with Clarks unit hydrograph theory.

Further more, a good number of authors have applied the kinematic wave theory to water flows and solute transport in porous media. Movement of water flow through an isothermal snow layer has been studied by Colbeck (1972). The theory was also applied to drainage (Sisson et al., 1980); infiltration and soil water movement (Beven, 1982; Smith and Hebbert, 1983; Beven and Germann, 1985); soil moisture and solute transport in porous media (Charbeneau, 1984); infiltration into soils with sorbing macropores (German and Beven, 1985; Germann, 1985); water flow in soil macropores and subsurface storm flow (Germann and Beven, 1986; Germann et al., 1986, Germann et al., 1987); moisture flux in the unsaturated zone (Bengtsson, 1988) etc.

In more recent applications of kinematic wave theory, it has been extended to some of the applied hydrologic problems viz. urbanization and its effect on storm runoff (Stephension, 1983); dam-break problem (Hunt, 1982); estimation of time of concentration (Ragan et al., 1972; Singh 1976; Singh and Agiralioglu, 1980; Akan, 1986); peak runoff estimates (Akan, 1985); effect of storm rainfall intensity (Woolhiser et al., 1988); unsaturated infiltration (Yamada et al., 1988); stochastic infiltration (Rayej et al., 1988); infiltrating parabolic shaped surfaces (De Lima et al., 1988); concentration of flow (Takasao et al., 1988); KW routing (Hromadka et al., 1988); dimensionless hydrographs (Constantinides and Stephension, 1982); urban drainage networks (Smith, 1983; Green, 1984; NIH, 1989); dynamic storms (Stephension, 1984a); detention storage (Stephension, 1984b); estimation of time of concentration (Agiralioglu, 1984 and 1988); estimation of overland roughness (Liong et al., 1989); rainfall-runoff modeling (Singh, 1988, 1989) etc.

Suitable KW models capable of meeting the objectives of the study are described in details in the next chapter.

CHAPTER - III

MODEL DEVELOPMENT

3.1 INTRODUCTION

In the previous chapter, basic concepts about kinematic wave (KW) equations and their solution techniques were discussed. Also, some of the important hydrologic models which were quite relevant to the theme of present study were briefed. In this work, it is proposed to study the mechanics of the runoff of small watersheds and the resulting surface runoff. This can better be studied by developing suitable models capable of meeting the objectives of the study.

A hydrologic model is an important tool for estimating and organizing quantitative hydrologic information. The main objectives for the development of a suitable surface hydrologic model are to study the movement of overland, (i.e. through its surface runff) as well as stream flow components of the hydrologic cycle.

It will not be out of place to mention that all the natural processes are very complicated. There are many factors influencing the runoff process which are widely responsible for the complex nature of rainfall-runoff relationships. There are different aspects of the hydrologic transformation process which have been widely investigated in order to get an insight into the complexities involved. But till now, no perfect methodologies or models are available for their universal applications to predict the runoff characteristics of small tropical watersheds having typical conditions of rainfall input and other controlling factors. However, Scientists, Hydrologists and Engineers are trying to find out the methods which will be the best suited to the small watershed under different natural conditions.

To achieve this objective various techniques, and available models were studied (Chapter - II). It was concluded that the dynamic approaches are the best to account for the physical processes associated with the runoff mechanics of the watersheds. Among these approaches (dynamic), the kinematic wave theory is the best suited to the prevailing conditions and also keeping in view the availability of data on the watersheds which are currently being investigated in this thesis.

Thus, the present study is aimed at developing mathematical models based on kinematic wave theory. As an initial step towards it, development of KW equations as derived from the hydrodynamic equations, is taken up first.

3.2 HYDRODYNAMIC THEORY AND KINEMATIC WAVE EQUATIONS

The hydrodynamic theory for incompressible fluid flows gives the following set of equations (also known as the Navier-Stokes' equations):

$$\rho\left(\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} + v\frac{\partial u}{\partial y} + w\frac{\partial u}{\partial z}\right) = X - \frac{\partial P}{\partial x} + \mu\nabla^2 u \qquad \ldots (3.1)$$

$$\rho\left(\frac{\partial v}{\partial t} + u\frac{\partial v}{\partial x} + v\frac{\partial v}{\partial y} + w\frac{\partial v}{\partial z}\right) = Y - \frac{\partial P}{\partial y} + \mu\nabla^2 v \qquad \ldots (3.2)$$

$$\rho\left(\frac{\partial w}{\partial t} + u\frac{\partial w}{\partial x} + v\frac{\partial w}{\partial y} + w\frac{\partial w}{\partial z}\right) = Z - \frac{\partial P}{\partial z} + \mu\nabla^2 w \qquad \ldots (3.3)$$

and continuity equation : $\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0$ ... (3.4)

where $\nabla^2 \equiv \frac{\partial^2}{\partial x^2} + \frac{\partial^2}{\partial y^2} + \frac{\partial^2}{\partial z^2}$;

ρ = the mass density; u, v, w are the velocity Components in the x, y, z directions respectively;
X, Y, Z are the body forces per unit volume;
P = pressure, and μ = viscosity.

The above four equations describe theoretically the fluid flow in any situation. However, in hydrology as well as in hydraulic engineering, viscous forces of the above equations may be replaced by turbulent momentum transfer or by a semi-empirical drag equation e.g. by Manning or Darcy equations as the case may be.

Keeping in view the difficulties involved in the application of these equations for the flow of water in a channel, the following one dimensional hydrodynamic equations were suggested by St. Venant (1871):

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \quad .. (3.5)$$

and $$\frac{1}{g}\frac{\partial u}{\partial t} + \frac{u}{g}\frac{\partial u}{\partial x} + \frac{\partial y_o}{\partial x} + S_f - S_o = 0 \quad ... (3.6)$$

where Q = flow rate; t = time (seconds); y_o=mean depth;
x = distance in down stream flow direction;
q = lateral inflows per unit length.
u = x-component of mean velocity; S_o = bed slope;
S_f = energy slope, and g = acceleration due to gravity.

A close inspection reveals that the St. Venant equation are similar in many respects to the Navier-Stokes' equations. For hydrologic problems, the exact solution of the St. Venant equations is also quite difficult. Now-a-days, these equations can be solved in their various forms with the help of powerful digital computers. Finite difference solution of these equations in its simplified form is also possible through micro-computers.

Another important and simplified form of St. Venant equations are the kinematic wave equations (Section 2.6). In the present situation, the derivatives of the energy and velocity terms in the momentum equation (3.6) are very small in comparison with gravity and frictional forces. This allows to assume that the bed slope is approximately equal to the friction slope (i.e. $S_o = S_f$). Under the above conditions and if there is no appreciable back water effect, for all x and t, the discharge can be described as the function of area of flow only and can be written as under :

$$Q = \alpha A^m \qquad \ldots \qquad (3.7)$$

Where α and m are known as kinematic wave routing parameters which are directly related to the watershed and flow characteristics. Equations (3.5) and (3.7) when taken together, are termed as 'Kinematic Wave (KW)' equations.

Thus, the general form of the KW equation can be written as under :

$$\frac{\partial A}{\partial t} + \alpha\, m\, A^{m-1} \frac{\partial A}{\partial x} = q \qquad \ldots (3.8)$$

provided α and m are independent variables.

In the past, the KW equations have already been successfully applied to the problems related to overland flows, small watersheds, slow-rising flood waves etc. It was found that under certain circumstances of overland flows as well as the channel and conduit flows the KW equations are much easy to work with. The KW equations are now recognised as a powerful tool which takes into account the physics of the flows on the surface, as well as in the channels of small watersheds.

3.3 ELEMENTS USED IN KINEMATIC WAVE MODELS

In this work, for computational purposes, the following two types of elements have been identified :

(i) Overland flow elements and

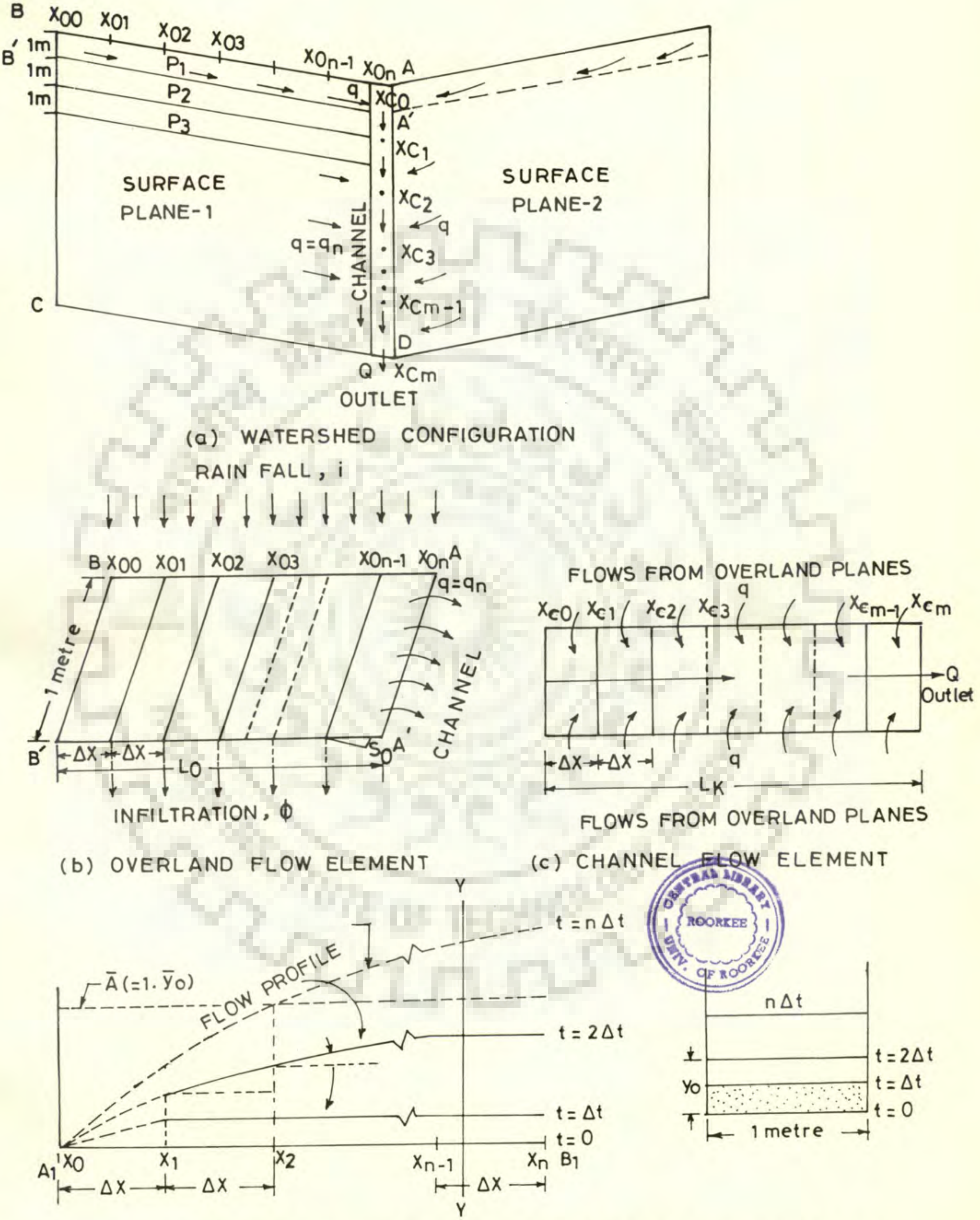
(ii) Channel flow elements (Figure 3.1)

Overview of simplified arrangement of the above mentioned two elements, in reference to a small watershed, is shown in figure 3.1(a), (b) & (c).

The one dimensional unsteady, nonlinear, uniform, deterministic, time variant first order partial differential for kinematic wave equations are presented in Section 3.2. Development of the KW equations for the overland and the channel flow elements is discussed in the following sections.

3.4 KINEMATIC WAVE EQUATION FOR THE OVERLAND FLOWS

It is assumed that the overland flow is in the form of sheet flow. A unit width of the plane has been considered for the computational aspects of the runoff generation. The KW equations can, therefore, be written as :



(a) WATERSHED CONFIGURATION

(b) OVERLAND FLOW ELEMENT

(c) CHANNEL FLOW ELEMENT

(d) FLOW PROFILE ON THE OVERLAND / CHANNEL FLOW ELEMENT

FIG. 3.1 GENERATION OF FLOW PROFILE

$$\frac{\partial y_o}{\partial t} + \frac{\partial q}{\partial x} = i_e = i - \emptyset \qquad \text{... (3.9)}$$

$$q = \alpha_o y_o^{m_o} \qquad \text{... (3.10)}$$

where α_o and m_o are kinematic wave routing parameters which are directly related to conveyance of particular surface (i.e. to the slope and its roughness);

q is discharge per unit width of overland flow;

y_o is the mean depth and i_e is the rainfall excess intensity [precipitation (i) - infiltration (Ø)].

3.4.1 Determination of Kinematic Wave Routing Parameters α_o and m_o for the Overland Flows

The overland flow on a wide plane has been considered as a sheet flow (very shallow flows). Therefore, the kinematic wave equations for the overland flow segments have been derived from equation (3.10) along with the Manning's equation. The steady state Manning's equation for discharge per unit width(q), can be written as :

$$q = \frac{1}{n} A_o R_o^{2/3} S_o^{1/2} \qquad \text{... (3.11)}$$

where n = Manning's roughness coefficient of overland flow

A_o = area of cross-section of water;

R_o = the hydraulic radius, and

S_o = average slope of overland flow element.

For a sheet flow on a plane of unit width A_o and R_o can be replaced by y_o i.e. the mean depth of flow (since $A_o = 1.y_o$ and $R_o = y_o.1/1$).

Substituting their values in equation (3.11) the discharge per unit width (q) can be wriitten as :

$$q = \frac{1}{n} y_o (y_o)^{2/3} S_o^{1/2}$$

$$\text{or } q \simeq \frac{1}{N} y_o^{5/3} S_o^{1/2} \qquad \ldots \qquad (3.12)$$

where the Manning roughness coefficient has been replaced by an appropriate coefficient N which is called here the 'Effective Roughness' parameter of overland flow. The parameter N is generally greater than the Manning's 'n' and it is a characteristic parameter of a watershed (DeVries et al. 1979).

Comparing of equations (3.10) and (3.12) reveals that

$$\alpha_o = \frac{1}{N} S_o^{1/2} \qquad \ldots \qquad (3.13)$$

$$\text{and } m_o = 5/3 \qquad \ldots \qquad (3.14)$$

The parameters N and S_o being known, the values of α_o is worked out from equation (3.13).

3.4.2 The Final Form of Kinematic Wave Equations for the Overland Flows :

Combining equations (3.9) & (3.10) and substituing m_o given in equation (3.14), the following complete form of kinematic wave equation for overland flow is derived as :

$$\frac{\partial y_o}{\partial t} + \frac{\partial(\alpha_o y_o^{5/3})}{\partial x} = i_e$$

$$\text{or } \frac{\partial y_o}{\partial t} + 5/3\alpha_o \, y_o^{5/3-1} \frac{\partial y_o}{\partial x} = i_e \qquad \ldots \qquad (3.15)$$

In equation (3.15), y_o is only dependent variable which is a function of x,t and rainfall excess intensity (i_e). Thus y_o can be determined explicitly by using equation (3.15). From computed values of y_o, the overland surface runoff per unit width (q) can be computed by using equation (3.10).

3.5 KINEMATIIC WAVE EQUATIONS FOR THE CHANNEL FLOWS

For the channel flows, the kinematic wave equations can be written as follows :

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \qquad \ldots \quad (3.16)$$

$$Q = \alpha_k A^{m_k} \qquad \ldots \quad (3.17)$$

where A = Cross sectional area of the channel; Q = discharge ;
q = lateral inflow per unit length of the channel; α_k and m_k are the kinematic wave routing parameters which are directly related to the watershed and the channel flow characteriistics (a particular channel cross-sectional shape, channel slope and roughness).

In order to apply the equations (3.16) and (3.17), it is necessary to know the values of the parameters α_k and m_k for the known channel physiography.

3.5.1 Determinaton of Kinematic Wave Routing Parameters α_k and m_k for Channel Flows

In channel flows, parameters α_k and m_k are different for different shapes of the cross sections. Their values are also dependent on the Manning roughness coefficient (n) and its slope (S). The cross-section of a natural channel can be approximated by the following geometric shapes.

(i) Trapezoidal, (ii) Rectangular, and (iii) Triangular.

Since, most of the natural channels nearly conform to trapezoidal shapes. Therefore, the same has been adopted in this work for the computation of the parameters α_k and m_k. This is an added advantage. The other two cross-sections are particular cases of the trapezoidal section under certain conditions as discussed next.

3.5.2 Trapezoidal Channel Cross Section

A trapezoidal cross-section is the most general type of channel cross-section. It is defined by the channel side slope (Z), and the channel bottom width (B) (Figure 3.2). The modifications of this section may result into the other two type of shapes mentioned earlier. For example, if the channel side slope is zero, it conforms to a rectangular section, whereas if its channel bottom width is zero, it becomes a triangular section. For the trapezoidal geometric shapes, it is not possible to derive a single simple relationship for determining α_k and m_k explicitly.

An indirect approach is adopted. The Manning's equation and KW equation are employed together to compute the values of parameters α_k and m_k.

The Manning's equation for discharge (Q) in a channel is given by

$$Q = \frac{1}{n} S^{1/2} AR^{2/3} \qquad \ldots \quad (3.18)$$

For the trapezoidal channel cross-section, equation (3.18) can be written as below :

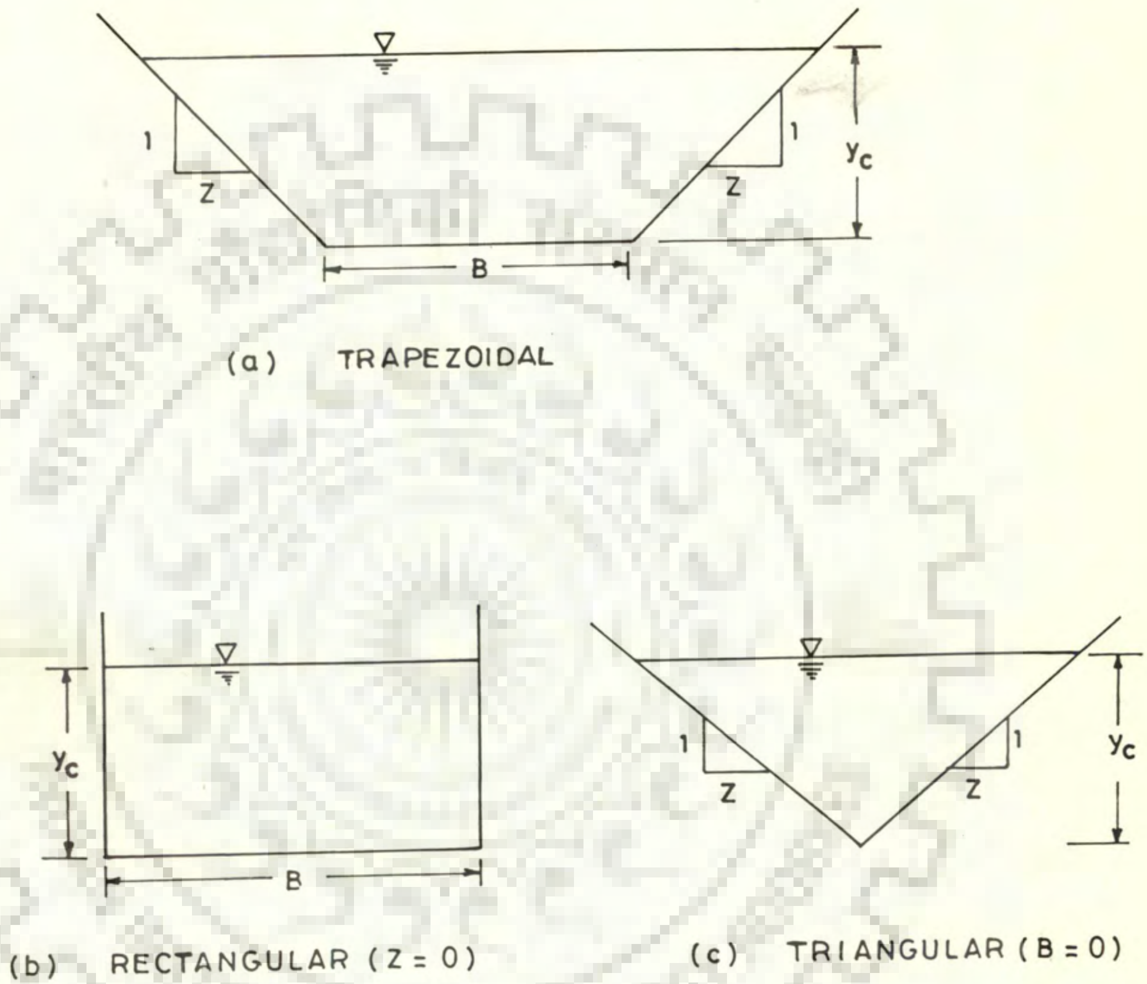


FIG. 3.2 CHANNEL SHAPES FOR KW CHANNEL ROUTING

$$Q = \frac{S^{1/2}}{n} (A_C)^{5/3} \left[\frac{1}{B + 2y_c \sqrt{(1 + Z^2)}}\right]^{2/3} \quad \ldots \quad (3.19)$$

where A_C = the area of the effective channel cross-section at depth y_c.

The KW equation for channel flow is given by (equation 3.17) and reproduced as under :

$$Q = \alpha_k A^{m_k} \quad \ldots \quad (3.20)$$

From equation (3.19) for two appropriate values of depths (y_c), (minimum, average or maximum depths may be adopted) corresponding Q values are obtained. By substituting them in equation (3.20), the two unknowns α_k and m_k are thus computed. It has been found that the values of m_k varies from 4/3 for a triangular channel cross-section to 5/3 for a wide rectangular channel cross-section. For the known parameters (α_k and m_k) the final form of KW equation for unknown discharge function Q can now be found out as discussed next.

3.5.3 The Final Form of Kinematic Wave Equations for the Channel Flows :

The unknown parameters for the channel shapes under consideration i.e. α_k and m_k being the unknown functions. The KW equatiion for the channel flow can be written by combining equations (3.16) and (3.17) as given below:

$$\frac{\partial A}{\partial t} + \frac{\partial(\alpha_k A^{m_k})}{\partial x} = q \quad \ldots \quad (3.21)$$

If α_k is independent of m_k, then the equation (3.21) becomes:

$$\frac{\partial A}{\partial t} + m_k \alpha_k A^{(m_k - 1)} \frac{\partial A}{\partial x} = q \qquad \ldots \quad (3.22)$$

Equation (3.22) is thus considered the final form of kinematic wave equation for channel flows. In this equation A is only the dependent variable whereas x, t and q are independent variables. If A is found out from equation (3.22) and is substituted in equation (3.17), the discharge (Q) from the channel is computed.

Appropriate numerical techniques are needed for solving the KW equations given for the overland flows as well as the channel flows.

3.6 NUMERICAL TECHNIQUE (FINITE DIFFERENCE METHODS) FOR SOLVING THE KINEMATIIC WAVE EQUATIONS

Different types of numerical solution techniques of kinematic wave equations have been discussed in Section 2.8 (Chapter - II). The fully off-centred, first order, explicit numerical finite diifference techniques (schemes) have been preferred for solving the KW equations characterising the flow of water in its overland and channel flow phases. Therefore, application of this technique in the context of this work has been discussed in details.

For solving the kinematic wave equations for the overland as well as channel components, the flow domain is characterised by x(for spatial coordinates) and t (for time coordinates). For the application of finite difference schemes to a section of the flow domain, space-time (x-t) plane is shown in figure 3.3. In this plane, the lines parallel to the x-axis (abscissa) are

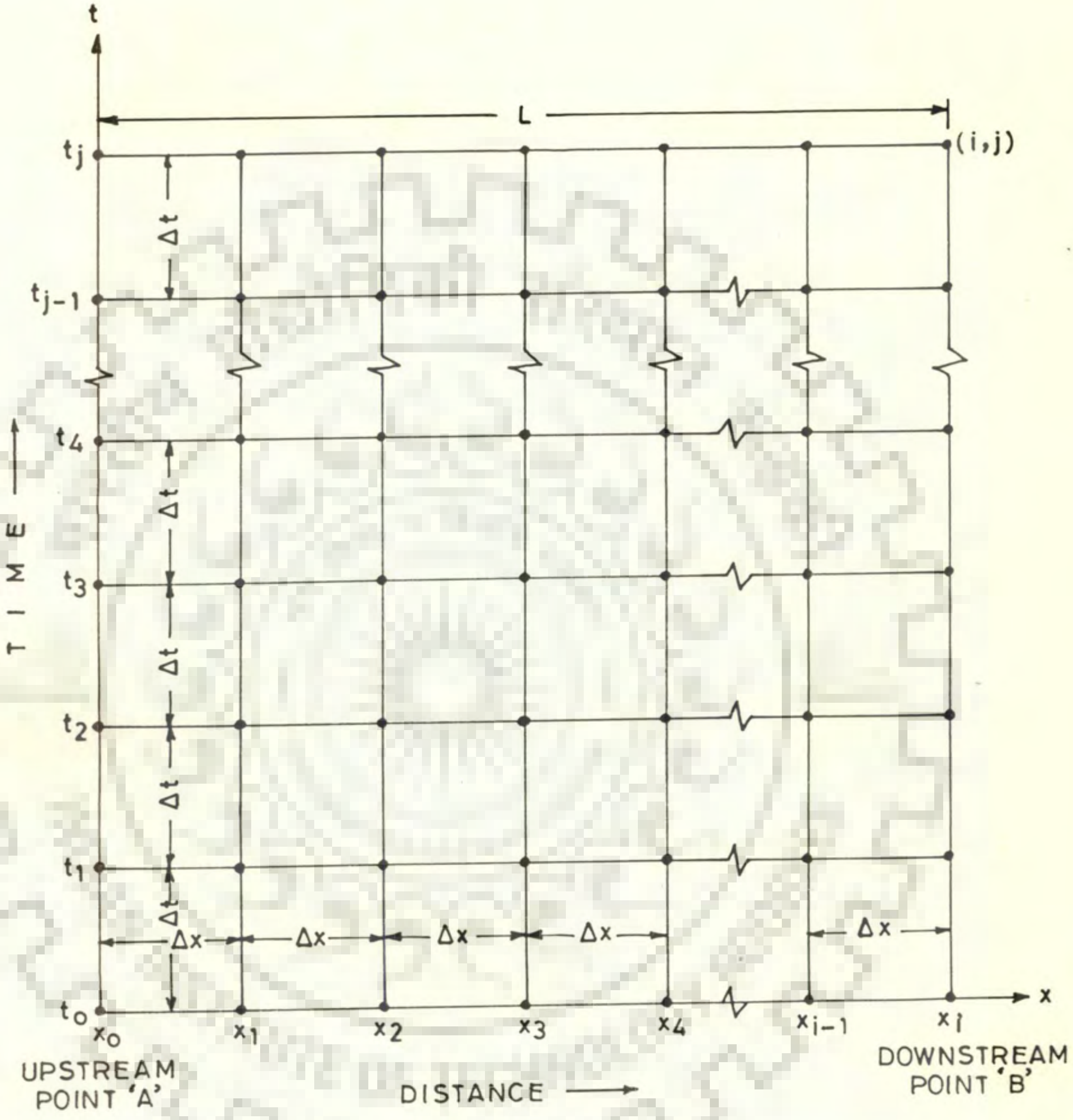


FIG. 3.3 FLOW DOMAIN REPRESENTATION THROUGH A FIXED $\Delta x - \Delta t$ GRID

the 'time - lines'. The ordinate is the t-axis and the lines parallel to it are the 'space-lines'. The intersections of these two lines are termed as nodes (or computational points). The solution domain in x-t plane is covered by a rectangular node (or grid) with grid spacing of Δx, Δt in the x-t directions respectively. The Δx and Δt values are being assumed constant in the present case. The grid consists of the set of lines parallel to the t-axis are given by

$$x = x_i, \qquad i = 0,1,2,3 \ldots\ldots$$

where $x_i = i\Delta x$.

And the set of lines parallel to the x-axis are given by

$$t = t_j, \quad j = 0,1,2,3 \ldots\ldots$$

where $t_j = j\Delta t$.

Therefore, the point $(i\Delta x, j\Delta t)$ is called the grid point (i,j) and is surrounded in neighbourhood by other grid points.

The computational scheme will require the initial values for the entire domain (X_i 's) and the upstream boundary conditions for all (tj's). Solutions of the governing equations through the finite difference scheme will be obtained at each of these grid point Computations advance along the downstream direction for a time step Δt, until all the discharges as well as water areas are computed at all the grid points in the entire longitudinal length L (Figure 3.3).

Next the computations are advanced ahead in time by another time step Δt and the computations proceed likewise.

The rainfall excess intensity (i_e) is assumed constant within a time step Δt. But it may change from one time step to the next time step, to account for the variation in rates of rainfall excess intensities occurring within a storm event.

3.7 APPROXIMATION OF THE DERIVATIVES OF KINEMATIC WAVE EQUATIONS THROUGH FINITE DIFFERENCES

The discharge of any point (x) at a given instant (t) is written as Q(x,t). If Q possesses a sufficient number of partial derivatives then at the two points (x,t) and ($x + \Delta x$, $t + \Delta t$), the values of Q are related by the Taylor's series expansion, as under :

$$Q(x + \Delta x,\ t + \Delta t) = Q(x,t) + (\Delta x\frac{\partial}{\partial x} + \Delta t\frac{\partial}{\partial t})Q(x,t)$$

$$+ \frac{1}{2!}(\Delta x\frac{\partial}{\partial x} + \Delta t\frac{\partial}{\partial t})^2 Q(x,t) + \ldots$$

$$\ldots\ldots + \frac{1}{(x-1)!}(\Delta x\frac{\partial}{\partial x} + \Delta t\frac{\partial}{\partial t})^{n-1} Q(x,t) + R_n \quad \ldots \quad (3.23)$$

where R_n is the remainder term and can mathematically be written as below :

$$R_n = \frac{1}{n!}(\Delta x\frac{\partial}{\partial x} + \Delta t\frac{\partial}{\partial t})^n Q(x + \xi\Delta x,\ t + \xi'\Delta t),\ 0 < \xi < 1 \quad \ldots (3.24)$$

That is,

$$R_n = O[(|\Delta x| + |\Delta t|)^n] \quad \ldots (3.25)$$

Here, 'O' represents the order (or degree) of the remainder.

Equation (3.25) suggests that there exists a positive constant M, such that $|R_n| \leq M(|\Delta x| + |\Delta t|)^n$ as $\Delta x \to 0$ and $\Delta t \to 0$.

The space point $(i\Delta x, j\Delta t)$ is already defined (Section 3.6) as the grid point (i,j) and is surrounded by the neighbouring grid points. Therefore, through neighbouring grid points, the Taylor's series expansion can be written for $Q_{i-1,j}$ and $Q_{i+1,j}$ about the central value $Q_{i,j}$ respectively as :

$$Q_{i-1,j} = Q_{i,j} - \Delta x \frac{\partial Q}{\partial x} + \frac{(\Delta x)^2}{2!} \frac{\partial^2 Q}{\partial x^2} - \frac{(\Delta x)^3}{3!} \frac{\partial^3 Q}{\partial x^3} + \frac{(\Delta x)^4}{4!} \frac{\partial^4 Q}{\partial x^4} + \ldots\ldots \tag{3.26}$$

and

$$Q_{i+1,j} = Q_{i,j} + \Delta x \frac{\partial Q}{\partial x} + \frac{(\Delta x)^2}{2!} \frac{\partial^2 Q}{\partial x^2} + \frac{(\Delta x)^3}{3!} \frac{\partial^3 Q}{\partial x^3} + \frac{(\Delta x)^4}{4!} \frac{\partial^4 Q}{\partial x^4} \ldots \tag{3.27}$$

All partial derivatives $\frac{\partial Q}{\partial x}$, $\frac{\partial^2 Q}{\partial x^2}$, $\frac{\partial^3 Q}{\partial x^3}$ ... etc.

are evaluated at the grid point (i,j).

The first order derivatives $\partial Q/\partial x$ at (i,j) in terms of finite difference relationships are obtained from the equations (3.27) and (3.26) and subsequently by subtracting the two :

$$\frac{\partial Q}{\partial x} = \frac{Q_{i+1,j} - Q_{i,j}}{\Delta x} + O(\Delta x) \quad \ldots \tag{3.28}$$

$$\frac{\partial Q}{\partial x} = \frac{Q_{i,j} - Q_{i-1,j}}{\Delta x} + O(\Delta x) \quad \ldots \tag{3.29}$$

$$\frac{\partial Q}{\partial x} = \frac{Q_{i+1,j} - Q_{i-1,j}}{2\Delta x} + O(\Delta x) \quad \ldots \tag{3.30}$$

The order term $O(\Delta x)$ accounts for the higher order terms (having order in two or more than two).

Equations (3.28), (3.29) and (3.30) are respectively known as the forward, backward and central difference forms of the finite differences.

In this work only the forward and the backward finite difference forms have been used. The first order partial derivative $\partial Q/\partial x$ is approximated by neglecting order term $O(\Delta x)$ from equations (3.28) and (3.29). Thus at the grid point (i,j) the partial derivates of Q in the forward and the backward finite difference forms are written as :

$$\frac{\partial Q}{\partial x} \simeq \frac{Q_{i+1,j} - Q_{i,j}}{\Delta x} \quad \ldots (3.31)$$

$$\frac{\partial Q}{\partial x} \simeq \frac{Q_{i,j} - Q_{i-1,j}}{\Delta x} \quad \ldots (3.32)$$

Similarly, the first order partial derivatives $\partial Q/\partial t$ can be written in forward and backward finite difference forms at the grid point (i,j) as below :

$$\frac{\partial Q}{\partial t} \simeq \frac{Q_{i,j+1} - Q_{i,j}}{\Delta t} \quad \ldots (3.33)$$

and

$$\frac{\partial Q}{\partial t} \simeq \frac{Q_{i,j} - Q_{i,j-1}}{\Delta t} \quad \ldots (3.34)$$

Likewise, the first order partial derivatives of cross sectional area (A) are approximated through the forward and the backward finite differences at the grid point (i,j) as under :

$$\frac{\partial A}{\partial x} \simeq \frac{A_{i+1,j} - A_{i,j}}{\Delta x} \quad \ldots \quad (3.35)$$

$$\frac{\partial A}{\partial x} \simeq \frac{A_{i,j} - A_{i-1,j}}{\Delta x} \quad \ldots \quad (3.36)$$

$$\frac{\partial A}{\partial t} \simeq \frac{A_{i,j+1} - A_{i,j}}{\Delta t} \quad \ldots \quad (3.37)$$

$$\frac{\partial A}{\partial t} \simeq \frac{A_{i,j} - A_{i,j-1}}{\Delta t} \quad \ldots \quad (3.38)$$

Equations (3.31) through (3.38) have been used for the finite difference approximation of the partial derivatives appearing the final form of KW equations. Suitable computational schemes are needed for the solution of these equations.

3.8 DIFFERENT SCHEMES FOR THE SOLUTION OF KINEMATIC WAVE EQUATIONS : Their Stabilities and Convergences

The three different kinds of computational schemes as briefed in Table 3.1 (and shown in Figure 3.4) have been used in this work for the solution of KW equations when applied to different watersheds under investigation.

TABLE 3.1 : The Three First Order Explicit Schemes of KW Equations Under Investigation

Description	Schemes
Forward-in-time and backward-in-space	I
Backward-in-time and forward-in-space	II
Backward-in-time and space	III

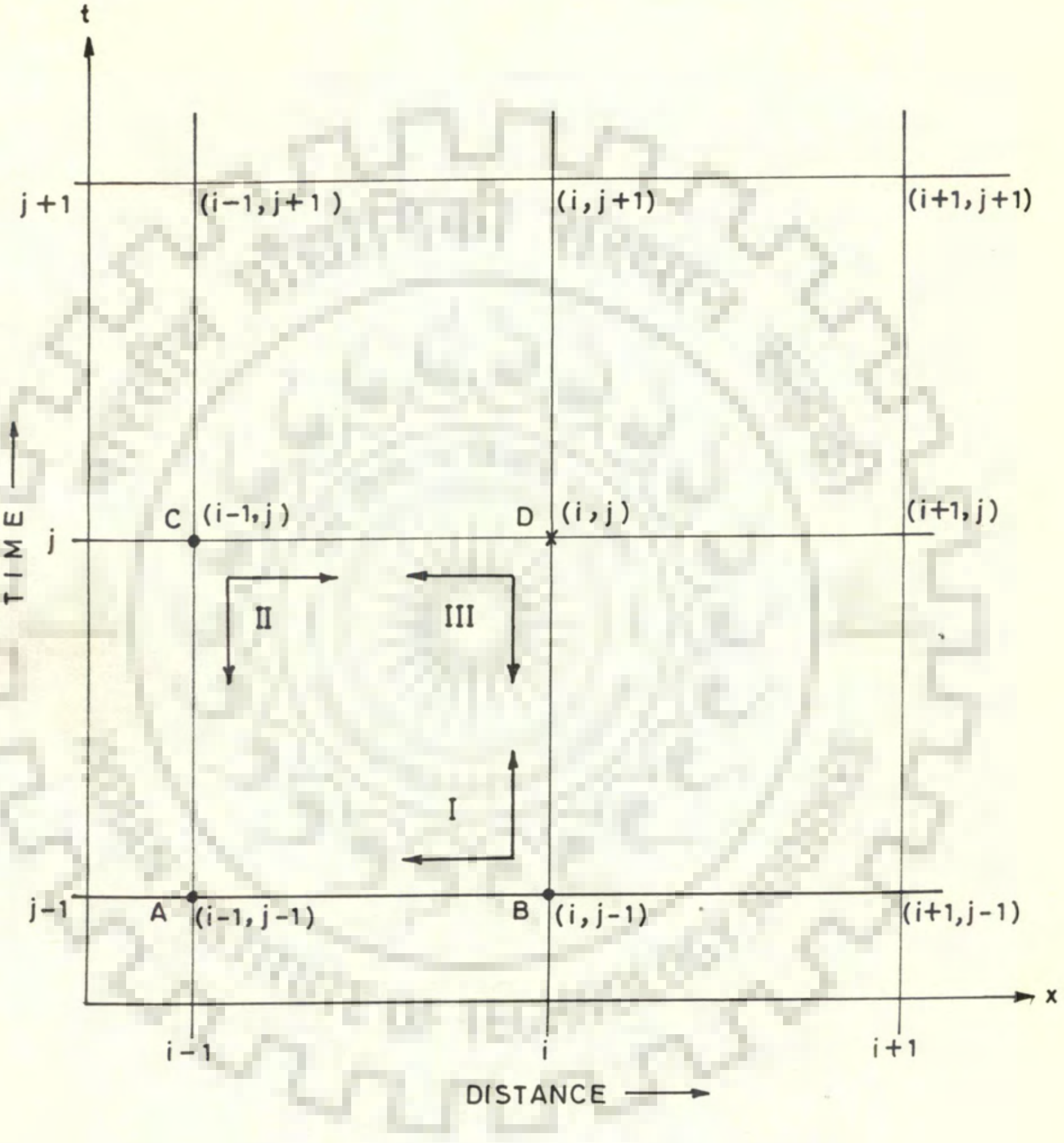


FIG. 3.4 SPACE - TIME DISCRETIZATION FOR THE COMPUTATIONAL SCHEMES I, II & III

The recommended use of one or more of these schemes will depend upon their stabilities and the convergences to the analytical solutions.

The criteria of stability and convergence for these three fully off-centred explicit schemes, (Figure 3.4) were spelled by Ponce (1979, 1986). Schemes I & II have been found conditionally stable and always converging to the analytical solutions if the Courant number approaches 1. The Courant number is defined as the ratio of the physical celerity C (in this case the KW celerity) to the grid celerity $\Delta x/\Delta t$. The Scheme I is reported to be stable for Courant number less than or equal to 1, whereas the Scheme II is said to be stable if Courant number equals to or is greater than 1. The Scheme I happens to the mirror image of Scheme II, therefore, for ascertaining the stability both the schemes I and II may be considered as one scheme and may thus be named as Scheme (I & II). Ponce (1979, 1986) has also stated that the Scheme III is unconditionally stable but non-convergent for any values of Courant number. Therefore, it should be taken as a separate computational scheme. Scheme III thus can be used only when it is found convergent to the analytical solution. The values of Δx and Δt which make the scheme convergent have to be ascertained by trial and error. It may be concluded that for ascertaining the stability of these computational schemes the Courant number has to be computed which requires estimation of KW celerity values.

3.8.1 Kinematic Wave Celerity

The kinematic wave celerity (C) values will depend upon as to how the mechanics of the generation of surface runoff and the channel flows have been taken care of. The manner in which the contributions of overland planes reach the channel, will affect the channel flows and so the channel velocities. Therefore, it is necessary to explain the mechanic of the overland and the channel flows, as accounted for in this work.

In this study, rectangular planes have been assumed to represent the watershed geometry in its simplified form. The assumed configuration of a watershed thus consists of the two rectangular planes and a channel in between (Figure 3.1(a)). The rainfall excess generated over the overland planes (on the two sides of the channel) causes the surface runoff to generate. This surface runoff contributes to the channel in the form of gradually varied flows (Fig. 3.1(b) and (c)). The overland flow elements are assumed to be perpendicular to the channel flow direction. The lateral contribution (q) from the planes is taken as inflow to the channel. By channel routing, the final flood hydrographs are estimated at the outlet.

For the computational purposes the surface planes are divided into the elements of unit width. In figures 3.1(a) and 3.1(b), ABB'A' is one of such overland flow element of one metre width (width AA' = BB' = 1 metre and length BA = B'A' = Lo i.e. the width of the plane. This overland plane element is further divided into n number of strips all along the downstream directions of flow. Each strip of equal length Δx_o (i.e. $L_o = n\Delta x_o$). By taking rainfall excess intensity (i.e. gross rainfall - abstractions) as input within a Δt, computations are performed for the flow per unit width (q_i) and flow area A_i ($=1.y_{oi} = y_{oi}$)

from one nodal point to other nodal point (i.e. from x_{oi-1} to x_{oi}) (where i = 1,2,... n). The discharge per unit width (q_i) and corresponding flow area (y_{oi}) for each of the nodal point so obtained have been used for the computation of q_i and y_{oi} as initial values for the next time step (i.e. Δt to $2\Delta t$). The flow profiles so obtained for different time steps (Δt) are shown in figure 3.1(d). In this way, the computations proceed and the final surface flow profile is generated.

In the case of channel flows, the computational scheme is similar to the overland flow scheme. Here, the channel is divided into m number of strips of length Δx_c (i.e. $m \ \Delta x_c = L_k$ length of the channel). From the overland plane elements the lateral contributions per unit width $q(= q_n)$, from the input to the channel. For computing the water area (A) and discharge (Q) at the outlet (or at any desired point in the channel), the computations proceed from the nodal point to the other in the downstream direction (i.e. from X_{co} to X_{c1} and onwards).

In the physiographic properties remain uniform over the entire surface plane, the surface runoff contributions (i.e. $q = q_n$) will also be uniform for all the elemental planes denoted by P_1, P_2, P_3 etc. [Figure 3.1(a)]. This final value of lateral contribution per unit width q is used in the channel flow routing.

For the earlier discussed mechanics of the runoff, the kinematic wave celerity (C) in its general form can be written as:

$$C = \frac{\partial Q}{\partial A} \qquad \ldots \quad (3.39)$$

A substitution from equation (3.7) in the above relationship gives the KW celerity (C) as below :

$$C = \alpha m A^{m-1} \quad \ldots \quad (3.40)$$

The average wave celerity $\bar{C}$ may thus be expressed as :

$$\bar{C} = \alpha m \bar{A}^{m-1} \quad \ldots \quad (3.41)$$

where $\bar{A}$ is the average waterflow area.
Equation (3.41) gives the average celerity of kinematic waves and may thus be used for the overland as well as the channel flows.

3.8.1.1 Average kinematic wave celerity for the overland flows

For overland flows, in equation (3.41), $\bar{A}$ will be the average water flow area. Since a unit width of overland element has been considered for the computations, average water area will be given by its average depth $\bar{y}_o$ (i.e. $\bar{A} = 1.\bar{y}_o$).

The average water area for the overland flow element at jth instant will be given by

$$\bar{A} = \bar{A}' + \text{Rainfall excess depth}$$

$$\text{or} \quad \bar{A} = \bar{y}_o + \text{Rainfall excess depth} \quad \ldots \quad (3.42)$$

Here $\bar{A}'$ (= $\bar{y}_o$) will be the average value of the flow area (y_{oi}, i = 0, 1, 2, 3 ... n) which are computed at (j-1)th instant (j = 1, 2, 3, 4, ...) at different nodal points (X_{oi}'s) over the entire length of plane L_o. Trapezoidal rule of numerical integration (Rajaraman, 1983) is used to compute the average flow areas which will be as under :

$$\bar{A}'(=\bar{y}_o) = \frac{1}{n}\left[\frac{y_{oo}}{2} + y_{o1} + y_{o2} + y_{o3} + \ldots + \frac{y_{on}}{2}\right] \quad \ldots \quad (3.43)$$

The rainfall excess depth (if it exists) is accounted for the duration Δt between (j-1)th and jth time steps.

Following the procedure, the average flow areas for all the time steps are computed and by using equation (3.41), the average KW celerity is computed for each time step.

3.8.1.2 Average kinematic wave celerity for the channel flows

The average kinematic wave celerity $\bar{C}$ of the channel flows varies from one time step to another time step and will depend upon the average flow area $\bar{A}$.

The average channel flow area, $\bar{A}$ is approximated by the following finite difference formula for the computations of $\bar{C}$:

$$\bar{A}_{i,j} = \frac{A_{i-1,j} + A_{i,j-1}}{2} \quad \ldots \quad (3.44)$$

where i and j respectively refer to the space and time steps (Figure 3.4).

Having computed the $\bar{A}_{i,j}$, the average KW celerity for the jth time step is computed by using equation (3.41). Similarly the KW celerity is computed for all the time steps till the channel flows continue. These values are then used to compute the Courant number for establishing the stability of the proposed computational schemes. These computational schemes are described in the following sections :

3.8.2 Computational Scheme I for the Solution of Kinematic Wave Equations

As mentioned in Table 3.1 and shown in figure 3.4, the Scheme I is defined as forward-in-time and backward-in-space. Also mentioned earlier, Scheme I is stable if Courant number is less than or equal to 1. That is the average kinematic wave celerity $\bar{C}$ should be less than or equal to, the ratio of the computational space strip Δx and time strip Δt (i.e. $\bar{C} \leqslant \frac{\Delta x}{\Delta t}$). For this scheme, the temporal derivative of cross sectional area is evaluated between two points B and D in figure 3.4, at the grid point (i, j-1) and thus can be written as (by the concept of Section 3.7):

$$\frac{\partial A}{\partial t} \simeq \frac{\Delta A}{\Delta t} = \frac{A_{i,j} - A_{i,j-1}}{\Delta t} \qquad \ldots \quad (3.45)$$

And the spatial derivative of area is evaluated between points B and A at the grid point (i,j-1) and gives

$$\frac{\partial A}{\partial x} \simeq \frac{\Delta A}{\Delta x} = \frac{A_{i,j-1} - A_{i-1,\, j-1}}{\Delta x} \qquad \ldots \quad (3.46)$$

The average area is taken between two points B and A and gives

$$\bar{A} \simeq \frac{A_{i,j-1} + A_{i-1,j-1}}{2} \qquad \ldots \quad (3.47)$$

By making use of equations (3.45), (3.46) and (3.47), the general KW equation (3.8) can be written in terms of cross sectional area in its complete finite difference forms at the grid point (i,j-1) as under :

$$\frac{A_{i,j} - A_{i,j-1}}{\Delta t} + \alpha m \left[\frac{A_{i,j-1} + A_{i-1,j-1}}{2}\right]^{m-1} \left[\frac{A_{i,j-1} - A_{i-1,j-1}}{\Delta x}\right]$$

$$= \bar{q} \quad \ldots \quad \ldots. \quad (3.48)$$

In this study, $\bar{q} = q$ ... ... (3.49)

The values of A are known at the grid points (i-1, j-1) and (i,j-1). Therefore, in equation (3.48), the only unknown is $A_{i,j}$ and its value may be computed for the adopted values of α & m (Sections 3.4.1 and 3.5.1), i.e. for the overland and channel flow, as the case may be.

Therefore, equation (3.48) can also be written in terms of $A_{i,j}$ in the following form explicitly:

$$A_{i,j} = \bar{q}\,\Delta t + A_{i,j-1} - \alpha m \frac{\Delta t}{\Delta x} \left[\frac{A_{i,j-1} + A_{i-1,j-1}}{2}\right]^{m-1}$$

$$\left[A_{i,j-1} - A_{i-1,j-1}\right] \quad \ldots \quad (3.50)$$

If $A_{i,j}$ is known from equation (3.50), then the corresponding $Q_{i,j}$ can be computed from equation (3.7) and is given by

$$Q_{i,j} = \alpha (A_{i,j})^{m} \quad \ldots \quad (3.51)$$

This is a direct method of computing the cross-sectional areas and the corresponding dischages. It is applied to the overland as well as the channel flow components with minor modifications in the relationships (3.50) and (3.51).

For overland flows, in equations (3.50) and (3.51), the following changes must be incorporated.

(i) q be replaced with i_e (i.e. rainfall excess intensity)

(ii) Q be replaced with y_o [i.e. $A(=y_o) = 1.y_o$]

(iii) α and m be substituted with α_o and m_o respectively (Section 3.4.1)

For the channel flows, q is the input due to overland flows. The parameters α and m be replaced with α_k and m_k respectively (Section 3.5.1).

3.8.3 Computational Scheme II for the Solution of Kinematic Wave Equations

It is seen from Table 3.1 and figure 3.4 that the Scheme II is defined as backward-in-time andforward-in-space in its finite difference forms. Scheme II is stable for Courant number greater than or equal to 1. That it, average kinematic wave celerity $\overline{C}$ must be greater than or equal to the ratio of the space step Δx and time step Δt (i.e. $\overline{C} \geqslant \Delta x/\Delta t$. In this scheme, the spatial derivative of disharge is evaluated between two points C and D, at the grid point (i-1,j) (Figure 3.4) and can be written as:

$$\frac{\partial Q}{\partial x} \simeq \frac{\Delta Q}{\Delta x} = \frac{Q_{i,j} - Q_{i-1,J}}{\Delta X} \qquad \ldots \quad (3.52)$$

And the temporal derivative of area of cross section is computed between points C and A at the grid point (i-1,j) as:

$$\frac{\partial A}{\partial t} \simeq \frac{\Delta A}{\Delta t} = \frac{A_{i-1,j} - A_{i-1,j-1}}{\Delta t} \qquad \ldots \quad (3.53)$$

Substituting theabove values in the continuity equation (3.5), the following equation is obtained.

$$\frac{Q_{i,j} - Q_{i-1,j}}{\Delta x} + \frac{A_{i-1,j} - A_{i-1,j-1}}{\Delta t} = \bar{q} \qquad \ldots (3.54)$$

In equation (3.54) all the values are known except $Q_{i,j}$. Therefore, this equation can be written in terms of $Q_{i,j}$ as :

$$Q_{i,j} = Q_{i-1,j} + \bar{q}\Delta x - \frac{\Delta x}{\Delta t}[A_{i-1,j} - A_{i-1,j-1}] \qquad \ldots (3.55)$$

Computing the value of $Q_{i,j}$ from equation (3.55) and substituting in equation (3.7), the area of cross section at the point (i,j) is to be determined by

$$A_{i,j} = (Q_{i,j}/\alpha)^{1/m} \qquad \ldots (3.56)$$

As stated in Section 3.8.2, with proper substitutions, the above mentioned two finite difference forms of equations (Equations 3.55 and 3.56) can be applied to the overland as well as the channel flow computations.

3.8.4 Computational Scheme III for the Solution of Kinematic Wave Equations

The Scheme III is defined (shown in Table 3.1 and Figure 3.4) as backward-in-space and time in its finite difference forms. This scheme is unconditionally stable but it is nonconvergent for any values of Courant number. In this scheme, the temporal derivative of cross sectional area is evaluated between two points D and B (Figure 3.4) at the grid point (i,j) and thus can be written as

$$\frac{\partial A}{\partial t} \simeq \frac{\Delta A}{\Delta t} = \frac{A_{i,j} - A_{i,j-1}}{\Delta t} \qquad \ldots (3.57)$$

And the spatial derivative of area is computed between points D and C at the grid point (i,j) and gives

$$\frac{\partial A}{\partial x} \simeq \frac{\Delta A}{\Delta x} = \frac{A_{i,j} - A_{i-1,j}}{\Delta x} \quad \ldots \quad (3.58)$$

By using equations (3.57) and (3.58), the KW equation (3.8) can be written as in terms of cross-sectional area in its finite difference forms at the grid point (i,j) as below :

$$\frac{A_{i,j} - A_{i,j-1}}{\Delta t} + \alpha m \bar{A}^{\,m-1} \frac{A_{i,j} - A_{i-1,j}}{\Delta x} = \bar{q} \quad \ldots \quad (3.59)$$

($\bar{q}$ as equation 3.49).

In equation (3.59), $A_{i,j}$ is unknown. Therefore, by putting $\alpha m\ \bar{A}^{m-1} = \bar{C}$ from equation (3.41) in equation (3.59), $A_{i,j}$ can explicitly be written as

$$A_{i,j} = \frac{\bar{q}.\Delta x.\Delta t + \Delta x.A_{i,j-1} + \bar{C}.\Delta t.\ A_{i-1,j}}{\Delta x + \bar{C}\Delta t} \quad \ldots \quad (3.60)$$

$$\text{or} \quad A_{i,j} = \frac{\bar{q}\Delta t + A_{i,j-1} + \bar{C}.A_{i-1,j}\cdot\ \frac{\Delta t}{\Delta x}}{1 + \bar{C}\ \frac{\Delta t}{\Delta x}} \quad \ldots \quad (3.61)$$

By knowing $A_{i,j}$ from equation (3.61) and then $Q_{i,j}$ can be computed from equation (3.7) as :

$$Q_{i,j} = \alpha(A_{i,j})^m \quad \ldots \quad \ldots \quad (3.62)$$

In the case of the overland flows, the values of $\bar{A}$ will be computed using equation (3.42) and for the channel flow by applying equation (3.44). With the proper substitutions of parameters as stated in Section 3.8.2, the above mentioned two finite difference forms of equations (Equations 3.61 and 3.62) can be applied to the overland and the channel flow computations.

The convergence of this scheme (Scheme III) is to be checked by adopting suitable values of Δx and Δt, and by following the trial and error procedure that set of values have to be determined which makes the scheme convergent to the analytical solution.

The KW approach is to be applied through the three finite difference schemes to different watershed models. This will help in ascertaining the suitability of each scheme under different conditions of physiography and land use.

3.9 PHYSIOGRAPHIC MODELS EMPLOYED

In nature, small watershed may be of numerous shapes and may differ in their slopes, roughnesses, channel configurations, etc. For the application of KW theory, some simplifications have to be resorted with respect to the watershed geometry. These are aimed at identifying a watershed in term of two basic elements namely, the 'overland plane element' and the 'channel element'. An oversimplification will result into a lumped approach, whereas a detailed description of the watershed through some arrangements of channels and the overland planes, may result into a distributed system. Thus for the present study, the following two broad categories of physiographic models have been considered for general applications.

(1) Lumped physiographic models, and

(2) Distributed parameters physiographic models.

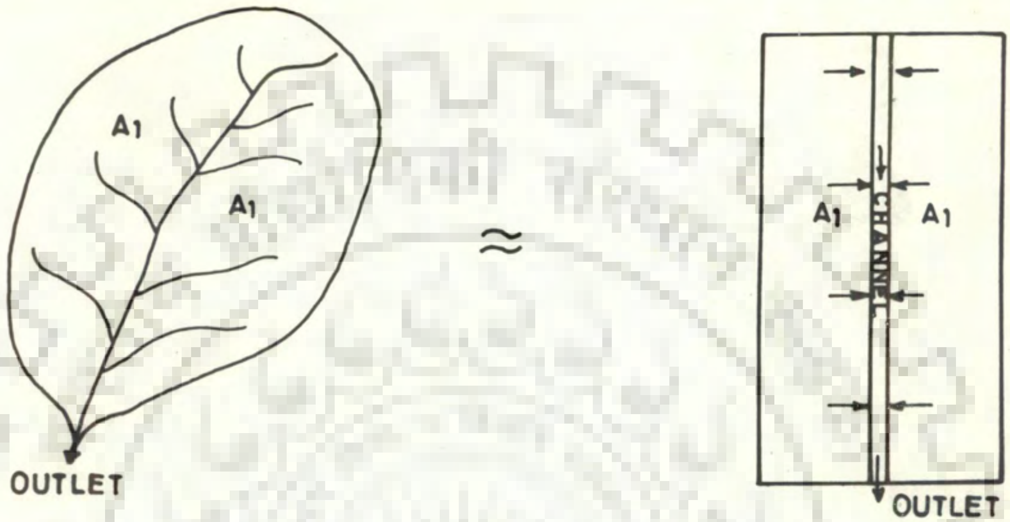
The main features of the above mentioned two categories of the models are briefly described in the following sections.

3.9.1 Lumped Physiographic Models

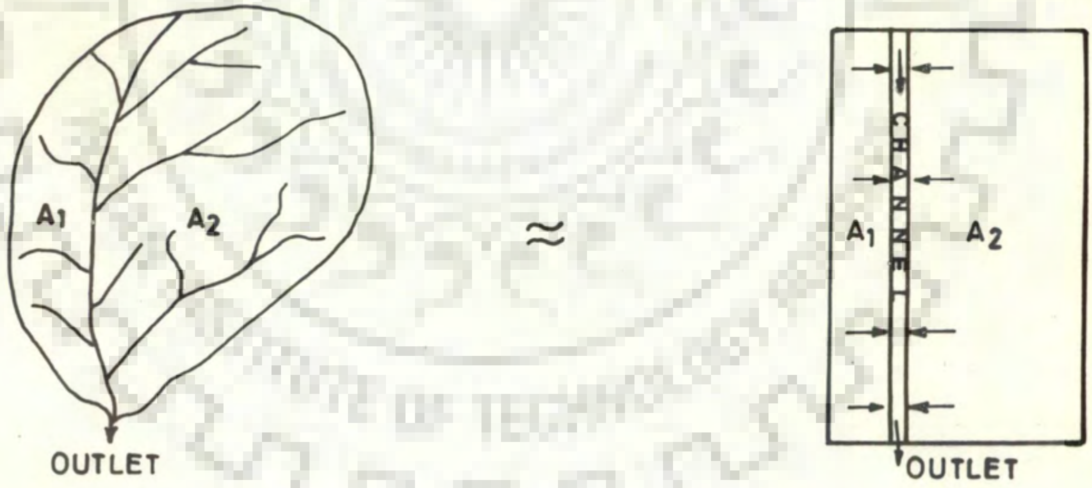
These models are characterized by a single main channel of length equal to that of the main drainage on the watershed. The contributing watershed on the two sides of the channel forms the two surface planes of this channel. If the main drainage is located nearly in the centre, the two surface planes may be equal in sizes (Figure 3.5(a)). The contributing surface planes are assumed as rectangular. The average values of the physiographic parameters (slopes i.e. overland and channel, Roughness etc.) are considered. The overland planes account for the total drainage area ɔf the watershed.

In many cases, the main drainage may not be centrally located in the watershed. In such cases, the contributing watershed areas on the two sides may differ significantly (Figure 3.5(b)). For such cases the physiographic models may have unequal widths of the two overland rectangular planes on the two sides of the channel. Both categories of the lumped physiographic models have been used in this work.

In general, the lumped parameter physiographic models are pro to establish the model parameters whcih may be fitted into the distributed parameter models which are best suited for watershed physiography.



(a) A LUMPED SYSTEM WITH EQUAL (SIMILAR) OVERLAND PLANES



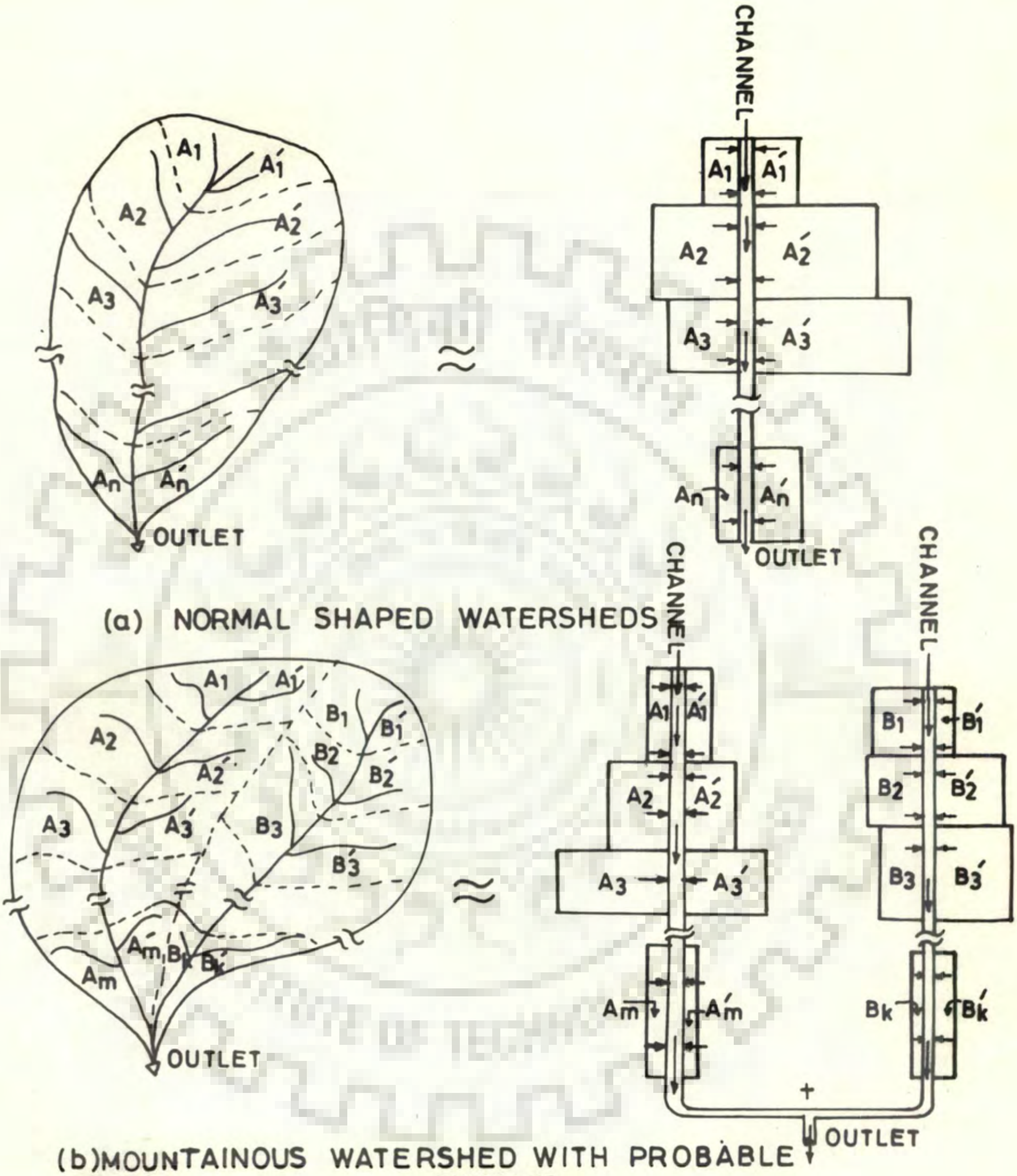
(b) A LUMPED SYSTEM WITH UNEQUAL OVERLAND PLANES

FIG. 3.5 LUMPED PHYSIOGRAPHIC MODELS

3.9.2 Distributed Physiographic Models

The lumped physiographic models discussed in the previous section, assumed average conditions of all the physiographic parameters related to the overland planes as well as the channels. In case of typical watersheds which may be a part of mountainous areas, agricultural lands, with varied land uses etc., the assumption of lumped approach may not be realistic. Also, approximation of the watershed area through the two rectangular planes will not take into account the shape of the watershed accurately. For this purpose watershed may be divided into number of subareas (6 or preferably more than that) keeping in view the overland drainages and their contributions to the channel system. Each subarea element is schematized by the two overland planes contributing to the channel located in between. The contributing overland planes may be unequal in sizes but are assumed to be rectangular in shapes. For a single main drainage, the schematic representation of the watershed physiography for a natural shaped watershed is given in figure 3.6(a). The cascade of elements (each element consisting of the two overland planes and a channel in between) thus represents the entire watershed and the watershed physiography is as closely followed as possible.

In case of hilly watersheds, parallel drainages are frequently found. In such cases, the main drainage channels are identified. Keeping in view, the channel system and the drainage patterns, their subareas are delineated. In the schematic representation, a watershed with two parallel main drainage has been shown in figure 3.6(b). Each drainage is represented by a cascade thus resulting into two parallel cascade systems



(a) NORMAL SHAPED WATERSHEDS

(b) MOUNTAINOUS WATERSHED WITH PROBABLE DYNAMIC

FIG. 3.6 DISTRIBUTED PHYSIOGRAPHIC MODELS contributing to the outlet. As discussed earlier, each element of the cascade will consist of two overland planes (may be unequal in sizes) contributing to the channel.

This is to remark that if the number of parallel drainages is restricted to two only, parameter estimaion through a lumped system may work well. However, if the number of parallel drainage exceeds (3 or more than that) estimation of physiographic parameter through the lumped physiographic model may not be realistic. It is due to the fact that the lengths of overland plane will be at much variance than what they eixst in practice. Therefore, parameter estimation in such cases has to be obtained through the distributed physiographic model as adopted for the analysis.

As discussed in Section 3.9 for the application to the proposed physiographic models of the KW theory, it is necessary to have the required information about the initial and the boundary conditions.

3.9.3 Initial and Boundary Conditions

In cases of lumped as well as distributed systems, for the application of KW theory, the three computational schemes discussed in Section 3.8 are to be applied. These explicit finite difference methods will require initial and upstream boundary informations for the overland as well as the channel flows.

3.9.3.1 Initial conditions

The present work is aimed at applying the KW theory to some of the typical small watersheds located in different regions of India through the proposed physiographic model to ascertain their general applicability. The stream pattern in these watersheds is generally emphameral. No appreciable base flows are found in these watershed. Thus, the channel remain dry prior to the onset of the storm as well as after the storm event. Therefore, the initial conditions of the channel for all the lumped as well as for the distributed systems are described as under :

$$\left.\begin{array}{l} Q(x,o) = 0 \\ \\ A(x,o) = 0 \end{array}\right\} \text{ for all } x \qquad \ldots \quad (3.63)$$

For the overland flows, the initial conditions refer to the depth of surface runoff at the instant t = 0 i.e. the instance when the rainfall excess just starts generating on the surface. So, the initial conditions applicable to both the types of the physiographic models (lumped and distributed) will be as below :

$$\left.\begin{array}{l} q(x,o) = 0 \\ \\ y_o(x,o) = 0 \end{array}\right\} \text{ for all } x \qquad \ldots \quad (3.64)$$

3.9.3.2 Boundary conditions

In the proposed physiographic models (lumped and distributed), the computational schemes discussed in Section 3.8 utilize only the upper boundary conditions. Since the watershed is bounded by the 'divide' no flow across the divide is possible.

Therefore, the upstream boundary conditions for the channel and the overland components are defined as under :

For channel flow :

$$\left.\begin{aligned} Q(o,t) &= 0 \\ A(o,t) &= 0 \end{aligned}\right\} \quad \text{for all } t \qquad \ldots \quad (3.65)$$

And for the overland flow :

$$\left.\begin{aligned} q(o,t) &= 0 \\ y_o(o,t) &= 0 \end{aligned}\right\} \quad \text{for all } t \qquad \ldots \quad (3.66)$$

3.10 COMPUTER PROGRAMS

For the physiographic models discussed in Section 3.9, the KW theory application was carried through the computational schemes described in Section 3.8. The computer programs for these applications were developed in FORTAN IV for the main frame computer (DECSYSTEM-20). The details of the programs are given in Appendix - I.

DeVries and MacArthur (1979) gave illustrative examples for the solution of finite difference applications to KW models. This data was used for the verification of the computer programs prepared and used in this thesis. A comparison of computed results with the illustrative examples of DeVaries et al., (1979) is given in Table A-1 of Appendix - I. The two results are in close agreement and this suggests and proves the correctness of the programs prepared and used in this work.

The computer programs were run for the available data of the four test watersheds located in different parts of India. A brief description of these test watersheds and availability of data is described in the following chapter.

CHAPTER - IV

WATERSHEDS UNDER STUDY AND AVAILABILITY OF DATA

4.1 INTRODUCTION

In developing countries, financial constraints put serious limitations on collection and compilation of detailed hydrologic data. Non-availabiliy of equipments, technical personnels and other facilities also adversely affect this programme. Because of the high expenditures involved, the data collection programme of small watersheds suffers the maximum, as the first priority normally goes to the problems relating to basin hydrology. India being a developing country with limited financial resources is also not an exception to it. Thus, the desired detailed data for the hydrologic studies of small watersheds are not easily available even for the problematic watersheds. Nonetheless, as discussed in Chapter-I, many a times small watersheds do pose threats and are responsible for major communication disruptions. Therefore, there is a need to study the runoff mechanics of small watersheds more carefully and to develop suitable methodologies or models best suited for the prevailing conditions.

The Ministry of Railways, Government of India (GOI) identified a semi-mountainous region in the midst of the 'Plateau of Decccan' which produced quick and varied runoff responses to the rainfall events. The flash floods so produced pose dangers to the numerous culverts and bridges of the railways which are located in this region. Consequently, need was felt and three small test watersheds were identified where short durationed rainfall and runoff data of major storm events were collected for in-depth study of the mechanics of the runoff generation. These bridges are identified as Bridge Nos. 319, 317 and 719. Also, the hydrologic data for a small test agricultural watershed, named as Kachwa Watershed, was available from a doctoral thesis (Singh, O.P. (1980)). This watershed was alkaline in nature and had undergone reclamation through the treatment of gypsum. It is located in the nearby flat lands of the State of Haryana. A brief description of three test watersheds of the Ministry of Railways, GOI as well as of the Kachwa watershed is presented in the following sections.

4.2 PHYSIOGRAPHIC DETAILS OF WATERSHEDS OF RAILWAY BRIDGE NOS. 319, 317 AND 719

The small watersheds of Railway Bridge Nos. 319 and 317 selected for this study are situated in Bangalore district of Karnataka State of India. These bridges are located on Arsikere-Bangalore Section of the Indian Railways. The index maps giving details of the two watersheds are given in figures 4.1 and 4.2. The Bridge No. 719 is situated on the Jalarpet-Bangalore Sections of the Indian Railways. The index map giving details of the watershed of this bridge is given in figure 4.3. Some important features, general information and the physiographic data as extracted from the available records and the topographic map of these bridges are given in Table 4.1.

4.2.1. The Storm Rainfall Data of Bridge Nos. 319, 317 and 719

No recording raingauges were installed on any of the three watersheds. Since the watersheds are small, one non-record-

TABLE 4.1 : General Information and Salient Features of Watershed of Railway Bridge Nos. 319, 317 and 719.

Sl. No.	Particulars		Units		
			Br. No. 319	Br. No. 317	Br. No. 719
1	2		3	4	5
1.	(a) Name of the zone		Deccan Plateau	Deccan Plateau	Deccan Plateau
	(b) Name of the subzone		3i	3i	3i
2.	Geographical location	(i)	Longitude(appr)		
			$77^{\circ}10'$ East	$77^{\circ}11'$ East	$78^{\circ}16'$ East
		(ii)	Latitude (appr)		
			$13^{0}18'$ North	$13^{0}19'$ North	$12^{0}52'$ North
3.	Terrain		Hilly	Hilly	Semi Hilly
4.	Shape of the basin		Oblong	Fan	Normal
5.	Climate		Humid	Humid	Humid
6.	Type of soil	(i)	Rocky	Rocky	Rocky
		(ii)	Red Earth	Red Earth	Red Earth
7.	Land use	Partly	Dry cultivation	Dry cultivation	Dry cultivation
8.	No. of raingauge station		1	1	1
9.	No. of discharge location		1	1	1
10.	Altitude (average elevation from MSL)		833 metre	833 metre	747 metre
11.	Watershed area		82.0 hectare	140.0 hectare	1400.0 hectare
12.	Length of the main channel(longest)		1650.0 metre	1475.0 metre	7200.0 metre
13.	Total average annual rainfall		500-1000 (mm)	500-1000 (mm)	500-1000 (mm)

ing rain gauge was installed for registering the storm rainfall data. The period of the data collection for the watersheds of these bridges are given in Table 4.2.

TABLE 4.2 : Period of Data Collection and Number of Storm Event for Each Watershed (Br. No. 319, 317 & 719)

Bridge No.	From	To	Onward	No. of Storm Events
319	1962	1964	1988	10
317	1961	1964	-	8
719	1964	1966	-	8

For these storms, the rainfall readings were usually taken at an interval of 30 minutes. This was often reduced to 15 minutes or 10 minutes in some cases. The original record was made available in FPS units which was converted to metric units. The storm rainfall data thus available for this study is given in Appendix - II-A, B and C.

4.2.2 **The Runoff Data of Bridge Nos. 319, 317 and 719**

The stage-discharge relationships (G.D - curves) were established for the main channels at the outlets (i.e. bridge sites). The flood discharge data was made available at intervals ranging between 10 to 30 minutes. Enquiries revealed that during working hours the current meter readings were taken and the discharges were computed. But during odd hours, only the channel stages were recorded. In such cases, the runoff was read off from stage-discharge relationships. The runoff data of various storm events is given in Appendix - II-A, B and C.

4.3 PHYSIOGRAPHIC DETAILS OF KACHWA WATERSHED

The Kachwa agricultural watershed (Figure 4.4) is situated near the Cental Soil Salinity Research Institute (CSSRI), Karnal in the Haryana State of India. It has a gross commanded watershed area of 1073.0 hectares. Its geographical location is at a Latitude 29°43' North and Longitude 76°50' East. The average elevation is 245 metres above the Mean Sea Level (M S L). The general average slope of the watershed was found to be 0.12 per cent. The maximum length of the watershed is 6.0 km. As a result, the terrain of the watershed is much flatter. The Kachwa watershed went under reclamation during 1974 to 1979. Prior to onset of the monsoon selected areas of the watershed were bounded and treated with gypsum. Therefore, such areas did not participate in the runoff process. In the subsequent rainy seasons, the treated lands were put under paddy cultivation to expedite the process of reclamation. By the year 1977, nearly 95 per cent land had been reclaimed and was put under paddy cultivations. This way the data was collected during the period 1977 onwards belonged primarily of an agricultural watershed with paddy fields.

In this watershed, the average yearly rainfall is about 740 mm. Out of which approximately 80 per cent of rainfall is normally received during the monsoon periods (June to September). The soil of this watershed is alluvial in nature. The texture of soil varies from sandy loam to silty clay loam.

4.3.1 The Availability of Data of Kachwa Watershed

The Kachwa watershed was gauged by the scientists of CSSRI, Karnal. The rainfall was measured with a non-recording raingauge as well as recording raingauge (the Siphon type). In the three years (1977-79) periods, the rainfall-runoff data was collected for two storm events each for the months of July as well as August except 1979. Thus, rainfall data of five storm events were available at smaller time intervals (i.e. in multiples of 15 minutes). The corresponding discharge data at hourly intervals were recorded for all the above mentioned storm events. Since the topography of the watershed is nearly flat, therefore, the time bases of the hydrographs were lengthy and these varied from sixty four hours to seventy hours or more.

The rainfall-runoff records of the five storm events is given in Appendix - II-D.

During the period 1977 and afterwards the reclamation process continued. As a result of it some areas did not participate and therefore, yearwise details of net area participating in the runoff process are given in Table 4.3.

Table 4.3 : Details of Net Area Participating in the Runoff Process (Kachwa Watershed)

Sl. No.	Year	Total area (ha)	Bounded area (ha)	Total area participating in the runoff process (ha)	% of total areas participating in the runoff process
1	2	3	4	5(=3-4)	6
1	1977	1073.0	56.60	1016.40	94.73
2	1978	1073.0	45.44	1027.56	95.77
3	1979	1073.0	-	1073.00	100.00

4.4 GENERAL REMARKS

In the past, the general practice in India was that hydrological data collection formed a part of time bound (generally three years and exceptional cases five years) research programmes which were taken up by various agencies. Therefore, as discussed in the introductory remarks of this chapter, limited availability of detailed hydrologic data is the biggest constraints for the development and testing of sophisticated techniques. For the application of KW theory, short durationed rainfall data study is needed. For most of the natural watersheds the rainfall data was available either by daily basis or in multiples of hours. With difficulty short duration rainfall data in multiples of 15 minutes could be procured through the Ministry of Railways. The data collected by the Ministry of Railways was primarily through non-recording gauges. Strictly speaking, it can not be considered to be very scientific or sophisticated. But no other alternatives were insight. All cares were taken to check the general consistencies with regard to the rainfall data and the corresponding flood data. However, a strict analysis pertaining to these aspects was not possible, keeping in view the limited number of storm events which were available. This data happens to be the only source of, for carrying out the research of the type as undertaken in this study.

Within the framework of this limitation, this study had to be taken up and the application of KW theory to some typical small watersheds through the different physiographic models is discussed in the following chapter.

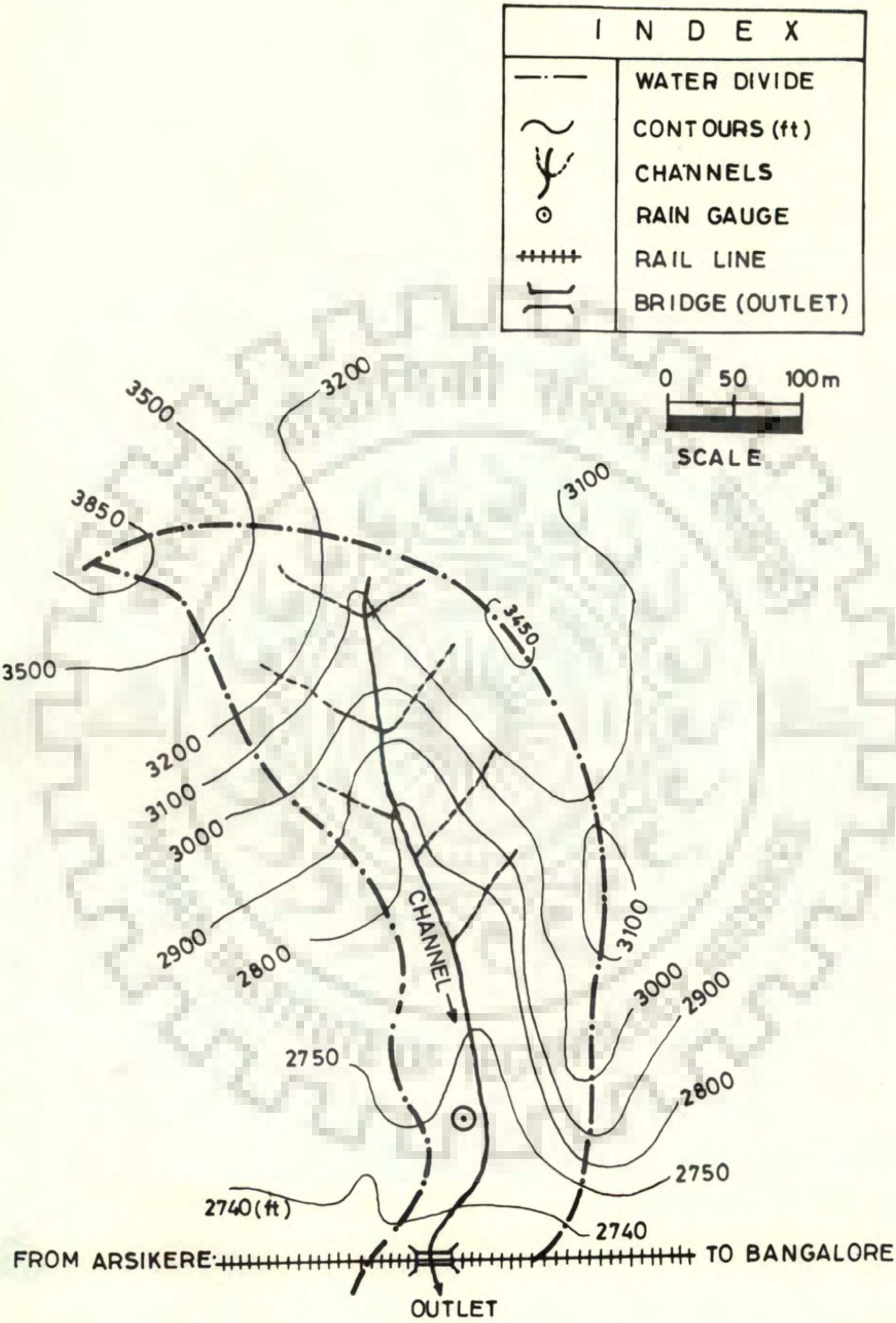


FIG. 4.1 THE INDEX MAP OF BRIDGE NO·319

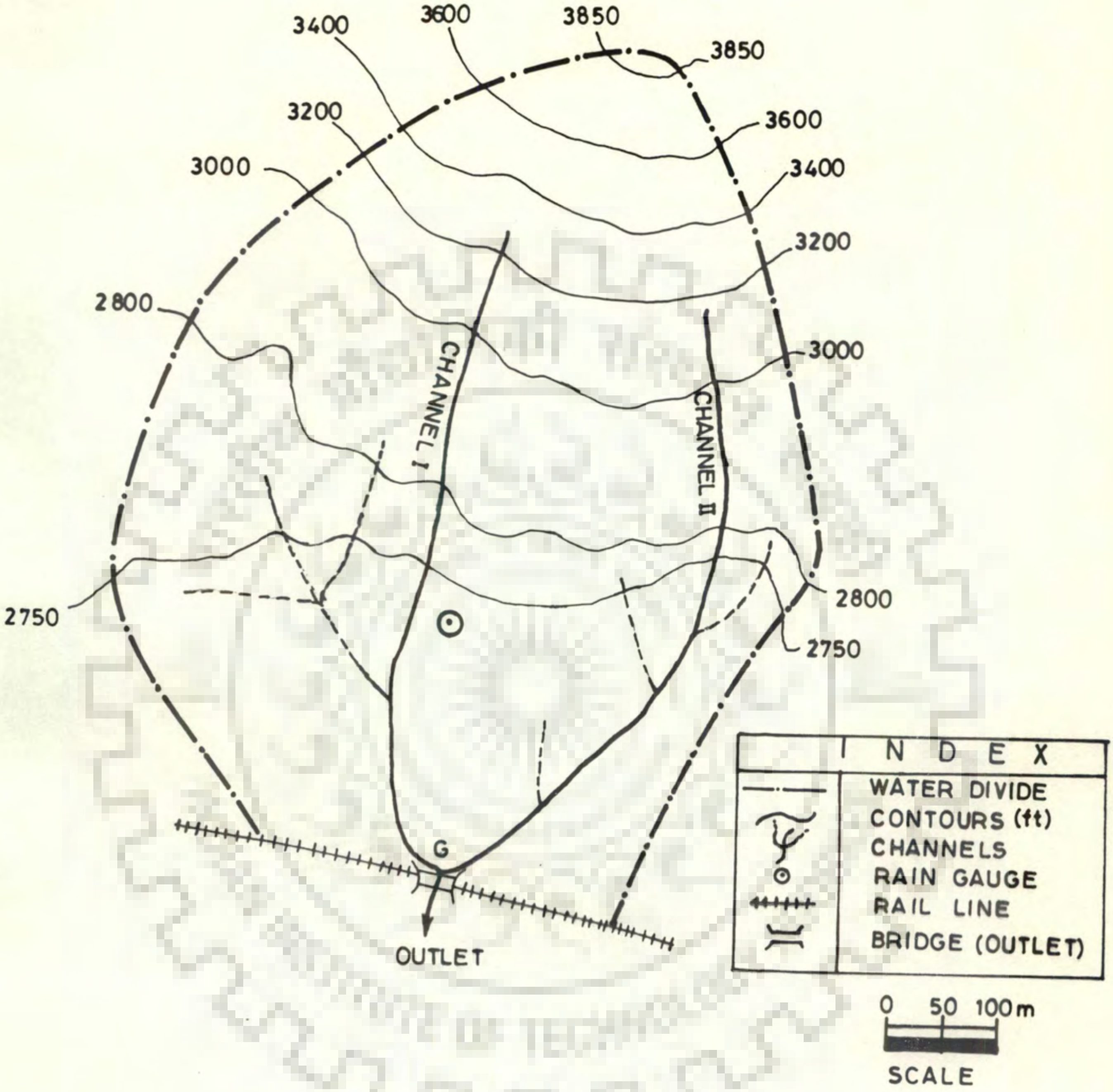


FIG. 4.2 THE INDEX MAP OF BRIDGE NO. 317

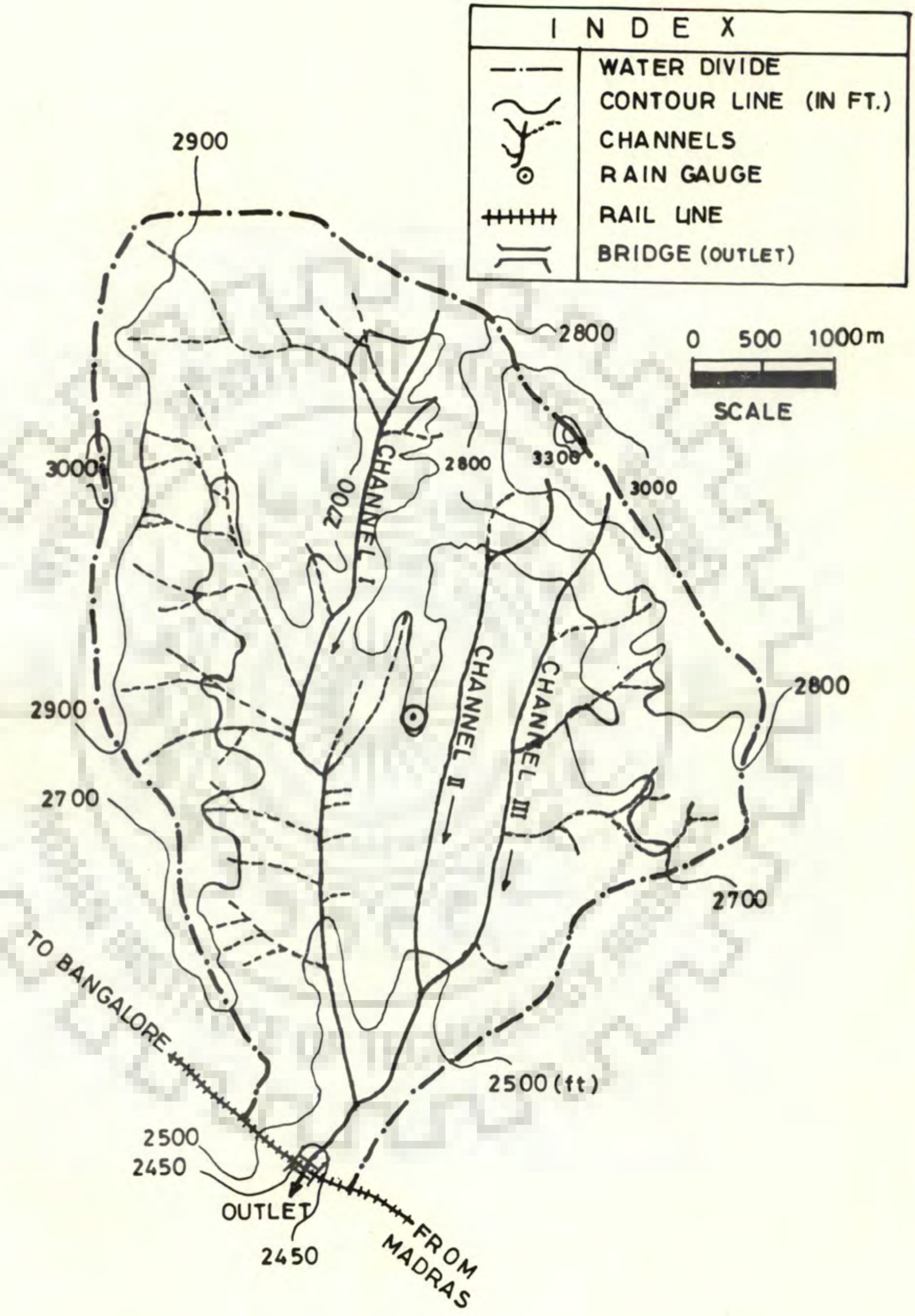


FIG. 4. 3 THE INDEX MAP OF BRIDGE NO. 719

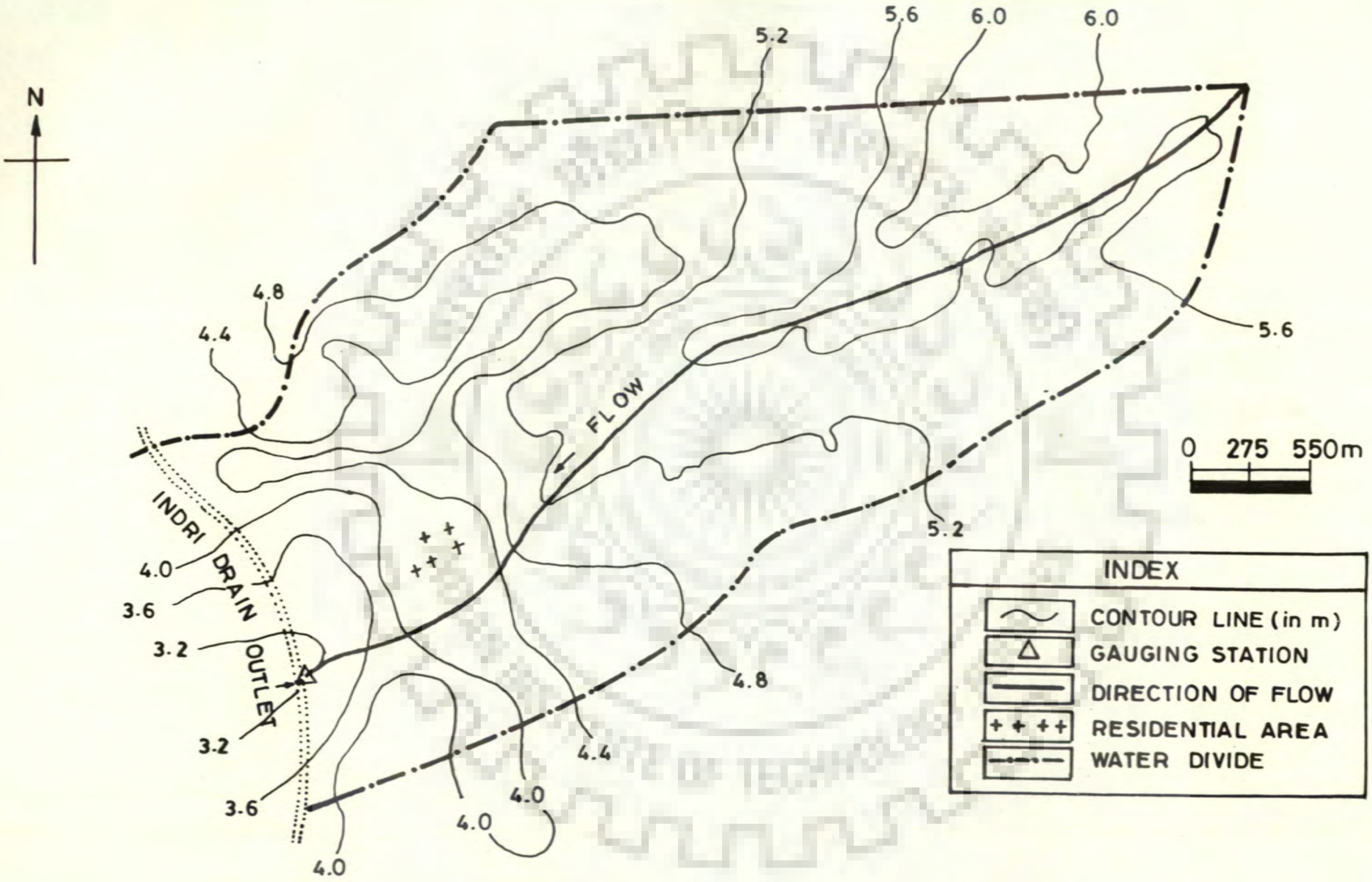


FIG. 4.4 THE INDEX MAP OF KACHWA AGRICULTURAL WATERSHED

CHAPTER - V

MODEL APPLICATION TO GAUGED SMALL WATERSHEDS

5.1 INTRODUCTION

Surface hydrologic models proposed for the application of KW theory, and the various finite difference schemes for solving the mathematical formulations have been discussed in details in Chapter - III. The availability of data on four test watersheds was described in Chapter - IV. In the current chapter, application of these models onto the four test gauged watersheds will be discussed. As a first step towards it, the procedures adopted for estimating the physiographic parameters, their sensitivity analysis (in general) as well as the rainfall excess computations are described.

5.2 ESTIMATION OF PHYSIOGRAPHIC PARAMETERS

Following is the list of physiographic parameters which are involved in the KW model applications.

(i) Overland slope, (ii) Channel bed slope, (iii) Channel roughness, (iv) Channel geometry (Section), (v) Overland roughness, (vi) Watershed geometry.

Procedures of estimation of the first four parameters are explained in the forthcoming sections. The overland roughness has to be computed for each watershed by trial and is explained in the section thereafter. The watershed geometry as explained in Sectiion 3.9, is taken care of keeping in view the type of physiographic model being handled i.e. lumped or distributed.

5.2.1 Determination of Overland Slope

The overland slope (S_o) for a watershed has been calculated by using Horton's formula (Viessman et al., 1977) which is given as under :

$$S_o = \frac{N_e \Delta h.\text{Sec}\,\theta}{l} \quad \ldots \quad (5.1)$$

The notations, as shown in figure 5.1, are explained below.

N_c = Total number of contour intersections with the horizontal and vertical grid lines

Δh = The contour interval (m)

l = The total length of grid line segments in metres (horizontal and vertical)

θ = The angle between a normal to the contours and the grid.

The term Sec θ is generally dropped to simplify the computations. Accordingly, overland slope (S_o) is written as below (Linsley et al., 1949):

$$S_o = \frac{N_e \,.\, \Delta h}{l} \quad \ldots \quad (5.2)$$

In this case, separate values of average slopes for the horizontal and vertical directions are computed. The mean overland slope (S_o) is computed by taking an average of these two values.

5.2.2 Determination of Channel Slope

As shown in figure 5.1, the channel slope (S) is taken as a ratio of the relief (Δh) (i.e. the elevation difference between the two reference points of the channel) with the channel length (L) and can be written as below :

$$\text{Average channel bed slope, } S = \frac{\text{Total relief } (\Delta h)}{\text{Total length of the channel (L)}} \qquad \ldots (5.3)$$

In lumped configurations, one of the reference point is the channel outlet whereas the other is marked by the remotest upstream end of the channel.

5.2.3 Channel Roughness Coefficient

In the KW theory, channel roughness coefficient (n) is one of the important parameters for routing the flows through the channel. In this work, the Manning's roughness is used as the channel roughness coefficient. From the available description of field investigations (relating to general physiography) its value is picked up from the published records (Chow, 1959).

5.2.4 Channel Geometry

For all the watersheds, the cross sections of the main drainage channels were found to be approximately trapezoidal in shape. Therefore, in this work, the main channel cross section for the adopted physiographic models (Section 3.9) has been considered as trapezoidal in nature. The channel bed width (B) is taken as the average value of the channel bed widths at different sections. Similarly, the side slopes have also been taken

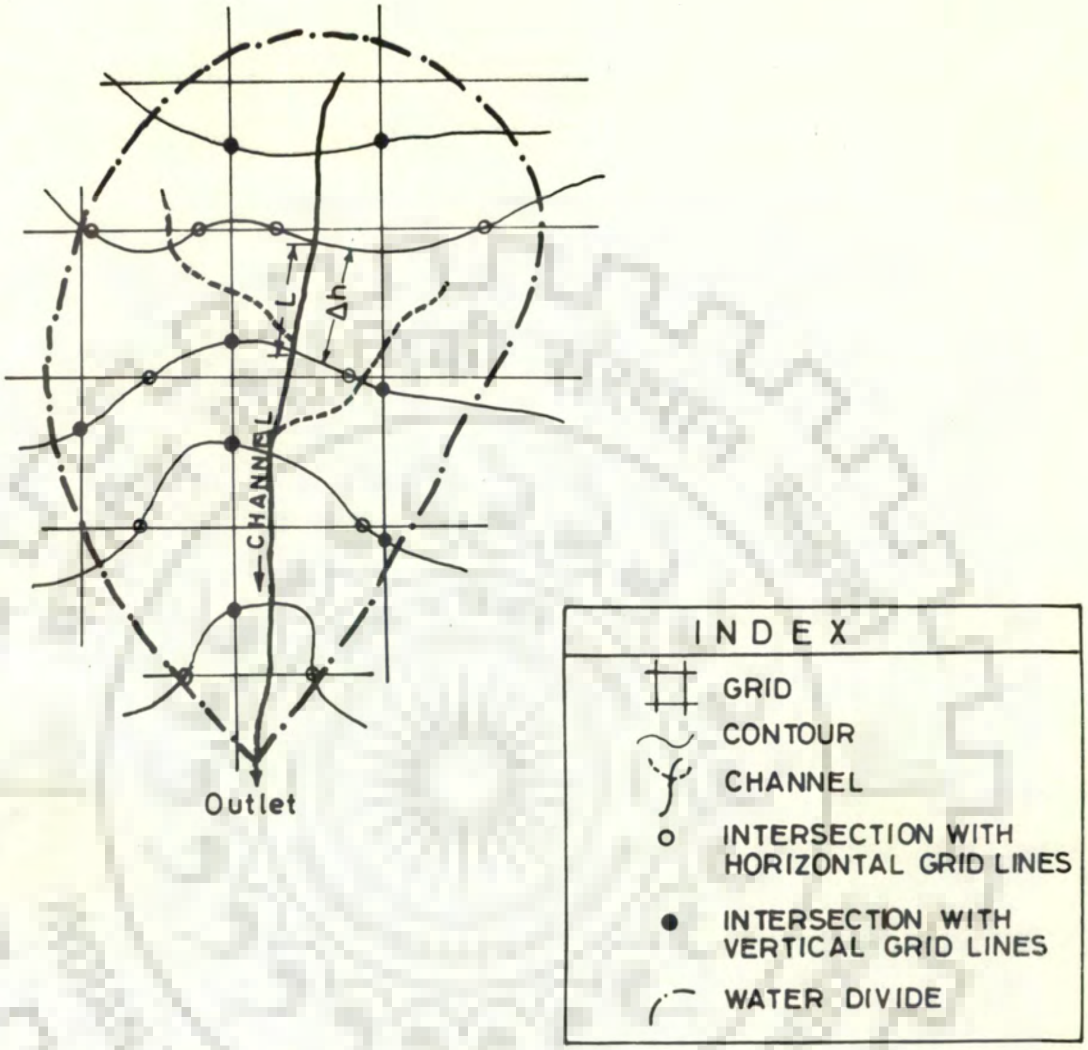


FIG. 5.1 DETERMINATION OF MEAN OVERLAND AND CHANNEL SLOPES

as the average of their values as measured at different section. In case of distributed models, representative values of channel bed widths were taken at the most upstream part of the watershed and these were subjected to some incremental increases for the down stream sections of the channel.

This is to remark that the impact of these physiographic parameters onto the watershed response in general, can better be ascertained through a 'parameter sensitivity analysis'. Some general aspects of this analysis are taken up in the next section.

5.3 SENSITIVITY ANALYSIS OF PHYSIOGRAPHIC PARAMETERS

It is of common knowledge that a number of physiographic parameters interact with the input rainfall function to produce the response i.e. the runoff. This interacton makes the rainfall-runoff process quite complex in nature. In different watersheds, the role of different physographic parameters i.e. the shape, soil type, land use, soil cover etc. may be different. Therefore, there is a need to conduct a sensitivity analysis for various physiographic parameters to ascertain the most effective ones.

A general mathematical treatment for the sensitivity analysis was suggested by McCuen (1973) in the following form:

$$s = \frac{\partial F_o}{\partial F_i} = [f(F_i+\Delta F_i;\ Fj,\ j\neq i)-f(F_1,F_2,\dots,F_n)]/\Delta F_i \quad \dots \quad (5.4)$$

where $F_o = f(F_1, F_2, F_3,\dots,F_n)$ and $i,j = 1,2,3 \dots n$.

The right hand side of above equation indicates that the sensitivity of F_o, to the change in F_i can be derived by incrementing F_i and computing the resulting change in the solution of dependent function F_o.

An inspection of KW equations for its overland phase (Equations 3.9 and 3.15) and for the channel phase (Equations 3.16 and 3.22) suggests that the two dependent functions i.e. discharge per unit width (q) and discharge (Q) (at the outlet) will be the functions of the following physiographic parameters.

$$q = q(N, S_o, t) \qquad \ldots \quad (5.5)$$

and

$$Q = f(N, S_o, n\ , S, B, Z, y_c, t) \qquad \ldots \quad (5.6)$$

Keeping in view the inter-dependence and inter-relationships of the parameters involved, no direct method of differentiation can be used in an explicit manner to satisfy the equation (5.4). Therefore, in this study, the method of perturbation is used to assess the impact of individual parameters on the response function i.e. the discharge. In this method, one parameter is varied while the others are kept constant. The various response functions values so obtained become an index for the effectiveness of the parameter.

5.4 RAINFALL EXCESS COMPUTATIONS

A study of the rainfall and runoff records of the four test watersheds reveals that in general a time lag exists between the starting timings of the rainfall and the runoff. In such cases, the rainfall that had fallen in between this period is assumed as 'initial loss'. The 'effective rainfall duration' is thus considered that period of storm rainfall which had actually contributed towards runoff. Since all the four watersheds are small in size, therefore, the physiographic characteristics relating to the soil type, landuse and soil cover have been assumed to be uniform.

As reported in Chapter - IV, in each test watershed one raingauge was installed. Therefore, the point rainfall data recorded at this rain gauge was taken as the representative value of the input rainfall function for the watershed.

For the three watersheds i.e. of the Railway bridge Nos. 319, 317 and Kachwa watershed, the field drains and main channel happened to be quite shallow and ground water table was found to be deep. Therefore, contributions from the ground water in the form of base flow (or interflow) are nil. Accordingly, the runoff in these three watersheds is totally derived from the rainfall. The watershed area of Bridge No. 719 is some what bigger (= 1400 ha) and traces of water were always seen in the main channel. Therefore, in this watershed, the base flows were accounted for to compute the 'Direct Runoff Hydrographs' (DRH). A linear distribution of base flows was considered between the 'rising point' and the 'point of cessation' in the observed hydrograph.

No detailed infiltration capacity curves for the storm events were available. Therefore, Ø-index approach (i.e assumption of constant rate of abstraction) has been adopted for the computation of rainfall excess distribution.

The KW models as reported in Chapter - III have been applied onto the available hydrologic data recorded on the four watersheds (Chapter - IV). To maintain logical sequences, the proposed model is first applied to the watershed of Bridge No. 319 which has much simpler physiography with a single main channel. Subsequently, brief descriptions of model applications to the other three watersheds are taken up.

5.5 APPLICATION OF PROPOSED MODEL ONTO THE WATERSHED OF BRIDGE No. 319

In order to apply the KW models to the watershed of Bridge No. 319, computations relating to the following would be necessary.

(i) rainfall excess, and

(ii) physiographic parameters and their sensitivities.

As discussed in Section 5.4, the rainfall excess computations were carried out after giving due consideration to the initial loss. Following the Ø-index approach, the time distributions of rainfall excess for all the storm events are computed. Their values are given in Appendix-II-A alongwith the data.

5.5.1 Estimation of Physiographic Parameters (Bridge No. 319)

The topographic details of this watershed are shown in figure 4.1. In this natural watershed, only one main drainage channel exists that too in the central part of the watershed. As discussed in Section 3.9.1, lumped physiographic model is used to compute the model parameters for the application of KW theory. For this purpose, a lumped model of the type given in figure 3.5(a) is adopted for the estimation of parameters.

The schematic representation of this model is shown in figure 5.2. The equivalent watershed has been obtained by dividing equally the total drainage area onto the two sides of 1650 metres long main channel. As discussed in Section 5.2, the physiographic parameters (namely, the slope, channel section, roughness etc.) were computed. the computed values of the physiographic parameters are given in Table 5.1.

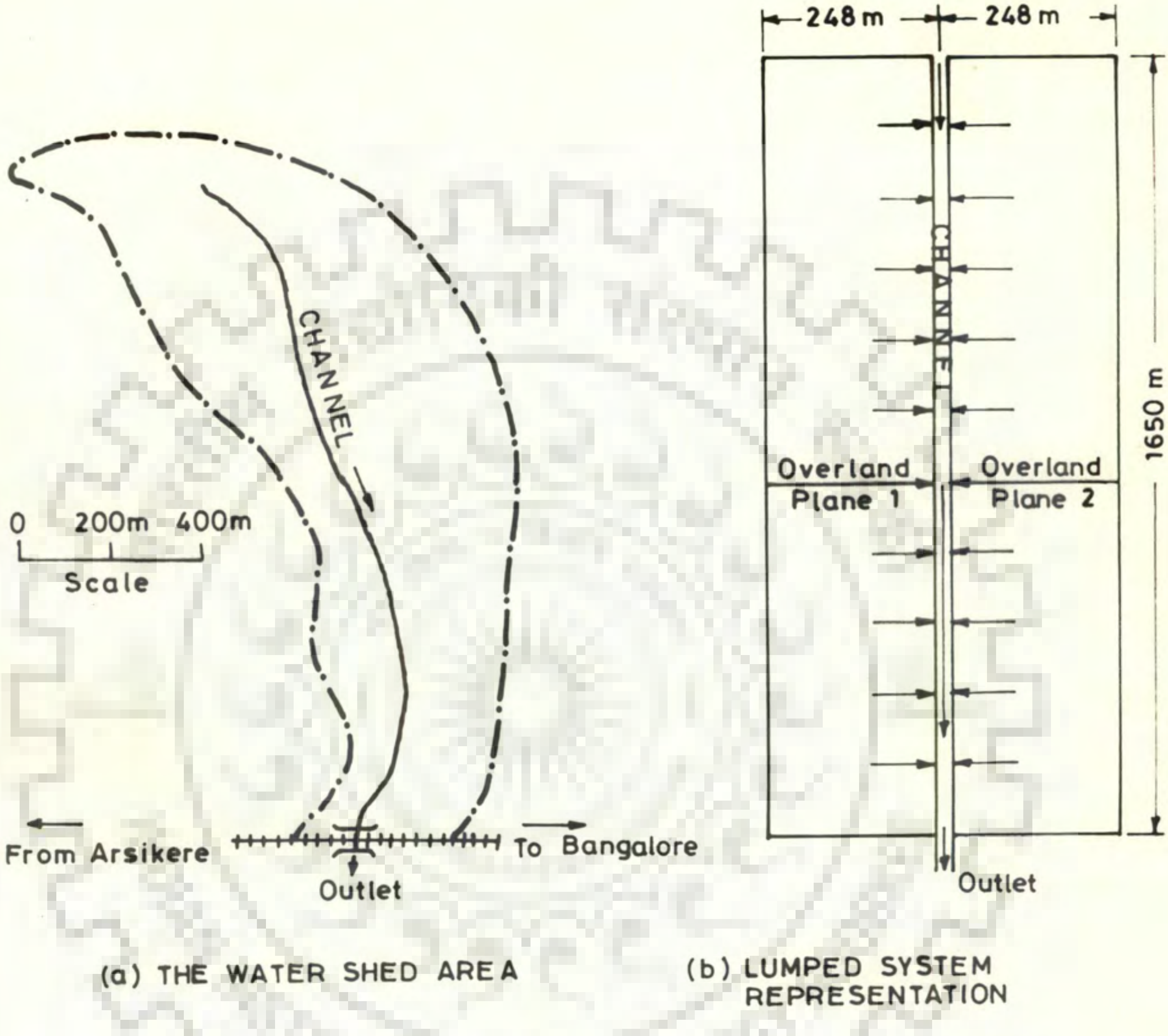


FIG. 5.2 SCHEMATIC REPRESENTATION THROUGH A LUMPED MODEL (BRIDGE NO. 319)

TABLE 5.1 : The Lumped Physiographic Parameter Values (Bridge No. 319)

Sl. No.	Particulars	Unit
(1)	(2)	(3)
1	Area	82.0 hectares
2	Overland (Plane) :	
	(a) Average length (each side)	248.0 metres
	(b) Average slope (each side)	0.092
3	Channel :	
	(a) Average length	1650.0 meters
	(b) Average slope	0.072
	(c) Average roughness	0.035
	(d) Average bed width	3.0 meters
	(e) Averag side slope	2.5 H : 1V

The KW routing parameters α_k and m_k (as dicsussed in details in Section 3.5.1) have been computed for three representative depths of flows (y_c) as 0.25 m, 0.55 m and 0.75 m. This is to mention that for the computation of these parameters only two values of depths are needed along with other related parameters. Therefore, out of these three values paired sets were taken for the computations α_k and m_k and their average values have been used in the computational schemes.

5.5.2 The Sensitivity Analysis of Physiographic Parameters (Bridge No. 319)

In Section 5.3, theoretical aspects of sensitivity analysis of model parameters were discussed. Following the method of perturbaton, the sensitivity analysis was conducted with respect to the following physiographic parameters.

(a) In Overland flow phase (i) Slope (ii) Effective roughness

(b) In channel flow phase (i) Slope, (ii) Manning's roughness, (iii) bed width, (iv) Side slope and (v) depths.

To compute the responses through the application of KW models, the computational scheme (I & II) is preferred. It is because stability and convergence of the scheme remain more-or-less ensured through a proper selection of smaller values of time and space steps.

It is proposed to conduct this analysis with a time step of 2.5 minutes and space step of 4.0 metres for the overland flow and 15.0 metres for the channel flow. However, a detailed analysis to ascertain the ranges over which this scheme remains stable as well as convergent has been carried out in the next section.

For the perturbation analysis, the values of physiographic parameters as worked out in Table 5.1 were varied over a wide range (50 to 150 per cent). The storm dated 5.8.64 is selected for this analysis because of its isolated nature, short duration (50.0 minutes) and nearly uniform intensity. The computations are carried out for a representative overland roughness value of 0.140. The computed values in the perturbation analysis are compared with the following parameters of the observed hydrograph for the flood event under consideration.

(a) Peak discharge = 3.651 cumecs

(b) Time to peak = 50.0 minutes, and

(c) Volume = 117.69 x 100 m^3

A close agreement in the computed and observed values of the volume of the runoff indicates that the computational scheme has taken care of nearly the entire runoff which is produced over the watershed.

The results of the senstivity analysis in respect of the bed width, side slope and depths of the channel are presented in Table 5.2(i), (ii) and (iii). Further, the effect of variations in channel slope and channel roughness are shown in Table 5.3(i) and (ii) respectiviely. A comparison of computed hydrograph parameters with the observed parameters mentioned earlier, reveals that inspite of large variations in the parameter values (50 to 150 per cent), the differences in the response parameters were found to be quite marginal. This suggests that these physiographic parameters are not very sensitive, with regard to their influence on the hydrograph parameters.

In Table 5.4, computed values of hydrograph parameters for different overland slopes and the overland roughness are given. Also as shown in figures 5.3 and 5.4, influence of these parameters on the peak discharge and the time to peak was found to be substantial. This led to a conclusion that the overland parameters namely the slope and roughness were very sensitive. The overland slope has been computed through a vigorous analysis as mentioned in Section 5.2.1 by selecting small contour intervals 3 m (10 ft) to 15 m (50 ft) . Therefore, all care should be taken for a proper accounting of the overland roughness values.

Through various sets of calculations, efforts to determine one single represnetative overland roughness value did not meet with success. Therefore, trial and error procedure was adopted to establish the following :

(i) Overland roughness values, and

(ii) Suitability of the two computational schemes (discussed in Section 3.8.2 to 3.8.4).

Detailed discussions and analysis into these aspects is reported in the following sections.

5.5.3 **Estimation of Effective Overland Roughness (Bridge No. 319)**

As reported in the previous section, no single unique value of the effective overland roughness could successfully be determined which could have produced well matched hydrographs for various storm events. Therefore, following the trial and

TABLE 5.2 : Sensitivity of (i) Channel Bed Width, (ii) Channel Side Slope and (iii) Channel Depths (Bridge No. 319, Storm dated : 5.8.1964)

(i) Effect of Channel Bed Width :

Sl. No.	Channel bed width (m)	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3) x 100
(1)	(2)	(3)	(4)	(5)	(6)
1	1.50	50.0	3.650	42.5	113.78
2	2.25	75.0	3.646	42.5	113.73
3	3.00	100.0	3.642	42.5	113.66
4	3.75	125.0	3.640	42.5	113.64
5	4.50	150.0	3.637	42.5	113.61

(ii) Effect of Channel Side Slope :

(1)	Channel Side Slope (2)	(3)	(4)	(5)	(6)
1	1.25	50.0	3.643	42.5	113.670
2	1.88	75.0	3.643	42.5	113.668
3	2.50	100.0	3.642	42.5	113.662
4	3.12	125.0	3.642	42.5	113.661
5	3.75	150.0	3.642	42.5	113.659

(iii) Effect of Channel Depths :

(1)	Channel Depths (m) (2)	(3)	(4)	(5)	(6)
1	0.25, 0.30 and 0.40		3.642	42.5	113.665
2	0.25, 0.55 and 0.75		3.642	42.5	113.662
3	1.25, 1.55 and 1.75		3.644	42.5	113.665
4	1.00, 2.55 and 3.75		3.643	42.5	113.661

TABLE 5.3 : Sensitivity of (i) Channel Slope and (ii) Channel Roughness
(Bridge No. 319, Storm dated : 5.8.1964)

(1) Effect of Channel Slope :

Sl. No.	Channel Slope	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3) x 100
(1)	(2)	(3)	(4)	(5)	(6)
1	0.036	50.0	3.605	42.5	113.40
2	0.054	75.0	3.630	42.5	113.58
3	0.072	100.0	3.642	42.5	113.66
4	0.090	125.0	3.649	42.5	113.75
5	0.108	150.0	3.652	42.5	113.80

(ii) Effect of Channel Roughness :

(1)	Channel Roughness (2)	(3)	(4)	(5)	(6)
1	0.0175	50.0	3.677	40.0	114.0
2	0.026	75.0	3.655	40.0	113.83
3	0.035	100.0	3.642	42.5	113.66
4	0.044	125.0	3.621	42.5	113.50
5	0.0525	150.0	3.601	45.0	113.35

TABLE 5.4 : Sensitivity of (i) Overland Slope and (ii) Overland Roughness
(Bridge No. 319, Storm dated : 5.8.1964)

(1) Effect of Overland Slope :

Sl. No.	Overland slope	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3) x 100
(1)	(2)	(3)	(4)	(5)	(6)
1	0.046	50.0	3.329	50.0	111.16
2	0.069	75.0	3.500	45.0	112.79
3	0.092	100.0	3.642	42.5	113.662
4	0.115	125.0	3.756	40.0	114.22
5	0.138	150.0	3.849	37.5	114.62

(ii) Effect of Overland Roughness :

(1)	Overland Roughness (2)	(3)	(4)	(5)	(6)
1	0.070	50.0	4.315	32.5	116.08
2	0.105	75.0	3.938	37.5	114.95
3	0.140	100.0	3.642	42.5	113.662
4	0.175	125.0	3.434	47.5	112.18
5	0.210	150.0	3.260	52.5	110.61

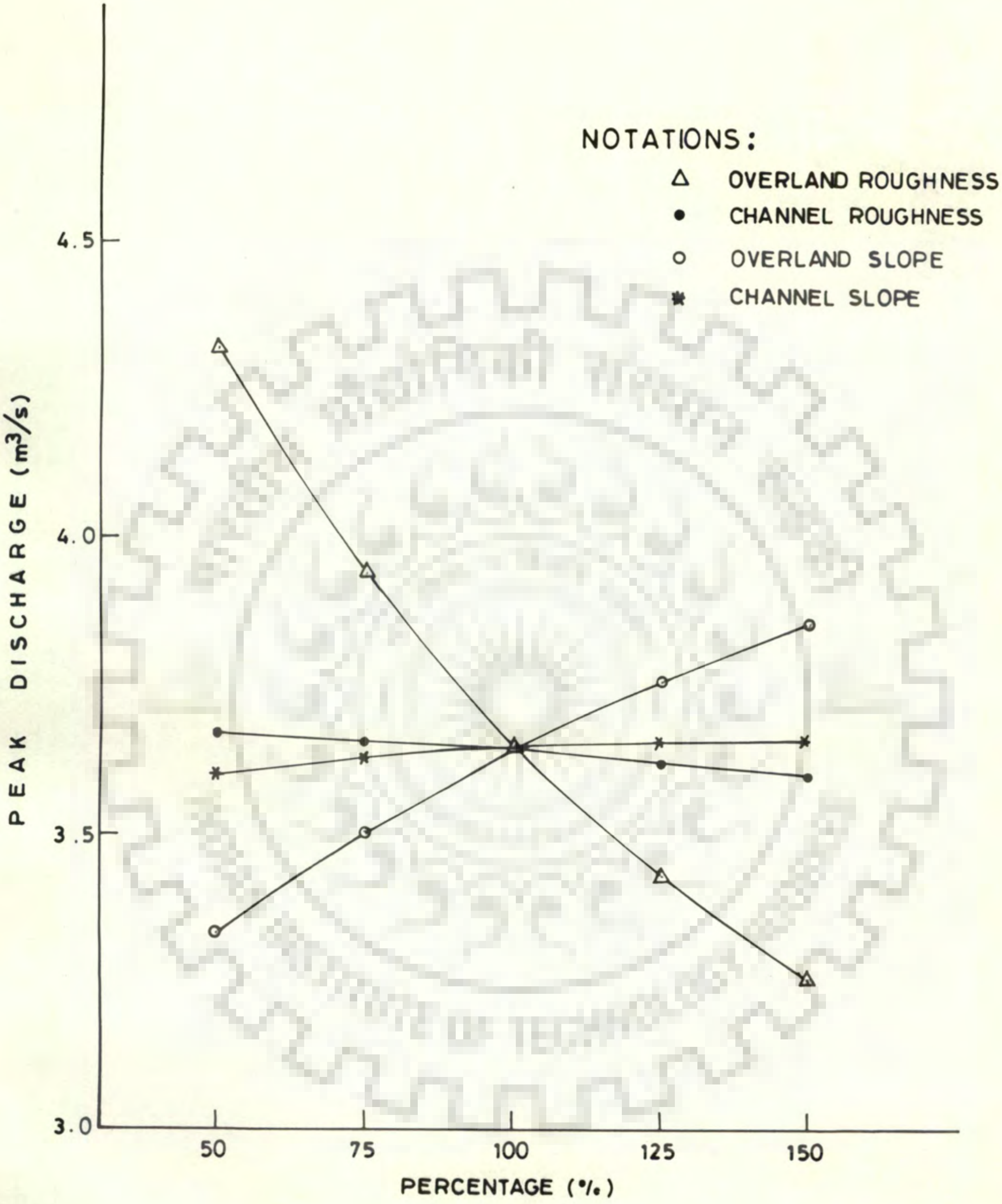


FIG. 5.3 SENSITIVITY OF OVERLAND AND CHANNEL ROUGHNESSES AND SLOPES w.r.t. PEAK DISCHARGES (BRIDGE NO. 319)

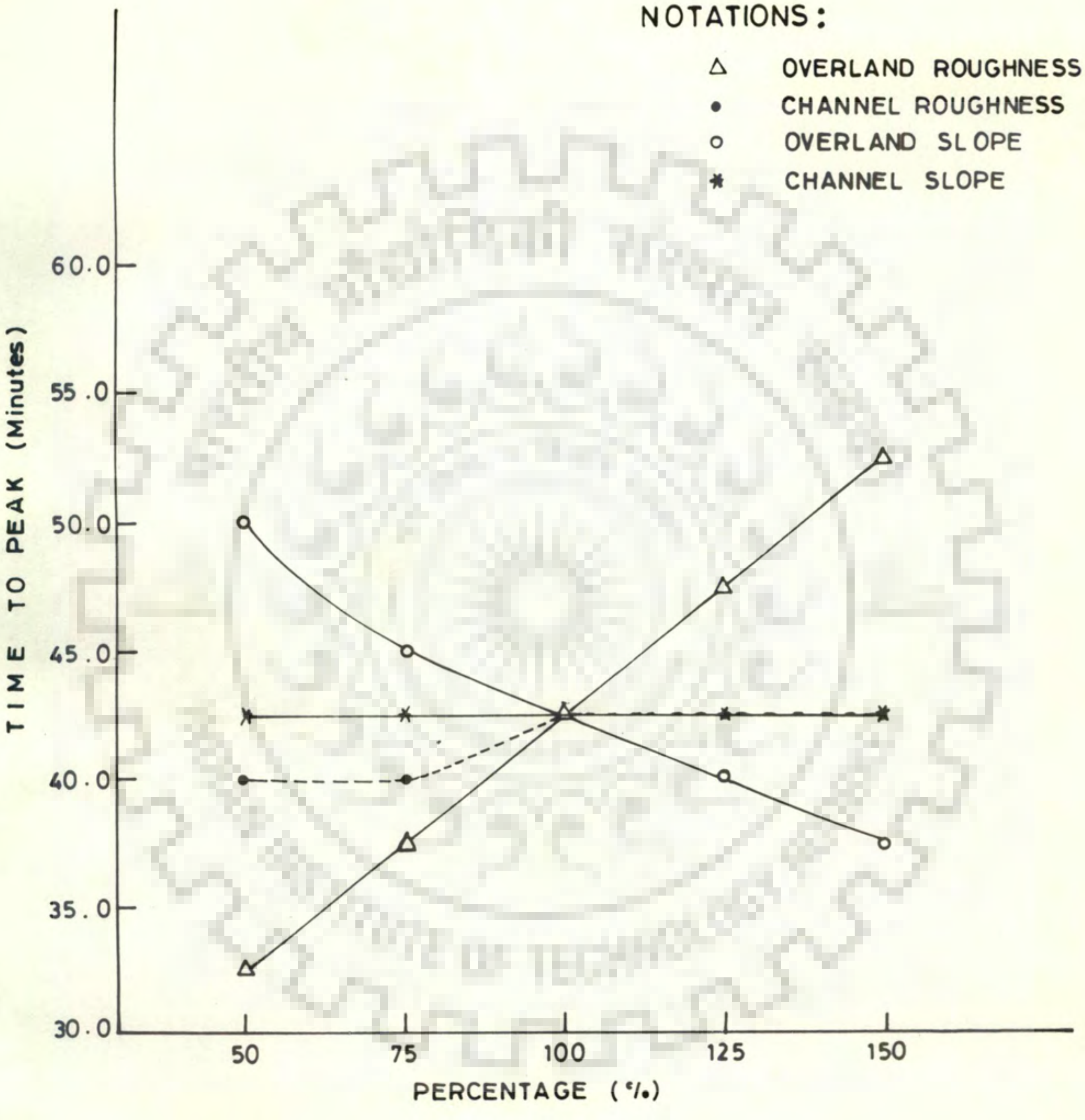


FIG. 5.4 SENSITIVITY OF OVERLAND AND CHANNEL ROUGHNESSES AND SLOPES w.r.t. TIME TO PEAKS (BRIDGE NO. 319)

error procedure, the exact value of effective overland roughnesses were tried withiin a range of 0.05 to 0.450. The upper and the lower limits of this range have been chosen from the description of the topography and landuse of this watershed. As Singh (1975a) has concluded, "For optimizing and estimating parameters, there is no need to consider the entire hydrograph in constructing an objective function. If the model structure is representative of physical reality, a peak matching criterion may suffice to reproduce and predict the entire hydrograph". This criteria has been adopted in this analysis.

For all the 10 storm events, the values of the unique effectiive overland roughnesses were determined which produced well matched peak discharges. For this purpose, computational scheme (I & II) was used for applying the KW theory for very small values of time and space steps (Δt = 2.5 minutes, Δx_o =4.0 m for overland and Δx_c = 15.0 m for channel). The lumped physiographic model (Section 3.9.1) was used for this case. The best suited computed effective overland roughnesses for the 10 storm events are givien in Table 5.5.

It may be seen from Table 5.5 that over the four monsoon months, the effective overland roughnesses varied from nearly 0.05 (for the months of July and October) to a very high value of 0.420 in the month of September. A plot of effective overland roughnesses Vs. time was tried and the same is reproduced in figure 5.5. The plot indicates a systematic rise and fall in overland roughness parameter values during the monsoon months. From this curve the ranges of the effective overland roughness coeffiiciient values for the four monsoon months were found

TABLE 5.5 : Computed Effective Overland Roughnesses (Bridge No. 319)

Sl. No.	Storm Date	Effective Overland Roughness (N)	Peak Discharge (m^3/s)	
			Observed	Computed
(1)	(2)	(3)	(4)	(5)
1	14.10.62	0.052	9.405	7.980
2	4.11.62	0.052	7.301	6.990
3	17.7.63	0.050	3.339	3.113
4	7.10.63	0.106	1.500	1.500
5	5.8.64	0.140	3.651	3.642
6	7/8.9.64	0.380	4.471	4.500
7	8/9.9.64	0.395	6.281	6.320
8	9.7.88	0.050	2.775	2.784
9	8.8.88	0.230	0.906	0.924
10	11.9.88	0.420	5.891	6.130

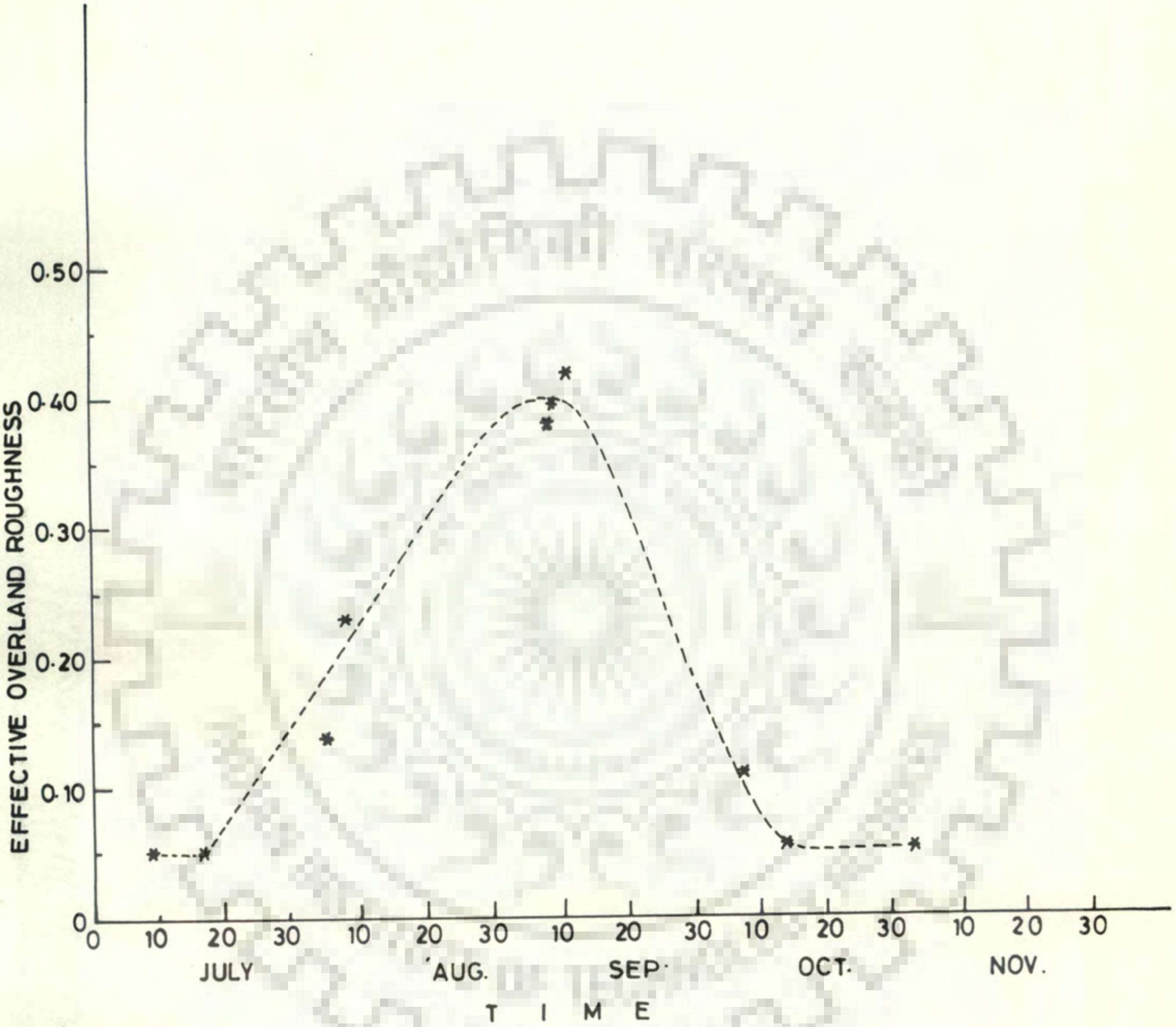


FIG. 5.5 VARIATION OF EFFECTIVE OVERLAND ROUGHNESS WITH TIME (BRIDGE NO.319)

TABLE 5.6 : Range of Effective Overland Roughnesses During the Four Monsoon Months (July to October)

(Bridge No. 319)

Sl. No.	Months	Range of Effective Overland Roughnesses	
		From	To
(1)	(2)	(3)	(4)
1	July	0.050	0.150
2	August	0.150	0.375
3	September	0.200	0.420
4	October	0.052	0.200

out and the same are given in Table 5.6. Having identified the various physiographic parameters as well as the rainfall excess disitribution, it is now possible to verify the applicability and relative advantages and disadvantages of the computational schemes which are given in Sections 3.8.2 to 3.8.4.

5.5.4 Applicability of Different Computational Schemes (Bridge No. 319)

Initially three computational schemes were identified (Sectons 3.8.2 to 3.8.4). As already reported (Section 3.8) the schemes I and II happen to be complementary to each other. These are clubbed together and identified as Scheme (I & II). The other scheme was termed as Scheme III. The various physiographic parameters worked out in the previous sections, were determined by using scheme (I & II) for very small time and space steps (i.e. Δt = 2.5 minutes, Δx_o = 4.0 m and Δx_c = 15.0 m). Validity of this criteria also needs confirmation. Further, there is a need to identify the ranges of the two step widths within which the schemes are to remain stable as well as convergent.

For this purpose, three storm events were selected for the analysis. One storm event dated 4.11.62 corresponds to the period having the lowest value of the effective overland roughness (i.e N = 0.052), whereas the second event of dated 5.8.64 belongs to the period of an effective overland roughness value as 0.140 (i.e. of medium order). The third storm event dated 7/8.9.64 corresponds to very large value of N (i.e. N = 0.380).

The following three sets of computatiions were carried out to decide :

(i) optimal time step (Δt),

(ii) maximum overland space step (Δx_o) and

(iii) best suited channel space step (Δx_c)

over which the schemes are to remain stable and convergent.

In the first set of calculations several time stpes have been chosen, starting with a very small initial value of 2.5 minutes. The other values were taken as integer multiples of 2.5 minutes i.e. 5.0, 10.0, 15.0 and in one case 30.0 minutes. For these runs the space step length for the overland and the channel flows is kept fixed at 4.0 metres and 15.0 metres respectively. A comparison of observed and computed hydrograph parameters [i.e peak discharge (Q_p), time to peak (t_p) and volume] for the two schemes along with the stability for the scheme (I & II) is shown in Table 5.7. It may be seen from this Table that the scheme (I & II) generally remains stable upto a time step 10 minutes for all the three storm events. Best convergence was seen for Δt = 2.5 minutes. Scheme III though stable did not converge satisfactorily even for a very small time step of 2.5 minutes.

The second set of computations were carried out to establish the range for the overland space step (Δx_o) over which the scheme (I & II) remains stable as well as convergent. The time step Δt and the channel space step (Δx_c) were fixed as 2.5 minutes and 15.0 metres respectively. Following the stability criteria mentioned in Section 3.8, the computations are carried out for all the three storm events and for both the schemes. The results are given in Table 5.8. The scheme (I & II) was found to be stable upto the step lengths of 4.0 to 8.0 metres in all the three storm events. Beyond it, the scheme showed unstable behaviour for the recession part of the hydrographs. The convergence of scheme III was not satisfactory for all the three storm events.

In the third set of calculations the time step Δt was kept as 2.5 minutes, overland Δx_o was fixed in 4.0 metres and the channel step length was varied from 15.0 to 150.0 metres. The computed results are presented in Table 5.9. For all the three storm events scheme (I & II) was found to be stable for all the space step lengths upto 150.0 metres. In this case too, the convergence of scheme III was not satisfactory.

A close inspection of the result obtained in Table 5.7, 5.8 and 5.9, leads to the following conclusions.

(a) Scheme III, though inherently stable yet did not show satisfactory convergence to peak values even for very small time step (i.e. Δt = 2.5 minutes) and small overland and channel space step of 4.0 metres and 15.0 metres respectively.

(b) The scheme (I & II) was found to be stable upto a time step of 10.0 minutes and overland space steps upto 4.0 to 8.0 metres. It was stable for all the channel space steps upto 150.0 metres.

The above analysis thereby confirms that the earlier exercises conducted for establishing the sensitivity of physiographic parameters (section 5.5.2) and for computation of effec-

TABLE 5.8 : Test Runs for Establishing the Range of Overland length Step (Δx_o) from Stability and Convergence Points of View for Computational Schemes (I & II) and III (Bridge No. 319)

Sl. No.	Overland Step (Δx_o) (m)	Observed: Peak (Q_p) (m^3/s)	Observed: Time to peak (t_p) (min)	Observed: Volume (m^3) $\times 10^2$	Computed Hydrograph Parameters, Scheme (I & II): Stability	Scheme (I & II): Q_p (m^3/s)	Scheme (I & II): t_p (min)	Scheme (I & II): Volume (m^3) x 100	Scheme III: Q_p (m^3/s)	Scheme III: t_p (min)	Scheme III: Volume (m^3) x 100
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
(a) Storm event dated 4.11.62											
	-	7.301	30.0	127.83	-	-	-	-	-	-	-
1	2.00	-	-	-	Stable	6.977	30.0	125.60	5.632	30.0	105.79
2	4.00	-	-	-	Stable	6.990	30.0	125.72	5.629	30.0	105.65
3	8.00	-	-	-	Stable	7.016	30.0	125.32	5.622	30.0	105.40
4	15.50	-	-	-	Unstable	-	-	-	5.602	30.0	104.97
5	30.00	-	-	-	Unstable	-	-	-	5.547	30.0	104.20
(b) Storm event dated 5.8.64											
	-	3.651	50.0	117.69	-	-	-	-	-	-	-
1	2.00	-	-	-	Stable	3.608	42.5	113.31	3.525	40.0	108.534
2	4.00	-	-	-	Stable	3.642	42.5	113.662	3.487	40.0	108.25
3	8.00	-	-	-	Unstable	-	-	-	3.417	40.0	107.70
(c) Storm event dated 7/8.9.64											
	-	4.471	60.0	234.66	-	-	-	-	-	-	-
1	2.00	-	-	-	Stable	4.45	60.0	217.36	4.546	60.0	211.29
2	4.00	-	-	-	Stable	4.50	60.0	217.27	4.472	60.0	210.92
3	8.00	-	-	-	Unstable	-	-	-	4.342	60.0	210.73

TABLE 5.7 : Test Runs for Establishing the Range of Time Step (Δt) from Stability and Convergence Points of View for Computational Schemes (I & II) and III (Bridge No. 319)*

(a) Storm event dated 4.11.62

Sl. No.	Time Step (Δt) (min)	Observed			Computed Hydrograph Parameters						
		Peak (Q_p) (m^3/s)	Time to peak (t_p) (min)	Volume (m^3) $\times 10^2$	Stability	Scheme (I & II) Q_p (m^3/s)	Scheme (I & II) t_p (min)	Scheme (I & II) Volume (m^3) within the base time x 100	Scheme III Q_p (m^3/s)	Scheme III t_p (min)	Scheme III Volume (m^3) within the base time x 100
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	7.301	30.0	127.83							
1	2.5	-	-	-	Stable	6.990	30.0	125.72	5.629	30.0	105.65
2	5.0	-	-	-	Stable	6.655	30.0	125.46	4.613	30.0	91.15
3	10.0	-	-	-	Stable	6.269	30.0	125.16	3.299	30.0	74.40
4	15.0	-	-	-	Unstable	-	-	-	-	-	-

(b) Storm event dated 5.8.64

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	3.651	50.0	117.69							
1	2.5	-	-	-	Stable	3.642	42.5	113.662	3.487	40.0	108.24
2	5.0	-	-	-	Stable	3.391	45.0	113.19	3.035	40.0	100.14
3	10.0	-	-	-	Stable	3.103	50.0	112.65	2.426	40.0	87.54
4	15.0	-	-	-	Unstable	-	-	-	-	-	-

(c) Storm event dated 7/8.9.64

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	4.471	60.0	234.66							
1	2.5	-	-	-	Stable	4.450	60.0	217.38	4.472	60.0	210.92
2	5.0	-	-	-	Stable	4.232	60.0	217.35	4.031	60.0	201.17
3	10.0	-	-	-	Stable	3.884	60.0	217.11	3.418	60.0	185.19
4	15.0	-	-	-	Stable	3.659	60.0	214.17	2.978	60.0	157.365
5	30.0	-	-	-	Unstable	-	-	-	-	-	-

* As mentioned in Section 2.7, applicability of KW theory has been checked through Kinematic Flow Number which always remained within permissible limits. One sample calculation set is shown in Ann.-I on page 308.

TABLE 5.9 : Test Runs for Establishing the Range of Channel length Step (Δx_c) from Stability and Convergence Points of View for Computational Schemes (I & II) and III (Bridge No. 19)

(a) Storm event dated 4.11.62

Sl. No.	Channel Step (Δx_c) (m)	Observed Q_p (m^3/s)	Observed t_p (min)	Scheme (I & II) Stability	Scheme (I & II) Q_p (m^3/s)	Scheme (I & II) t_p (min)	Scheme (I & II) Volume m^3 x 100	Scheme III Q_p (m^3/s)	Scheme III t_p (min)	Scheme III Volume (m^3) x 100
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
-	-	7.301	30.0	-	-	-	-	-	-	-
1	15.0	-	-	Stable	6.990	30.0	125.72	5.629	30.0	105.65
2	25.0	-	-	Stable	6.993	30.0	125.73	5.626	30.0	105.61
3	50.0	-	-	Stable	6.999	30.0	125.74	5.619	30.0	105.51
4	75.0	-	-	Stable	7.000	30.0	125.74	5.612	30.0	105.41
5	150.0	-	-	Stable	7.020	30.0	125.745	5.590	30.0	105.12

(b) Storm event dated 5.8.64

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
-	-	3.651	50.0	-	-	-	-	-	-	-
1	15.0	-	-	Stable	3.642	42.5	113.662	3.487	40.0	108.25
2	25.0	-	-	Stable	3.643	42.5	113.667	3.484	40.0	108.21
3	50.0	-	-	Stable	3.645	42.5	113.67	3.478	40.0	108.09
4	75.0	-	-	Stable	3.648	42.5	113.68	3.473	40.0	107.99
5	150.0	-	-	Stable	3.656	42.5	113.68	3.457	42.5	107.667

(c) Storm event dated 7/8.9.64

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
-	-	4.471	60.0	-	-	-	-	-	-	-
1	15.0	-	-	Stable	4.500	60.0	217.28	4.472	60.0	210.92
2	25.0	-	-	Stable	4.508	60.0	217.29	4.470	60.0	210.83
3	50.0	-	-	Stable	4.509	60.0	217.35	4.465	60.0	210.618
4	75.0	-	-	Stable	4.511	60.0	217.38	4.459	60.0	210.403
5	150.0	-	-	Stable	4.512	60.0	217.39	4.450	60.0	210.393

tive overland roughnesses (Section 5.5.3) stand vindicated as the time step 2.5 minutes and the overland and the channel space steps values were adopted as 4.0 and 15.0 metres respectively. The best stability and convergence were found for these values, therefore, there is no need for any reconsideration.

All the analysis pertaining to the parameter estimation in this section as well as in the earlier sections were based on the concept of 'peak matching criteria'. There is a need to test that the computed parameters do reproduce the hydrographs satisfactorily. This text is performed in the following section.

5.5.5 Application of KW Theory to the Proposed Distributed Parameter Model (Bridge No. 319)

As discussed in Section 3.9.2, the distributed parameter model for the watershed of Bridge No. 319 has been obtained by dividing the watershed into 7 subwatershed areas as shown in figure 5.6(a). The subwatersheds were delineated by following the general drainage patterns of the overland flow. For schematic repesentation the subareas are approximated through a rectangular plane as shown in figures 5.6(b). The physiographic parameters for the subareas so formed have been worked out in accordance with the concepts discussed in Section 5.2. The measured and computed values of these parameters are given in Table 5.10. The subwatershed areas which are located on the right bank of the channel are marked with 'A' whereas those located on left bank are marked with 'B'. The channel bed widths at different sections were of the order of 3.0 metres. Therefore, the bed widths are given a variation from 2.5 to 3.5 metres. The parameters α_k and m_k are computed in the computational schemes as detailed in Section 5.5.1.

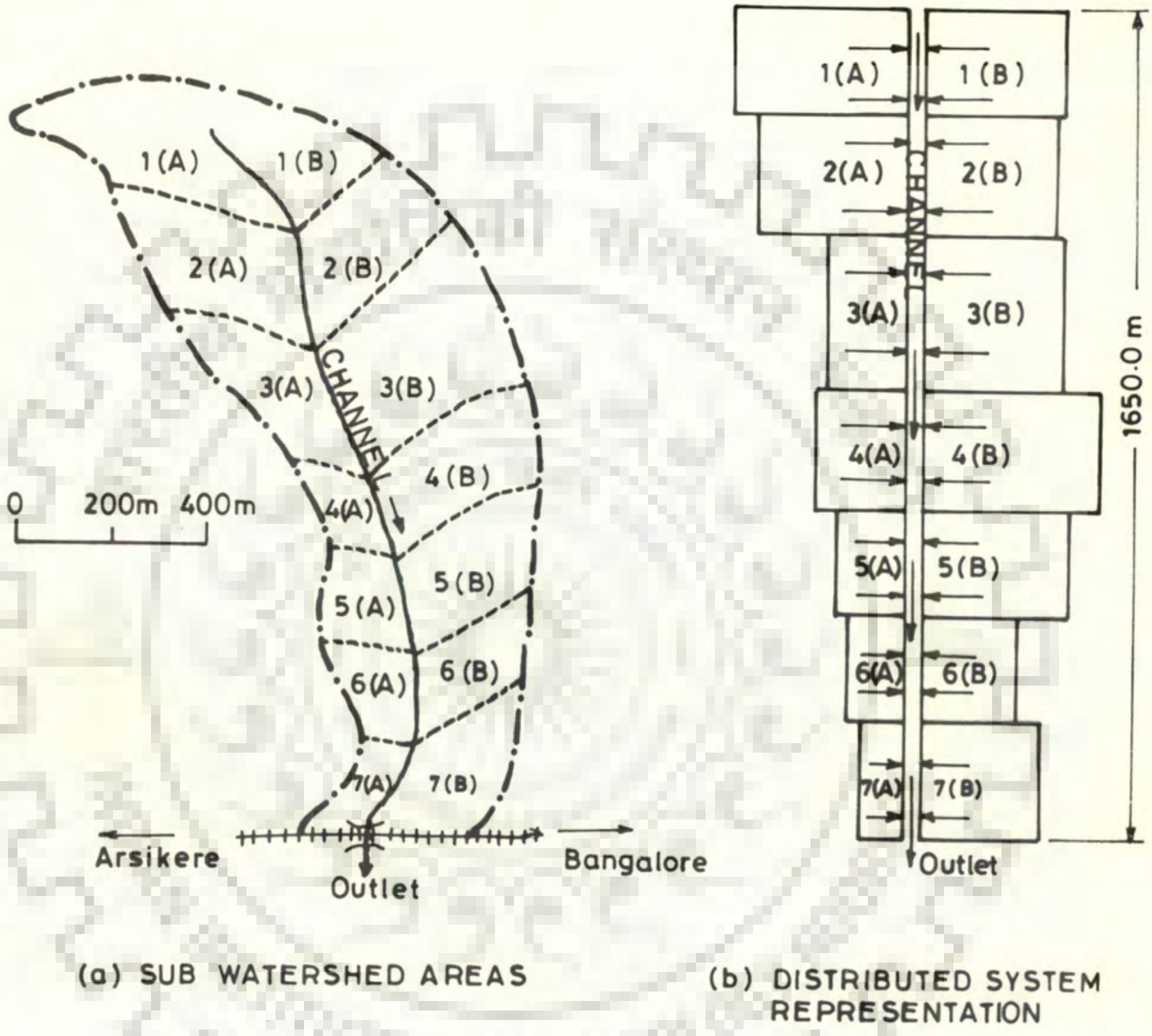


FIG. 5. 6 SCHEMATIC REPRESENTATION THROUGH A DISTRIBUTED MODEL (BRIDGE NO. 319)

TABLE 5.10 : Computed Physiographic Parameter Values for the Distributed System of Bridge No. 319

Sl. No. of sub watershed	Overland length (m)	Right bank sub watershed areas (A) (ha)	Left bank sub watershed areas (B) (ha)	Overland slope	Channel length (m)	Channel slope	Channel bed width (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1(A)	368.0	7.728	-	0.291	210.0	0.123	2.5
1(B)	296.0	-	6.216	0.282			
2(A)	324.0	7.776	-	0.212	240.0	0.116	2.5
2(B)	280.0	-	6.720	0.259			
3(A)	184.0	5.520	-	0.167	300.0	0.061	3.0
3(B)	292.0	-	8.760	0.266			
4(A)	200.0	4.800	-	0.080	240.0	0.0126	3.0
4(B)	368.0	-	8.832	0.270			
5(A)	156.0	3.276	-	0.067	210.0	0.036	3.0
5(B)	312.0	-	6.552	0.261			
6(A)	132.0	2.772	-	0.074	210.0	0.0145	3.5
6(B)	204.0	-	4.284	0.192			
7(A)	104.0	2.496	-	0.026	240.0	0.0127	3.5
7(B)	260.0	-	6.240	0.027			

The effective overland roughnesses have been computed in Section 5.5.3 and are given in Table 5.5. As remarked in Section 5.5.4 the following values of KW parameters are used for computing the discharges at the outlet.

(i) Time step, Δx = 2.5 minutes,

(ii) Overland space step, Δx_o = 4.0 metres and

(iii) Channel space step Δx_c = 15.0 metres.

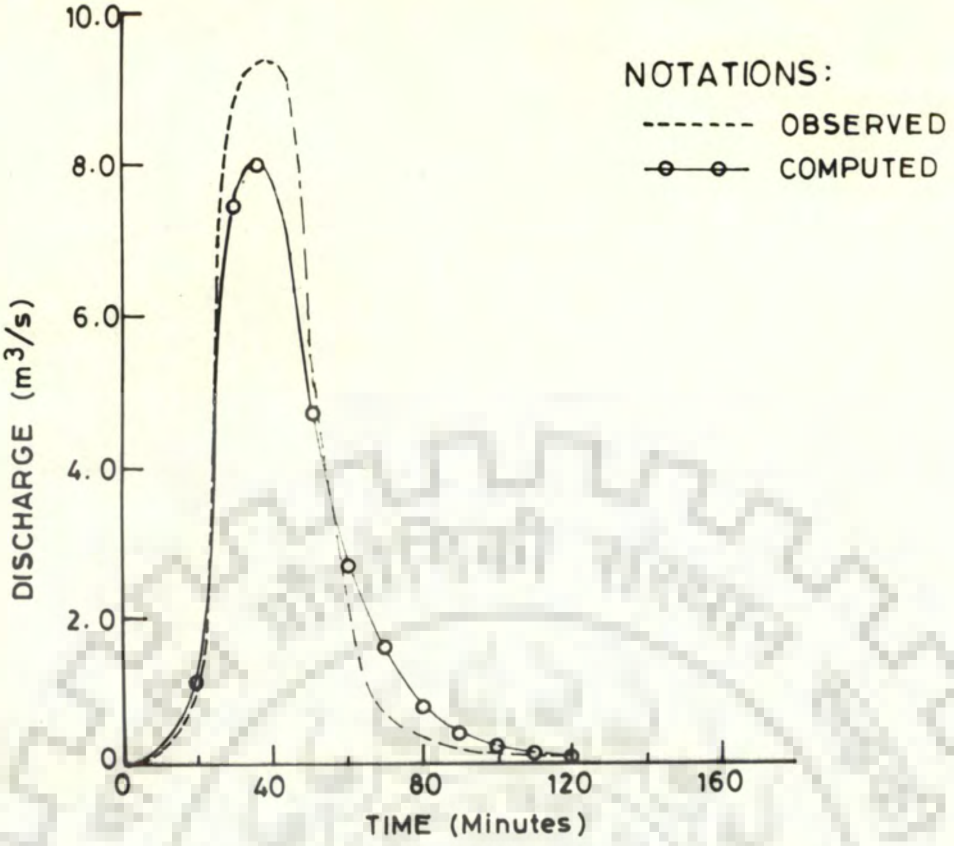
The scheme (I & II) has been used for the runoff computation. As shown in previous section for these values of the parameters, the scheme was found to be stable as well as convergent. The scheme III (Section 3.8.4), since was not found to be convergent has been dropped.

The rainfall excess function for all the 10 storm events has already been computed and given in Appendix-II. The computer programme for KW model of scheme (I & II) (Appendix-I) was run on DECSYSTEM-20 computer. A comparison of the observed and the computed hydrograph parameters is presented in Table 5.11. Also the computed hydrographs have been compared with the respective observed hydrographs. This comparison is shown in figures 5.7 throuh 5.11. The plot of observed and computed values of the peak discharges and the time to peaks are shown in figure 5.12. Further in columns 7 through 9 of Table 5.11, the observed, computed and percentage errors in volumes are given. It may be seen that the errors lie in a range from 0.98 per cent to 6.84 per cent. This is well within the acceptable limits for such an analysis.

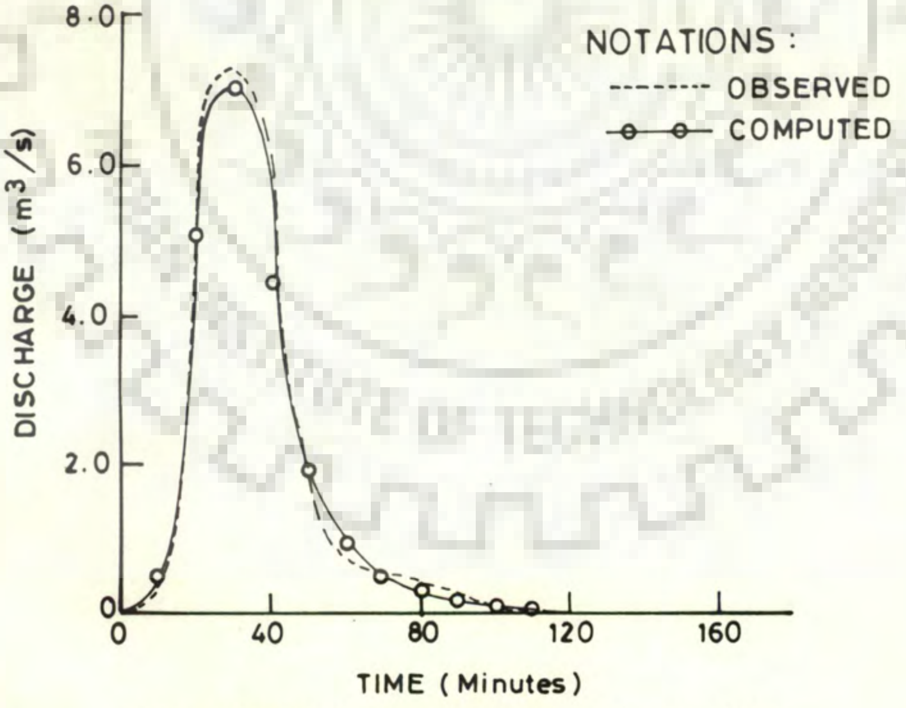
TABLE 5.11 : Observed and Computed Hydrograph Parameters Using KW Model Scheme (I & II)

(Bridge No. 319)

Sl. No.	Storm Date	Peak Discharge (m^3/s)		Time to Peak (min)		Volume (m^3) x 100		Volume error (%)	Model Efficiency R^2 (%)
		Observed	Computed	Observed	Computed	Observed	Computed		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	14.10.62	9.405	7.938	40.0	32.50	170.844	168.90	1.13	90.1
2	4.11.62	7.301	6.950	30.0	30.0	127.820	125.96	1.46	99.1
3	17.7.63	3.339	3.212	20.0	20.0	50.420	48.920	2.90	93.5
4	7.10.63	1.50	1.665	40.0	40.0	45.834	43.725	4.60	76.3
5	5.8.64	3.651	3.758	50.0	40.0	117.69	113.783	3.31	68.8
6	7/8.9.64	4.471	4.598	60.0	57.5	234.66	218.595	6.84	96.4
7	8/9.9.64	6.281	6.612	80.0	75.0	324.10	305.13	5.85	62.3
8	9.7.88	2.775	2.790	60.0	40.0	157.01	155.475	0.98	79.5
9	8.8.88	0.906	1.085	60.0	55.0	49.18	-	-	90.5
10	11.9.88	5.891	6.379	90.0	62.5	314.51	305.355	2.9	70.8

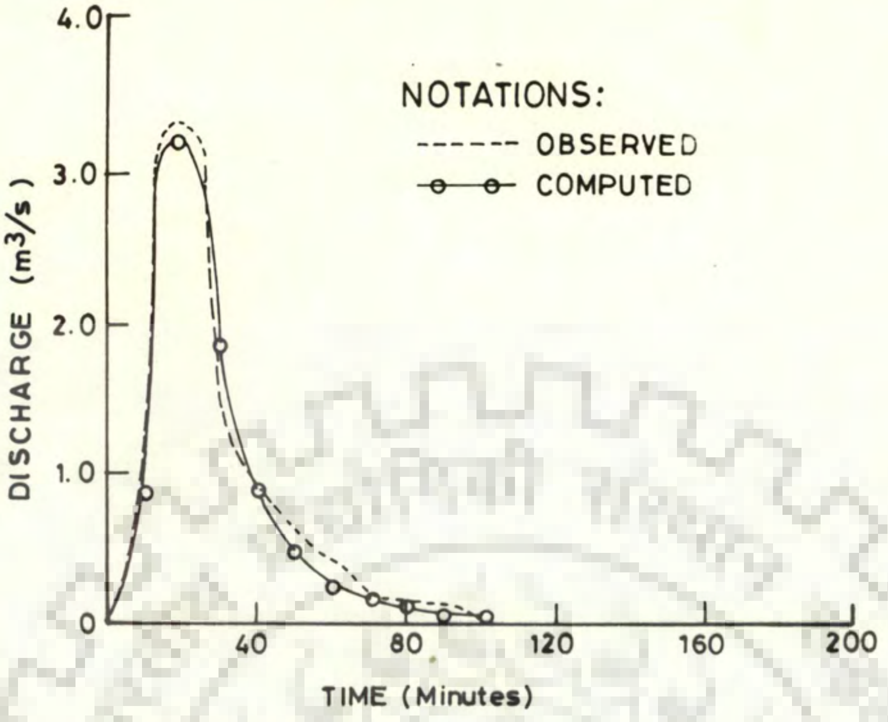


(a) STORM EVENT, DATED : 14 · 10 · 1962

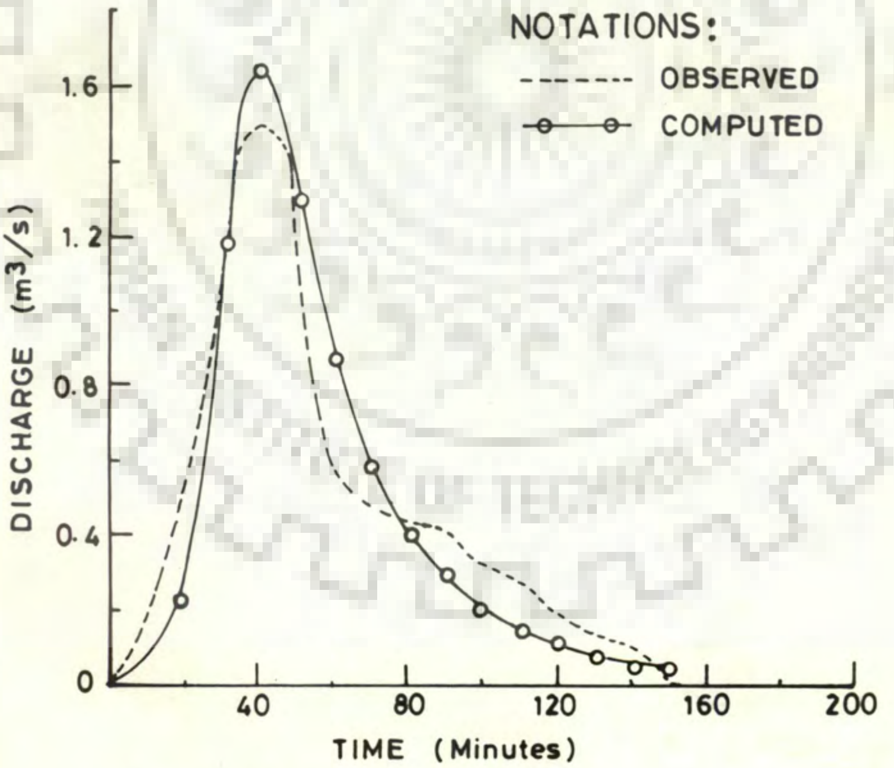


(b) STORM EVENT, DATED : 4 · 11 · 1962

FIG. 5.7 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 319)



(a) STORM EVENT , DATED : 17 . 7 . 1963



(b) STORM EVENT , DATED : 7 . 10 . 1963

FIG. 5.8 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 319)

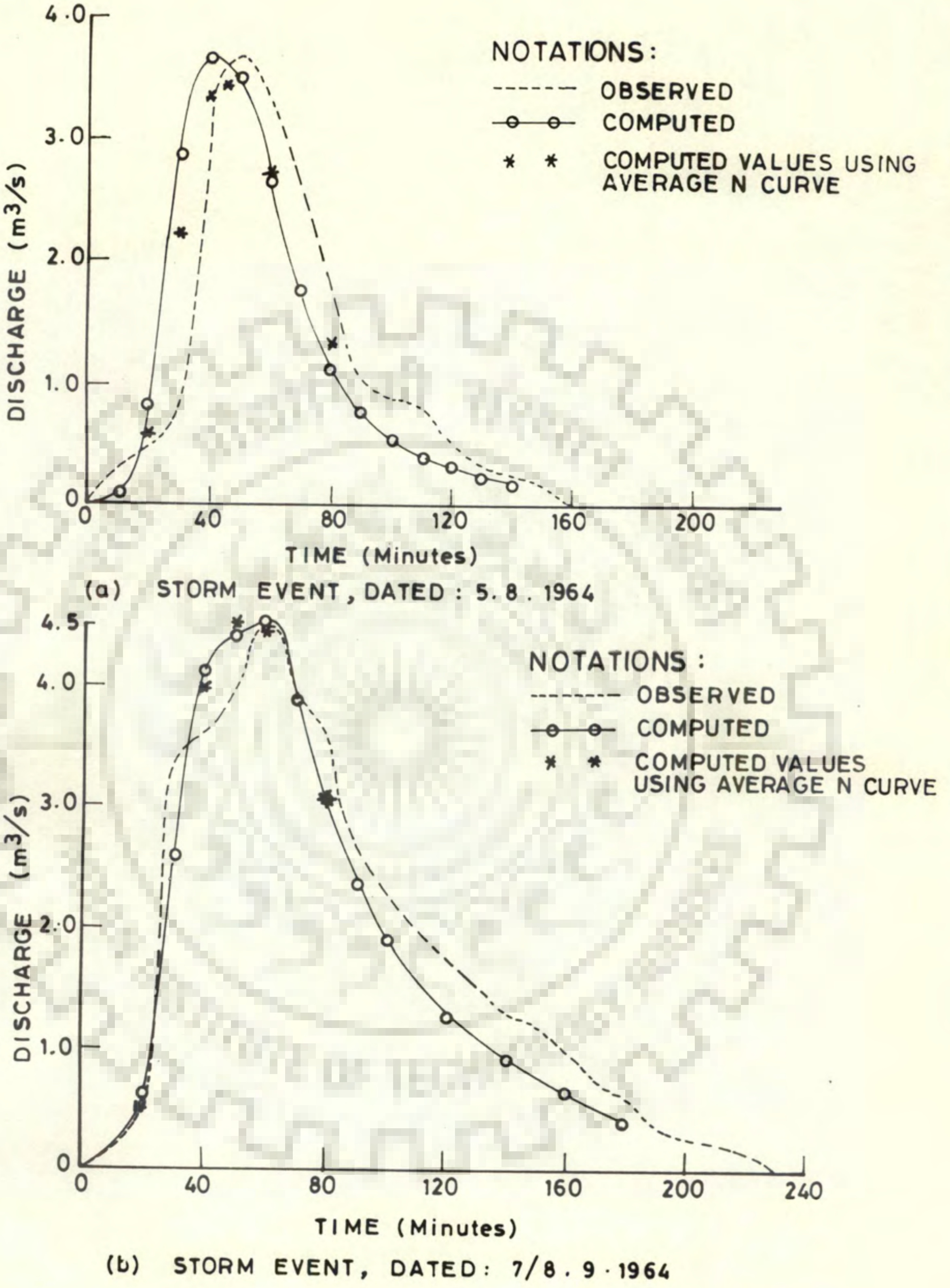
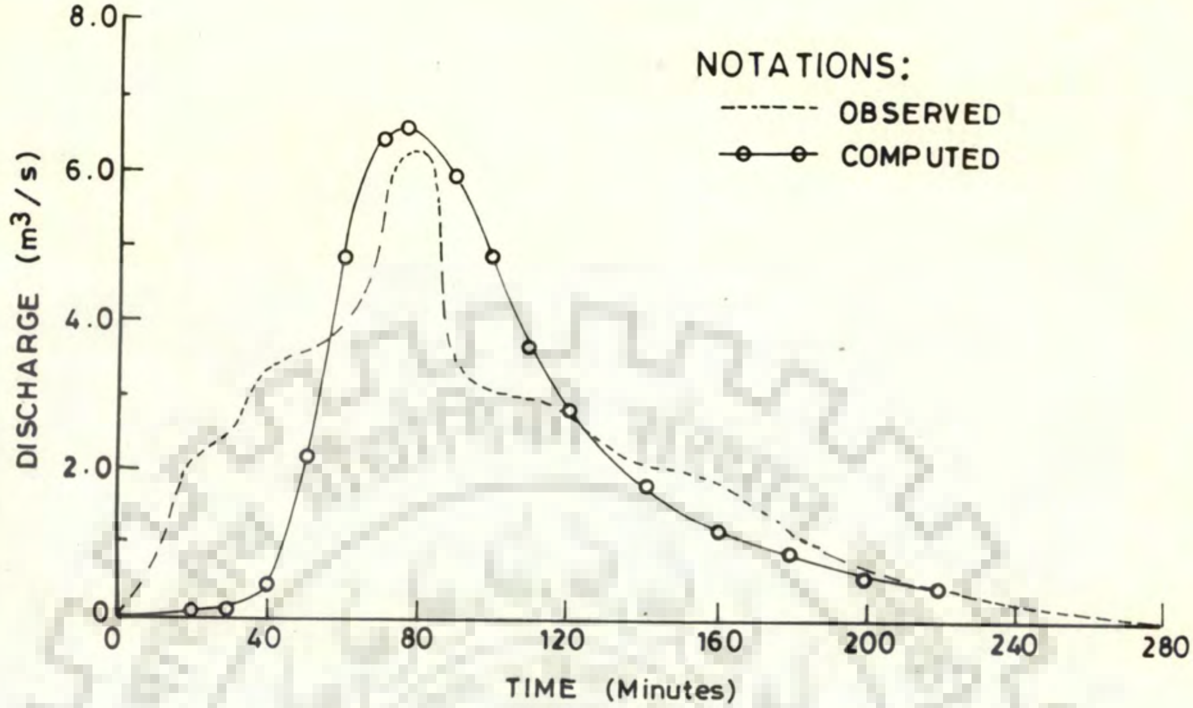
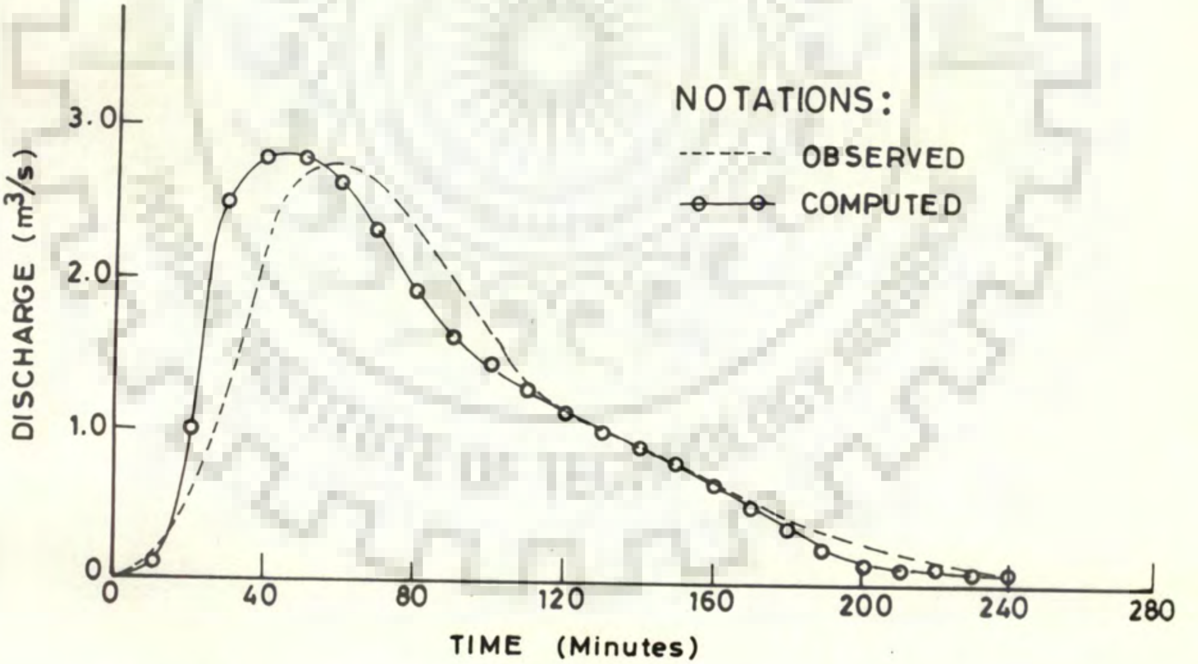


FIG. 5.9 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 319)

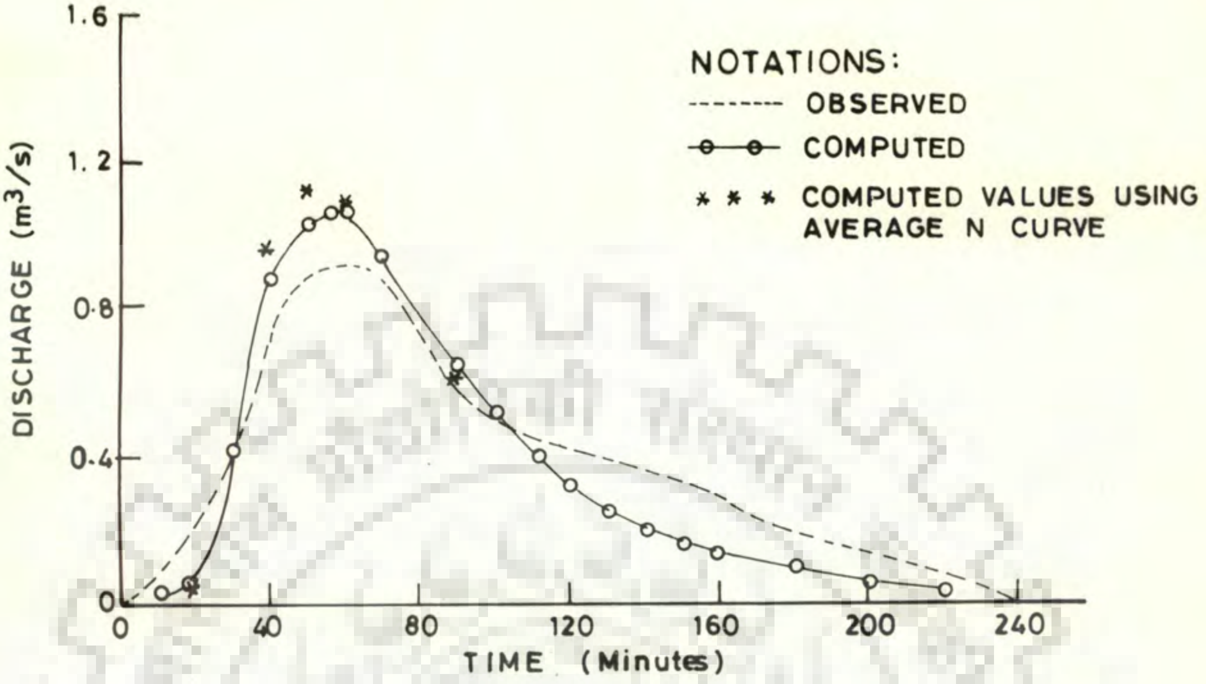


(a) STORM EVENT, DATED: 8/9.9.1964

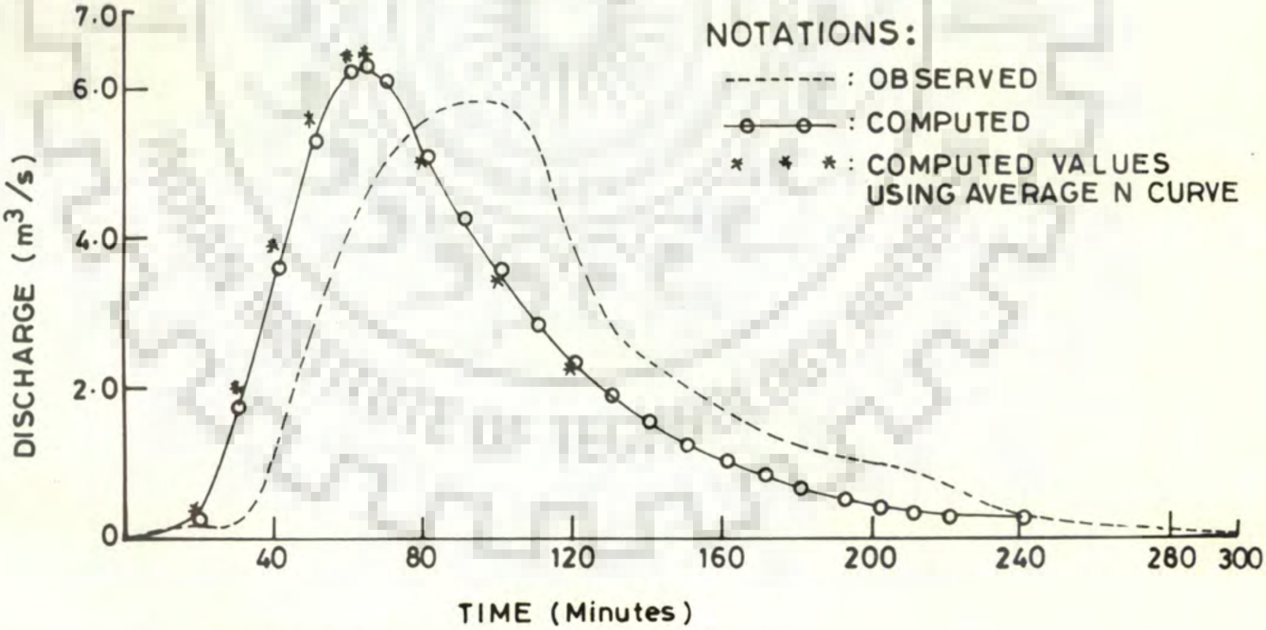


(b) STORM EVENT, DATED : 9.7.1988

FIG. 5.10 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO.319)



(a) STORM EVENT, DATED: 8.8.1988



(b) STORM EVENT, DATED: 11.9.1988

FIG. 5.11 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 319)

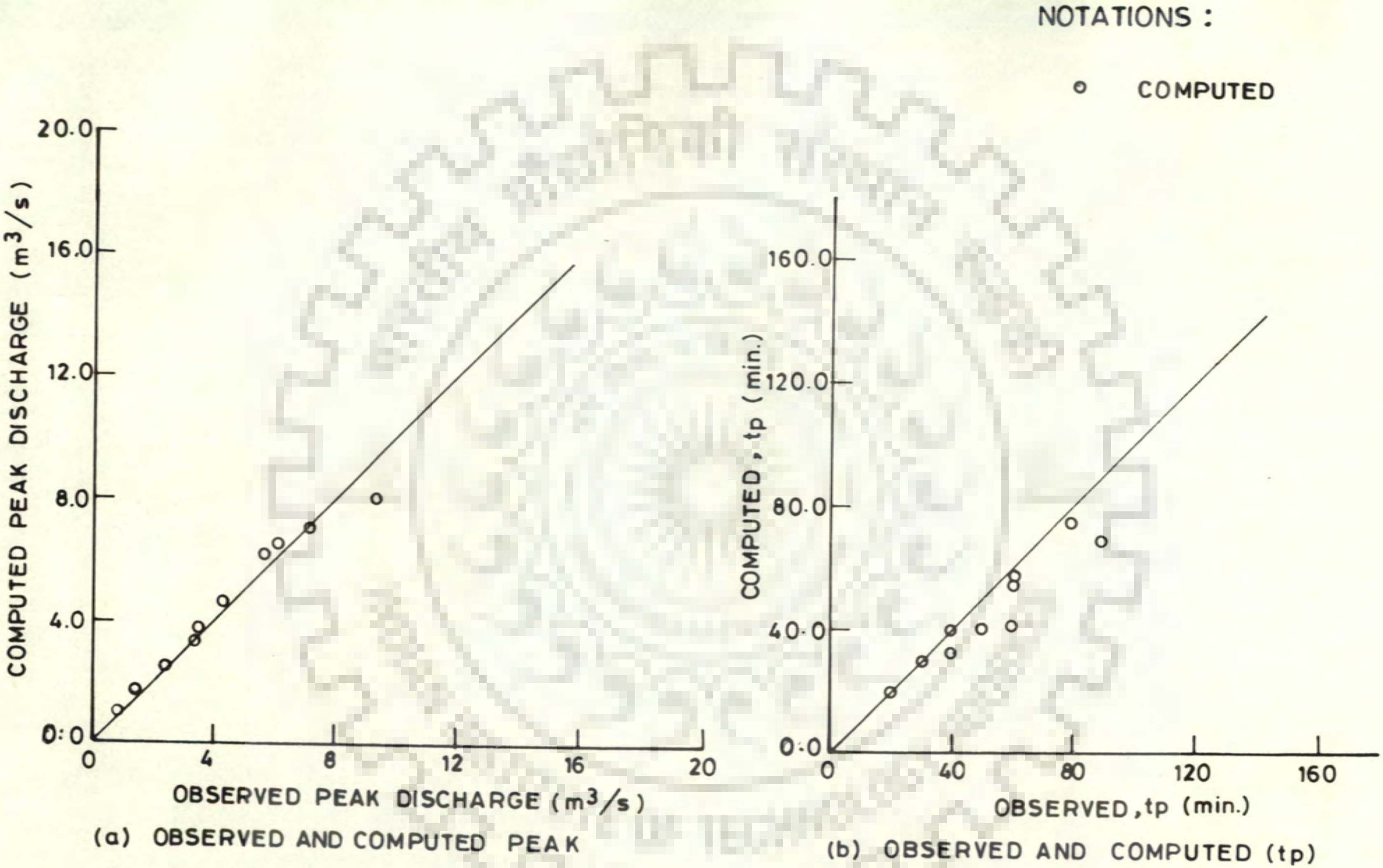


FIG. 5.12 COMPARISON OF OBSERVED AND COMPUTED PEAK DISCHARGES AND TIME TO PEAKS FOR 10-STORM EVENTS (BRIDGE NO. 319)

Criteria for compaison of single event models have been discussed by Green et al. (1986). It was recommended that to assess the performance of a model over a number of different events, a more general dimensionless ordinate independent measure of fit is required. The criteria for model efficiency suggested by Nash and Sutcliffe (1970) is reportedly the most appropriate. The same has been applied for assessing the model efficiencies for the different flood events. These values are given in Table 5.11 in the last column. In most of the cases, the close agreement between the observed and the computed hydrographs suggests that the proposed distributed physiographic model is well suited for the application of KW model for this watershed.

Application of KW models to the other three watersheds has been discussed in the forthcoming sections.

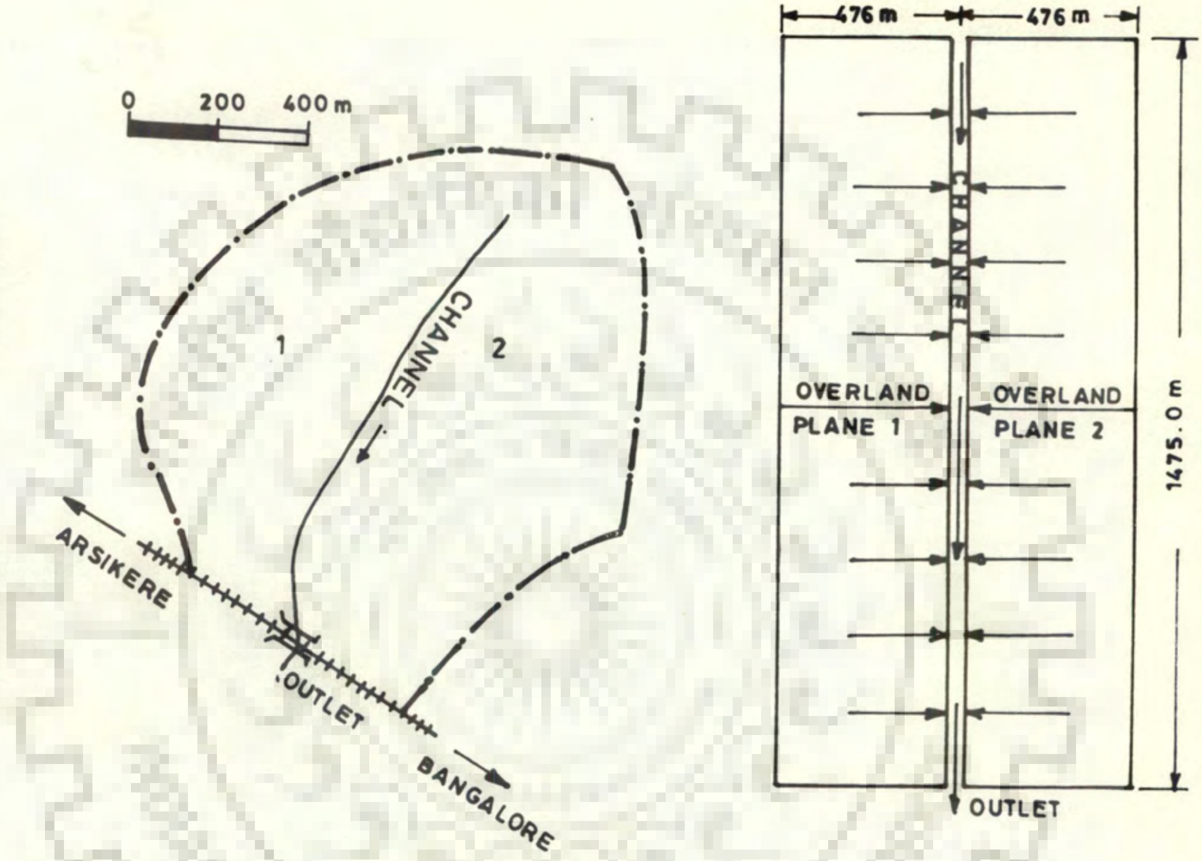
5.6 APPLICATION OF PROPOSED KW THEORY MODELS TO THE NATURAL WATERSHEDS OF BRIDGE NOS. 317, 719 AND TO THE KACHWA AGRICULTURAL WATERSHED.

In the previous section, details of application of the proposed KW theory models were explained by applying them on to the natural watershed of Bridge No. 319. Following the similar lines, application of the proposed models is discussed for the other three watersheds (viz. of the Bridge Nos. 317, 719 and the Kachwa agricultural watershed).

As discussed in Section 5.4, based on the Ø-index approach, the time distributions of rainfall excesses for various storms were computed. The same are presented in Appendix-II-B,C and E along with the gross rainfall data. In the following sections, various steps involved in the application of KW theory to these watersheds are discussed and the results obtained are presented.

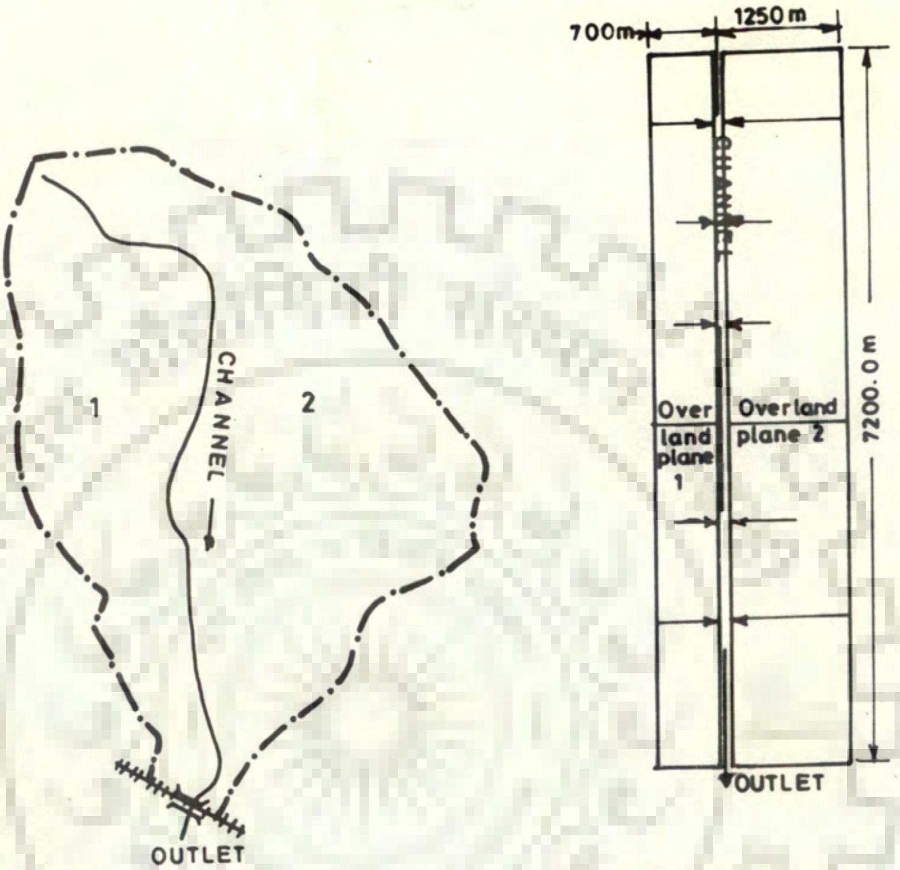
5.6.1 Estimation of Physiographic Parameters (Bridge Nos. 317, 719 and Kachwa Watershed)

Figures 4.2, 4.3 and 4.4 give the topographic feature of the watersheds of the Bridge Nos. 317, 719 and the Kachwa watershed. There are two parallel main drainages in the watershed of Bridge No. 317. Each of these channels has its small tributaries. The watershed is hilly, registering a fall from 1173.0 m (3850 ft) to 835.0 m (2740 ft). As discussed in Section 3.9, it was considered appropriate to estimate physiographic parameters of this watershed through the lumped physiographic model given in figure 5.13. In the case of natural watershed of Bridge No. 719, nealy three parallel drainages exist. Firstly, a lumped physiographic model of the type given in figure 5.14 was tried for this hilly watershed which registered a fall from nearly 914.0 m (3000.0 ft) to 747.0 m (2450.0 ft). In the Kachwa watershed the area is nearly flat with contours registering the fall from 6 metres to 3.2 metres (w.r.t. an arbitrary datum). The lumped physiographic model proposed and tried for the watershed is shown in figure 5.15. The physiographic parameters computed through lumped physiographic models are given in Tables 5.12, 5.13 and 5.14. The distributed parameters models for the three watershed are shown in figures 5.16, 5.17 and 5.18. The distributed physiographic parameters computed for the watershed of Bridge No. 719 are given in Table 5.15.



(a) THE WATERSHED AREA (b) LUMPED SYSTEM REPRESENTATION

FIG. 5.13 SCHEMATIC REPRESENTATION THROUGH A LUMPED MODEL (BRIDGE NO. 317)



(a) THE WATERSHED AREA

(b) LUMPED SYSTEM REPRESENTATION

FIG.5.14 SCHEMATIC REPRESENTATION THROUGH A LUMPED MODEL (BRIDGE NO.719)

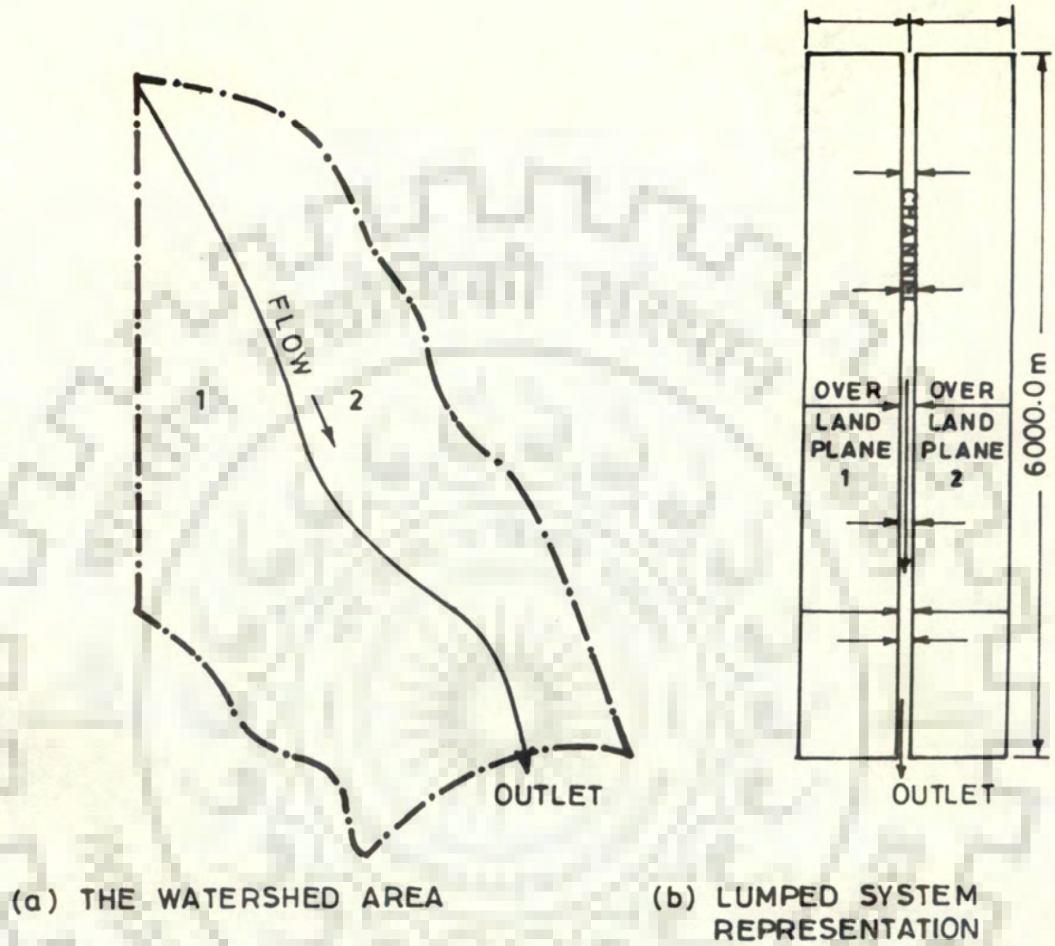


FIG. 5.15 SCHEMATIC REPRESENTATION THROUGH A LUMPED MODEL (KACHWA WATERSHED)

TABLE 5.12 : **The Lumped Physiographic Parameter Values (Bridge No. 317)**

Sl. No.	Particulars	Unit
(1)	(2)	(3)
1	Area	140.00 hectares
2	Overland Plane :	
	(a) Average length (each side)	476.00 metres
	(b) Average slope (each side)	0.081
3	Channel :	
	(a) Average length	1475.00 metres
	(b) Average slope	0.105
	(c) Average roughness	0.040
	(d) Average bed width	4.00 metres
	(e) Average side slope	2.5 H : 1V

TABLE 5.13 : The Lumped Physiographic Parameter Values (Bridge No. 719)

Sl. No. (1)	Particulars (2)	Unit (3)
1	Area	1400.00 hectares
2	Overland Plane :	
	(a) Average length (Right bank side)	700.00 metres
	(b) Average length (left bank side)	1250.00 metres
	(c) Average slope (Right bank side)	0.071
	(d) Average slope (Left bank side)	0.065
3	Channel :	
	(a) Average length	7200.00 metres
	(b) Average slope	0.0210
	(c) Average roughness	0.045
	(d) Average bed width	15.00 metres
	(e) Average side slope	2.00 H : 1V

TABLE 5.14 : The Lumped Physiographic Parameter Values (Kachwa Watershed)

Sl. No.	Particulars	Unit
(1)	(2)	(3)
1	Area	1073.00 hectares
2	Overland Plane :	
	(a) Average length (each side)	894.00 metres
	(b) Average slope (each side)	0.0012
3	Channel :	
	(a) Average length	6000.00 metres
	(b) Average slope	0.00047
	(c) Average roughness	0.0520
	(d) Average bed width	2.00 metres
	(e) Average side slope	2.00 H : 1V

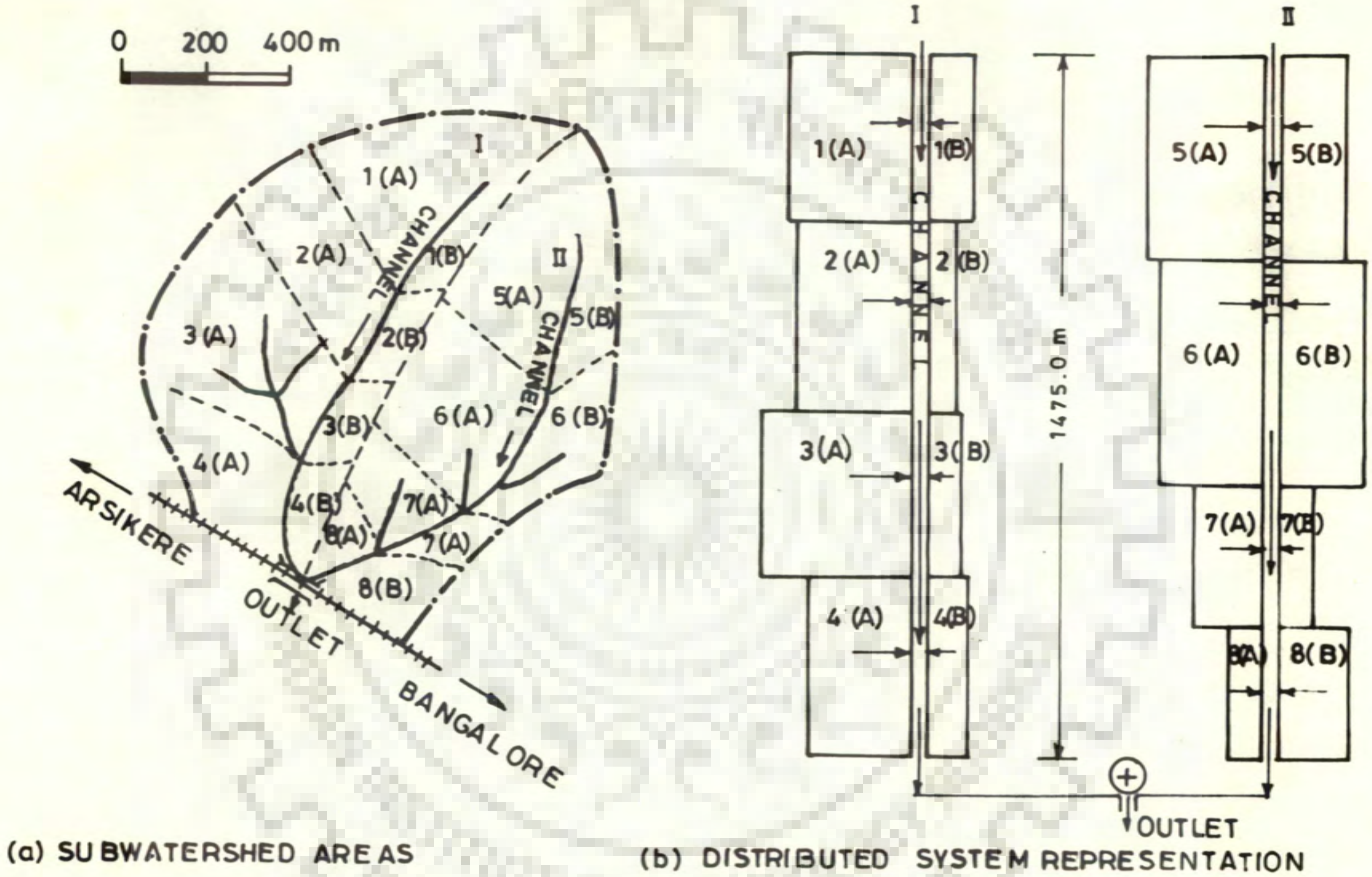
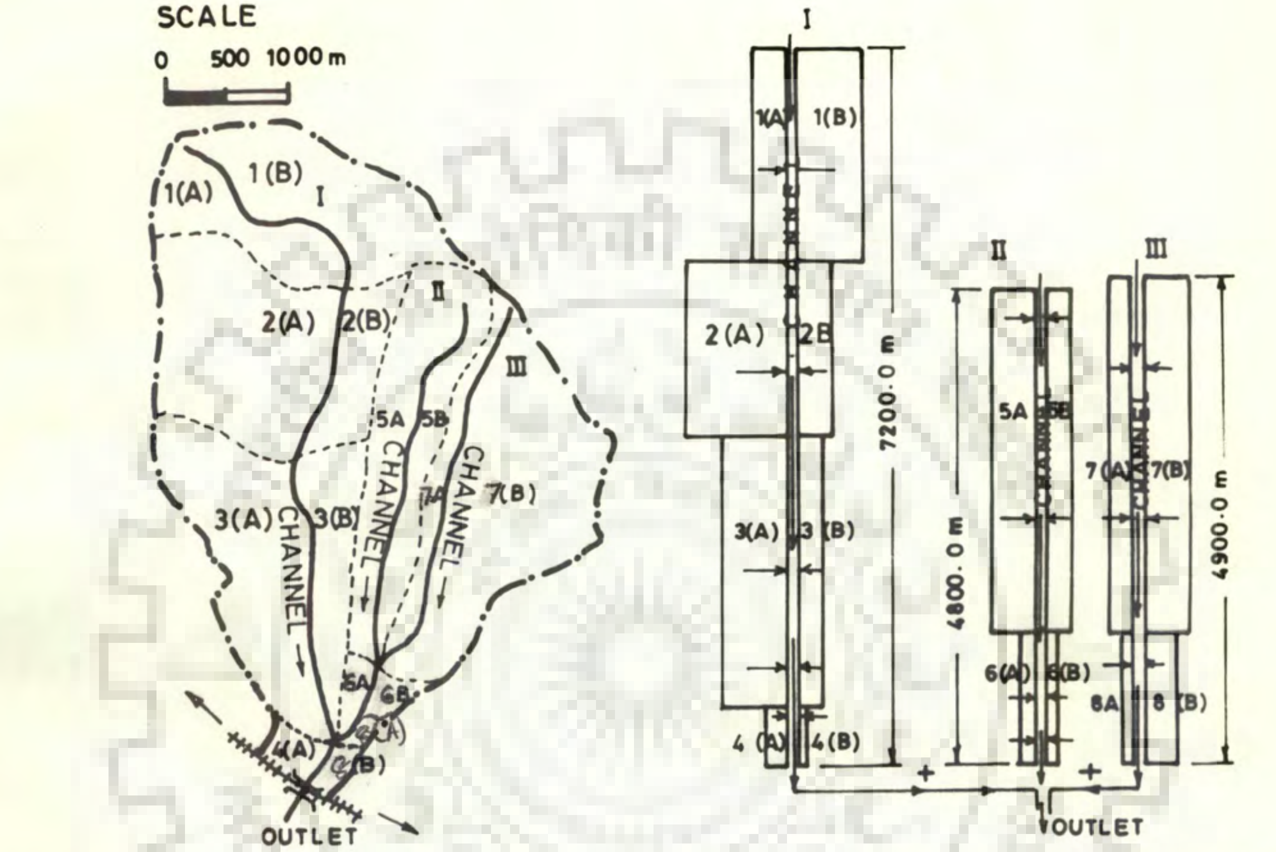
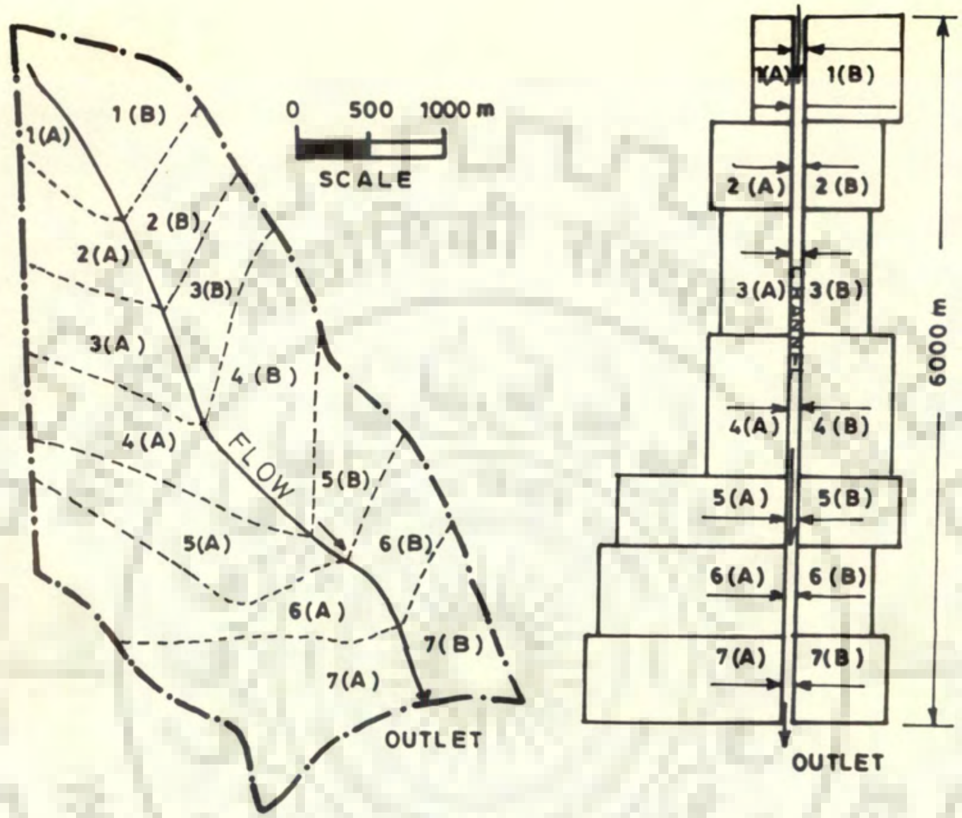


FIG. 5.16 SCHEMATIC REPRESENTATION THROUGH DISTRIBUTED MODEL (BRIDGE NO.317)



(a) SUBWATERSHED AREAS (b) DISTRIBUTED SYSTEM REPRESENTATION

FIG. 5.17 SCHEMATIC REPRESENTATION THROUGH A DISTRIBUTED MODEL (BRIDGE NO-719)



(a) SUBWATERSHED AREA (b) DISTRIBUTED SYSTEM REPRESENTATION

FIG. 5.18 SCHEMATIC REPRESENTATION THROUGH A DISTRIBUTED MODEL (KACHWA WATERSHED)

TABLE 5.15 : Computed Physiographic Parameter Values for the Distributed System of Scheme (I & II), (Bridge No. 719)

Sl. No. of chann-el	Sl. No. of sub water-shed	Overland length (m)	Right bank sub water-shed areas (A) (ha)	Left bank sub water-shed areas (B) (ha)	Overland slope	Channel length (m)	Channel slope	Channel bed width (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
I	1(A)	350.0	73.5000	-	0.071	2100.0	0.032	5.0
	1(B)	720.0	-	151.2000	0.043			
	2(A)	1100.0	198.0000	-	0.056	1800.0	0.012	5.0
	2(B)	390.0	-	70.2000	0.038			
	3(A)	680.0	183.6000	-	0.082	2700.0	0.018	10.0
	3(B)	270.0	-	72.9000	0.040			
	4(A)	230.0	13.8000	-	0.028	600.0	0.0076	15.0
	4(B)	60.0	-	3.6000	0.019			
II	5(A)	520.0	179.4000	-	0.071	3450.0	0.036	5.0
	5(B)	300.0	-	103.5000	0.070			
	6(A)	60.0	8.1000	-	0.017	1350.0	0.009	10.0
	6(B)	60.0	-	8.1000	0.017			
III	7(A)	200.0	71.000	-	0.071	3550.0	0.040	5.0
	7(B)	580.0	-	205.9000	0.053			
	8(A)	60.0	8.1000	-	0.017	1350.0	0.009	10.0
	8(B)	350.0	-	47.2500	0.017			
Total			735.500	662.6500				

The KW routing parameters α_k and m_k (Section 5.5.1) for the above three watersheds have been computed by using three representative depths of flows given in Table 5.16.

TABLE 5.16 : Three Representative Depths for the Computations of α_k and m_k of three Watersheds.

Sl. No.	Watersheds	Depths (m)		
		(i)	(ii)	(iii)
1	Bridge No. 317	0.25	0.50	0.70
2	Bridge No. 719	0.15	0.50	0.90
3	Kachwa	0.25	0.40	0.50

The physiographic parameters arrived at in this section are subjected to the sensitive analysis to determine their sensitiveness.

5.6.2 The Sensitivity Analysis of Physiographic Parameters (Bridge No. 317, 719 and Kachwa Watershed)

The sensitivity analysis was carried out for the various physiographic parameters (viz. channel bed width, channel side slope, channel depths) on similar lines as proposed in Section 5.5.2. The computed results of this analysis for one selected storm on each of the watersheds are given in Appendix-III-A,B and C. Also, the sensitivity of channel slopes, channel roughnesses, overland slopes and its roughnesses were tested for all the three watersheds. Their computed results are given in Appendix-III-D, E, F and G, H, I respectively. The computed values of the hydrograph parameters indicate that for these watersheds also the overland slope and the overland roughnesses happened to be the most sensitive.

This is also evident from the plots given in figures 5.19, 5.20 and 5.21. The overland roughnesses have been computed by following the comprehensive procedure given in Section 5.5.3. Therefore, a detailed analysis is needed for the estimation of effective overland roughnesses of all the three watersheds.

5.6.3 Estimation of Effective Overland Roughnesses (Bridge Nos. 317, 719 and Kachwa Watershed)

As reported in Section 5.5.3, following the similar lines the effective overland roughnesses have been worked out using the data of all the storm events of the watersheds. The scheme (I & II) is applied to the Bridge Nos. 317 and 719 whereas scheme III has been used for the Kachwa watershed. Reasons for this selection of schemes are explained through a comprehensive analysis given in the next section.

For the computational scheme (I & II), very small values of time and space steps were used (i.e. Δt = 5.0 minutes, Δx_o = 4.0 meters and Δx_c = 25.0 metres for the Bridge No. 317 and Δt = 10.0 minutes, Δx_o = 10.0 metres and Δx_c = 50.0 metres for the Bridge No. 719). For the watershed of Bridge No. 719, both the lumped and distributed physiographic models (Section 3.9) have been used. The Kachwa watershed is very flat and in most of the cases, the time base for hydrographs was found to be more than 60.00 hours. Therefore, for the application of scheme III, a moderate value of time steps (Δt = 60.0 minutes) and small values of space steps (Δx_o = 22.35 metres and Δx_c = 75.0 metres) were selected.

Satisfying the criteria stated in Section 5.5.3, the computed effective overland roughness values for the three watershed are given in Tables 5.17, 5.18 and 5.19. Using these values, plots of effective overland roughnesses Vs. time were tried and the same are produced in figures 5.22, 5.23 and 5.24. For the Bridge No. 719, the distributed physiographic model has also been used for computing the effective overland roughness parameter values. The plots of figures 5.22 and 5.23, do suggest the existence of a systematic rise and fall in effective overland roughness parameter values over the monsoon months.

In the figure 5.23, it is clear that the effective overland roughnesses differ significantly for the lumped and distributed physiographic models. The ranges of the effective overland roughness values for different months of monsoon period for the Bridge Nos. 317 and 719 are given in Tables 5.20 and 5.21.

For the Kachwa agricultural watershed most of the storm events belonged to a period starting from the last week of July to the first week of August. A plot of effective overland roughness factors w.r.t. their times of occurrences is given in figure 5.24. Since all the storm events belonged to a narrow period, therefore, no definite trend in the change of roughness values over the four monsoon months could be established.

5.6.4 Applicability of Different Computational Schemes (Bridge Nos. 317, 719 and Kachwa Watershed)

The suitability of the two computational schemes (Section 3.8) for each of the three watersheds has been ascertained on the lines reported in the section 5.5.4. Firstly, for different values of time steps Δts, applicability of the scheme (I & II) as well as of the scheme III has been tried keeping the criteria of the stability and convergence in mind. Subsequently the space steps for the overland and channel segments are worked out.

For selecting appropriate time steps, the results of the test runs for one selected storm on each of the three watersheds are presented in Table 5.22. The comparison of computed and observed hydrograph parameters (i.e. Q_p, t_p and volume) suggests that the scheme (I & II) which is inherently convergent is also stable for Δt upto 30.0 minutes whereas scheme III though stable did not converge for any value of Δt ranging (i.e. from 5.0 minutes to 30.0 minutes) in cases of the two watersheds i.e. of Bridge Nos. 317 and 719. Therefore, for further computations of runoff on the two watersheds, scheme (I & II) has been used with a time step of 5.0 minutes for the Bridge No. 317 and 10.0 minutes for the Bridge No. 719. In the case of Kachwa agricultural watershed, the Scheme (I & II) remained mostly unstable. A time step value of less than 15.0 minutes was not found to be practical as well as economical from computer time considerations because the time base of the hydrographs happened to be more than 60.0 hours. However, the scheme III did converge for a time step value as large as 60.0 minutes. Therefore, the scheme III with one hour time step has been chosen for further calculations.

As shown in Table 5.23, for the adopted values of time steps, scheme (I & II) remained stable upto an overland space

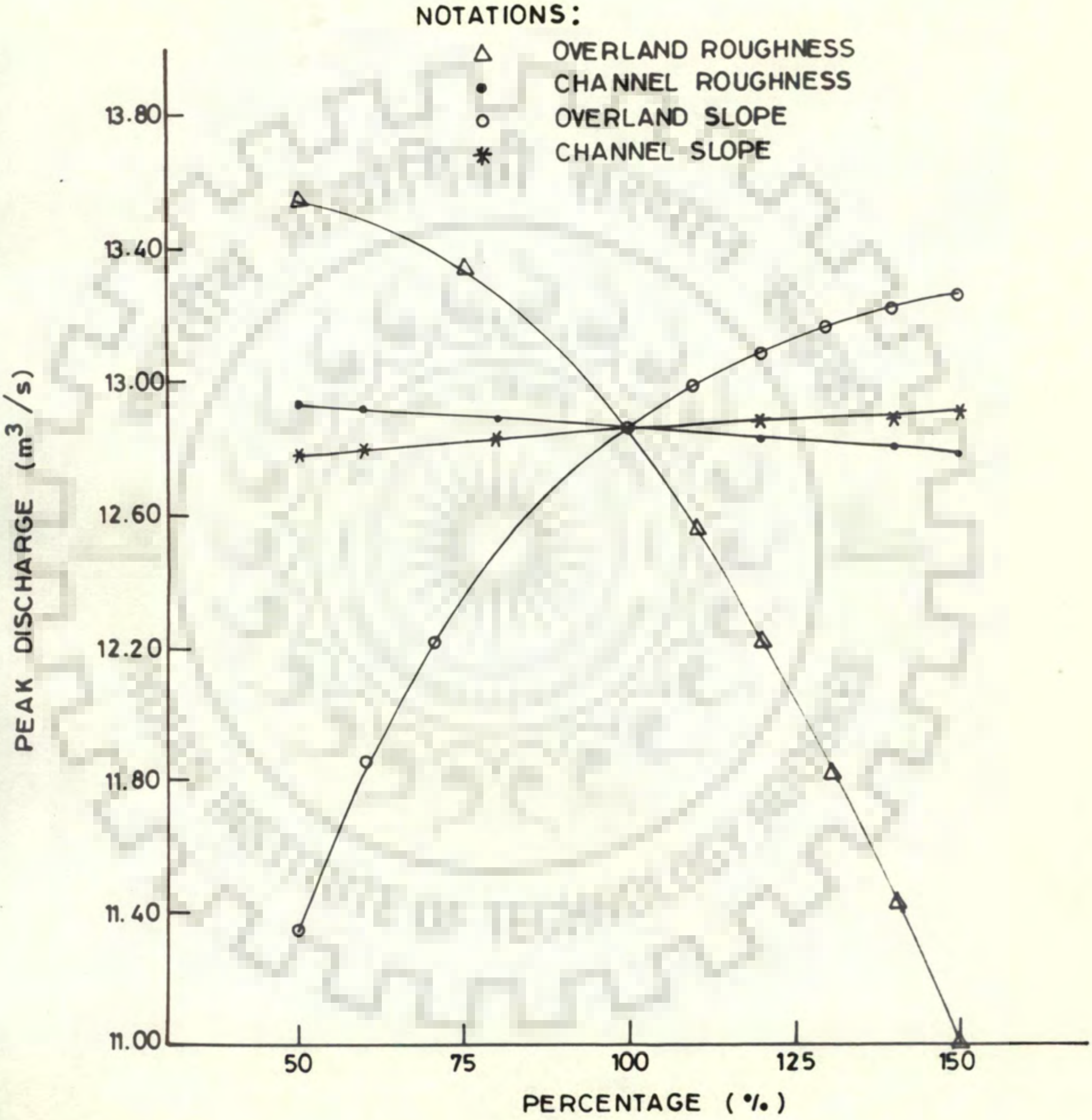


FIG.5.19 SENSITIVITY OF OVERLAND AND CHANNEL ROUGHNESSES AND SLOPES w.r.t PEAK DISCHARGE (BRIDGE NO. 317)

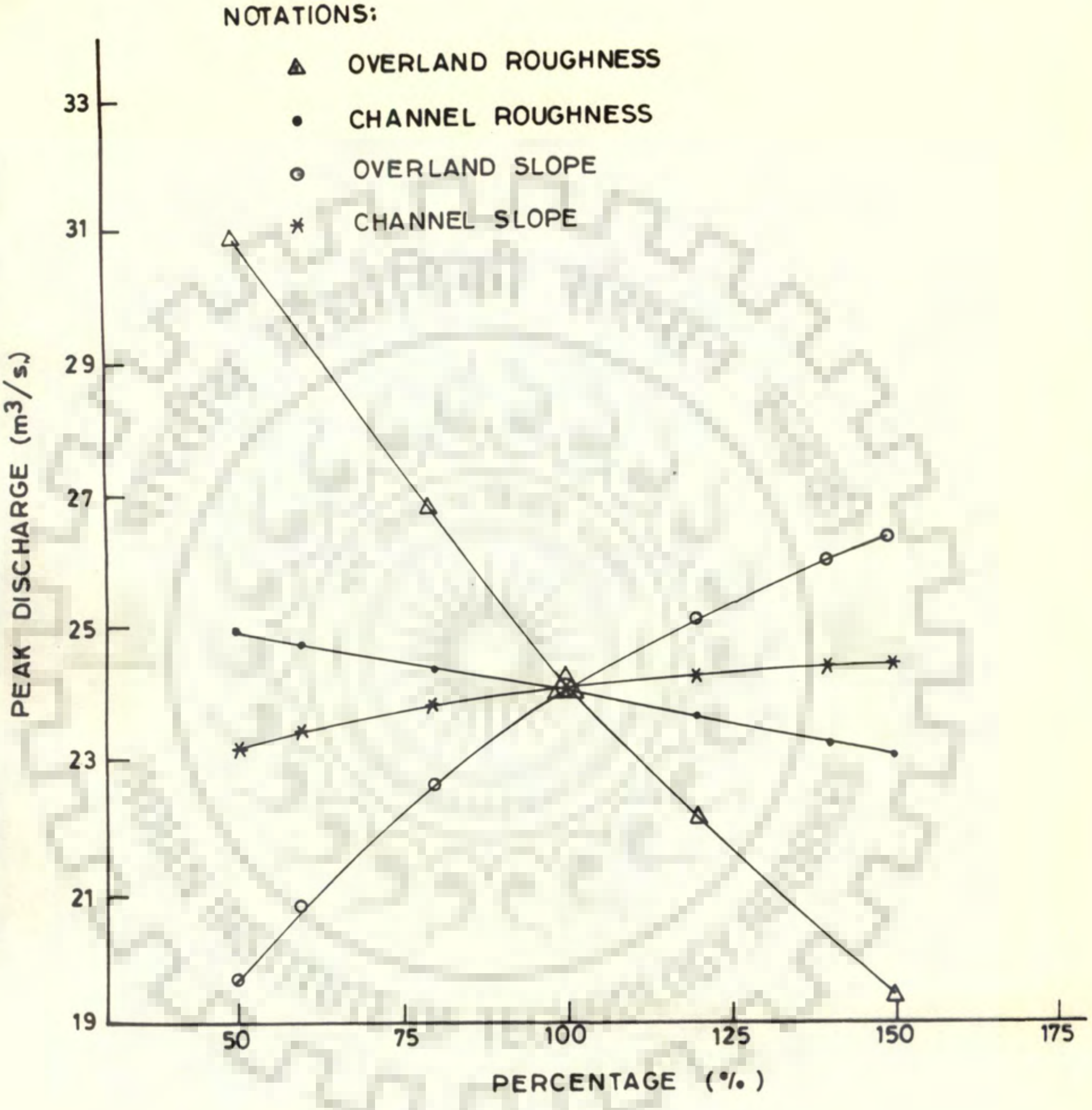


FIG. 5.20 SENSITIVITY ANALYSIS OF OVERLAND AND CHANNEL ROUGHNESSES AND SLOPES w.r.t PEAK DISCHARGE (BRIDGE NO. 719)

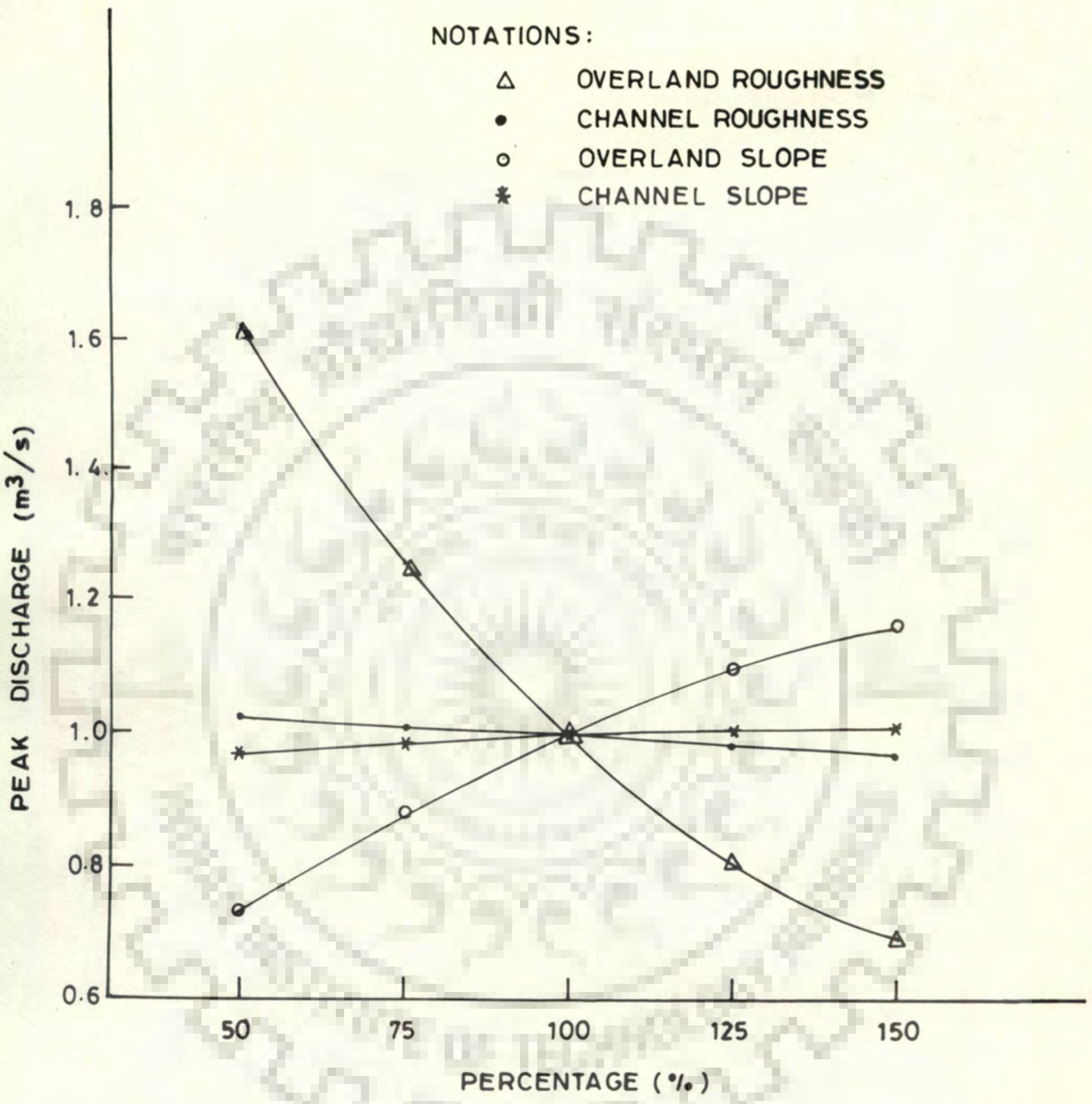


FIG. 5.21 SENSITVITY OF OVERLAND AND CHANNEL ROUGHNESSES AND SLOPES w.r.t PEAK DISCHARGE (KACHWA WATERSHED)

TABLE 5.17 : Computed Effective Overland Roughness (Bridge No. 317)

Sl. No.	Storm Date	Effective Overland Roughness (N)	Peak Discharge (m^3/s)	
			Observed	Computed
(1)	(2)	(3)	(4)	(5)
1	25.10.1961	0.192	12.82	12.864
2	24.9.1962	0.169	15.10	11.691
3	14.10.1962	0.150	5.773	5.770
4	18.7.1963	0.070	9.905	9.862
5	26.7.1964	0.083	6.481	6.382
6	7/8.9.1964	0.292	4.245	4.243
7	8.9.1964	0.336	5.098	5.088
8	15.9.1964	0.270	3.424	3.426

TABLE 5.18 : Computed Effective Overland Roughness
(Bridge No. 719)

Sl. No.	Storm Date	Effective Overland Roughness (N)		Peak Discharge (m^3/s)		
		Lumped	Distributed	Observed	Computed (lumped)	Computed (Distributed)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
1	24/25.7.1964	0.168	0.251	24.100	24.050	24.20
2	26/27.7.1964	0.215	0.345	32.709	32.740	32.68
3	3.9.1964	0.110	0.189	10.480	10.470	10.50
4	11.8.1965	0.200	0.345	40.022	40.060	39.95
5	18.9.1965	0.042	0.072	9.350	9.286	9.420
6	6.7.1966	0.044	0.073	10.365	10.293	10.284
7	16.9.1966	0.132	0.230	33.98	33.890	33.890
8	20.9.1966	0.040	0.070	10.30	9.726	10.480

TABLE 5.19 : Computed Effective Overland Roughness (Kachwa Watershed)

Sl. No.	Storm Date	Effective Overland Roughness (N)	Peak Discharge (m^3/s)	
			Observed	Computed
(1)	(2)	(3)	(4)	(5)
1	27.7.1977	0.094	0.851	0.852
2	4.8.1977	0.119	0.993	0.993
3	2/3.8.1978	0.196	1.178	1.179
4	8/9.8.1978	0.158	0.918	0.921
5	2.8.1979	0.106	0.417	0.418

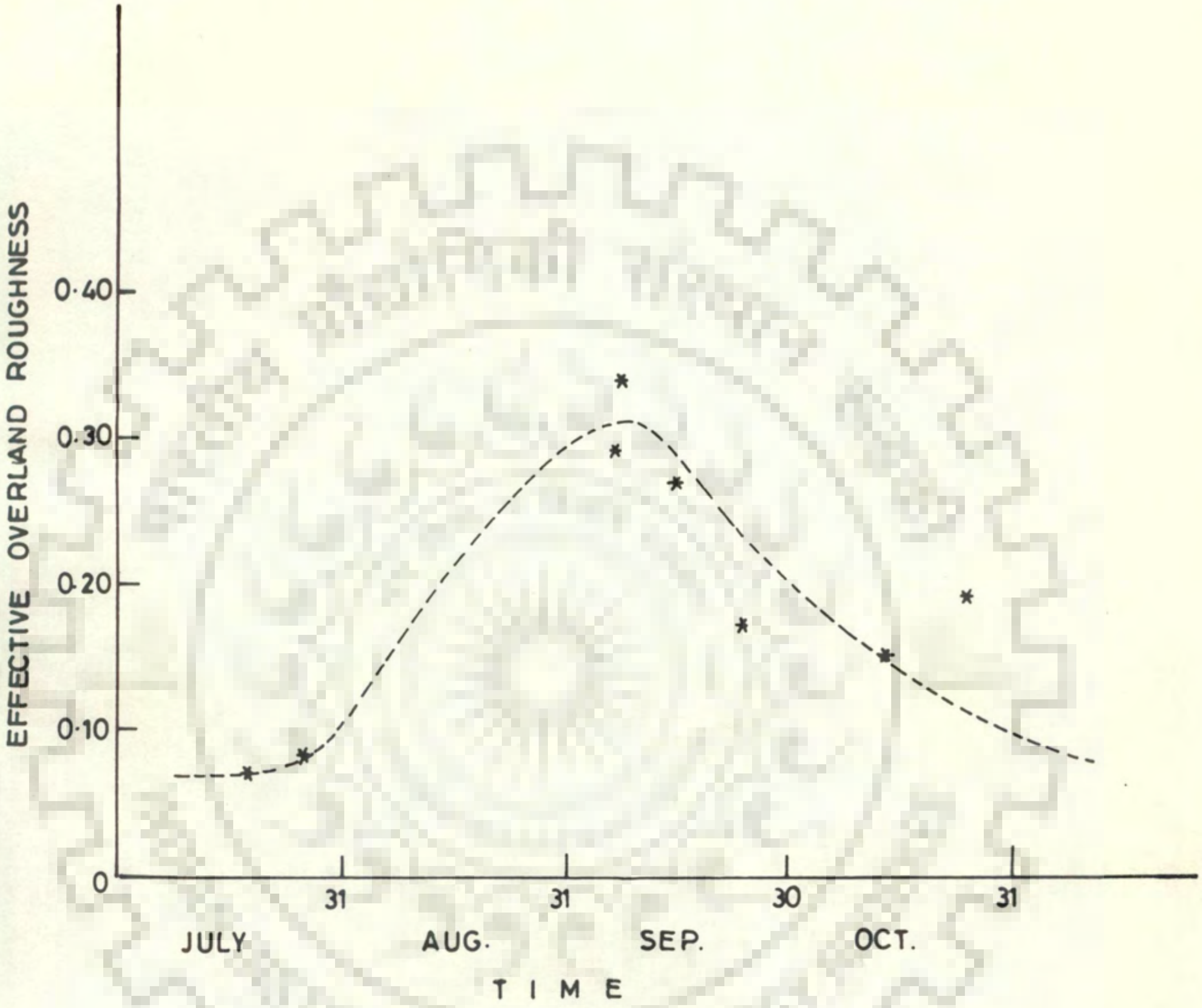


FIG. 5.22 VARIATION OF EFFECTIVE OVERLAND ROUGHNESS WITH TIME (BRIDGE NO. 317)

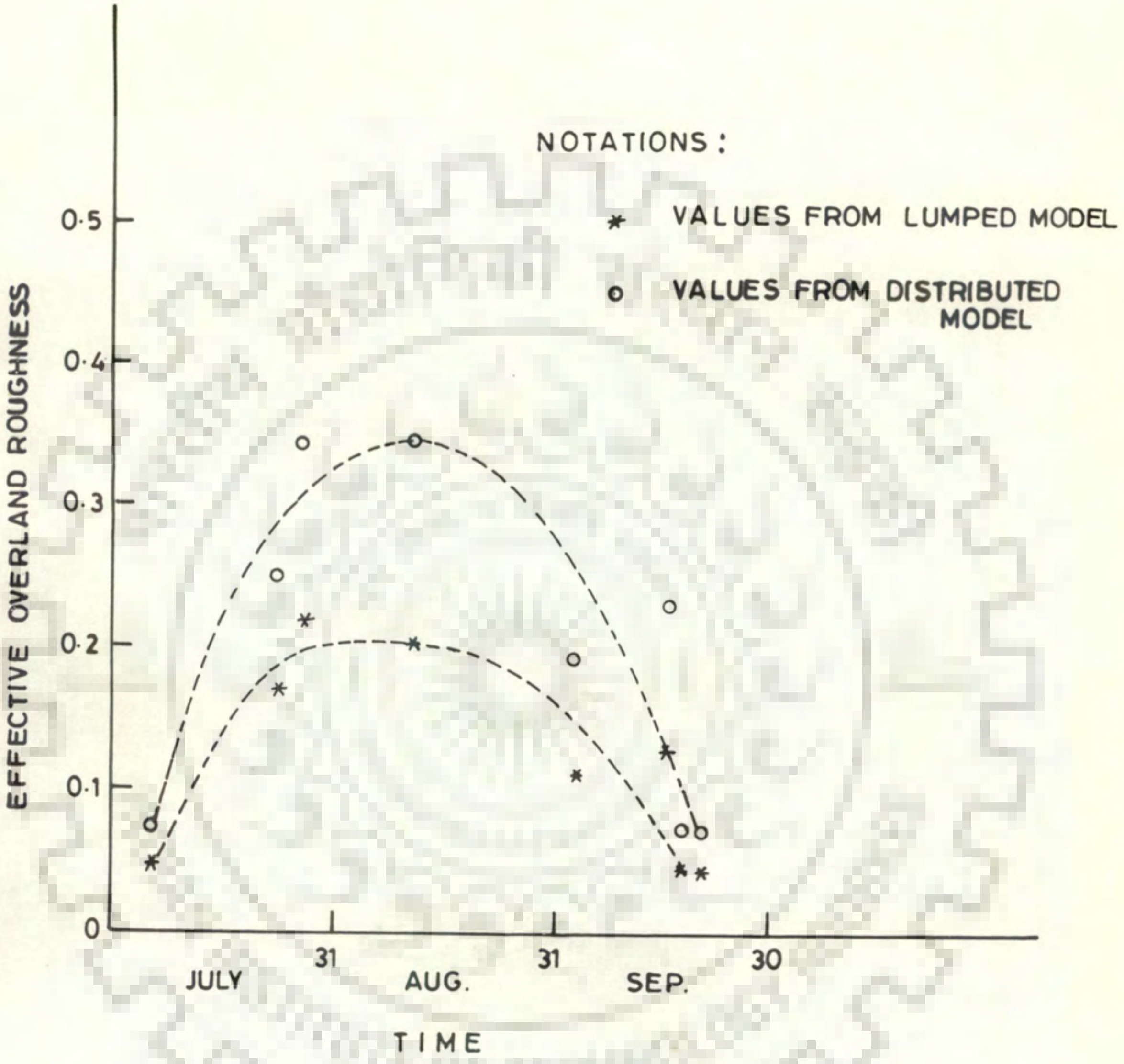


FIG. 5·23 VARIATION OF EFFECTIVE OVERLAND ROUGHNESS WITH TIME (BRIDGE NO. 719)

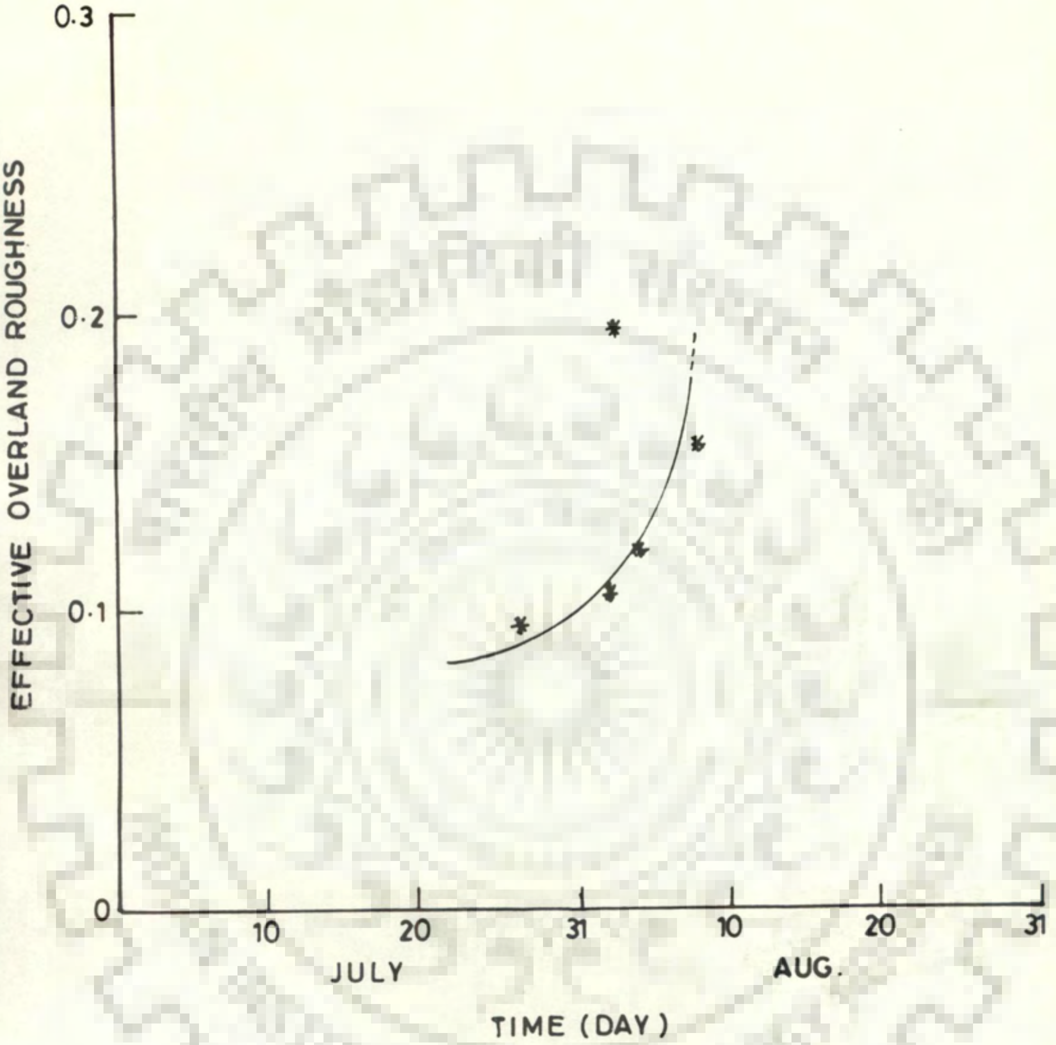


FIG. 5.24 VARIATION OF EFFECTIVE OVERLAND ROUGHNESS WITH TIME (KACHWA WATERSHED)

TABLE 5.20 : Range of Effective Overland Roughnesses During the Monsoon Months (July to October). (Bridge No. 317).

Sl. No.	Months	Range of Effective Overland Roughnesses	
		From	To
1	July	0.07	0.10
2	August	0.10	0.290
3	September	0.210	0.290
4	October	0.10	0.210

TABLE 5.21 : Range of Effective Overland Roughnesses During the Monsoon Months (July, August and September) (Bridge No. 719)

Sl. No.	Months	Range of Effective Overland Roughnesses			
		Lumped		Distributed	
		From	To	From	To
1	July	0.044	0.200	0.073	0.325
2	August	0.165	0.200	0.280	0.325
3	September	0.040	0.165	0.070	0.280

TABLE 5.22 : Test Run for Establishing the Range of Time Step (Δt) from Stability and Convergence Points of View for Computational Schemes (I & II) and III

(i) Bridge No. 317 (Storm event, dated 25.10.1961, N = 0.192)

Sl. No.	Time Step (Δt) (min)	Observed Peak (Q_p) (m^3/s)	Observed Time to peak (t_p) (min)	Observed Volume (m^3) x 10^3	Computed Hydrograph Parameters: Scheme (I & II) Stability	Scheme (I & II) Q_p (m^3/s)	Scheme (I & II) t_p (min)	Scheme (I & II) Vol. (m^3) x 10^3	Scheme III Q_p (m^3/s)	Scheme III t_p (min)	Scheme III Vol. (m^3) x 10^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	12.82	90.0	53.586	-	-	-	-	-	-	-
1	5.0	-	-	-	Stable	12.864	90.0	52.960	11.940	90.0	50.097
2	10.0	-	-	-	Stable	12.026	90.0	52.960	10.160	90.0	44.633
3	15.0	-	-	-	Stable	11.360	90.0	52.767	8.844	90.0	41.076
4	30.0	-	-	-	Stable	9.877	90.0	52.725	6.437	90.0	32.868

(ii) Bridge No. 719 (Storm event, dated 24/25.7.1964, N = 0.168)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	24.10	180.0	254.83							
1	10.0	-	-	-	Stable	24.05	130.0	237.24	23.10	130.0	231.61
2	15.0	-	-	-	Stable	22.73	135.0	236.34	21.25	120.0	219.45
3	30.0	-	-	-	Stable	19.93	120.0	234.23	17.34	120.0	190.62

(iii) Kachwa Watershed (Storm event, dated 4.8.1977, N = 0.119)

(1)	(2)	(3)	(4) (hr)	(5)	(6)	(7)	(8)	(9)	(10)	(11) (hr)	(12)
-	-	0.993	8.0	78.264							
1	15.0	-	-	-	Stable	1.049	10.25	72.251	1.044	9.0	78.524
2	30.0	-	-	-	Unstable	-	-	-	1.033	9.0	77.900
3	60.0	-	-	-	Unstable	-	-	-	0.993	9.0	76.802

TABLE 5.23 : Test Run for Establishing the Range of Overland length step (Δx_o) from Stability and Convergence Points of View for Computational Schemes (I & II) and III

(i) Bridge No. 317 (Storm dated 25.10.1961, N = 0.192)

Sl. No.	Overland space step Δx_o (m)	Observed Peak (Q_p) (m^3/s)	Observed Time to peak (tp) (min)	Observed Volume (m^3) x 10^3	Computed Hydrograph Parameters: Stability	Scheme (I & II) Q_p (m^3/s)	Scheme (I & II) t_p (min)	Scheme (I & II) Vol. (m^3) x 10^3	Scheme III Q_p (m^3/s)	Scheme III t_p (min)	Scheme III Vol. (m^3) x 10^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	12.82	90.0	53.566							
1	[illegible]	-	-	-	Stable	12.864	90.0	52.960	11.940	90.0	50.097
2	[illegible]	-	-	-	Stable	12.864	90.0	52.960	11.926	90.0	50.085
3	16.0	-	-	-	Unstable	-	-	-	11.791	90.0	49.856
4	32.0	-	-	-	Unstable	-	-	-	11.495	90.0	49.442

(ii) Bridge No. 719 (Storm dated 24/25.7.1964, N = 0.166)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	24.10	180.0	254.83							
1	5.0	-	-	-	Stable	23.94	130.0	236.87	23.31	130.0	232.44
2	10.0	-	-	-	Stable	24.05	130.0	237.24	23.18	130.0	231.93
3	20.0	-	-	-	Stable	24.26	130.0	238.16	22.91	130.0	229.29
4	50.0	-	-	-	Unstable	-	-	-	22.29	130.0	226.97

(iii) Kachwa Watershed (Storm dated 4.8.1977, N = 0.119)

(1)	(2)	(3)	(4) (hr)	(5)	(6)	(7)	(8)	(9)	(10)	(11) (hr)	(12)
-	-	0.993	8.0	78.264							
1	21.20	-	-	-	Unstable	-	-	-	0.993	9.0	76.802
2	42.40	-	-	-	Unstable	-	-	-	0.976	8.0	74.408
3	84.60	-	-	-	Stable	1.046	10.0	70.743	0.945	8.0	70.884

TABLE 5.24 : Test Run for Establishing the Range of Channel length Step (Δx_c) from Stability and Convergence Points of View for Computational Schemes (I & II) and III

(i) Bridge No. 317 (Storm dated 25.10.1961, N = 0.192)

Sl. No.	Channel space step (Δx)	Observed			Computed Hydrograph Parameters						
		Peak (Q_p) (m^3/s)	Time to peak (t_p) (min)	Volume (m^3) x 10^3	Scheme (I & II)				Scheme III		
					Stability	Q_p (m^3/s)	t_p (min)	Vol. (m^3) x 10^3	Q_p (m^3/s)	t_p (min)	Vol. (m^3) x 10^3
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	12.82	90.0	53.586	-	-	-	-	-	-	-
1	15.05	-	-	-	Stable	12.867	90.0	52.981	11.945	90.0	50.106
2	25.0	-	-	-	Stable	12.864	90.0	52.960	11.940	90.0	50.097
3	49.167	-	-	-	Stable	12.867	90.0	52.957	11.920	90.0	49.992
4	98.334	-	-	-	Stable	12.876	90.0	52.953	11.900	90.0	49.890
5	[illegible]	-	-	-	Stable	12.890	90.0	52.950	11.851	90.0	49.716

(ii) Bridge No. 719 (Storm dated 24/25.7.1964, N = 0.168)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
-	-	24.10	180.0	254.63	-	-	-	-	-	-	-
1	25.0	-	-	-	Stable	24.032	130.0	237.21	23.204	130.0	231.87
2	50.0	-	-	-	Stable	24.05	130.0	237.24	23.180	130.0	231.97
3	100.0	-	-	-	Stable	24.09	130.0	237.30	23.150	130.0	231.69
4	150.0	-	-	-	Stable	24.13	130.0	237.35	23.110	130.0	231.57
5	[illegible]	-	-	-	Stable	24.17	130.0	237.42	23.070	130.0	231.14

(iii) Kachwa Watershed (Storm dated 4.8.1977, N = 0.119)

(1)	(2)	(3)	(4) (hr)	(5)	(6)	(7)	(8)	(9)	(10)	(11) (hr)	(12)
-	-	0.993	8.0	78.264	-	-	-	-	-	-	-
1	50.0	-	-	-	Unstable	-	-	-	0.994	9.0	76.824
2	75.0	-	-	-	Unstable	-	-	-	0.993	9.0	76.802
3	100.0	-	-	-	Unstable	-	-	-	0.992	9.0	76.777
4	150.0	-	-	-	Unstable	-	-	-	0.990	9.0	76.726
5	300.0	-	-	-	Unstable	-	-	-	0.984	9.0	76.582

step of 10.0 metres for the Bridge Nos. 317 and 719. For the Kachwa agricultural watershed, scheme III could be satisfactorily run for the overland steps of about 85.0 metres. For the channel space step (Δx_c) (Table 5.24) scheme (I & II) remained stable upto nearly 200.0 metres for the two watersheds of Bridge Nos. 317 and 719. For the Kachwa watershed the scheme III did converge even for large values of channel space step length 300.0 metres. Summarisingly, for the three watersheds, the adopted schemes alongwith their chosen parameter values are given in Table 5.25.

TABLE 5.25 : Adopted Schemes and Their Parameters (Δt, Δx_o and Δx_c) for the Three Watersheds.

Sl. No.	Watershed	Scheme	Δt (min)	Δx_o (m)	Δx_c (m)
1	Bridge No. 317	(I & II)	5.0	4.0	25.0
2	Bridge No. 719	(I & II)	10.0	10.0	50.0
3	Kachwa	III	60.0	22.35	75.0

Using the parameters of Table 5.25, the KW models has now been applied to three watersheds to compute their responses.

5.6.5 Application of KW theory to the Proposed Distributed Parameters Model (Bridge Nos. 317, 719 and Kachwa Watershed)

The distributed parameter models (discussed in Section 3.9.2) for the three watersheds of Bridge Nos. 317, 719 and Kachwa have been evolved by dividing the watersheds into a number of subwatersheds (Figures 5.16, 5.17 and 5.18). The distributed physiographic parameters computed for the two watersheds of

Bridge Nos. 317 and Kachwa are given in Tables 5.26 and 5.27. For the watershed of Bridge No. 719, the distributed physiographic parameters have already been computed in Section 5.6.1 (Table 5.15). The computational schemes to be applied on these watersheds and parameter values to be used are given in Table 5.25 of the previous section.

The subwatershed areas which are located on the right bank of the channel are marked with 'A' whereas those located on the left bank are marked with 'B'. Bed widths, channel side slopes and parameters α_k and m_k etc. are computed on the lines as reported in the Section 5.5.5.

For the watershed of Bridge No. 719, the physiographic parameters worked out on the basis of the lumped model (Figure 5.14 and Table 5.13) were used to compute the responses (Figures 5.29 to 5.32). The visual comparison of the observed and the computed responses suggests that the two did not match satisfactorily. The possible reason that the three main streams located in the watershed form nearly parallel drainages and a lumped parameter approach did distort the real existing situation. Consequently, the physiographic parameters worked out from the distributed parameter model (Figure 5.17 and Table 5.15) and the effective overland roughness worked out for this system were used. The computer programmes used are given in Appendix-I.

The comparisons of the observed and the computed hydrographs for 21 storm events recorded on the three watersheds are shown in figures 5.25 through 5.35 (i.e. Figures 5.25 through 5.28 for the Bridge No. 317, figures 5.29 through 5.32 for the Bridge No. 719 and figures 5.33 through 5.35 for the Kachwa watershed). The computed hydrograph parameters are also compared with their respective observed hydrograph parameters of these three watersheds (Tables 5.28, 5.29 and 5.30). Plots of observed and computed values of the peak discharges and the time to peaks are shown in figures 5.36 and 5.37. Percentage of errors in volume and model efficiencies are also computed (Tables 5.28, 5.29 and 5.30).

Concludingly, it may be remarked that the proposed KW model approach has produced quite satisfactory results. Attempts were made to explore the possibility of extending this work to ungauged watersheds as well. Also, further investigations relating to time of concentration, time-area-concentration applications were carried out, as reported in the next chapter.

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TABLE 5.26 : Computed Physiographic Parameter Values for the Distributed System of Bridge No. 317

Sl. No. of chann-el	Sl. No. of sub water-shed	Overland length (m)	Right bank sub water-shed areas (A) (ha)	Left bank sub water-shed areas (B) (ha)	Overland slope	Channel length (m)	Channel slope	Channel bed width (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	1(A)	376.0	13.1600	-	0.223	350.0	0.174	2.0
	1(B)	144.0	-	5.0400	0.224			
	2(A)	360.0	14.4000	-	0.06	400.0	0.144	3.0
	2(B)	72.0	-	2.8800	0.05			
I	3(A)	456.0	15.9600	-	0.052	350.0	0.017	3.0
	3(B)	96.0	-	3.3600	0.043			
	4(A)	320.0	12.0000	-	0.0036	375.0	0.004	3.0
	4(B)	104.0	-	3.9000	0.0040			
	5(A)	352.0	14.9600	-	0.169	425.0	0.179	2.0
	5(B)	192.0	-	8.1600	0.152			
II	6(A)	320.00	15.2000	-	0.067	475.0	0.064	3.0
	6(B)	264.0	-	12.5400	0.077			
	7(A)	232.0	6.9600	-	0.044	300.0	0.054	3.0
	7(B)	104.0	-	3.1200	0.056			
	8(A)	104.0	2.8600	-	0.0076	275.0	0.0033	3.0
	8(B)	224.0	-	6.1600	0.0064			
Total			95.5000	45.1600				

TABLE 5.27 : Computed Physiographic Parameter Values for the Distributed System of Kachwa Watershed

Sl. No. of chann-el	Sl. No. of sub water-shed	Overland length (m)	Right bank sub water-shed areas (A) (ha)	Left bank sub water-shed areas (B) (ha)	Overland slope	Channel length (m)	Channel slope	Channel bed width (m)
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
	1(A)	400.00	36.00	-	0.0012	900.00	0.00044	1.50
	1(B)	900.00	-	81.00	0.0013			
	2(A)	750.00	56.25	-	0.0013	750.0	0.00044	1.50
	2(B)	750.00	-	56.25	0.0010			
I	3(A)	650.00	68.25	-	0.0010	1050.0	0.00045	2.00
	3(B)	650.0	-	68.25	0.0012			
	4(A)	750.0	90.00	-	0.0013	1200.0	0.00045	2.00
	4(B)	850.0	-	102.00	0.00099			
	5(A)	1450.0	87.00	-	0.0013	600.0	0.00046	2.00
	5(B)	950.0	-	57.00	0.0012			
	6(A)	1650.0	123.50	-	0.0014	750.0	0.00047	2.50
	6(B)	700.0	-	52.50	0.00091			
	7(A)	1750.0	131.25	-	0.0014	750.0	0.00055	2.50
	7(B)	850.0	-	63.75	0.0014			
Total			592.25	480.75				

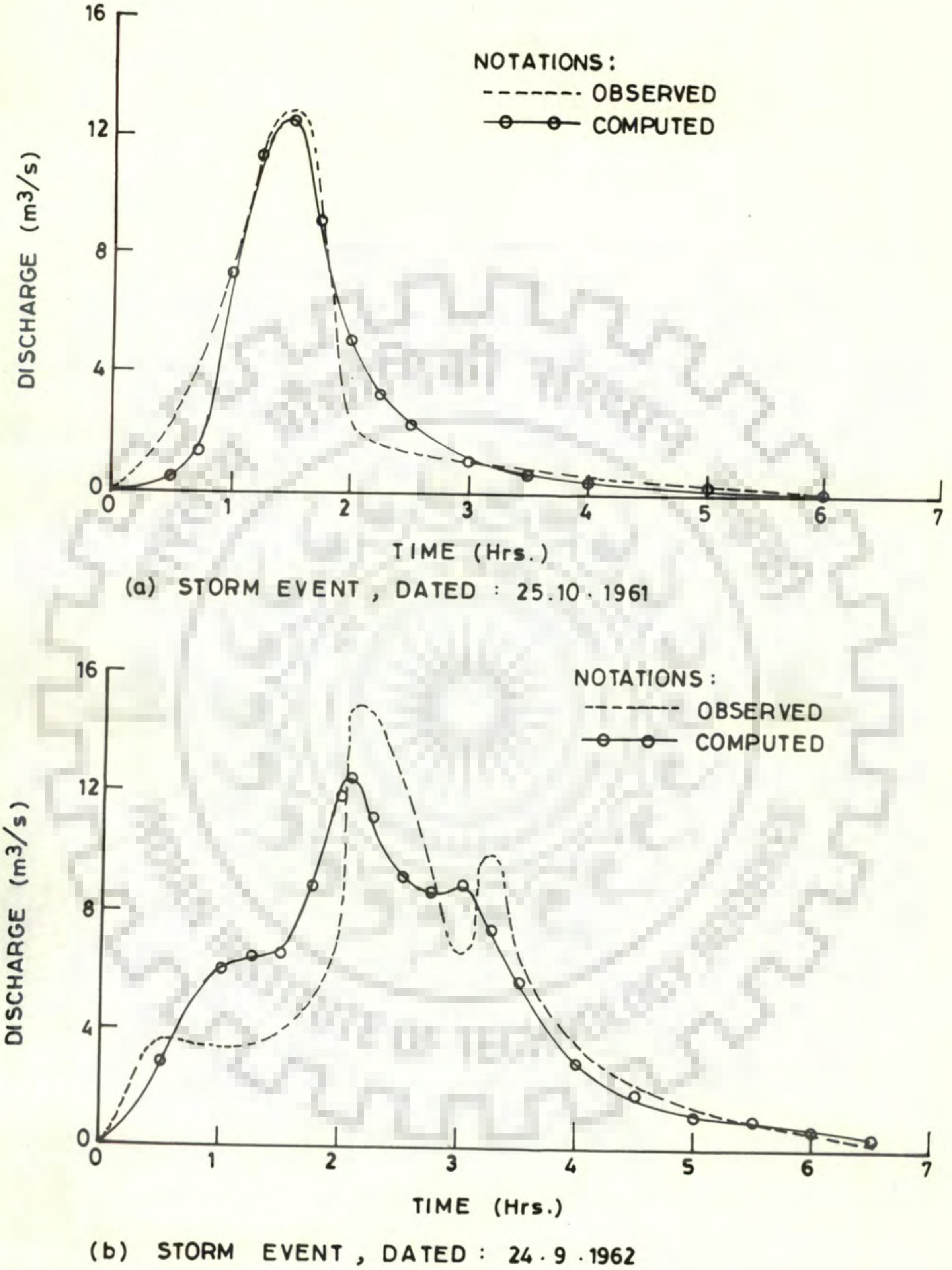
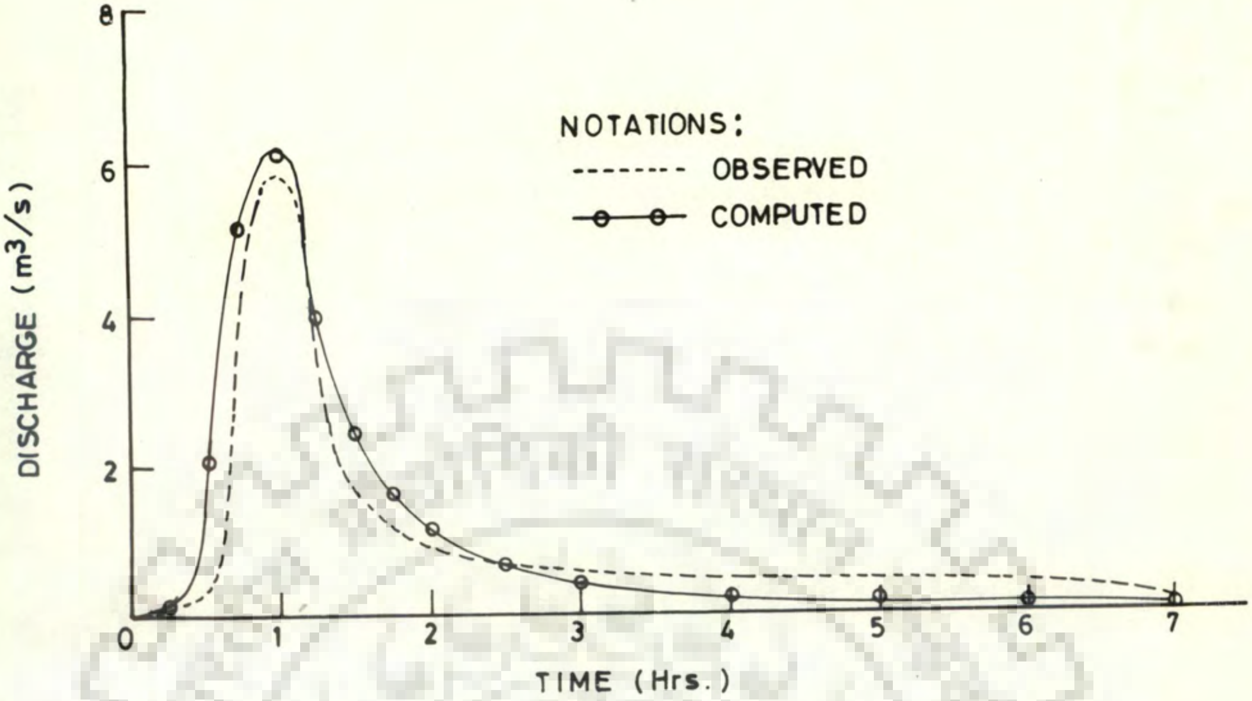
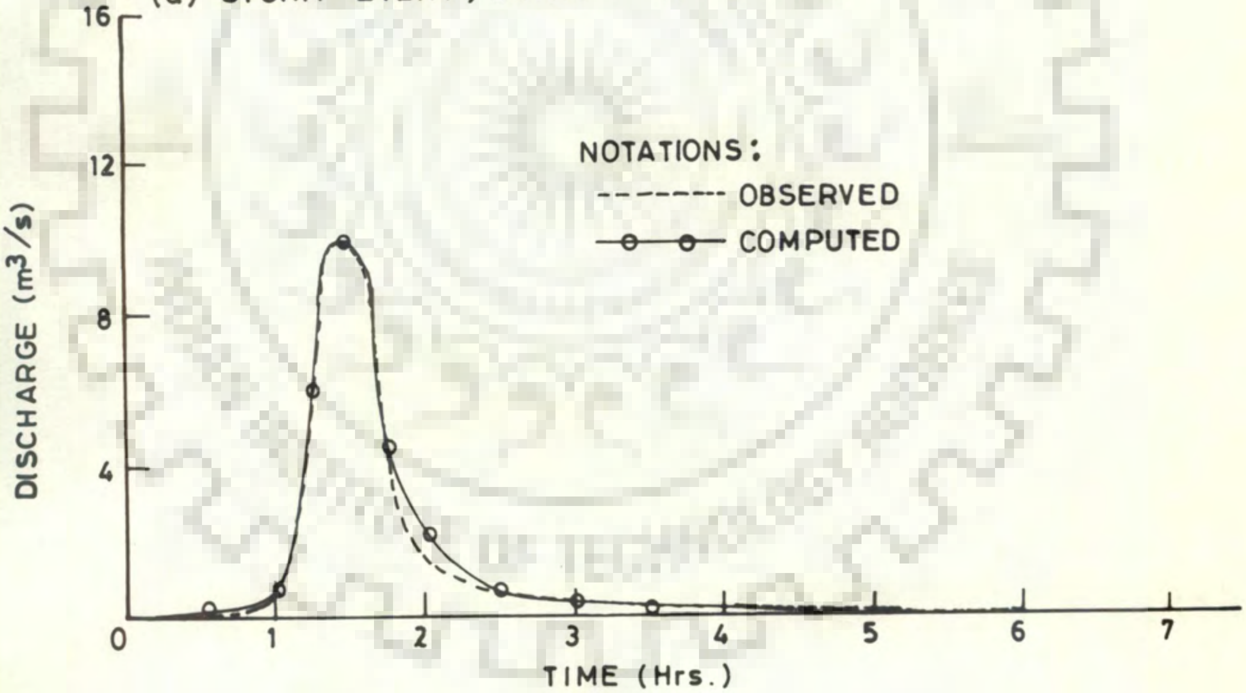


FIG. 5.25 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 317)

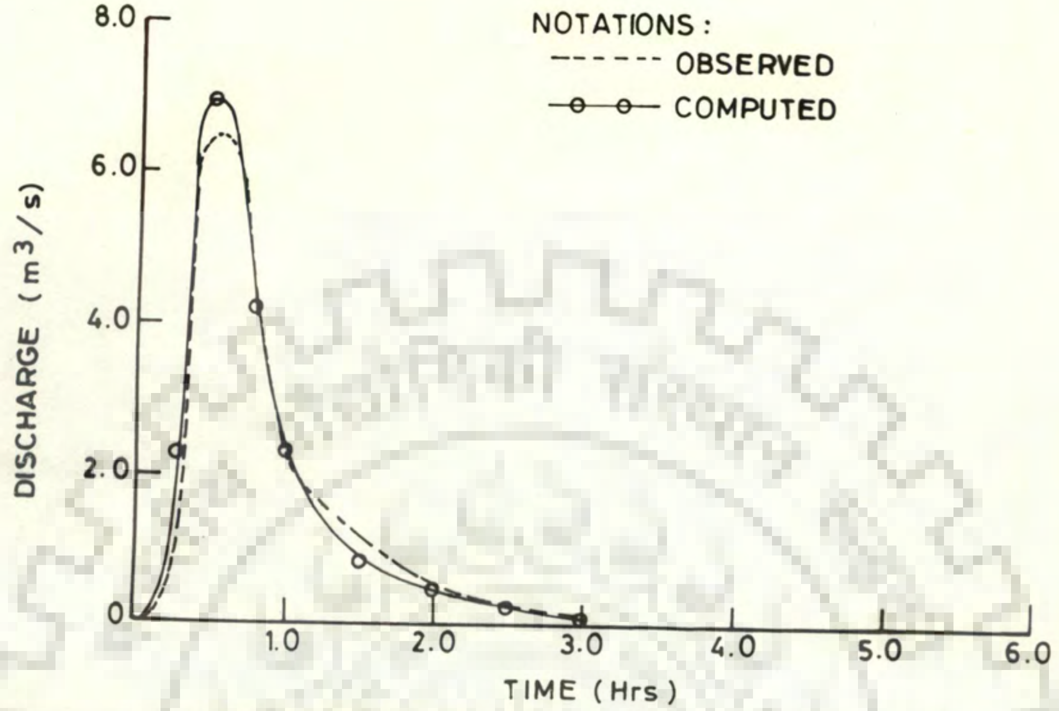


(a) STORM EVENT, DATED : 14 · 10 · 1962

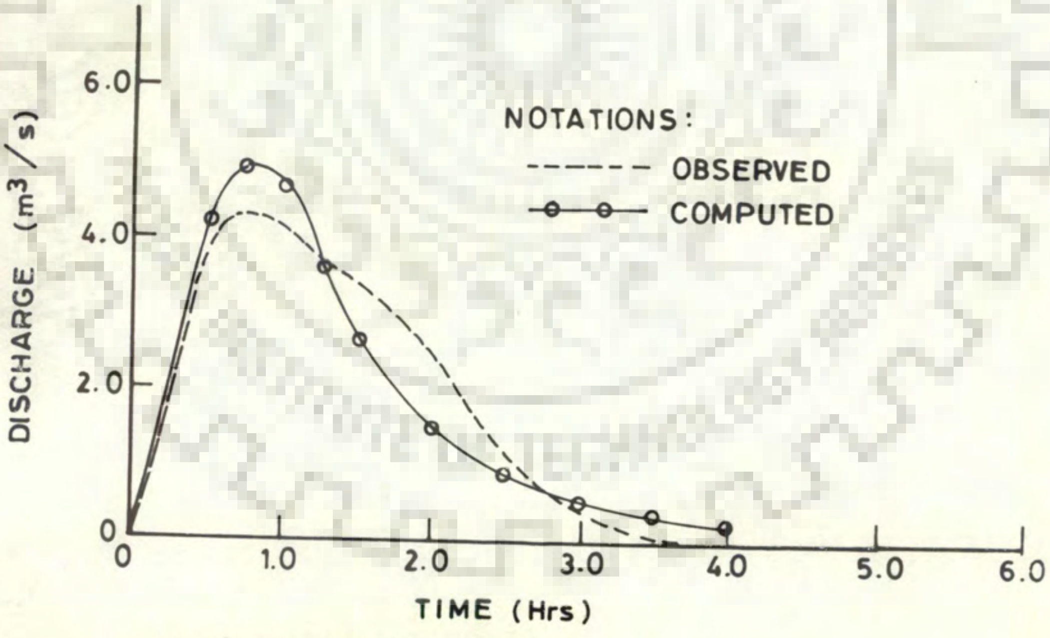


(b) STORM EVENT, DATED : 18 . 7 . 1963

FIG. 5.26 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 317)



(a) STORM EVENT, DATED : 26 . 7 . 1964



(b) STORM EVENT, DATED : 7/8 . 9 . 1964

FIG. 5. 27 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 317)

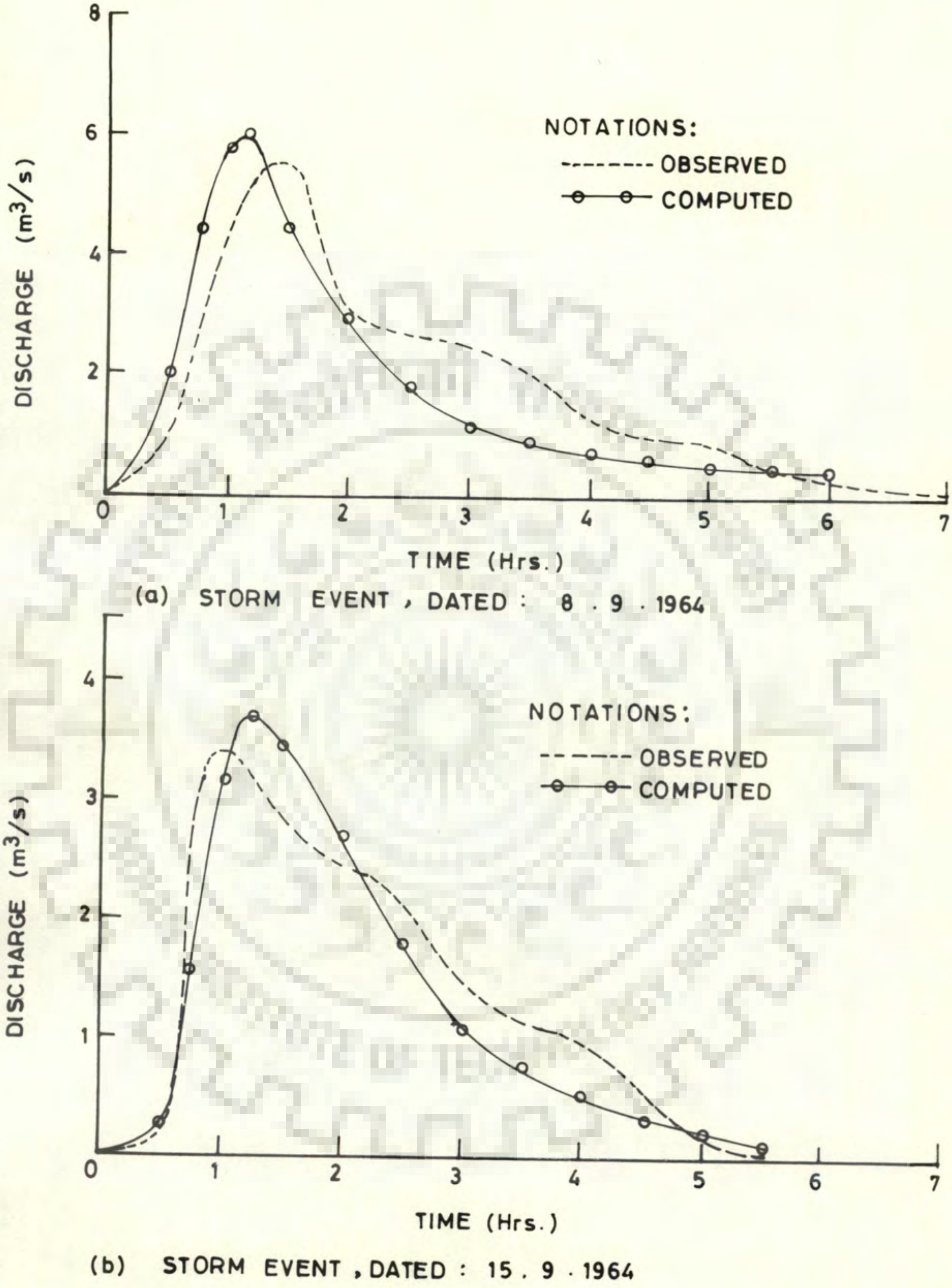
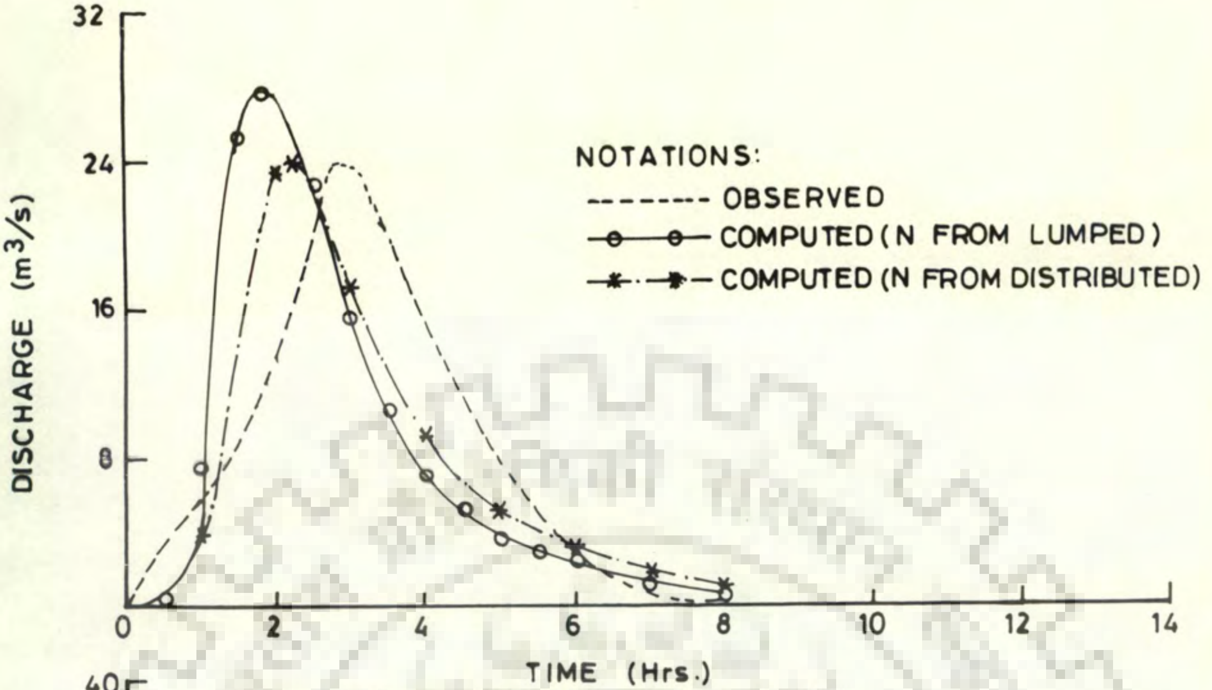
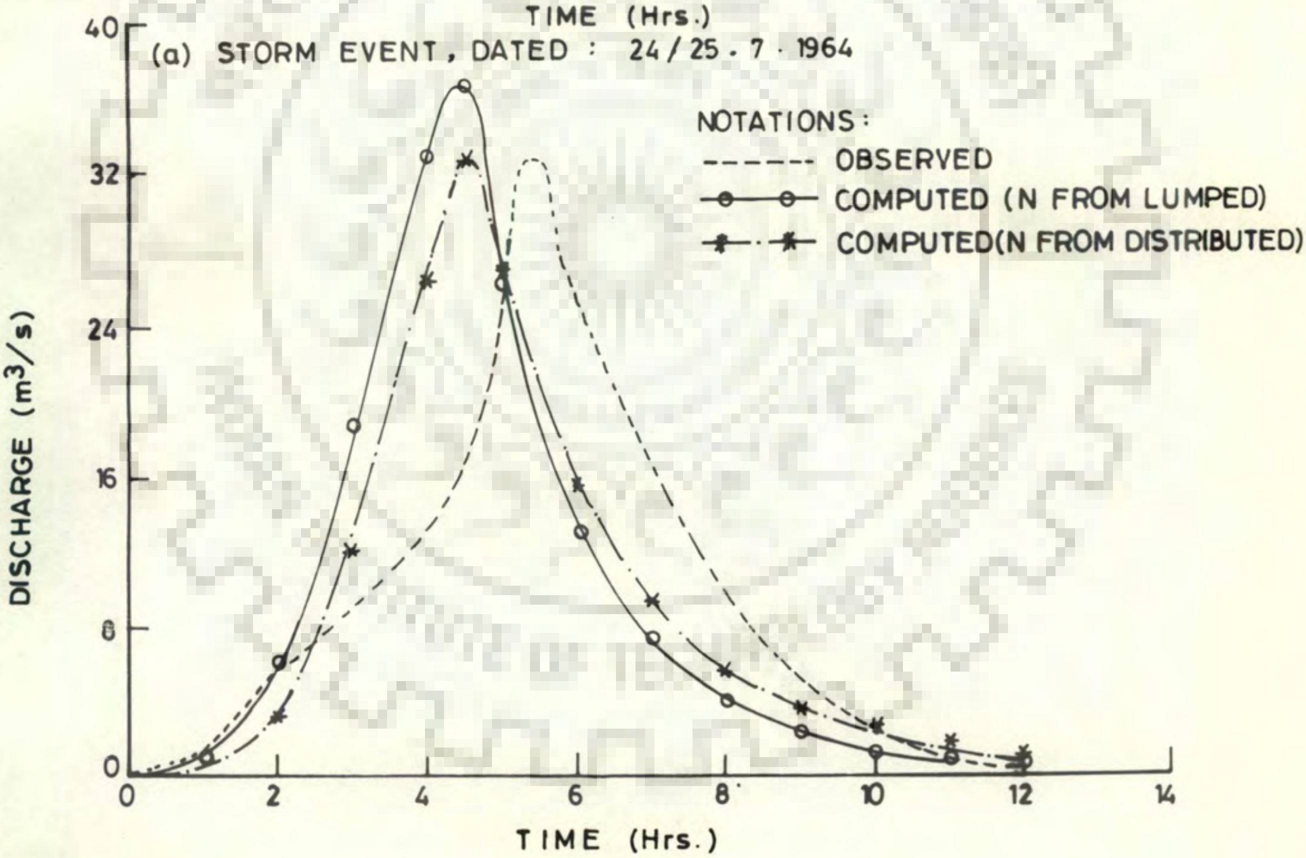


FIG. 5.28 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO.317)



(a) STORM EVENT, DATED : 24/25 . 7 . 1964



(b) STORM EVENT, DATED : 26/27 . 7 . 1964

FIG. 5. 29 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 719)

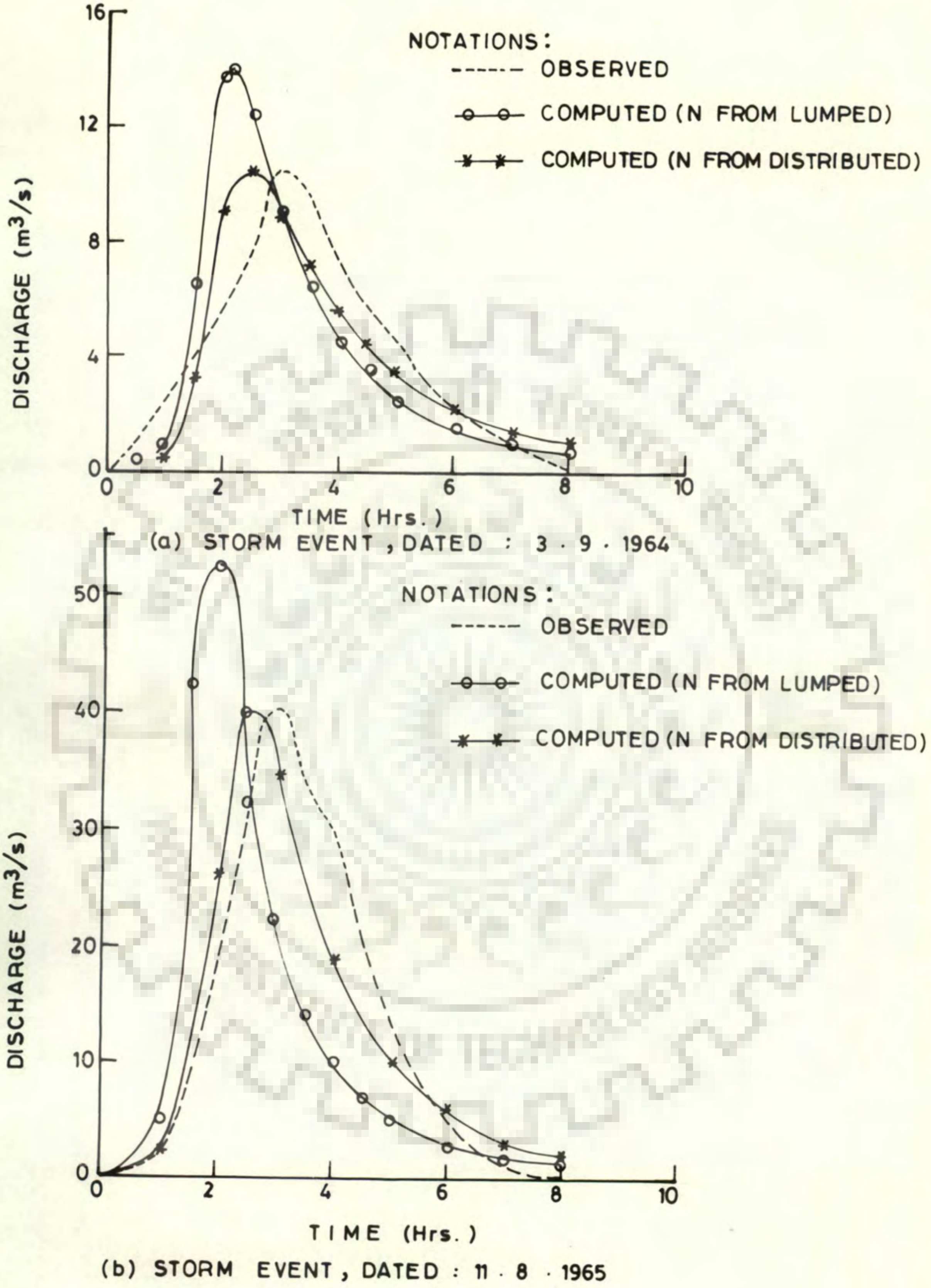
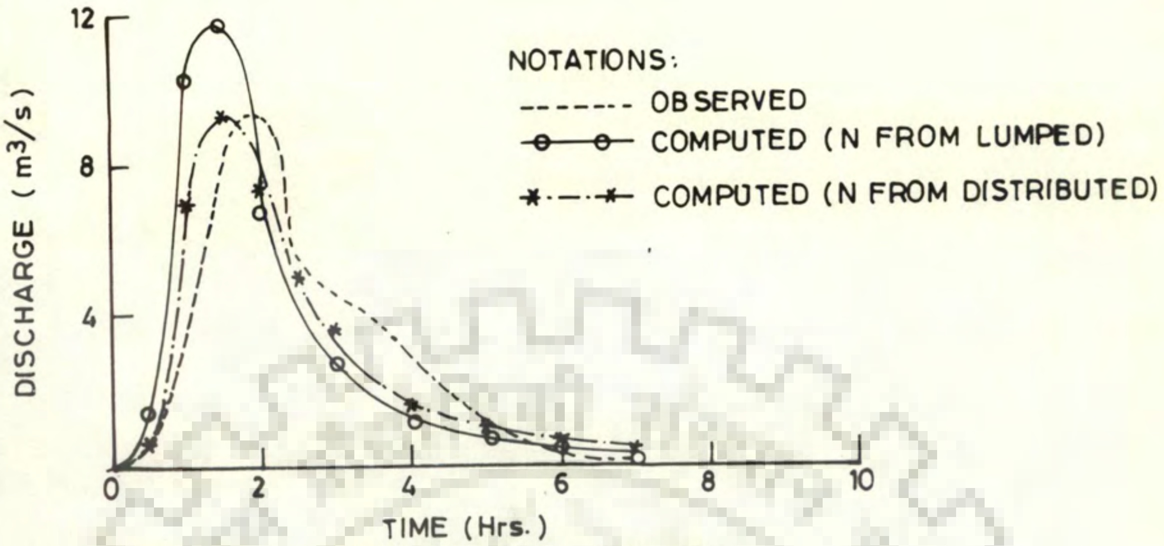
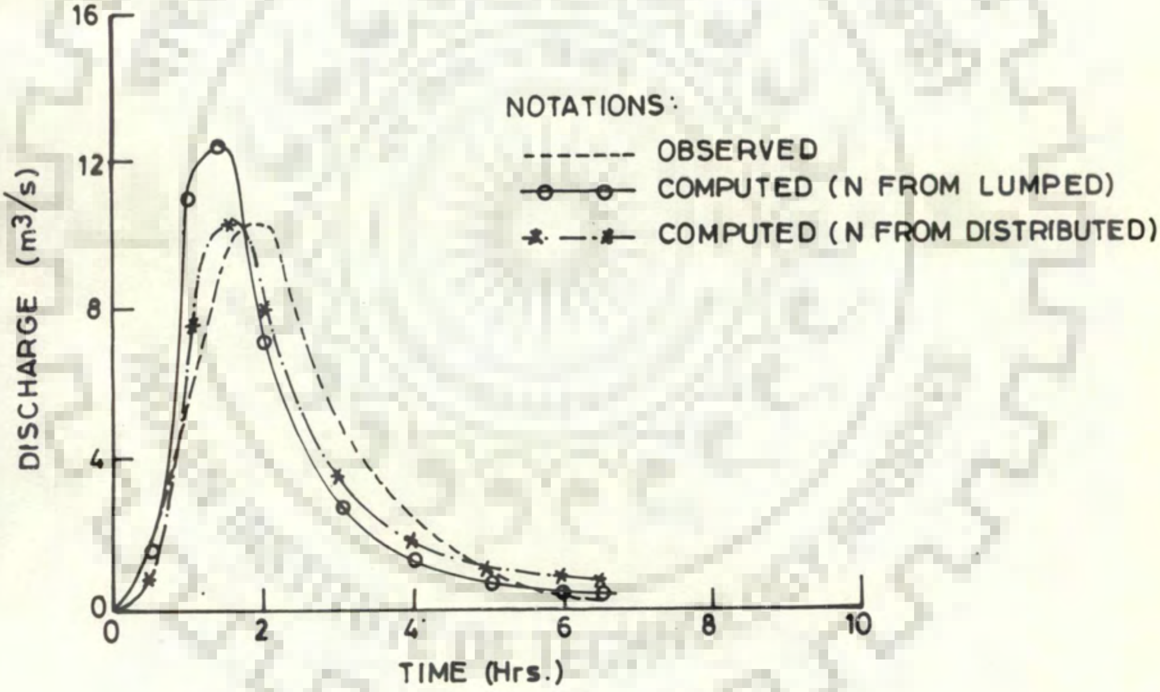


FIG. 5.30 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 719)

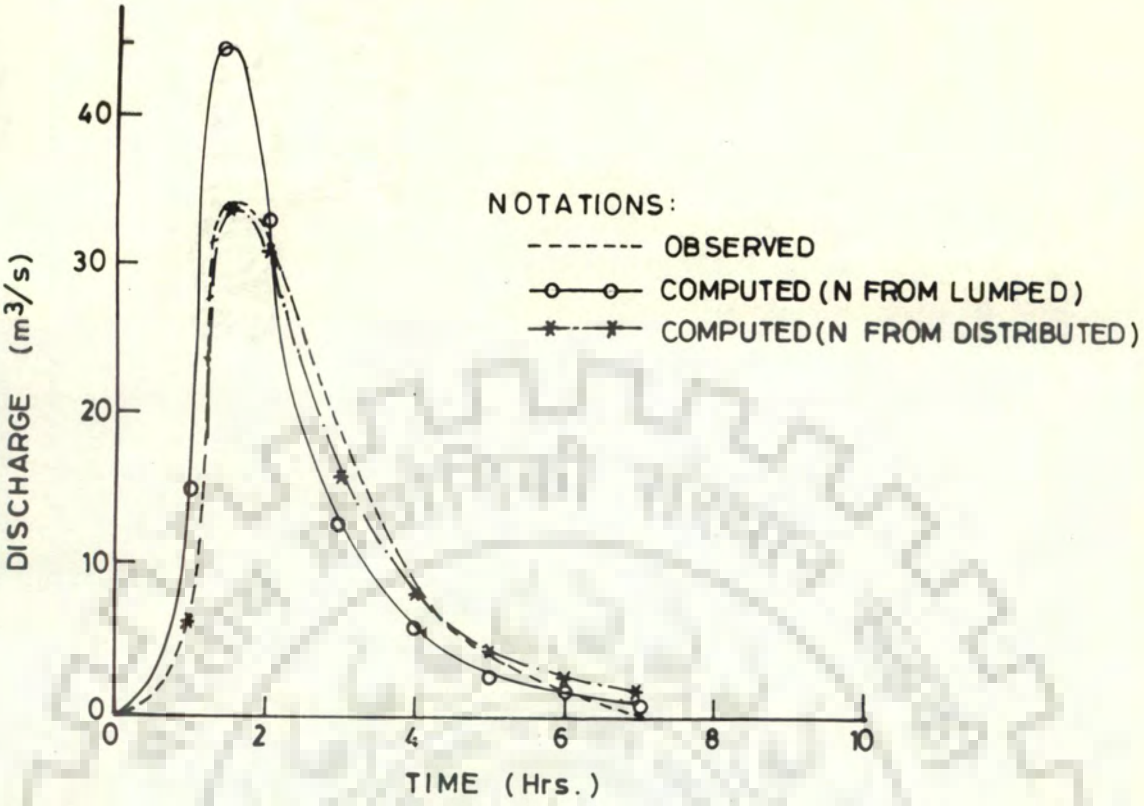


(a) STORM EVENT, DATED : 18.9.1965

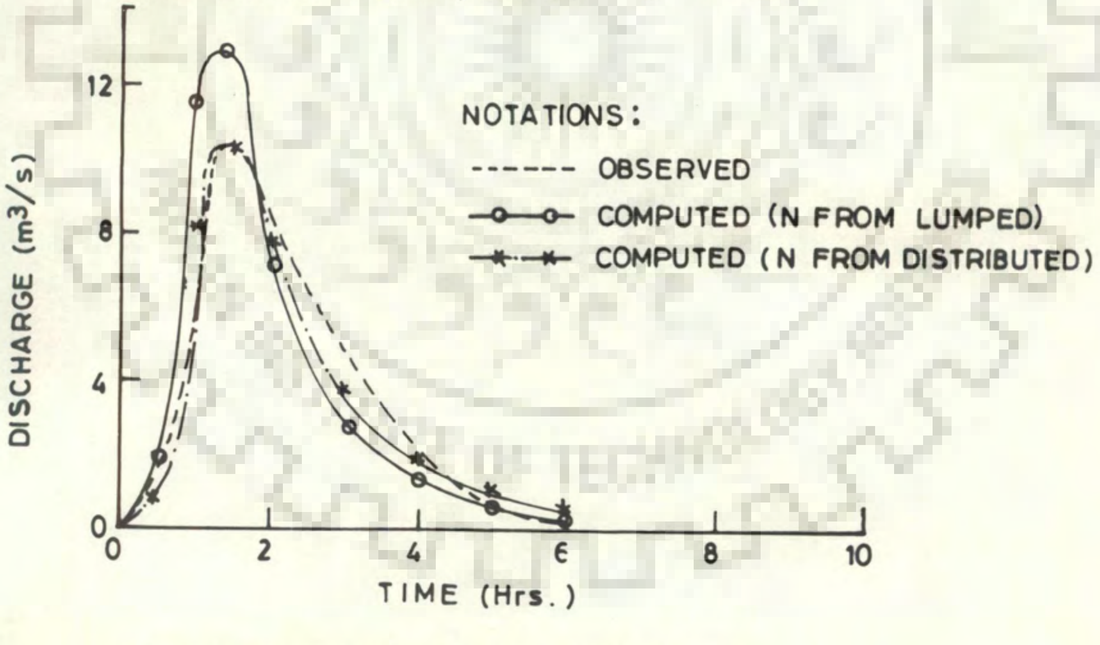


(b) STORM EVENT, DATED : 6.7.1966

FIG. 5.31 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 719)



(a) STORM EVENT, DATED : 16 . 9 . 1966



(b) STORM EVENT, DATED : 20 .9 . 1966

FIG. 5.32 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 719)

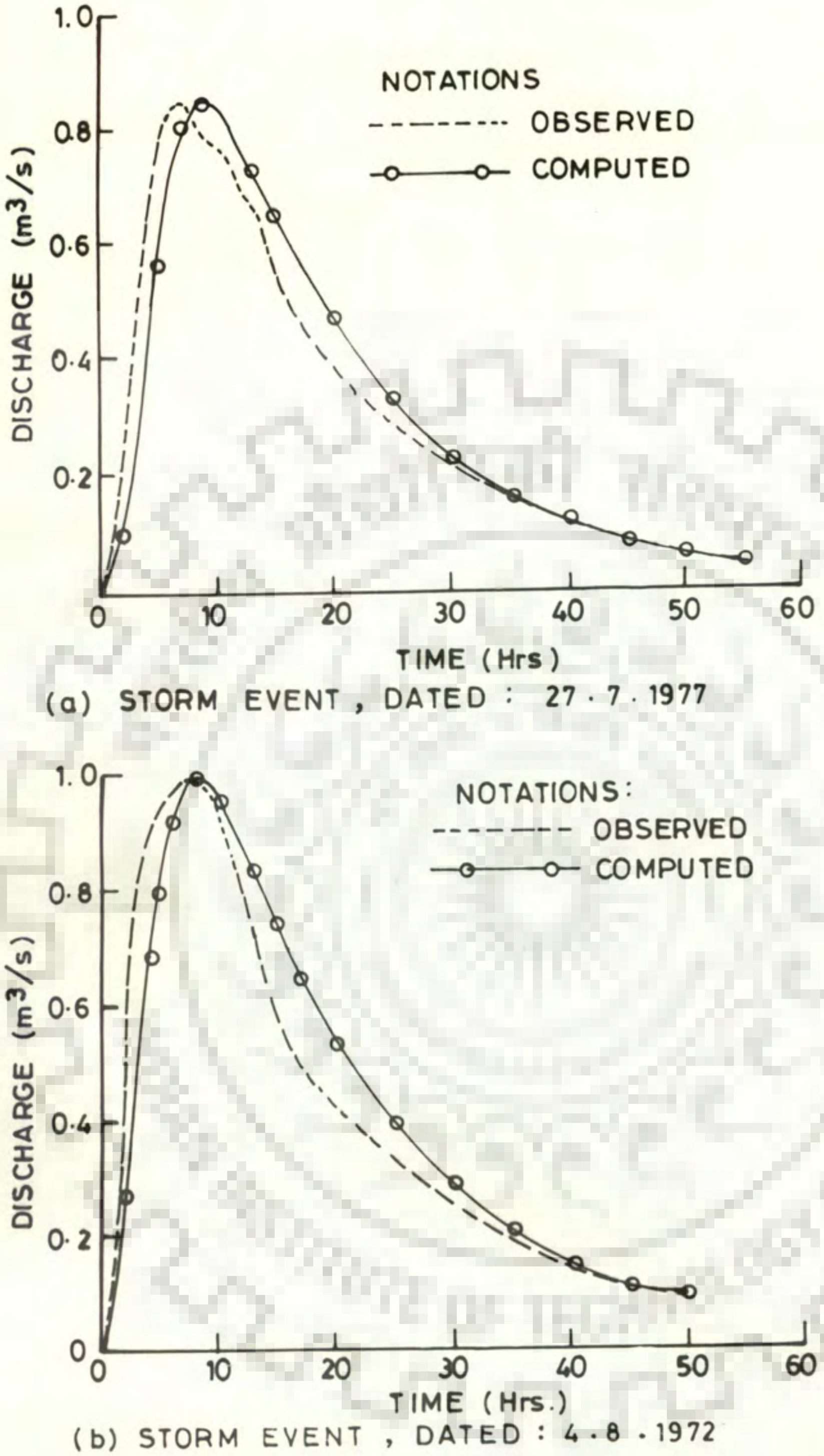
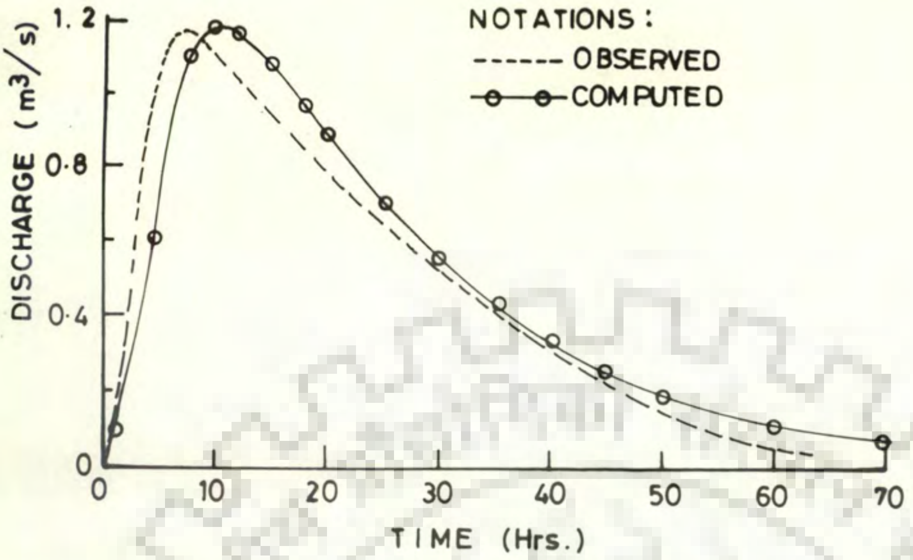
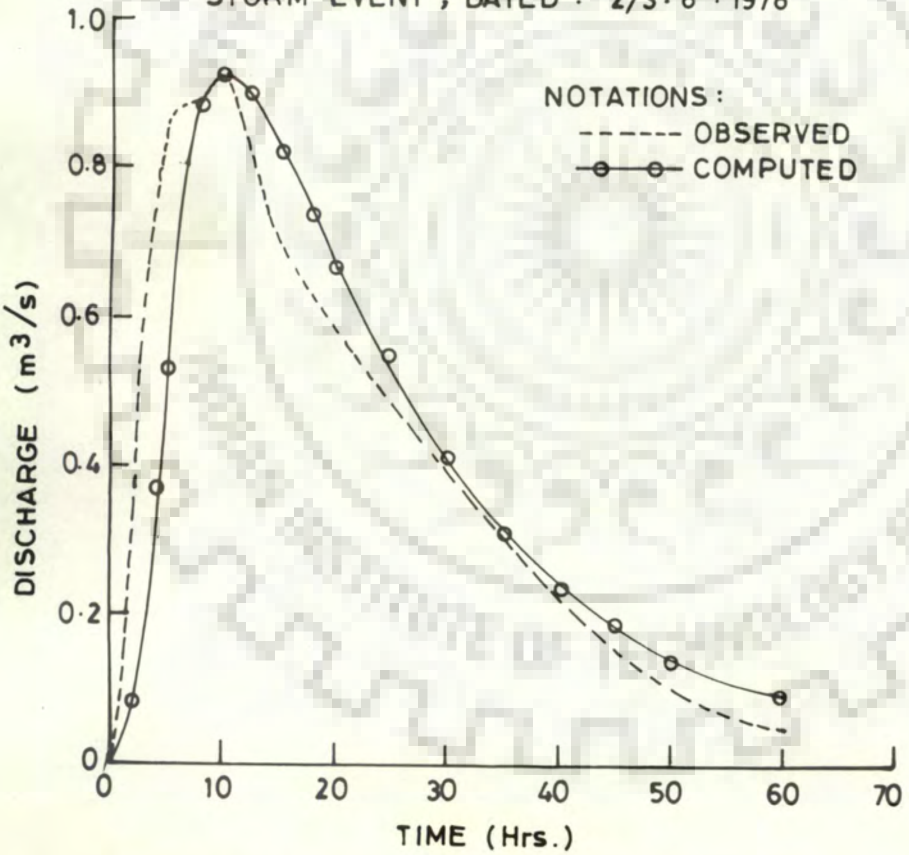


FIG. 5.33 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (KACHWA WATERSHED)



STORM EVENT, DATED : 2/3 . 8 . 1978



STORM EVENT, DATED : 8/9 . 8 . 1978

FIG. 5.34 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (KACHWA WATERSHED)

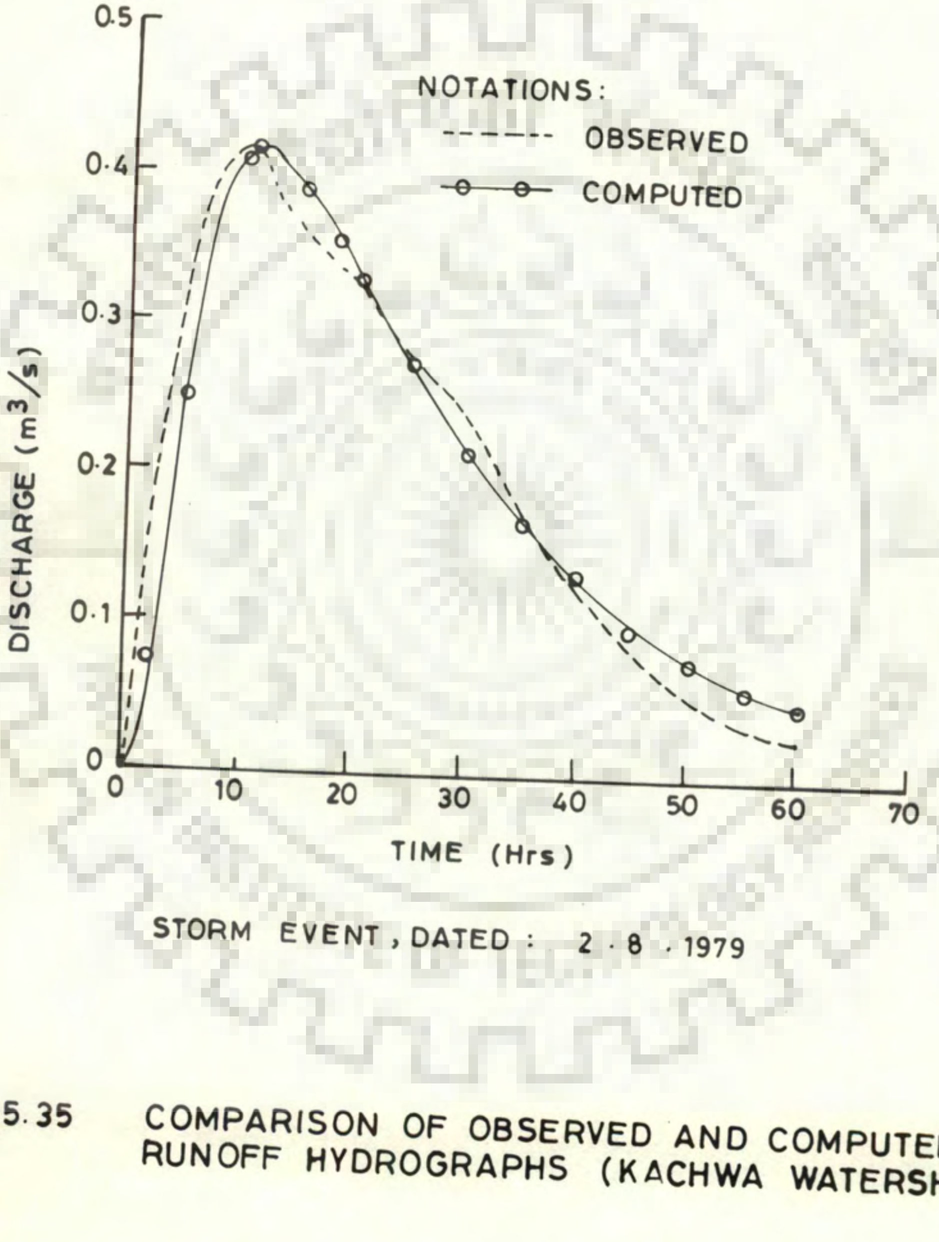


FIG. 5.35 COMPARISON OF OBSERVED AND COMPUTED DIRECT RUNOFF HYDROGRAPHS (KACHWA WATERSHED)

TABLE 5.28 : Observed and Computed Hydrograph Parameters Using KW Model of Scheme (I & II) (Bridge No. 317)

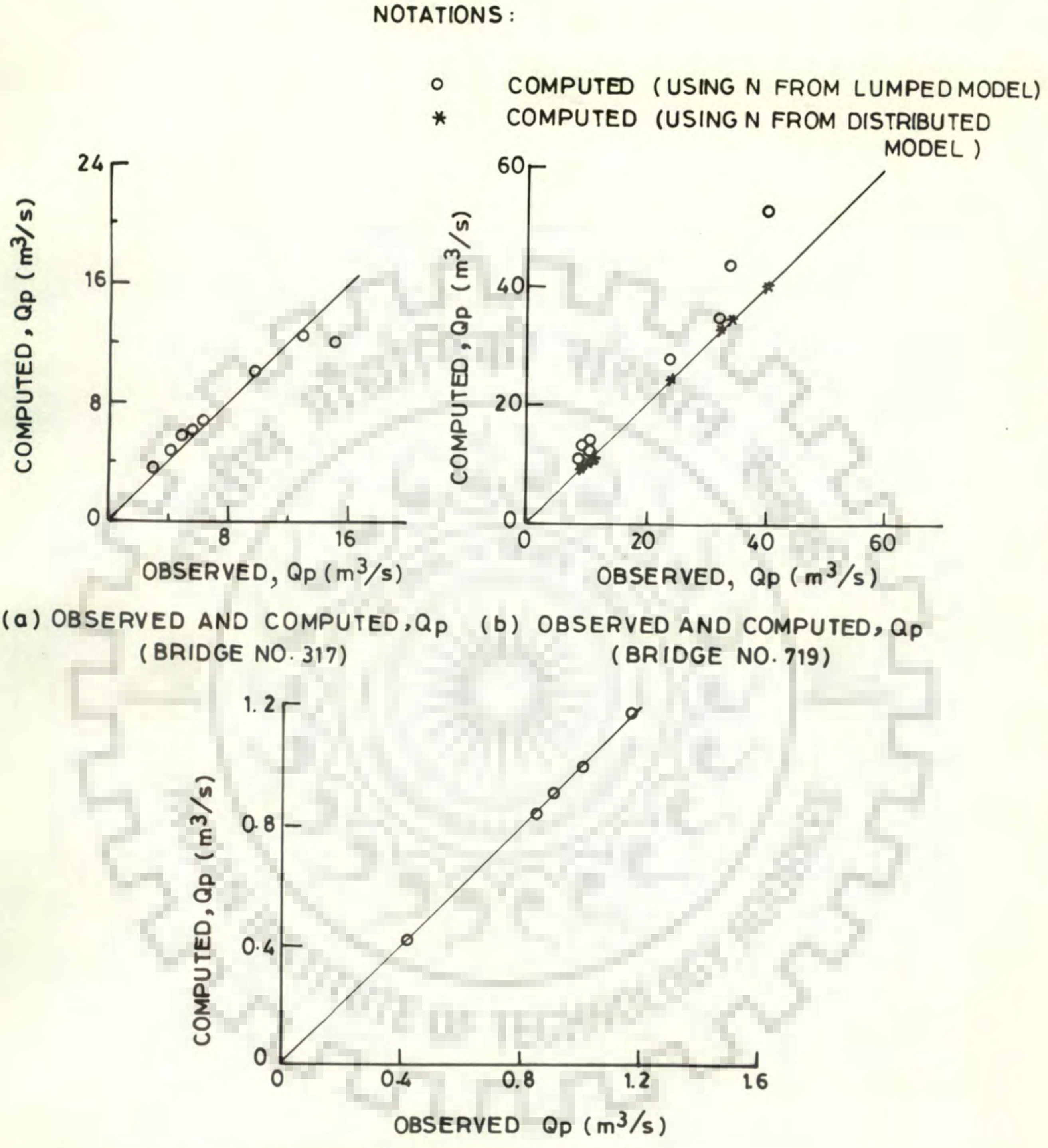
Sl. No.	Storm Date	Peak Discharge (m^3/s)		Time to Peak (min)		Volume (m^3) x 10^3		Volume error (%)	Model Efficiency R^2 (%)
		Observed	Computed	Observed	Computed	Observed	Computed		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	25.10.1961	12.82	12.44	90.0	90.0	178.62	177.074	0.86	90.2
2	24.9.1962	15.10	12.03	130.0	125.0	340.734	339.489	0.36	62.0
3	14.10.1962	5.773	6.212	60.0	60.0	81.110	80.983	0.156	77.8
4	18.7.1963	9.905	10.19	80.0	85.0	81.846	81.680	0.20	98.0
5	26.7.1964	6.481	7.08	30.0	30.0	60.776	60.532	0.40	96.9
6	7/8.9.1964	4.245	5.11	50.0	50.0	94.96	89.450	5.80	81.5
7	8.9.1964	5.098	6.00	90.0	65.0	145.356	-	-	85.3
8	15.9.1964	3.424	3.792	60.0	75.0	93.040	89.051	4.2	90.3

TABLE 5.29 : Observed and Computed Hydrograph Parameters Using KW Model of Scheme (I & II), (Bridge No. 719)

Sl. No.	Storm Date	Peak Discharge (m^3/s)			Time to Peak (min)			Volume (m^3) x 10^3			Volume error (%)		Model Efficiency (%)
		Observed	Computed		Observed	Computed		Observed	Computed				
			Lumped	Distri-buted		Lumped	Distri-buted		Lumped	Distri-buted	Lumped	Distri-buted	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	24/25.7.1964	24.100	28.09	24.20	180.0	110.0	130.0	254.83	244.013	241.432	4.2	5.2	48
2	26/27.7.1964	32.709	35.80	32.68	330.0	260.0	270.0	422.58	409.146	405.329	3.1	4.1	46
3	3.9.1964	10.480	14.02	10.50	180.0	130.0	150.0	125.06	119.290	116.660	4.6	6.7	69
4	3.9.1964	40.022	54.176	39.95	180.0	120.0	150.0	391.07	376.302	368.435	3.7	5.7	79
5	18.9.1965	9.350	11.80	9.42	120.0	80.0	90.0	78.584	77.314	75.331	1.6	4.1	78
6	6.7.1966	10.365	12.706	10.284	120.0	80.0	90.0	85.27	81.853	79.649	4.0	6.5	75
7	16.9.1966	33.980	45.80	33.89	90.0	80.0	90.0	285.59	276.503	268.104	3.2	6.1	97
8	20.9.1966	10.300	13.119	10.48	90.0	80.0	90.0	79.72	78.595	77.40	1.4	2.9	88

TABLE 5.30 : Observed and Computed Hydrograph Parameters Using KW Model of Scheme III (Kachwa Watershed)

Sl. No.	Storm Date	Peak Discharge (m^3/s)		Time to Peak (hr.)		Volume (m^3) x 10^3		Volume error (%)	Model Efficiency R^2 (%)
		Observed	Computed	Observed	Computed	Observed	Computed		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
1	27.7.1977	0.851	0.847	7.0	9.0	60.08	60.04	0.07	95.0
2	4.8.1977	0.993	0.992	8.0	8.0	63.00	62.64	0.57	92.0
3	2/3.8.1978	1.178	1.194	7.0	10.0	45.07	44.90	0.38	86.0
4	8/9.8.1978	0.918	0.928	10.0	10.0	115.91	115.826	0.07	86.0
5	2.8.1979	0.417	0.422	10.0	11.0	88.164	88.200	-0.04	95.0

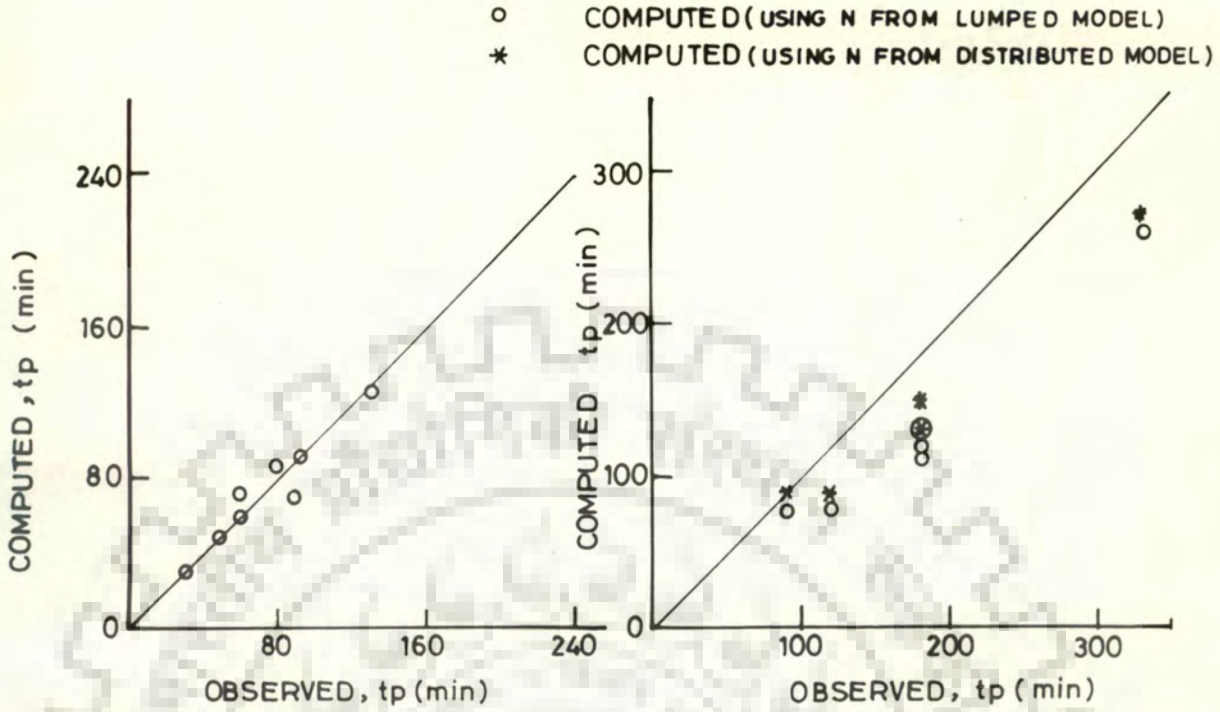


(a) OBSERVED AND COMPUTED, Qp (BRIDGE NO. 317)

(b) OBSERVED AND COMPUTED, Qp (BRIDGE NO. 719)

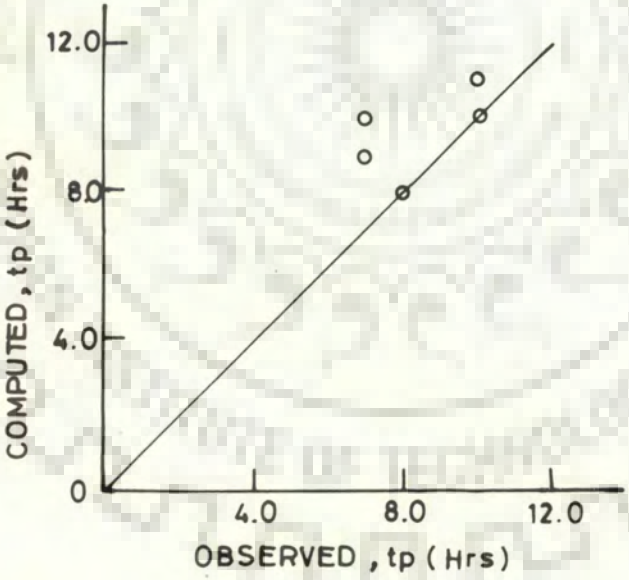
(c) OBSERVED AND COMPUTED, Qp (KACHWA WATERSHED)

FIG. 5.36 COMPARISON OF OBSERVED AND COMPUTED PEAK DISCHARGE OF THE THREE WATERSHEDS (BRIDGE NO. 317 , 719 AND KACHWA)



(a) OBSERVED AND COMPUTED, tp (BRIDGE NO. 317)

(b) OBSERVED AND COMPUTED, tp (BRIDGE NO. 719)



(c) OBSERVED AND COMPUTED, tp (KACHWA WATERSHED)

FIG. 5.37 COMPARISON OF OBSERVED AND COMPUTED TIME TO PEAK OF THE THREE WATERSHEDS (BRIDGE NO. 317 719 AND KACHWA)

CHAPTER - VI

EXTENSION OF PROPOSED CONCEPTS TO UNGAUGED WATERSHEDS

6.1 INTRODUCTION

In the previous chapter, proposed KW models developed for distributed systems were applied to three small gauged natural hilly watersheds. Also these applications were discussed for an agricultural watershed. Such attempts mainly remain of accademic interest and do not find their practical field applications unless they can be suitably applied to the ungauged watersheds. For this purpose, appropriate relationships are to be developed. The term "Ungauged" is used in a sense that information pertaining to physiography, land use, soil cover is available. The rainfall excess data is either available for the storm events recorded in the past or their future reliable estimates can be had from the information of the nearby larger system to which the ungauged watershed belongs. However, in any case, no gauged runoff responses are availbale for identification of the system. The usual attempts are directed towards correlating the vital hydrograph parameters with the watershed physiography and the causative excitation function i.e. the rainfall in this case. For such attempts, it is usual that the proposed models should be applied on to a large number of watersheds. Having identified these systems correctly, their hydrologic responses and the corresponding physiographic details along with the input functions are utilised to develop suitable relationships for their extension to ungauged watersheds.

This work is handicapped because data are available for only three gauged hilly small natural watersheds of the 'Plateau of Deccan'. Nevertheless, attempts are made to develop suitable relationships in the forms of nomogram and regression equations. As and when data of more gauged watersheds would be available, the proposed KW models can be extended to them and the relationships arrived at in the forthcoming sections can further be modified.

6.2 THE PHYSIOGRAPHIC DATA

Normally topographic sheets of Survey of India are available for most parts of the Indian main land. Following information can suitably be extracted by appropriately selecting a topo-sheet of the desired scale. Usually 1:50,000 scale is the right choice for such studies.

(1) The water divide of the watershed and its watershed area, width of the watershed etc.

(2) The channel network, length of the longest (main) channel, length of the main channel from the outlet to the point opposite the C.G. of watershed.

(3) Contributing overland areas for various channels

(4) Contours, spot levels for computing the watershed slopes (i.e. the overland as well as for the channels).

In the proposed concepts, the watershed effective overland roughness plays an important role as it has been found to be the most sensitive parameter. A correct description of effective overland roughness will be needed for the ungauged watersheds for suitably applying the proposed concepts to them. For this purpose, the effective overland roughnesses worked out in the previous applications in 3 watershed (Figures 5.5, 5.22 and 5.23) have been used as indices. The fortnightly average effective overland roughnesses as worked out for the three watersheds are given in figures 6.1(a) through 6.1(c).

Keeping in view, the size of the watershed as well as the rainfall period appropriate values of the effective overland roughnesses can be picked up from these three curves.

6.3 PROPOSED APPROACHES

The peak discharge ordinate has been considered to be a dependent function. This is to be determined in terms of independent physiographic and meteorologic variables. In the proposed theory, the effective physiographic parameters are the watershed area (A), length of the main channel (L_c), effective overland plane length (L_o), effective overland roughness (N), overland slope (S_o), channel roughness (n), channel slope (S) and cross-section of the water area in channel. The meteorologic parameters used are rainfall excess intensity (I) and its duration (D).

Keeping in view, the limited data base, the choice of the independent variables have been restricted to the following :

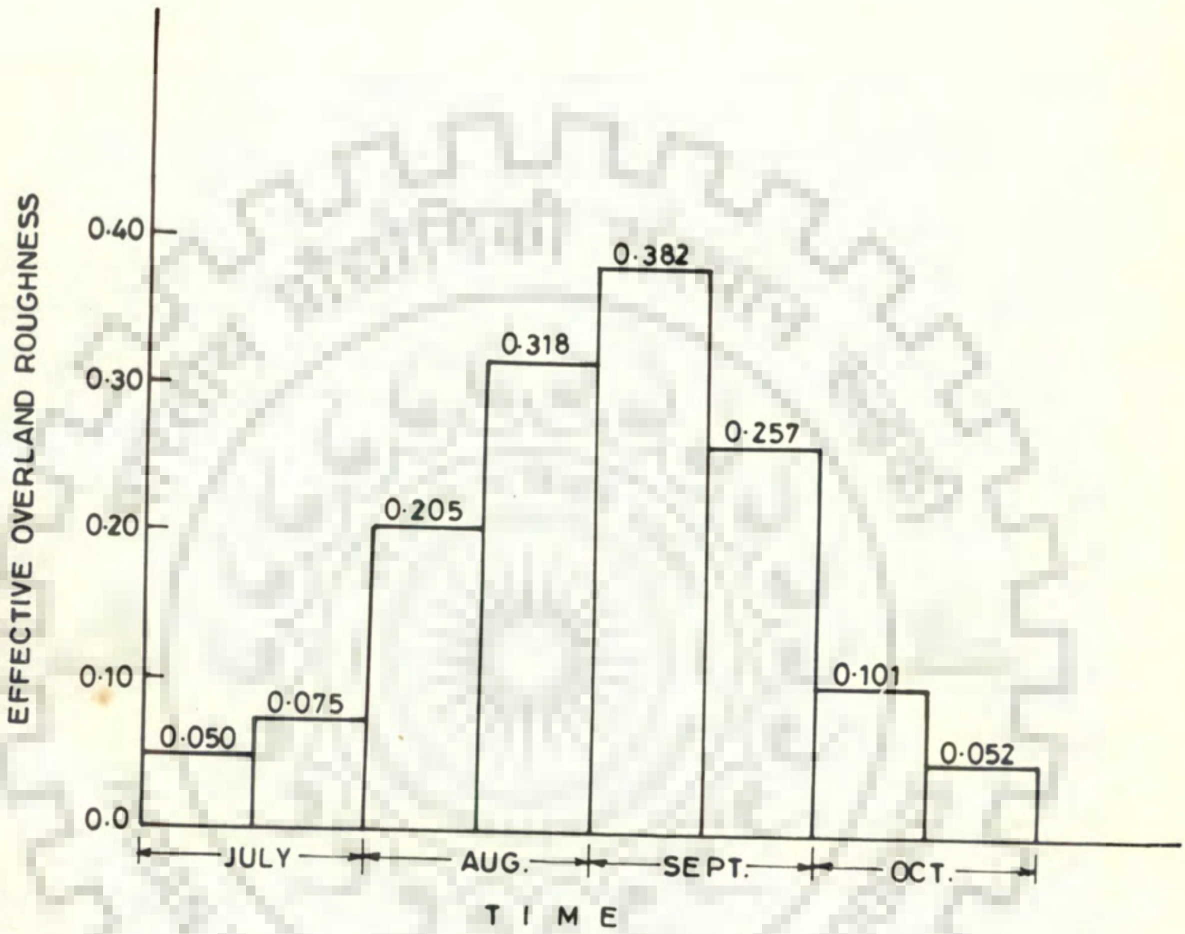


FIG. 6.1 (a) FORTNIGHTLY AVERAGE EFFECTIVE OVERLAND ROUGHNESS FOR 4 MONSOON MONTHS (JULY TO OCTOBER, BRIDGE NO. 319)

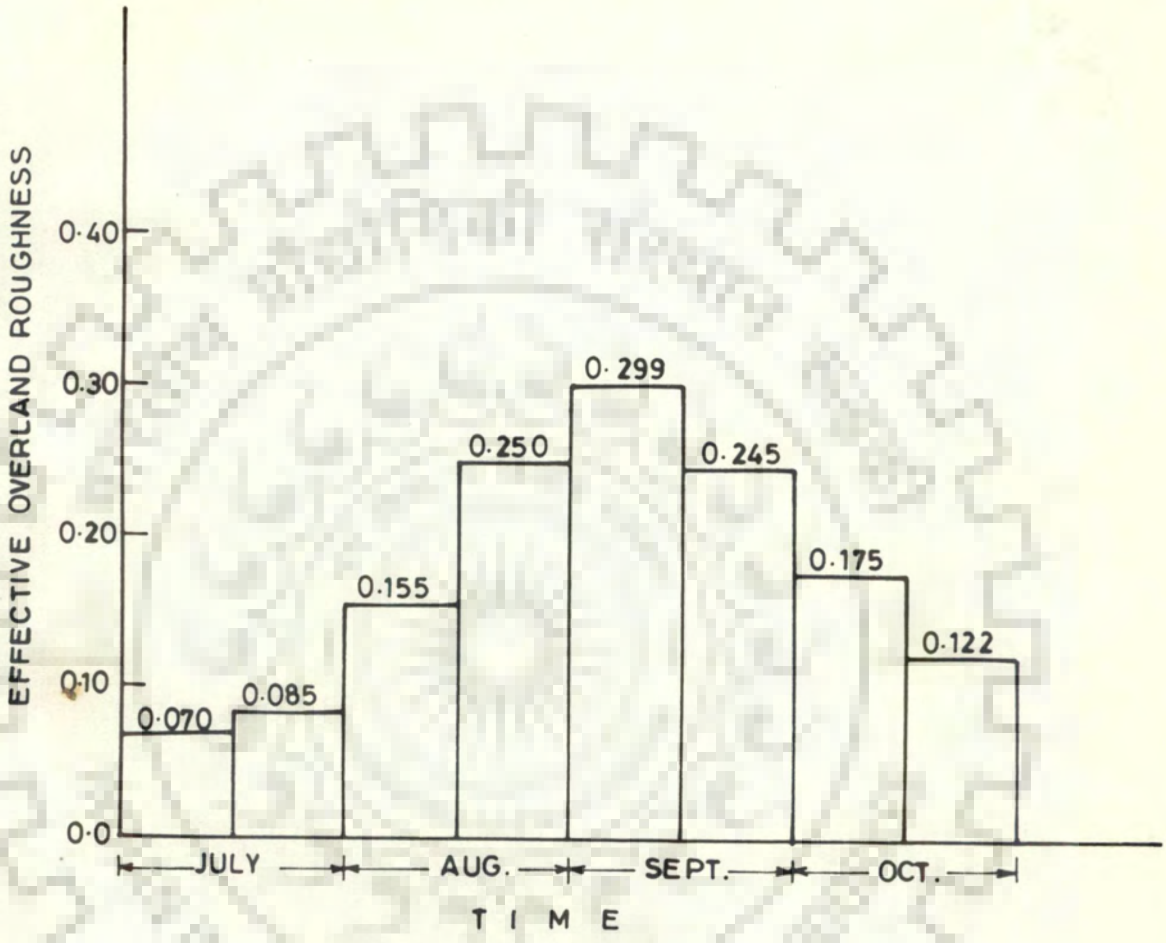


FIG. 6.1(b) FORTNIGHTLY AVERAGE EFFECTIVE OVERLAND ROUGHNESS FOR 4 MONSOON MONTHS (JULY TO OCTOBER, BRIDGE NO.317)

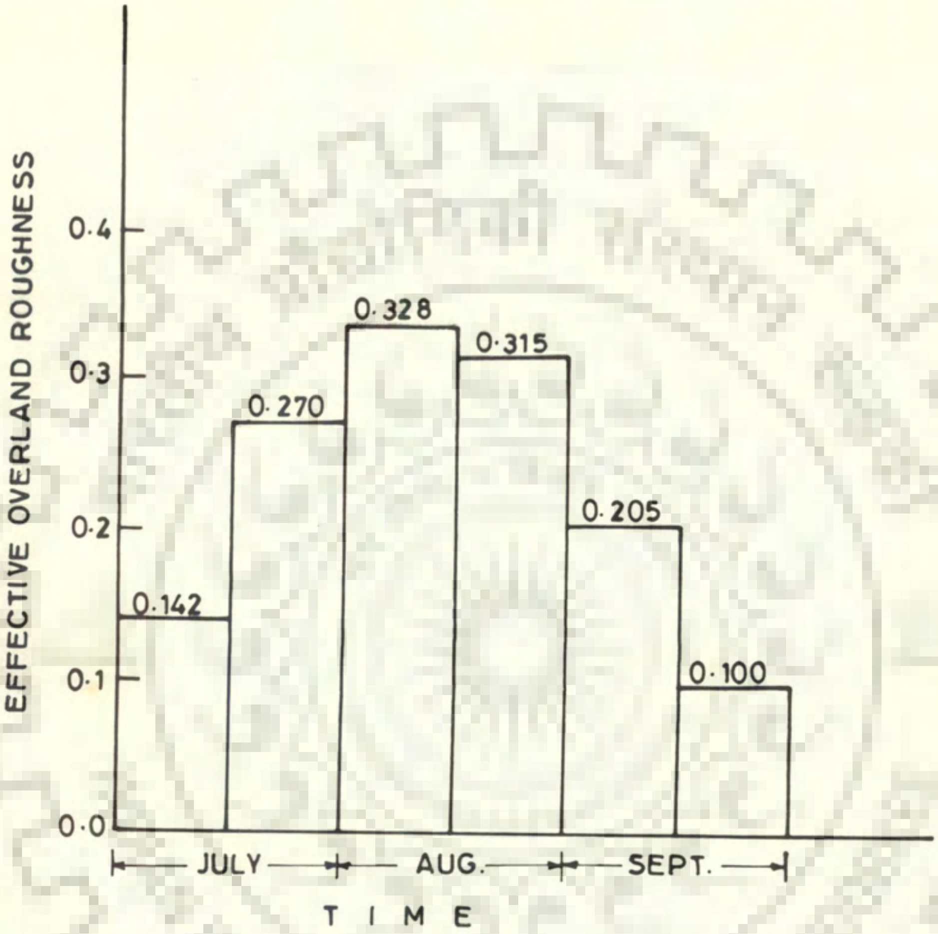


FIG. 6.1 (c) FORTNIGHTLY AVERAGE EFFECTIVE OVERLAND ROUGHNESS FOR 3 MONSOON MONTHS (JULY TO SEPT., BRIDGE NO. 719)

(1) Physiographic parameters : Watershed area (A), effective overland roughness (N), length of the main channel (L_c).

(2) Meteorologic parameters : Intensity of rainfall excess (I) and its duration (D) [or depth of rainfall excess (d)].

It will not be out of place to mention that the computation of rainfall excess function for the given gross rainfall continues to be the weakest link in surface hydrologic computations. Like most analyses, in this work too, the rainfall excess functions have been computed by assuming constant rate of abstractions (i.e. the Ø-index). Thus, it is necessary to have an idea of Ø-index along with the gross rainfall data to compute the rainfall excesses functions.

The following approaches have been tried for obtaining the responses for the ungauged watersheds.

(1) Nomogram approach, and
(2) Regression approach.

A brief description of the above approaches is given in the forthcoming sections.

6.4 NOMOGRAM APPROACH

Owing to limited data availability (i.e. of only 3 watersheds and total number of 26 storm events), the watershed area could not be taken up as variable. The dependent variable i.e.

the peak discharge (Q_p) is considered a function of effective overland roughness (N), rainfall excess intensity (I) and its duration (D).

For the watershed of Bridge No. 319 (82.0 ha), the relationships in the form of nomograms are plotted in figures 6.2(a) through 6.2(c). In the relationships, curve drawn by the firm lines are for the watershed of Bridge No. 319. The relationships given by the dotted lines refer to some hypothetical cases of watersheds having the same area (82.0 ha) but different ratios of L_c (length of the channel) and L_o (length of the overland plane). The very high intensity rainfall excesses (20.0, 50.0 and 100.0 mm/hr) have been considered. It is interesting to note the following :

(1) For the same effective overland roughness, as the rainfall excess intensity increases, the time of concentration decreases.

(2) For the same rainfall excess intensity, the increase in effective overland roughness, increases the time of concentration (T_c).

Thus, according to the revealations, the time of concentration (T_c) is not a fixed property (characteristic) of the watershed. It needs further investigations which have been carried out in the forthcoming section.

Concludingly, it may be remarked that similar curves can be prepared for the watersheds of different sizes, different

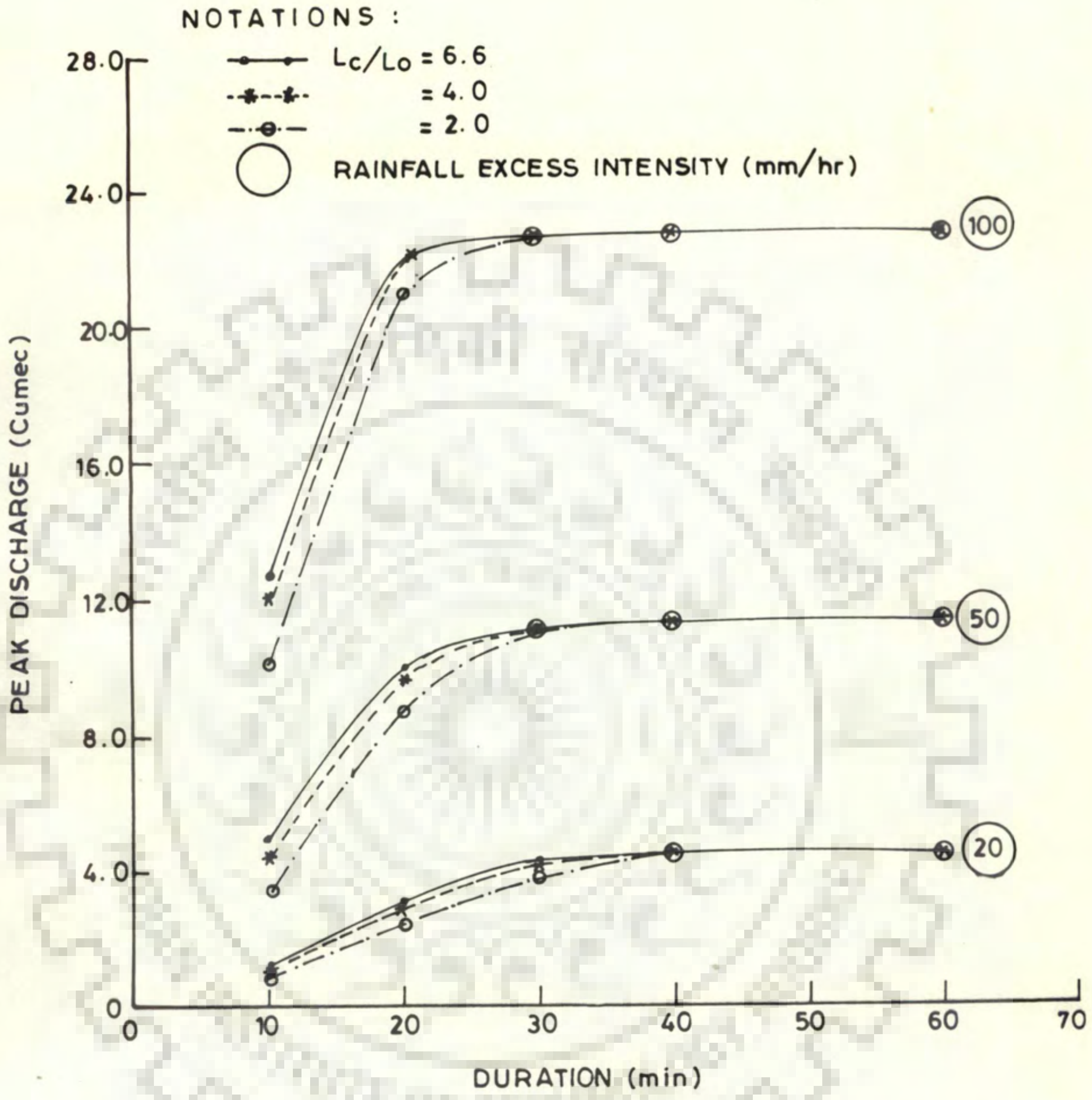


FIG. 6.2 (a) NOMOGRAM OF PEAK DISCHARGE FOR N = 0.05 (BRIDGE NO. 319)

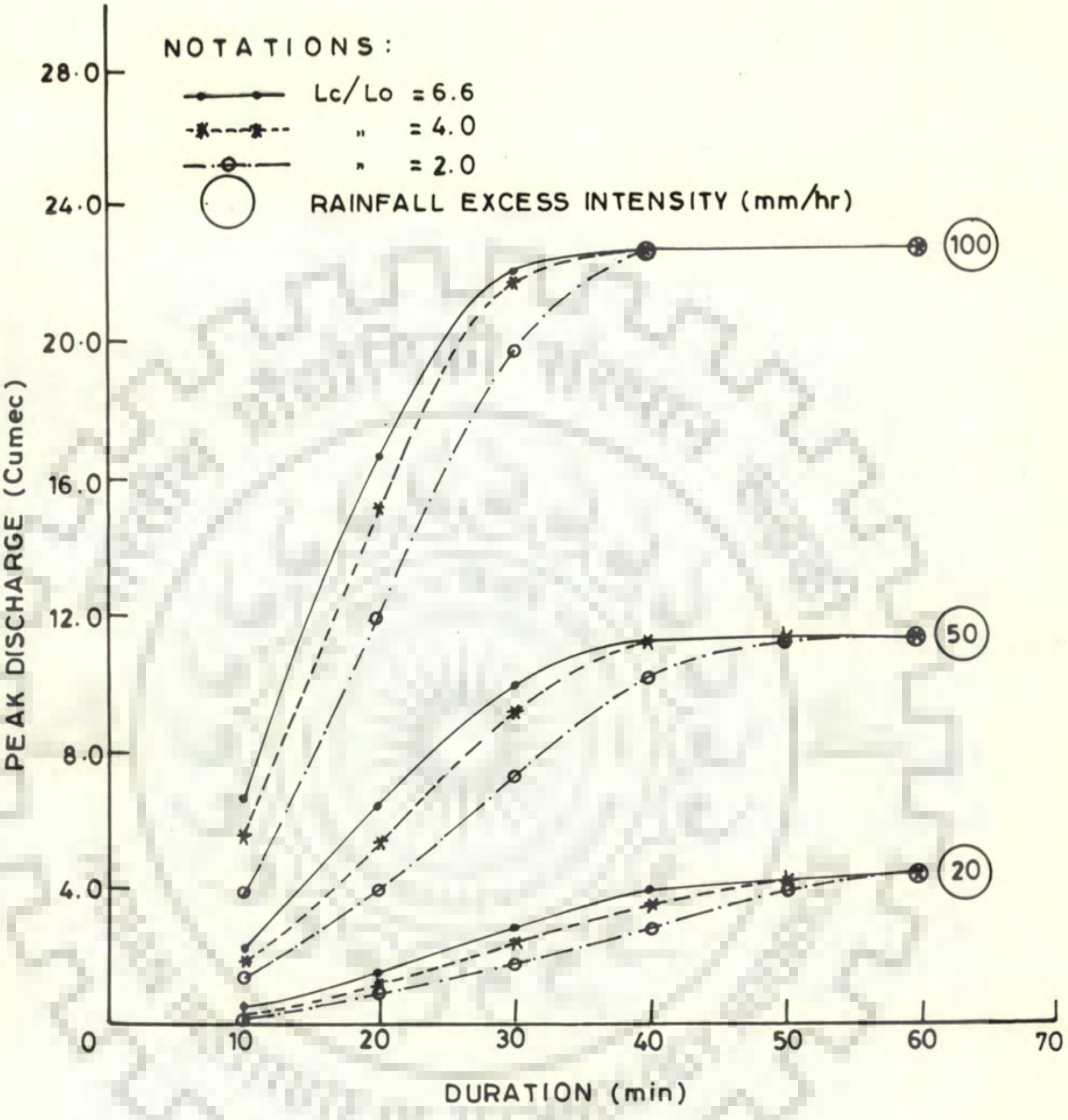


FIG. 6.2 (b) NOMOGRAM OF PEAK DISCHARGE FOR N = 0.150 (BRIDGE NO. 319)

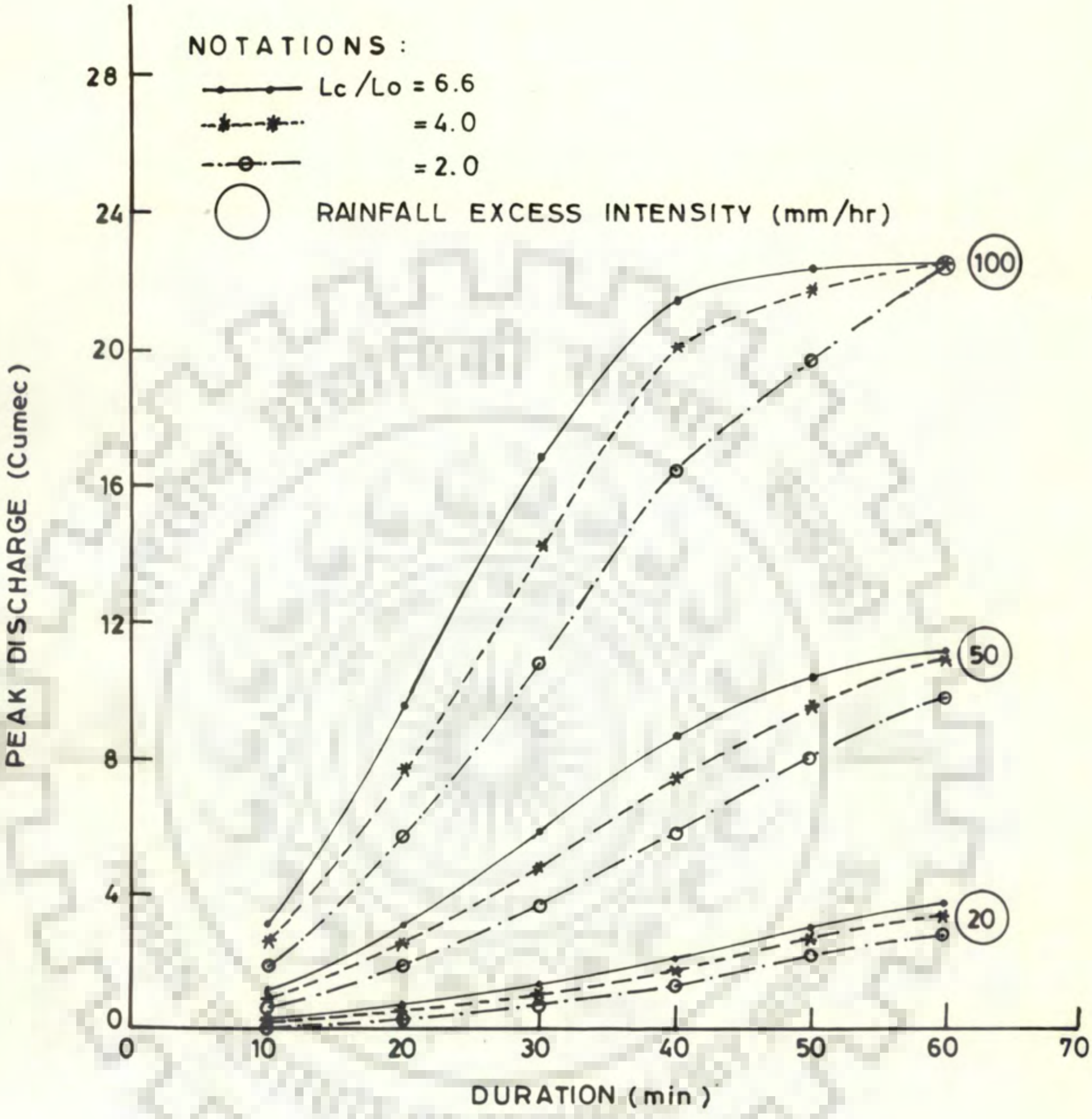


FIG. 6. 2(c) NOMOGRAM OF PEAK DISCHARGE FOR N = 0.350 (BRIDGE NO. 319)

effective overland roughnesses, of the similar physiographic characteristics of the watersheds in the vicinity.

These curves have a limitation that the another sensitive parameter overland slope could not be included and its effect is not reflected in the relationships. For this purpose, the regression analysis have been tried to arrive at suitable relationships.

6.5 MULTIPLE NONLINEAR REGRESSION ANALYSIS FOR PEAK DISCHARGE COMPUTATIONS

The use of nonlinear multiple regression equations for the transformation of the desired hydrologic dependent variable from a few gauged stations to many ungauged ones is a practical and handy tool. In such relationships, the dependent variable is expressed as a function of independent variables. Since, the hydrologic processes are mostly nonlinear in nature, therefore, a nonlinear fit is preferred. With the advent of digital computers, it is now easy to consider a number of independent variables which effectively influenced the hydrological processes.

In this analysis, the peak discharge (Q_p) is the dependent function. In the proposed KW models application, the independent physiographic parameters and the input meteorological parameters which influence Q_p have been listed in the previous section. Large quantity of data are required to incorporate all the parameters in the regression analysis. Further, the experience tells that, it is better to restrict the number of independent variables to a minimum. Since, for developing such relationships, data of only three watersheds was available and the total number of 26 storm events, therefore, choice has been restricted to only 4 independent variables. The following two alternatives have been considered by making use of a total number of 6 independent variables (viz. A, N, S_o, I, D, d).

$$Q_p = f(A, I, D, N) \quad \ldots \quad (6.1)$$

and

$$Q_p = f_1(A, d, N, S_o) \quad \ldots \quad (6.2)$$

Data pertaining to independent variables have been listed in Tables 5.1, 5.5, 5.12, 5.13, 5.17, 5.18 and Appendix - II. The average rainfall excess intensity (I) has been considered for the storm events. The two nonlinear multiple regression equations worked out for the dependent variable Q_p are as below :

$$Q_p = 0.9336 \, A^{0.688} \, I^{1.261} \, D^{0.848} \, N^{-0.304} \quad \ldots \quad (6.3)$$

and

$$Q_p = 2.113 \, A^{1.060} \, d^{1.055} \, S_o^{\,4.292} \, N^{-0.401} \quad \ldots \quad (6.4)$$

where Q_p = Peak discharge in cumec

A = Area in sq.metre

I = Average rainfall excess intensity in m/s

D = Duration of rainfall excess in second

d = Depth of rainfall excess in metre.

It is necessary that the regression equations be used with care and caution. Best results are obtained when such equations are applied for the values of independent variables which lie within the ranges for which the relationships have been obtained. For equations (6.3) and (6.4), the range of independent variables are given in Table 6.1.

TABLE 6.1 : Range of Indpendent Variables Used for Nonlinear Multiple Regression Analysis

Independent Variable Range	I (mm/hr)	D (min)	S_o	N	d (mm)	A (ha)
(1)	(2)	(3)	(4)	(5)	(6)	(7)
Maximum	40.68	240.0	0.092	0.420	73.02	1400.0
Minimum	5.976	10.0	0.071	0.05	5.6	82.0

Equation (6.3) and (6.4) have been used to compute the peak discharges for the available data sets of 26 storm events registered on the three natural watersheds. The comparisons of computed and the observed peak discharges are shown in figure 6.3 [from multiple regression equation (6.3)] and in figure 6.4 [from multiple regression equation (6.4)].

It is to be seen that the equation (6.3) fits into the data well and is therefore, the same is recommended for use of ungauged watersheds of the vicinity.

6.6 ANALYSIS OF TIME OF CONCENTRATION

In Section 6.4, while discussing the nomograms given in figures 6.2, findings regarding the effects of overland rough-

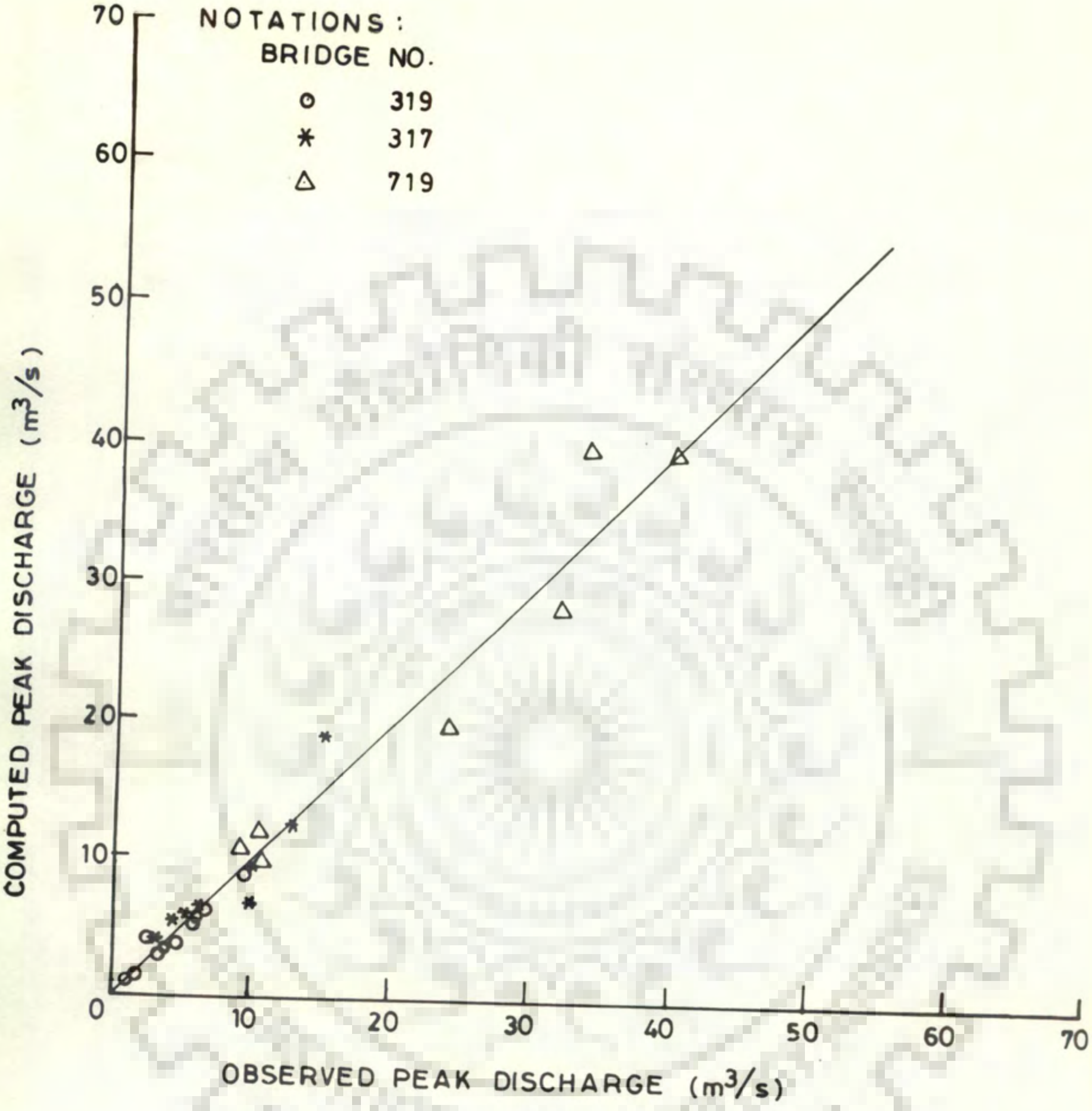


FIG. 6.3 COMPARISON OF OBSERVED AND COMPUTED PEAK DISCHARGES BY USING MULTIPLE REGRESSION EQUATION (6.3)

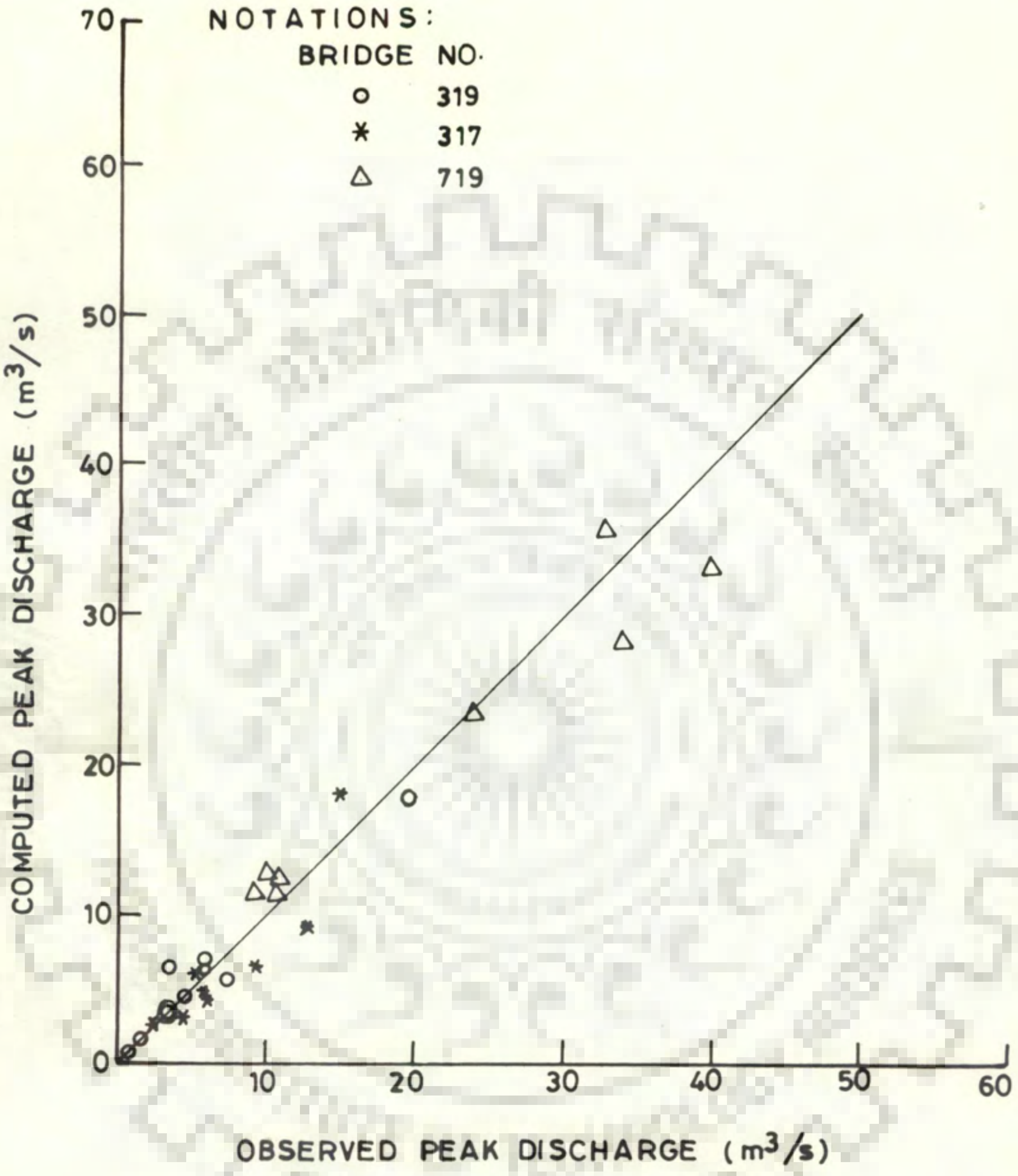


FIG. 6.4 COMPARISON OF OBSERVED AND COMPUTED PEAK DISCHARGES BY USING MULTIPLE REGRESSION EQUATION (6.4)

ness and the rainfall excess intensity on the time of concentration were stated. Singh (1976) and Agiralioglu (1988) have given mathematical formulations of KW theory for the time of concentration for varying rainfall input conditions, watershed geometry and physiographic conditions. Inspite of these findings, no research work could be traced where there were translated into practice for use of the 'Time-Area-Concentration' (TAC) models. TAC based models generally assume linearity and the time of concentration is considered to be a fixed characteristic. Attempts were made in this work to introduce the variability in the time of concentration with rainfall excess intensity and effective overland roughness.

For three representative overland roughness values of N = 0.05 (i.e. the least value), 0.15 (i.e. average value) and 0.35 (i.e. maximum value), runoff hydrographs were computed for the watershed of Bridge No. 319 (Section 5.5.5) for seven uniform rainfall excess intensities varying in the range of 5.0 to 60.0 mm/hr. The time of concentrations were marked on these hydrographs when the discharges became steady. A plot of time of concentration (T_c) Vs. rainfall excess intensity (I) was drawn as a function of effective overland roughness (N) for the watershed of Bridge No. 319 and the same is produced in figure 6.5. From this plot, it is evidently clear that the time of concentration very much depended upon the rainfall excess intensity as well as effective overland roughness.

6.7 MULTIPLE NONLINEAR REGRESSION ANALYSIS FOR COMPUTATIONS OF TIME OF CONCENTRATION

As suggested by various researchers, the time of concen-

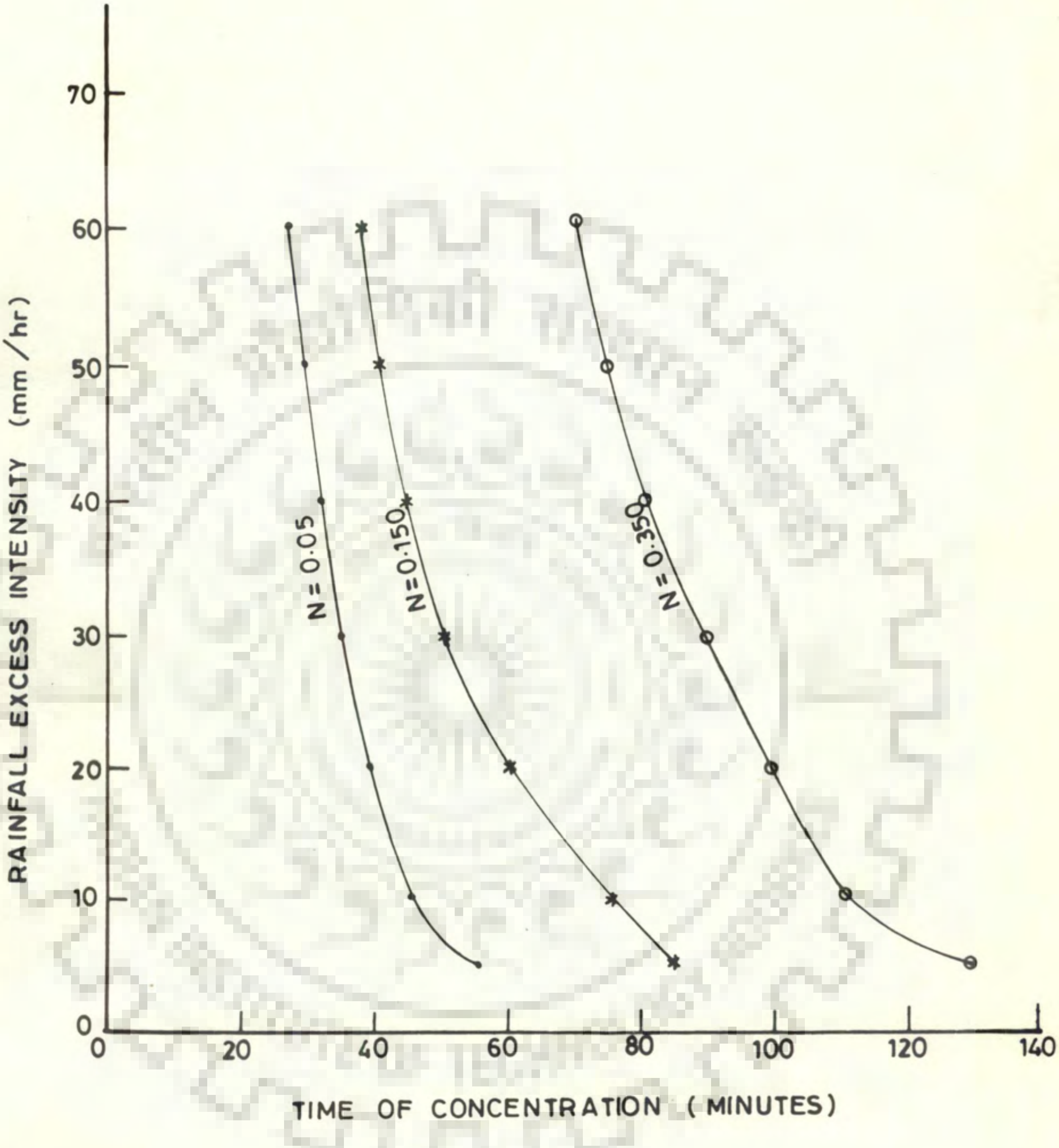


FIG. 6.5 RELATIONSHIP OF TIME OF CONCENTRATION AND RAINFALL EXCESS INTENSITY AS A FUNCTION OF EFFECTIVE OVERLAND ROUGHNESS (BRIDGE NO. 319)

tration was considered to be a function of physiographic characteristics as well as the rainfall excess intensities. The following three alternatives were investigated.

$$T_c = f(A, N, I) \quad \ldots \quad (6.5a)$$

$$T_c = f_1(A, N, I, S_o) \quad \ldots \quad (6.5b)$$

and

$$T_c = f_2(A, N, I, S) \quad \ldots \quad (6.5c)$$

The physiographic data of the three watersheds and computed time of concentrations for five different rainfall excess intensities as given in Table 6.2 were used to develop nonlinear regression equations. The equations so arrived at are given as below :

$$T_c = 6.465\ A^{0.409}\ I^{-0.130}\ N^{0.347} \quad \ldots \quad (6.6a)$$

$$T_c = 0.0369\ A^{0.086}\ I^{-0.138}\ N^{0.312}\ S_o^{-3.88} \quad \ldots \quad (6.6b)$$

and

$$T_c = 0.602\ A^{0.657}\ I^{-0.138}\ N^{0.312}\ S^{0.50} \quad \ldots \quad (6.6c)$$

where T_c = Time of concentration in seconds

A = Area in m^2

I = Rainfall excess intensity in m/s.

To check the performance of these equations, the time of concentration computed by equations and through the models were compared. The comparisons are shown in figures 6.6(a), (b) and (c). Though all the three comparisons are very close, yet performance of equation (6.6c) may be said better and the same is suggested for future works.

TABLE 6.2 : DATA Used for Regression Analysis for the Time of Concentration

(a) Bridge No. 319 (A = 82.0 ha, S_o = 0.092, S_c = 0.072):

Sl. No.	I mm/hr	T_c (min) for Effective Overland Roughness (N)			
		N=0.05	N=0.075	N=0.200	N=0.380
1	10.0	52.5	65.0	87.5	112.5
2	20.0	40.0	50.0	70.0	90.0
3	30.0	40.0	45.0	62.5	77.5
4	50.0	37.5	40.0	50.0	65.0
5	100.0	35.0	35.0	50.0	62.5

(b) Bridge No. 317 (A = 140.0 ha, S_o = 0.081, S_c = 0.105):

Sl. No.	I mm/hr	T_c (min) for Effective Overland Roughness (N)			
		N=0.07	N=0.155	N=0.255	N=0.300
1	10.0	85.0	120.0	150.0	160.0
2	20.0	85.0	115.0	135.0	150.0
3	30.0	80.0	105.0	125.0	135.0
4	50.0	70.0	85.0	105.0	115.0
5	100.0	60.0	75.0	85.0	95.0

(c) Bridge No. 719 (A = 1400.0 ha, S_o = 0.071, S_c = 0.021):

Sl. No.	I mm/hr	T_c (min) for Effective Overland Roughness (N)		
		N = 0.07	N = 0.150	N = 0.338
1	10.0	200.0	250.0	350.0
2	20.0	180.0	230.0	290.0
3	30.0	180.0	230.0	280.0
4	50.0	170.0	220.0	270.0
5	100.0	130.0	160.0	200.0

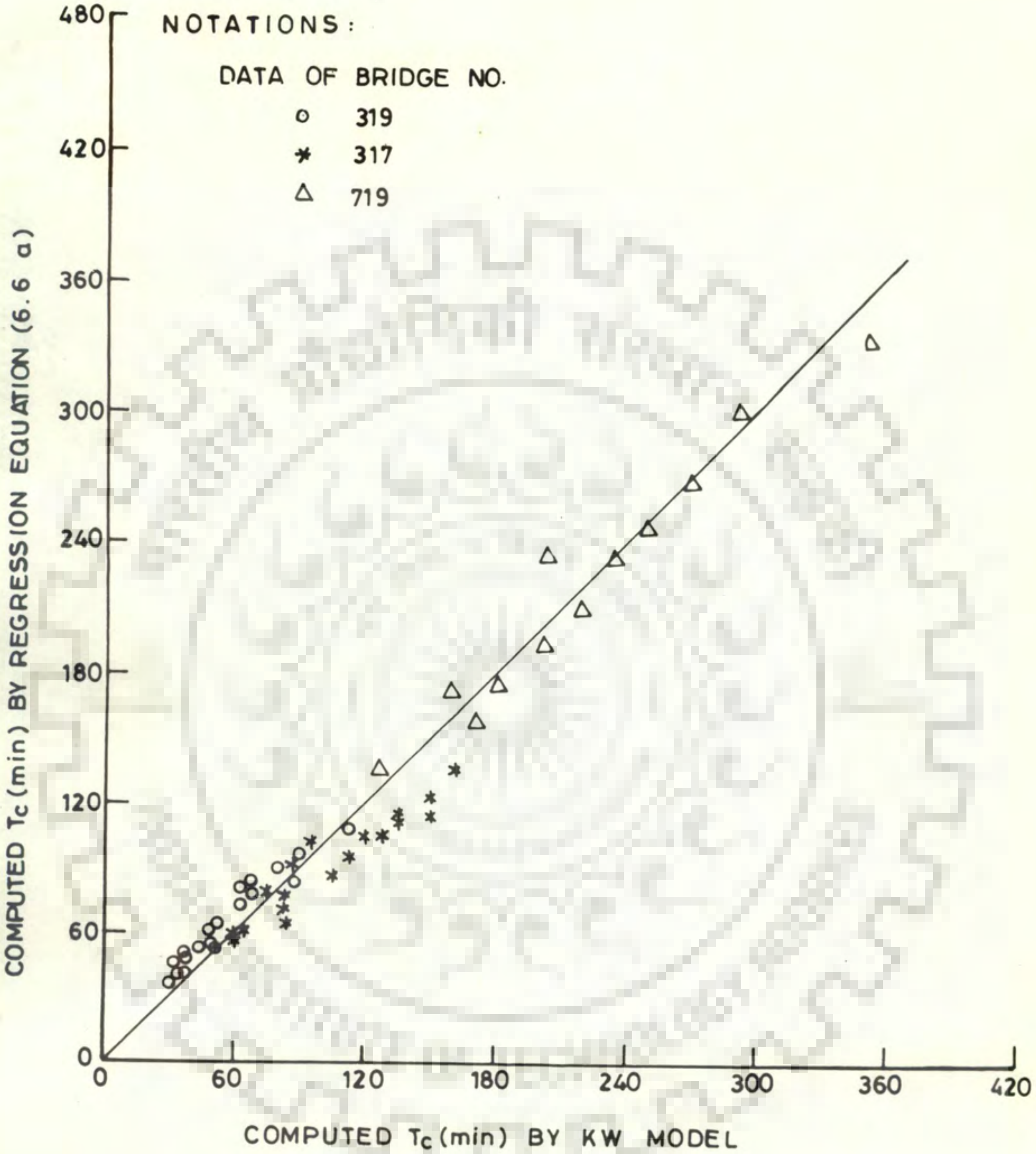


FIG. 6.6(a) COMPARISON OF COMPUTED VALUES OF TIME OF CONCENTRATION USING THE MODEL (SECTION 3.9.1) AND THE EQUATION (6.6 a)

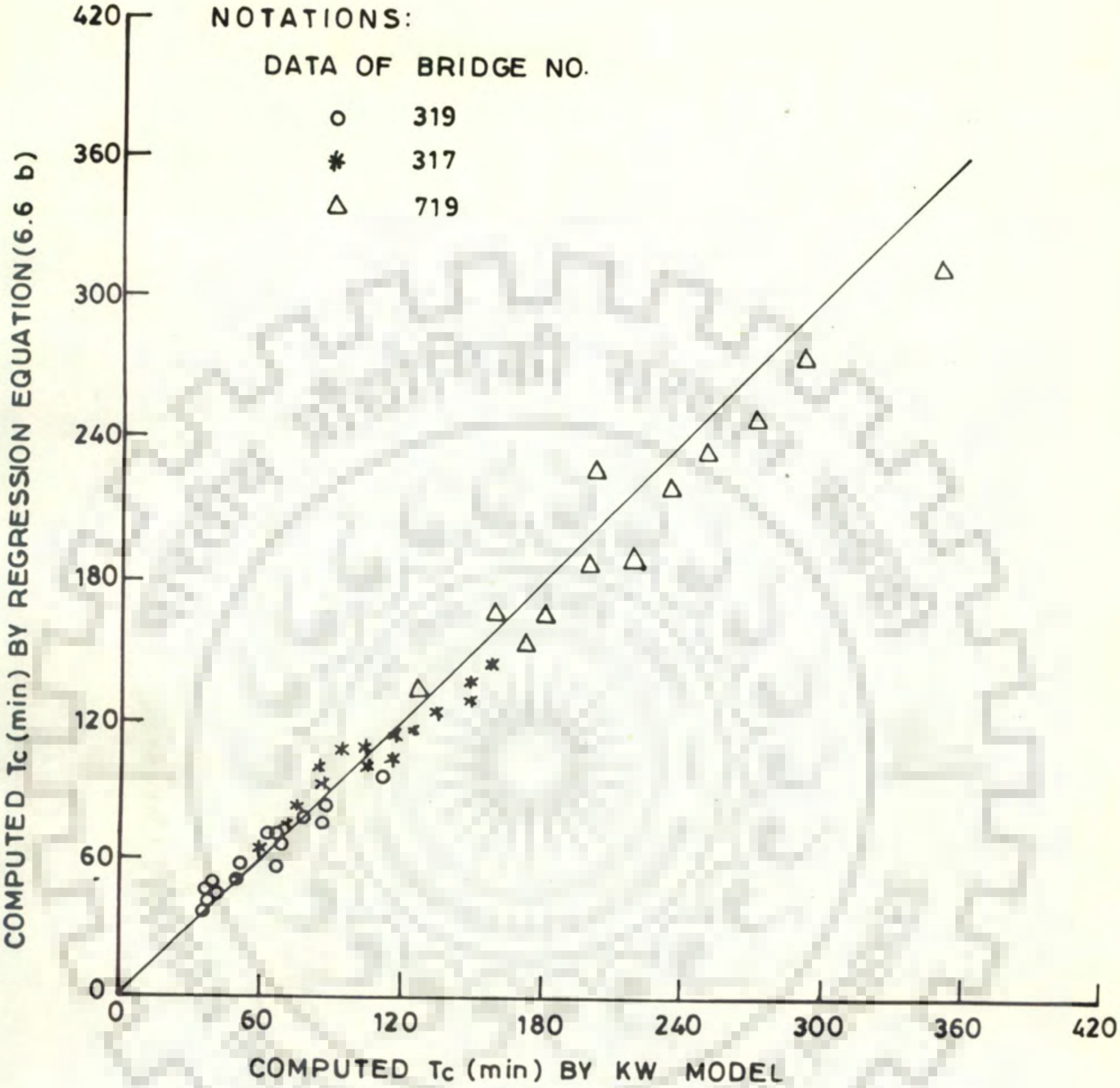


FIG. 6.6(b) COMPARISON OF COMPUTED VALUES OF TIME OF CONCENTRATION USING THE MODEL (SECTION 3.9.1) AND THE EQUATION (6.6 b)

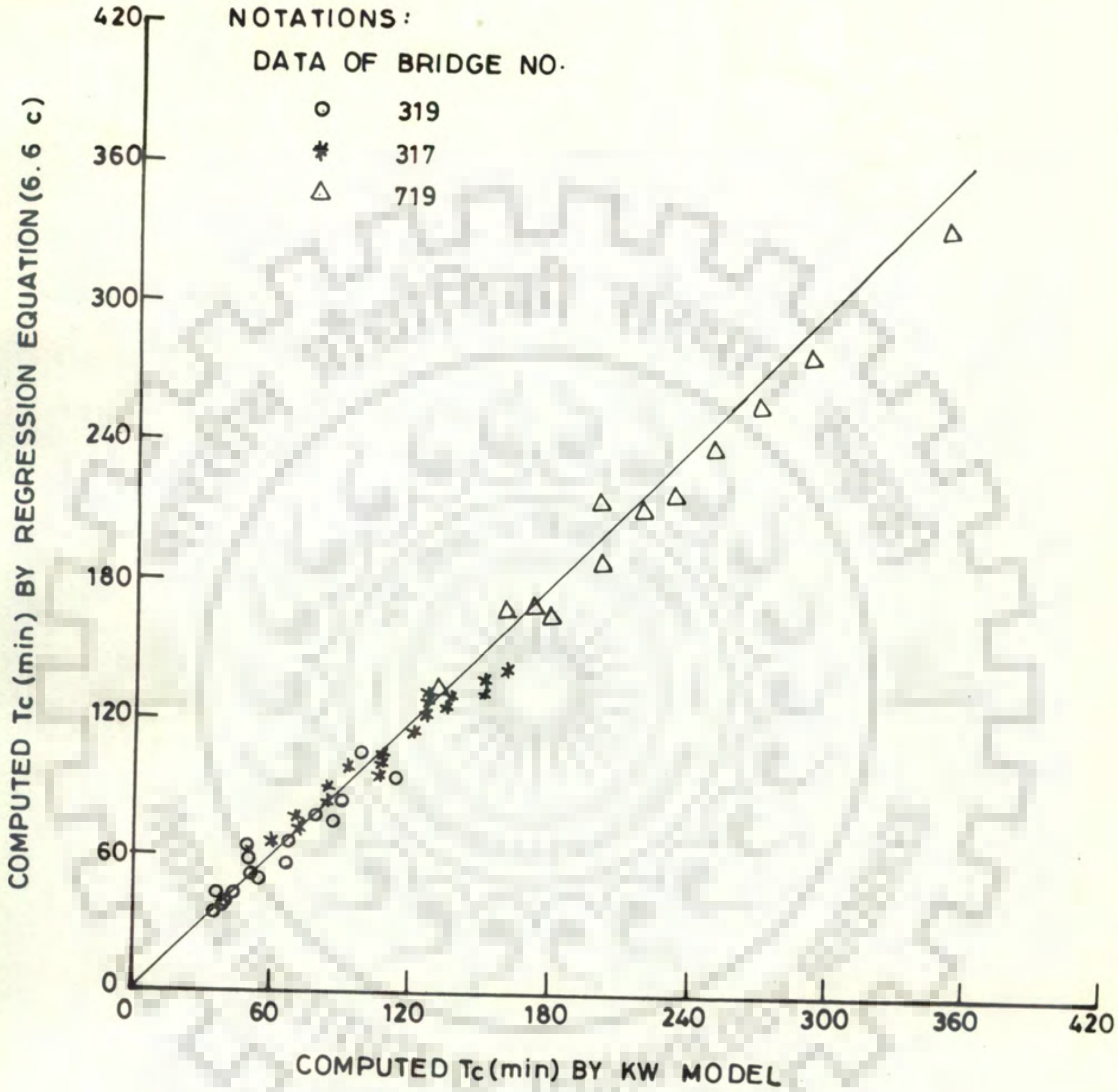


FIG.6.6(c) COMPARISON OF COMPUTED VALUES OF TIME OF CONCENTRATION USING THE MODEL (SECTION 3.9.1) AND THE EQUATION (6.6 c)

6.8 VARIABILITY IN TIME OF CONCENTRATION AND ITS APPLICATION TO TIME-AREA-CONCENTRATION(TAC) BASED MODELS

In the previous sections dependence of the time of concentration on rainfall excess intensity and the physiographic parameters has been described. In this section attempts are made to translate the variability in time of concentration into practical applications for TAC based models. For this purpose, the watershed of Bridge No. 319 is selected. It is mainly because, this watershed is oblong in shape. To such watershed the TAC based models have been found to be the most practical and useful. Three representative effective overland roughness values viz. n = 0.05 (i.e. minimum), 0.150 (i.e. average) and 0.350 (i.e. maximum) have been chosen. Six uniform rainfall excess intensities (viz. 5.0, 10.0, 20.0, 30.0, 40.0 and 60.0 mm/hr) have been tried. By using the proposed KW model for this watershed (Section 5.5.5), the six runoff responses have been computed.

The genetic principle of runoff (Figure 6.7) suggests the following relationship for the computation of runoff.

$$Q(t) = \int_{o}^{t} \frac{dA_{t-\tau}}{dt} \cdot I(\tau)d\tau \qquad \ldots \quad (6.7)$$

By using KW models for this relationship, for particular values of N and rainfall excess intensities, Q(t) at different times became known functions. Thus for variable times (0.0 to T_c), the cumulative contributing areas are worked out. By appropriately adopting a uniform incremental values (say, 5.0 minutes in this case), the inter-isochronal areas are obtained. For each of the three effective overland roughnesses, the relationships between the inter-isochronal areas and 'contributing times' are shown in figures 6.8(a), (b) and (c).

In figure 6.9, for the two selected rainfall excess intensities (i.e. 10.0 and 60.0 mm/hr), the contributing inter-isochronal areas are plotted against time for the three adopted N values. This gives a clear picture of the impact of effective overland roughness and rainfall excess intensities on the time of concentration for this watershed.

6.9 RESPONSES OF THE TAC MODEL

The relationship arrived at in figures 6.8(a), (b) and (c) have been utilized with appropriate interpolation for working out the inter-isochronal areas at 5.0 minutes interval for the following three storm events with effective overland roughnesses shown against them.

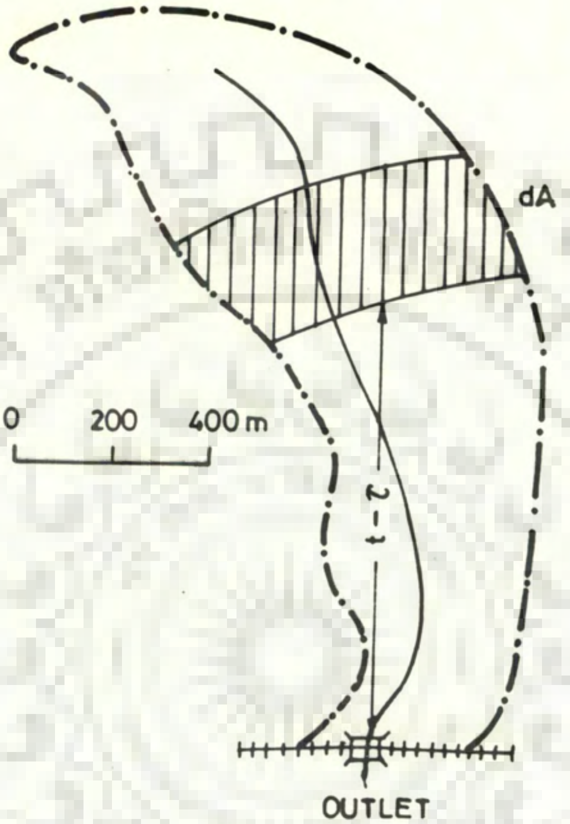
(1) Storm event dated 4.11.1962 (N=0.052)
(2) Storm event dated 5.8.1964 (N=0.140)
(3) Storm event dated 7.10.1963 (N=0.106)

The rainfall excess functions are given for the storm events in Appendix-II(A).

Comparison of observed hydrographs with those of computed through the TAC model as well as KW model is shown in figure 6.10. Though the comparison between the computed and the observed is quite satisfactory but this can further be refined for a still closer match by appropriately drawing increased number of design curves for a number of N values in the close ranges (i.e. to minimize the approximation due to interpolation).

This leads enough scope for further studies and research in different aspects of the TAC based model. Perhaps, various conceptual models proposed by different researchers (Dooge (1959), Laurensow (1964), Mathur (1972) etc.) can further be improved.

Concludingly, it may be remarked that the KW theory is an excellent tool to analyse the runoff mechanics. The effect of various physiographic and meteorologic parameters on the runoff can best be studied and various unexplored aspects can be studied in depths and details to improve upon the existing simpler techniques and models.



$$Q(t) = \int_{0}^{t} \frac{dA_{t-\tau}}{dt} \, I(\tau) \, d\tau$$

FIG. 6.7 TIME-AREA CONCENTRATION APPROACH FOR COMPUTATION OF RUNOFF (BRIDGE NO. 319)

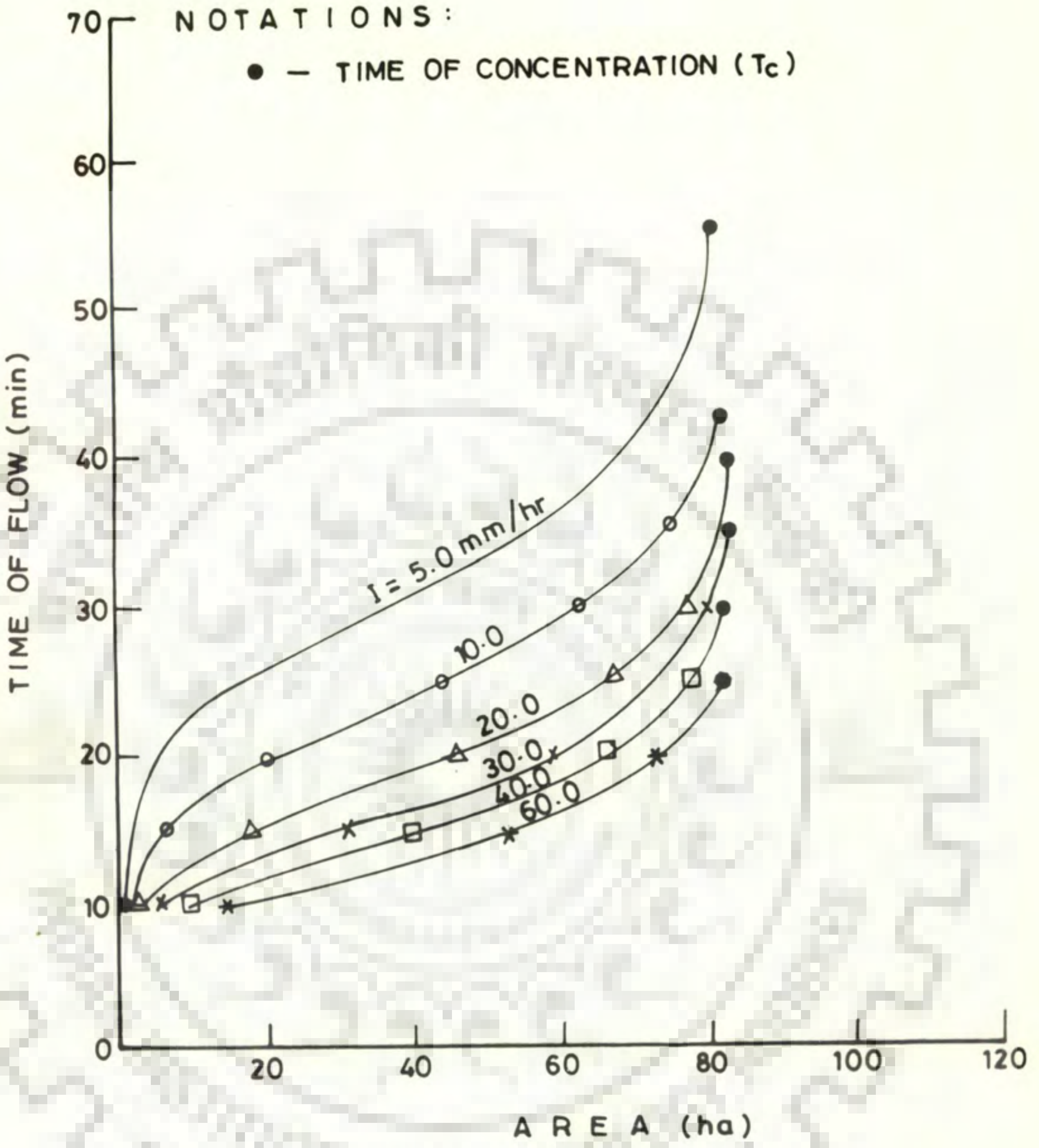


FIG.6.8(a) CONTRIBUTING INTER-ISOCHRONAL AREAS FOR SELECTED RAINFALL EXCESS INTENSITIES AND N= 0.05 (BRIDGE NO.319)

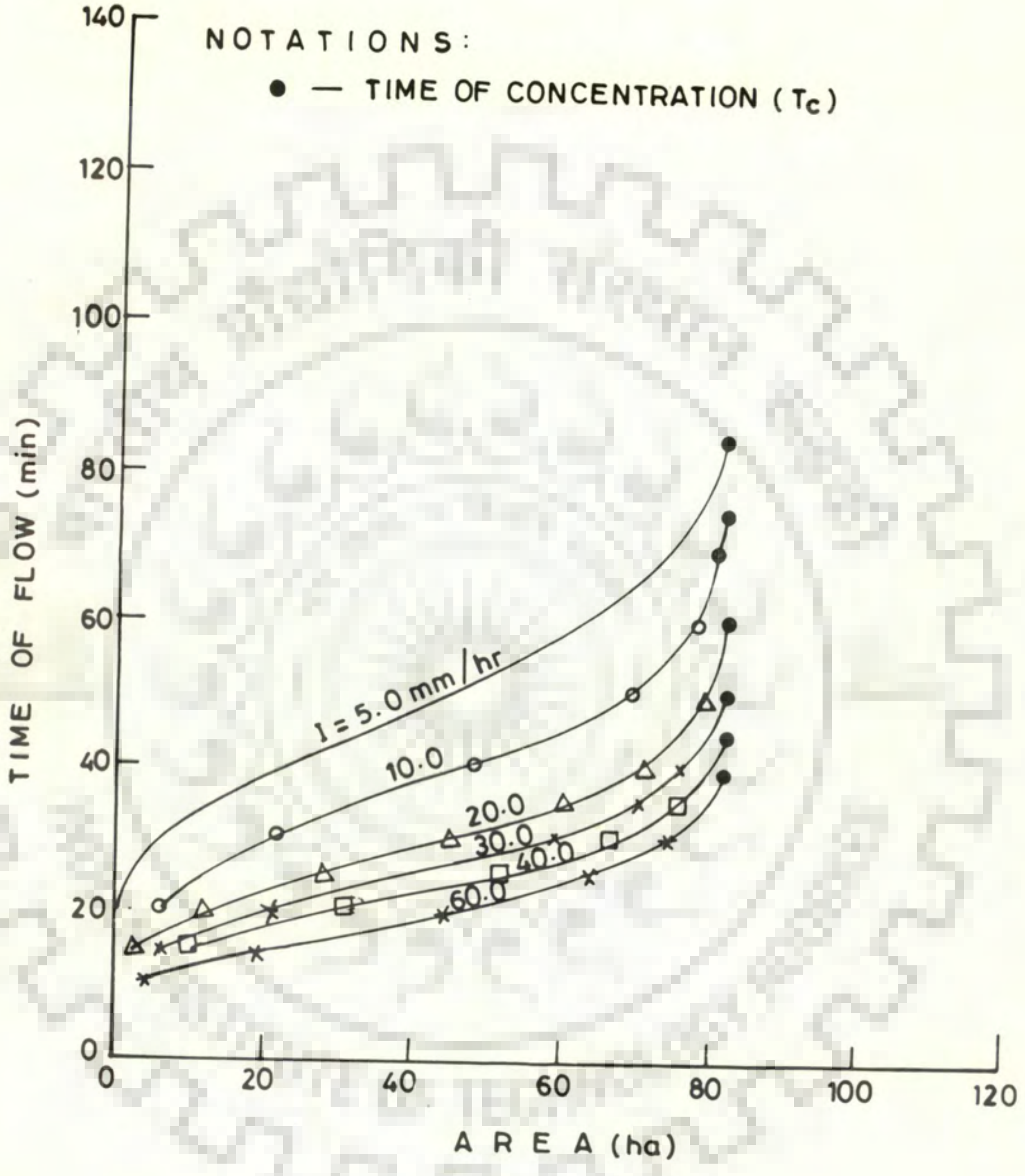


FIG. 6.8(b) CONTRIBUTING INTER-ISOCHRONAL AREAS FOR SELECTED RAINFALL EXCESS INTENSITIES AND N ≈ 0.150 (BRIDGE NO. 319)

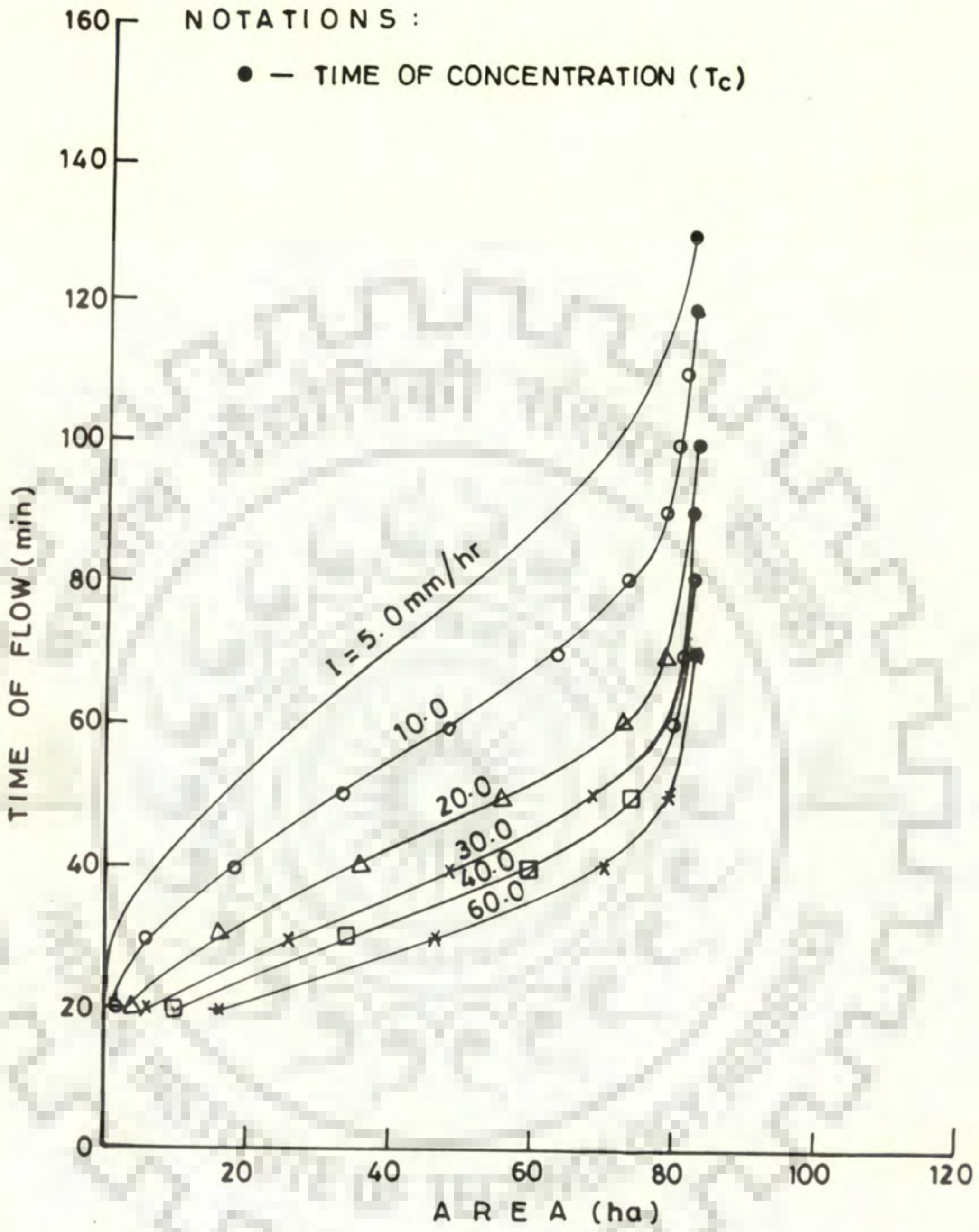


FIG. 6.8(c) CONTRIBUTING INTER-ISOCHRONAL AREAS FOR SELECTED RAINFALL EXCESS INTENSITIES AND N= 0.350 (BRIDGE NO. 319)

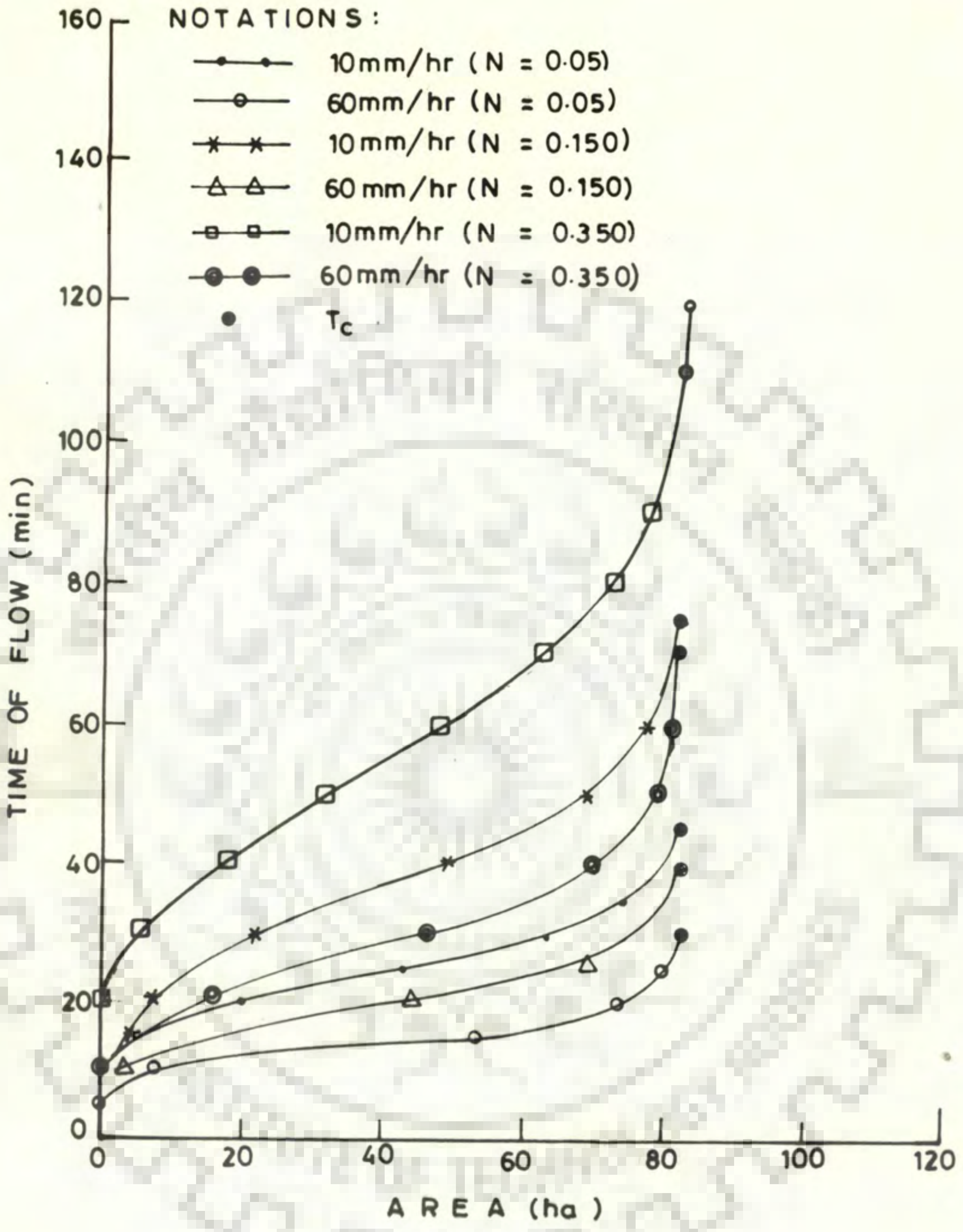


FIG. 6.9 CONTRIBUTING INTER-ISOCHRONAL AREAS FOR THREE N (i.e N = 0.05, 0.150, 0.350) AND TWO I (i.e I = 10.0, 60.0 mm/hr) (BRIDGE NO. 319)

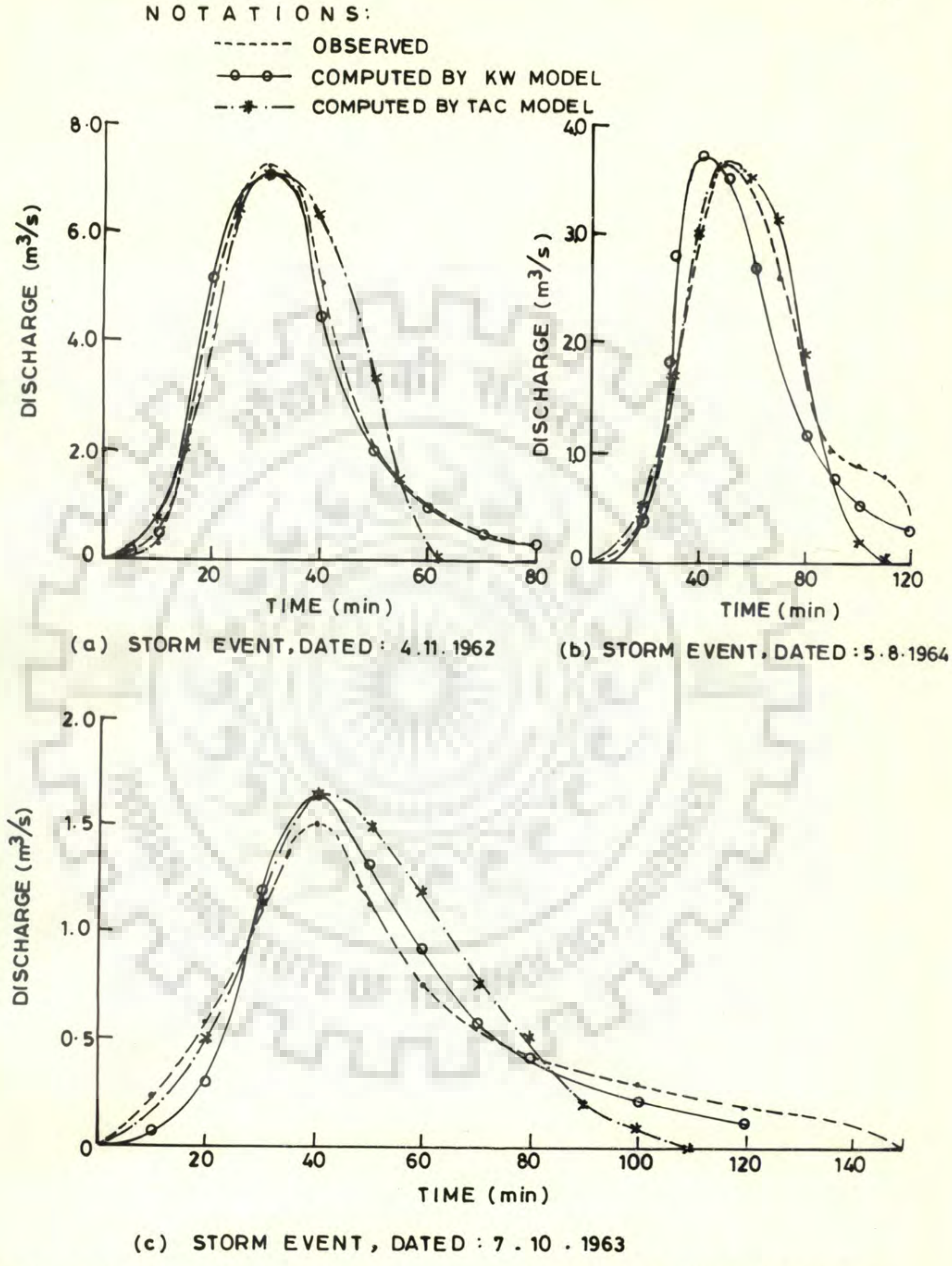


FIG. 6.10 COMPARISON OF OBSERVED AND COMPUTED (BY KW MODEL AND TAC MODEL) DIRECT RUNOFF HYDROGRAPHS (BRIDGE NO. 319)

CHAPTER - VII

C O N C L U S I O N S

In this chapter, the results of the KW theory applications to three natural hilly watersheds and one agricultural watershed (as detailed in the previous chapters) are discussed. Suitable conclusions have been drawn from these discussions. In the last, some proposals relating to the possible extension of the present work have also been mentioned.

7.1 DISCUSSION OF RESULTS AND CONCLUSIONS

(1) In Section 3.8, two schemes for the solution of KW equations were proposed. The scheme (I & II) was found to be stable and convergent upto a time step of 10.0 minutes and therefore, the same was adopted for solving the proposed distributed configurations of the three natural watersheds discussed in Chapter V. The scheme III inherently stable did not show convergence for these watersheds and therefore, the same has not been used for solving the model equations.

(2) In case of the agricultural watershed (Kachwa), the proposed model scheme (I & II) which is inherently convergent did not exhibit stability. On the other hand, the scheme III which is inherently stable showed convergence even for large time step value of 60.0 minutes. Since the watershed is nearly flat and the runoff durations exceeded 60.0 hours, in most of the storm events, this scheme was found to be advantageous from

economic considerations i.e. the total computer run time was very less.

(3) Among the physiographic parameters considered for this analysis, the effective overland roughness (N) was found to be the most sensitive for all the watersheds (Figures 5.3, 5.4, 5.19, 5.20 and 5.21).

(4) Subsequent to N values, the overland slope (S_o) were found to be effective. Therefore, a detailed analysis (Section 5.2.1 as given in Horton's formula and discussed by Viessman et al., 1977) was adopted for the computation of overland slope.

(5) In order to identify the systems, it was necessary to have correct estimates of effective overland roughness values as this parameter was found to be the most sensitive. In case of natural watersheds of Bridge Nos. 319, 317 and 719, interesting variations were found in the effective overland roughnesses with time. As the monsoon season proceeded, the N values registered a rise from 0.05 to 0.420 for the watershed of Bridge No. 319; 0.07 to 0.336 for Bridge No. 317 and 0.07 to 0.345 for Bridge No. 719 (disributed). The value of N was found to be maximum in the mid-monsoon season and subsequently, it again registered infall in their values. The distribution of effective overland roughnesses over the monsoon months was to be approximatley parabolic in nature (Figures 5.5, 5.22 and 5.23).

In case of agricultural watershed, most of the storm events belonged to a period which ranged from last week of July to first week of August. Therefore, the overall variation of N values could not be ascertained.

(6) The average fortnightly N values worked out for the three watersheds are shown in figures 6.1(a), (b) and (c) and the same were used for prediction purposes. These were also used of ungauged watersheds as well as in the application of TAC based models.

(7) The N values obtained from the best fit curves (Figures 5.5, 5.22, 5.23 and 5.24) were used for computing the discharges by the application of KW theory in the proposed models. The comparison of observed and computed hydrographs was found to be quite close. For most of the storm events of four tests watersheds, model efficiencies were found to vary from 80% to 99%. The volume error was in the range of 0.04% to 6.84% which is also within the acceptable limits.

(8) In order to apply the proposed concepts to the ungauged watersheds, attempts were made to arrive at some nomograms. For watersheds having areas under a hundred hectares (i.e. nearly 80.0 ha or so), the relationship arrived at for Bridge No. 319 can be used (Section 6.4). In this case some sample plots of peak discharges versus rainfall excess durations (upto 60.0 minutes) were drawn for heavy rainfall excess intensities (20.0, 50.0 and 100.0 mm/hr) of three representative effective overland roughness values [N = 0.05 (i.e. minimum); 0.15 (i.e. average) and 0.35 (i.e. maximum)]. Similar plots can be developed for a variety of ranges of rainfall excess intensi-

ties and effective overland roughness to have quick estimates of peak discharges of watershed areas in the vicinity of 80.0 hectares and having similar physiographic conditions.

(9) Nonlinear multiple regression analysis was conducted, considering the peak discharges to be the functions of area (A), effective overland roughness (N), overland slope (S_o), rainfall excess intensity (I), its duration (D) and depth (d) [Section 6.5]. The two alternatives of independent variables were attempted. The comparison of computed and observed peaks for various storm events on the three watersheds are shown in figures 6.3 and 6.4. Both the comparisons give nearly similar results. However, equation (6.3) has been found to be better and therefore, is preferred.

(10) For the watershed of Bridge No. 319, the KW model applications were made for various uniform rainfall excess intensities (ranging from 5.0 to 60.0 mm/hr) of infinite durations ($>T_c$) and the three representative effective overland roughness values (i.e. N = 0.05, 0.15 and 0.35). The time of concentration were marked on the hydrographs. A plot of T_c Vs. rainfall excess intensity as a function of effective overland roughness is shown in figure 6.5. T_c was found to be a function of I and N. Time of concentration decreased with increase in rainfall excess intensity (I) values for the same effective overland roughness value (N). Also, it increased with increase in N values for the same rainfall excess intensity.

(11) In order to determine the time of concentration of ungauged watershed, T_c was considered to be a function of

A, N, S_o, S and I. Three alternatives were tried for different combinations of independent variables. The relationships arrived at are given in Section 6.7 [Equations 6.6(a), (b) and (c)]. Using these equations the T_c values were computed for the three watersheds and they were compared with their computed values obtained from the KW models [Figures 6.6(a), (b) and (c)]. This comparison shows that all the three equations perform quite well. However, performance of the following equation was found better and therefore, is preferred.

$$T_c = 0.602\ A^{0.657}\ I^{-0.138}\ N^{0.312}\ S^{0.50} \qquad \dots (7.1)$$

(12) Variation in the T_c values open up new options to develop time-area-concentration (TAC) based models for variable T_c values. For different rainfall excess intensities (ranging from 5.0 to 60.0 mm/hr), for three N values (N = 0.05, 0.15 and 0.35) and for different T_c values, the inter-isochronal areas were worked out for watershed of Bridge No. 319. Plots of inter- isochronal areas Vs. time of flows as the function of different rainfall excess intensities are given in figures 6.8(a), (b) and (c). The effect of rainfall excess intensity variations on the inter-isochronal areas for different effective overland roughness values is evident from these figures.

The three figures 6.8(a), (b) and (c) were used as the design curves. For the three storm events of Bridge No. 319 (Section 6.9) dated 4.11.1962, 7.10.1963 and 5.8.1964, the inter-isochronal areas were interpolated for the prevailing effective overland roughness values as well as for the rainfall excess intensities. The discharges were computed by using the time-area method. Comparisons of observed and computed discharges through variable T_c and KW models are presented in figure 6.10. The performance of TAC model is quite encouraging. However, it can further be improved by developing more numbers of design curves of the type given in figures 6.8 for a number of N values and different rainfall excess intensities. This will minimize the errors due to interpolations.

7.2 SCOPE OF FURTHER WORK

Three fully off-centred first order explicit finite difference methods have been used for the solution of proposed KW models. A single step, second order explicit, Lax-Wendroff scheme (Singh, 1979) can also be used with advantage for the above mentioned models. Also, implicit finite difference methods and the finite element methods may be used for this purpose. The applicability of these methodologies in the context of small watersheds of tropical regions need be investigated in details.

In the proposed models, the contributing surface plane elements have been assumed rectangular and perpendicular to the channels. The shape and the alignment of these planes need be oriented according to the prevailing overland slopes. An altogether, different approach will have to be developed for such cases of modelling.

For the ungauged watersheds, the nomogram appraoch and the regression models are proposed. These are to be further refined with data of larger number of watersheds as and when the same are available.

The concept of a variable time of concentration and its use for different TAC based models need be investigated in much more details. However, dependence of T_c on rainfall excess intensity and effective overland roughness will be of practical significance for the prediction of flood peaks at the outlet.

7.3 CONCLUDING REMARKS

Applicatioin of KW theory for surface hydrologic computations has been tried in different parts of the world with success. Perhaps, this is the first detailed research work which has been carried out on the small watersheds of tropical regions in India. This sophisticated methodology needs a detailed data which should be collected for short intervals and must be accurate as well as reliable. Unfortunately, in most of the developing countries such elaborate data are not available. With great difficulty data on three natural watersheds and of one agricultural watershed would be procured. With all these limitations and constraints, it was quite enterprisinig experience to work for such sophisticated and detailed techniques. The various conclusions which have been discussed in the last section need be further verified in future when more detailed data are available.

Concludingly, it may be remarked that in the computational schemes used in this work uniform values of steps Δx and Δt have been used. Schemes which take variable time steps may be tried to make computations more efficient. Further, in one or two cases computed values differed appreciably from observed values. This is partly due to computations of rainfall excess function which was based on ϕ-index approach. This aspect of surface hydrology continues to be the weakest link in the present day technology. It effects the magnitudes of computed discharges as well as the time of their occurrences. Thus affecting the shape of hydrograph in general. This limitation remained beyond the dealings of the present work and it is hoped that in future some solution will be available some day.

```
C     CAL. OF CANNEL FLOW BY K.W.T.MODEL(P2.FOR)
*************************************************************************
C*******  SCHEME I AND II  *****'LUMPED'*************
C        BY MD. MUBARAK HOSSAIN
C        -------------------------------------
C        --------------------------------------
C        CHANNEL FLOW ROUTING:
C        ----------------------------------
C        DX1=LENGTH STRIP(METRE)
C        DT1=TIME  STEP(SECOND)
C        AN1=CHANNEL BED ROUGHNESS
C        SO1=CHANNEL BED SLOPE
C        M1=NOS. OF TIME STEPS
C        N1=NOS. OF LENGH STRIPS
C        AL1=LENGH OF THE MAIN CHANNEL(METRE)
C        Y1,Y2,Y3=DEPTHS OF WATER ON CHANNEL
C        B=BED WIDTH(METRE)
C        BZ=CHANNEL SIDE SLOPE
C        FR1=FRUDE NO.(CHANNEL)
C        AK=KINEMATIC WAVE NO.
C        Q=DISCHARGE(M**3/S) AT THE CHANNEL
C        A=AREA OF CROSS-SECTION AT THE CHANNEL
C        -------------------------------------
C        OVERLAND FLOW ROUTING:
C        ----------------------------------
C        DX=LENGTH STRIP(METRE)
C        DT=TIME  STEP(SECOND)
C        AN= ROUGHNESS
C        SO= SLOPE
C        M=NOS. OF TIME STEPS
C        N=NOS. OF LENGH STRIPS
C        AL=LENGH OF THE OVER LAND(METRE)
C        FR2=FRUDE NO.
C        AK2=KINEMATIC WAVE NO.
C        Q=DISCHARGE(M**2/S) ON THE OVERLAND
C        QR=RAINFALL EXCESS INTENSITY(METRE/SEC.)
C        A=AREA OF CROSS-SECTION OF OVER LAND DEPTH
C        ----------------------------------------------------------
```

TABLE - A-1 OF APPENDIX - I

ILLUSTRATIVE EXAMPLE
(DeVaries and MacArthur, 1979)

(i) DATA

Overland Flow Element	Channel Element
Overland Flow Length = 50 ft	Length = 1,600 ft
Slope = 0.06 ft/ft	Slope = 0.003 ft/ft
Roughness Coefficient = 0.3	Roughness ('n') = 0.025
Loss Rate = 0.0	Shape : Trapezoidal
Rainfall	Bottom width = 2 ft
(i) 1st five minutes = 1"/hr	Side slope = 2 to 1
(ii) 2nd five minutes = 2 "	
(iii) 3rd five minutes = 1 "	

(ii) OVERLAND AND CHANNEL FLOW CALCULATIONS

Time (min)	Flow to the channel (Cfs/ft)		Channel outflow (Cfs)	
	DeVries	Mine	DeVries	Mine
0	0	0	0	0
5	0.000308	0.000308	0.057	0.0563
10	0.00161	0.00 161	0.658	0.662
15	0.00159	0.001593	1.369	1.385
20	0.000844	0.000845	1.542	1.551
25	0.000041	0.000041	1.162	1.167
30	0.000025	0.000026	0.808	0.807
35	0.000017	0.000017	0.534	0.532

```
      DIMENSION A(140,50),Q(140,50),QR(50),ABR(50),QI(140,50)
        DIMENSION V(100,100)
        OPEN(UNIT=1, DEVICE='DSK',FILE='DA.DAT')
        OPEN(UNIT=2, DEVICE='DSK',FILE='DAH.OUT')
      READ(1,*)DX1,DT1,AN1,SO1,M1,N1,AL1,Y1,Y2,Y3,B,BZ
      READ(1,*)DX,DT,AN,SO,AM,M,N,NR,AL
      READ(1,*)( QR(J),J=1,NR)
      WRITE(2,950)DX1,DT1,AN1,SO1,M1,N1,AL1,Y1,Y2,Y3,B,BZ
      WRITE(2,960)(DX,DT,AN,SO,AM,M,N,NR,AL)
      CALL LAND (DX,DT,AN,SO,AM,M,N,NR,AL,QR,QI)
      WRITE(2,953)
      WRITE(2,955)
      DXT1=DX1/DT1
      DTX1=DT1/DX1
       CALL SALP (Y1,Y2,Y3,B,BZ,SO1,AN1,EC,ALP)
C     SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
      DO 3J=1,M1
      A(1,J)=0.0
      Q(1,J)=0.0
    3 CONTINUE
      DO 4I=1,N1
      A(I,1)=0.0
      Q(I,1)=0.0
    4 CONTINUE
        T=0.00
      DO 5J=2,M1
      T=T+DT1
        TT=T/60.0
C         TT=T/3600.0
      DO 50 I=2,N1
        ABR1=(A(I,J-1)+A(I-1,J))*0.5
      QBR=QI(N,J)*2.0
      CBR=ALP*EC*(ABR1**(EC-1.0))
      A1=A(I,J-1)+A(I-1,J-1)
      A2=A(I,J-1)-A(I-1,J-1)
      A3=A(I-1,J)-A(I-1,J-1)
      A4=QBR*DT1+A(I,J-1)
```

```
      A5=ALP*EC*DTX1*(A1/2)**(EC-1.0)*A2
      IF(CBR.LE.DXT1)GO TO 200
      IF(CBR.GT.DXT1) GOTO 300
 200  A(I,J)=A4-A5
      Q(I,J)=ALP*(A(I,J))**EC
        GO TO 50
        GO TO 5
  300 Q(I,J)=Q(I-1,J)+QBR*DX1-DXT1*A3
      A(I,J)=(Q(I,J)/ALP)**(1./EC)
 50      CONTINUE
        V(N1,J)=Q(N1,J)/A(N1,J)
        BS=SQRT(B*B+4.0*BZ*A(N1,J))
        YS=(-B+BS)/(2.*BZ)
        Y=ABS(YS)
        TP=B+2.*BZ*Y
        XP=9.81*A(N1,J)
C       XP=32.2*A(N1,J)
        FR1=SQRT(V(N1,J)*V(N1,J)*TP/XP)
        SK=SO1*AL1
        ADC1=SK/Y
        FK=Y*FR1*FR1
        AK=SK/FK
      WRITE(2,954)TT,Q(N1,J),A(N1,J),V(N1,J),Y,ADC1,FR1,AK
        J1=J-1
        TYPE*,J1,Q(N1,J),A(N1,J),V(N1,J),Y
 953  FORMAT(9X,'TIME(MINS)',6X,'DISCH',6X,'AREA',12X,'VEL',
     19X,'DPTH',14X,'ADC1',12X,'FR1',16X,'AK')
 954  FORMAT(5X,8F15.7)
   5  CONTINUE
C    QO=OBSERVED DISCHARGE IN CUMECS
        QO=51.516
        QC=0.0
C       VC=0.0
        DO 20 J=2,M1
        QC=QC+Q(N1,J)
C       VC=VC+V(N1,J)
        IF(J.EQ.50) QC8=QC
```

```
            IF(J.EQ.61) QC9=QC
C           IF(J.EQ.11) VC10=VC
C           IF(J.EQ.16) VC15=VC
C           IF(J.EQ.37) VC36=VC
20           CONTINUE
            WRITE(2,240) QO,QC8,QC9,QC
C           WRITE(2,240) VC10,VC15,VC36
240         FORMAT(5X,'QO=',F10.3,4X,'QC8 AFTER(50.HINS)='
            1,F10.3,4X,'QC9(60.MIS)=',F10.3,4X,'TOTAL  CAL DISCH
            2.IN CUMES=',F10.3)
C240        FORMAT(5X,'VC10=',F10.3,4X,'VC15='F10.3,4X,'VC36=',F10.3)
            TYPE*,QO,QC8,QC9,QC
C           TYPE*,VC10,VC15,VC24
 950      FORMAT(2X,'DX1=',F9.4,4X,'DT1=',F9.4,4X,'AN1='
            1,F9.4,4X,'SO1=',F9.4,4X,'M1=',I9,4X,'N1=',I9
            2,4X,'AL1=',F14.4,4X,'Y1=',F10.4,4X,'Y2='
            3,F9.4,4X,'Y3=',F9.4,4X,'B=',F9.4,4X,'BZ=',F9.4)
 960      FORMAT(2X,'DX=',F9.4,4X,'DT=',F9.4,4X,'AN='
          1,F9.4,4X,'SO=',F9.4,4X,'AM=',F9.4,4X,'M=',I9,4X,'N=',I9
          2,4X,'NR=',I9,4X,'AL=',F14.4)
955         FORMAT(6X,'CANNEL FLOW ROUTING')
          CLOSE(UNIT=1)
          STOP
          END
           SUBROUTINE SALP (Y1,Y2,Y3,B,BZ,SO1,AN1,EC,ALP)
C::::::*********************************************
C            CAL. OF ALP. AND EC. (ALP.FOR)
            A1=(B+BZ*Y1)*Y1
            P1=B+2.*Y1*SQRT(1+BZ*BZ)
            R1=A1/P1
            HR1=R1**(2./3.)
            AR1=SQRT(SO1)/AN1
C           AR1=1.486*SQRT(SO1)/AN1
            Q1=AR1*HR1*A1
C           PRINT*,A1,P1,R1,AR1,Q1
            A2=(B+BZ*Y2)*Y2
            P2=B+2.*Y2*SQRT(1+BZ*BZ)
```

```
      R2=A2/P2
      HR2=R2**(2./3.)
      Q2=AR1*HR2*A2
C     PRINT*,Q1,R1,A2,P2
      A3=(B+BZ*Y3)*Y3
      P3=B+2.*Y3*SQRT(1+BZ*BZ)
      R3=A3/P3
      HR3=R3**(2./3.)
      Q3=AR1*HR3*A3
      Q4=Q1/Q2
      A4=A1/A2
      A5=A1/A3
      Q5=Q1/Q3
      XX=ALOG(Q4)
      YY=ALOG(A4)
      XX2=ALOG(Q5)
      YY2=ALOG(A5)
      EC1=XX/YY
      EC2=XX2/YY2
      EC=(EC1+EC2)*0.5
      AB1=A1**EC
      AB2=A2**EC
      AB3=A3**EC
      ALP1=Q1/AB1
      ALP2=Q2/AB2
      ALP3=Q3/AB3
C     PRINT*,Q2,AB2,MC,AB1,AB2,ALP1,ALP2
      ALP=(ALP1+ALP2+ALP3)/3.
      WRITE(2,751)EC, ALP
 751  FORMAT(5X,'EC=',F14.9,4X,'ALP=',F10.6)
      RETURN
      END
C  ************************************************
      SUBROUTINE LAND (DX,DT,AN,SO,AM,M,N,NR,AL,QR,Q)
C    CAL. OF OVER LAND FLOW BY KW MODEL(HEC.FOR)
     DIMENSION A(140,50),Q(140,50),QR(50),ABR(50)
      WRITE(2,956)
```

```
      WRITE(2,963)
      DXT=DX/DT
      DTX=DT/DX
       ALP=SQRT(SO)/AN
C       ALP=1.486*SQRT(SO)/AN
C     SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
      DO 3J=1,M
      A(1,J)=0.0
      Q(1,J)=0.0
    3 CONTINUE
      DO 4I=1,N
      A(I,1)=0.0
      Q(I,1)=0.0
    4 CONTINUE
        T=0.00
      DO 5J=2,M
      T=T+DT
          TT=T/60.0
C       TT=T/3600.0
      DO 50 I=2,N
       ABJ=0.0
        DO 6 II=2,N-1
        ABJ=ABJ+A(II,J-1)
    6   CONTINUE
         ABJ1=ABJ+(A(1,J-1)+A(N,J-1))*0.5
         ABRJ=DT*QR(J)
C       ABR1=(A(I,J-1)+A(I-1,J))*0.5
         ABR(J)=(DX/((N-1)*DX))*ABJ1+ABRJ
      QBR=QR(J)
      CBR=ALP*AM*(ABR(J)**(AM-1.0))
      A1=A(I,J-1)+A(I-1,J-1)
      A2=A(I,J-1)-A(I-1,J-1)
      A3=A(I-1,J)-A(I-1,J-1)
      A4=QBR*DT+A(I,J-1)
      A5=ALP*AM*DTX*A2*(A1/2)**(AM-1.0)
C       A6=A4-A5
C       IF(A6.LE.0.0) GO TO 32
```

```
C       TYPE*,A1,A2,A3
      IF(CBR.LE.DXT)GO TO 200
      IF(CBR.GT.DXT) GOTO 300
200     A(I,J)=A4-A5
C200    A(I,J)=A6
        Q(I,J)=ALP*(A(I,J))**AM
        GO TO 50
        GO TO 5
300    Q(I,J)=Q(I-1,J)+QBR*DX-DXT*A3
      A(I,J)=(Q(I,J)/ALP)**(1./AM)
 50      CONTINUE
        V=Q(N,J)/A(N,J)
C       Y1=A(N,J)/1.
        XP=9.81*A(N,J)
C       XP=32.2*A(N,J)
C       FR2=SQRT(V*V*TP/XP)
        FR2=V/SQRT(XP)
        SK1=SO*AL
        ADC=SK1/A(N,J)
        FK1=A(N,J)*FR2*FR2
        AK2=SK1/FK1
      WRITE(2,964)TT,Q(N,J),A(N,J),V,QR(J),ADC,FR2,AK2
 963  FORMAT(9X,'TIME(MINS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL',
     19X,'QR',16X,'ADC',16X,'FR2',12X,'AK2')
 964  FORMAT(2X,8F16.7)
   5  CONTINUE
956     FORMAT(6X,'OVER LAND FLOW ROUTING')
         RETURN
        END
```

```
C     CAL. OF CANNEL FLOW BY K.W.T. MODEL(R2.FOR)
*****************************************************************************
C******** SCHEME III **********'LUMPED'********
C        BY MD. MUBARAK HOSSAIN
C        ------------------------------------
C        -------------------------------------
C        CHANNEL FLOW ROUTING:
C        ----------------------------------
C        DX1=LENGTH STRIP(METRE)
C        DT1=TIME  STEP(SECOND)
C        AN1=CHANNEL BED ROUGHNESS
C        SO1=CHANNEL BED SLOPE
C        M1=NOS. OF TIME STEPS
C        N1=NOS. OF LENGH STRIPS
C        AL1=LENGH OF THE MAIN CHANNEL(METRE)
C        Y1,Y2,Y3=DEPTHS OF WATER ON CHANNEL
C        B=BED WIDTH(METRE)
C        BZ=CHANNEL SIDE SLOPE
C        FR1=FRUDE NO.(CHANNEL)
C        AK=KINEMATIC WAVE NO.
C        Q=DISCHARGE(M**3/S) AT THE CHANNEL
C        A=AREA OF CROSS-SECTION AT THE CHANNEL
C        ------------------------------------
C        OVERLAND FLOW ROUTING:
C        ----------------------------------
C        DX=LENGTH STRIP(METRE)
C        DT=TIME  STEP(SECOND)
C        AN= ROUGHNESS
C        SO= SLOPE
C        M=NOS. OF TIME STEPS
C        N=NOS. OF LENGH STRIPS
C        AL=LENGH OF THE OVER LAND(METRE)
C        FR2=FRUDE NO.
C        AK2=KINEMATIC WAVE NO.
C        QR=RAINFALL EXCESS INTENSITY(METRE/SEC.)
C        Q=DISCHARGE(M**2/S) ON THE OVERLAND
C        A=AREA OF CROSS-SECTION AT THE OVER LAND DEPTH
C        ---------------------------------------------------
```

```
      DIMENSION A(130,201),Q(130,201),QR(201),ABR(201),QI(130,201)
      OPEN(UNIT=1, DEVICE='DSK',FILE='K44.DAT')
      OPEN(UNIT=2, DEVICE='DSK',FILE='K44T.OUT')
      READ(1,*)DX1,DT1,AN1,SO1,M1,N1,AL1,Y1,Y2,Y3,B,BZ
      READ(1,*)DX,DT,AN,SO,AM,M,N,NR,AL
      READ(1,*)( QR(J),J=1,NR)
      WRITE(2,950)DX1,DT1,AN1,SO1,M1,N1,AL1,Y1,Y2,Y3,B,BZ
      WRITE(2,960)(DX,DT,AN,SO,AM,M,N,NR,AL)
      CALL LAND (DX,DT,AN,SO,AM,M,N,NR,AL,QR,QI)
      WRITE(2,953)
      WRITE(2,955)
      DXT1=DX1/DT1
      DTX1=DT1/DX1
       CALL SALP (Y1,Y2,Y3,B,BZ,SO1,AN1,EC,ALP)
C     SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
      DO 3J=1,M1
      A(1,J)=0.0
      Q(1,J)=0.0
    3 CONTINUE
      DO 4I=1,N1
      A(I,1)=0.0
      Q(I,1)=0.0
    4 CONTINUE
        T=0.00
      DO 5J=2,M1
      T=T+DT1
C       TT=T/60.0
        TT=T/3600.0
      DO 50 I=2,N1
        QBR=QI(N,J)*2.
       A1=(A(I,J-1)+A(I-1,J))*0.5
        CBR=ALP*EC*((A1)**(EC-1.))
        CRR=CBR*DTX1
        A2=QBR*DT1
        A(I,J)=(A2+A(I,J-1)+CRR*A(I-1,J))/(1+CRR)
        Q(I,J)=ALP*(A(I,J))**EC
   50    CONTINUE
```

```
         V=Q(N1,J)/A(N1,J)
         BS=SQRT(B*B+4.0*BZ*A(N1,J))
         YS=(-B+BS)/(2.*BZ)
         Y=ABS(YS)
         TP=B+2.*BZ*Y
         XP=9.81*A(N1,J)
         FR1=SQRT(V*V*TP/XP)
         SK=SO1*AL1
         ADC1=SK/Y
         FK=Y*FR1*FR1
         AK=SK/FK
         WRITE(2,954)TT,Q(N1,J),A(N1,J),V,QBR,ADC1,FR1,AK
         J1=J-1
         TYPE*,J1,Q(N1,J),Y,ADC1
  953  FORMAT(9X,'TIME(HRS)',6X,'DISCH',6X,'AREA',12X,'VEL',
      19X,'QBR',14X,'ADC1',12X,'FR1',16X,'AK')
  954  FORMAT(5X,8F15.7)
    5  CONTINUE
C     QO=OBSERVED DISCHARGE IN CUMECS
         QO=32.039
         QC=0.0
         DO 20 J=2,M1
         QC=QC+Q(N1,J)
         IF(J.EQ.51) QC8=QC
         IF(J.EQ.66) QC9=QC
20        CONTINUE
C        PC=(QC/PC)*100
         WRITE(2,240) QO,QC8,QC9,QC
240      FORMAT(5X,'QO=',F10.3,4X,'QC AFTER(50.0HR)=',F10.3,4X
         1,'QC9(65HR)',F10.3,4X,'TOTAL  CAL DISCH.IN CUMECS=',F10.3)
         TYPE*,QO,QC8,QC9,QC
  950  FORMAT(2X,'DX1=',F9.4,4X,'DT1=',F9.4,4X,'AN1='
      1,F9.4,4X,'SO1=',F9.4,4X,'M1=',I9,4X,'N1=',I9
      2,4X,'AL1=',F14.4,4X,'Y1=',F10.4,4X,'Y2='
      3,F9.4,4X,'Y3=',F9.4,4X,'B=',F9.4,4X,'BZ=',F9.4)
  960  FORMAT(2X,'DX=',F9.4,4X,'DT=',F9.4,4X,'AN='
      1,F9.4,4X,'SO=',F9.4,4X,'AM=',F9.4,4X,'M=',I9,4X,'N=',I9
```

```
     2,4X,'NR=',I9,4X,'AL=',F14.4)
955    FORMAT(6X,'CANNEL FLOW ROUTING')
      CLOSE(UNIT=1)
C      CLOSE(UNIT=2)
      STOP
      END
       SUBROUTINE SALP (Y1,Y2,Y3,B,BZ,SO1,AN1,EC,ALP)
C:::::*********************************************
C         CAL. OF ALP. AND EC. (ALP.FOR)
        A1=(B+BZ*Y1)*Y1
        P1=B+2.*Y1*SQRT(1+BZ*BZ)
        R1=A1/P1
        HR1=R1**(2./3.)
        AR1=SQRT(SO1)/AN1
        Q1=AR1*HR1*A1
C       PRINT*,A1,P1,R1,AR1,Q1
        A2=(B+BZ*Y2)*Y2
        P2=B+2.*Y2*SQRT(1+BZ*BZ)
        R2=A2/P2
        HR2=R2**(2./3.)
        Q2=AR1*HR2*A2
C       PRINT*,Q1,R1,A2,P2
        A3=(B+BZ*Y3)*Y3
        P3=B+2.*Y3*SQRT(1+BZ*BZ)
        R3=A3/P3
        HR3=R3**(2./3.)
        Q3=AR1*HR3*A3
        Q4=Q1/Q2
        A4=A1/A2
        A5=A1/A3
         Q5=Q1/Q3
        XX=ALOG(Q4)
        YY=ALOG(A4)
        XX2=ALOG(Q5)
        YY2=ALOG(A5)
        EC1=XX/YY
        EC2=XX2/YY2
```

```
              EC=(EC1+EC2)*0.5
              AB1=A1**EC
              AB2=A2**EC
              AB3=A3**EC
              ALP1=Q1/AB1
              ALP2=Q2/AB2
              ALP3=Q3/AB3
C             PRINT*,Q2,AB2,MC,AB1,AB2,ALP1,ALP2
              ALP=(ALP1+ALP2+ALP3)/3.
              WRITE(2,751)EC, ALP
  751         FORMAT(5X,'EC=',F14.9,4X,'ALP=',F10.6)
              RETURN
              END
C    ******************************************************
              SUBROUTINE LAND (DX,DT,AN,SO,AM,M,N,NR,AL,QR,Q)
C         CAL. OF OVER LAND FLOW BY KW MODEL(HEC.FOR)
          DIMENSION A(130,201),Q(130,201),QR(201),ABR(201)
              WRITE(2,956)
          WRITE(2,963)
          DXT=DX/DT
          DTX=DT/DX
          ALP=SQRT(SO)/AN
C         SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
          DO 3J=1,M
          A(1,J)=0.0
          Q(1,J)=0.0
    3     CONTINUE
          DO 4I=1,N
          A(I,1)=0.0
          Q(I,1)=0.0
    4     CONTINUE
C             T=13.00*3600.00
              T=0.00
          DO 5J=2,M
          T=T+DT
C               TT=T/60.0
              TT=T/3600.0
```

```
      DO 50 I=2,N
       ABJ=0.0
        DO 6 II=2,N-1
        ABJ=ABJ+A(II,J-1)
 6       CONTINUE
          ABJ1=ABJ+(A(1,J-1)+A(N,J-1))*0.5
          ABRJ=DT*QR(J)
         ABR(J)=(DX/((N-1)*DX))*ABJ1+ABRJ
       CBR=ALP*AM*(ABR(J)**(AM-1.0))
      QBR=QR(J)
C      A1=(A(I,J-1)+A(I-1,J))*0.5
        CRR=CBR*DTX
        A2=QBR*DT
C       Q(I,J)=(A2*CBR+Q(I,J-1)+Q(I-1,J)*CRR)/(1+CRR)
C       A(I,J)=(Q(I,J)/ALP)**(1./AM)
        A(I,J)=(A2+A(I,J-1)+CRR*A(I-1,J))/(1+CRR)
        Q(I,J)=ALP*(A(I,J))**AM
 50      CONTINUE
        V=Q(N,J)/A(N,J)
C       Y1=A(N,J)/1.
        XP=9.81*A(N,J)
C       FR2=SQRT(V*V*Tp/XP)
        FR2=V/SQRT(XP)
        SK1=SQ*AL
        ADC=SK1/A(N,J)
        FK1=A(N,J)*FR2*FR2
        AK2=SK1/FK1
      WRITE(2,964)TT,Q(N,J),A(N,J),V,QR(J),ADC,FR2,AK2
 963  FORMAT(9X,'TIME(HRS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL',
     19X,'QR',16X,'ADC',16X,'FR2',12X,'AK2')
 964  FORMAT(2X,8F16.7)
 5    CONTINUE
 956    FORMAT(6X,'OVER LAND FLOW ROUTING')
         RETURN
      END
```

```
*******************************************************************************
C**CAL. OF CHANNEL FLOW BY KINEMATIC WAVE (DISTR) THEORY(AKK4.FOR)
C:::*************SCHEME I AND II******(1ST)***************
C        BY MD. MUBARAK HOSSAIN
C        ------------------------------------
C        -------------------------------------
C        CHANNEL FLOW ROUTING:
C        ----------------------------------
C        DX1=LENGTH STRIP(METRE)
C        DT1=TIME  STEP(SECOND)
C        AN1=CHANNEL BED ROUGHNESS
C        SO1=CHANNEL BED SLOPE
C        M1=NOS. OF TIME STEPS
C        N1=NOS. OF LENGH STRIPS
C        AL1=LENGH OF THE MAIN CHANNEL(METRE)
C        Y1,Y2,Y3=DEPTHS OF WATER ON CHANNEL
C        B=BED WIDTH(METRE)
C        BZ=CHANNEL SIDE SLOPE
C        FR1=FRUDE NO.(CHANNEL)
C        AK=KINEMATIC WAVE NO.
C        Q=DISCHARGE(M**3/S) AT THE CHANNEL
C        A=AREA OF CROSS-SECTION AT THE CHANNEL
C        -----------------------------------
C        OVERLAND FLOW ROUTING:
C        ----------------------------------
C        DX=LENGTH STRIP(METRE)
C        DT=TIME  STEP(SECOND)
C        AN= ROUGHNESS
C        SO= SLOPE
C        M=NOS. OF TIME STEPS
C        N=NOS. OF LENGH STRIPS
C        AL=LENGH OF THE OVER LAND(METRE)
C        FR2=FRUDE NO.
C        AK2=KINEMATIC WAVE NO.
C        AN2= ROUGHNESS
C        SO2= SLOPE
C        N2=NOS. OF LENGH STRIPS
C        AL2=LENGH OF THE OVER LAND(METRE)
```

```
C       Q=DISCHARGE(M**2/S) ON THE OVERLAND
C       A=AREA OF CROSS-SECTION OF THE OVERLAND DEPTH
        QR=RAINFALL EXCESS INTENSITY(METRE/SEC.)
C       ----------------------------------------------------------
        DIMENSION N1(4),AL1(4),QBR1(85,4),QBR2(85,4)
        DIMENSION Q1(85),Q2(85),Q3(85),QC(85)
        COMMON/AAA/L1
        COMMON/AA1/Y1,Y2,Y3,BZ
        COMMON/AA2/SO1(4),AN1(4),B(4)
        COMMON/AA3/EC(4),ALP(4)
        COMMON/BB1/DX,DT,AM,M,NR,QR(5,85)
        COMMON/BB2/SO(4),AN(4),N(4),AL(4)
        COMMON/BB3/Q(112,85,5),A(112,85,5)
        COMMON/CC1/SO2(4),AN2(4),N2(4),AL2(4)
        OPEN(UNIT=1, DEVICE='DSK',FILE='NAA.DAT')
        OPEN(UNIT=2, DEVICE='DSK',FILE='NAAH.OUT')
        READ(1,*)DX1,DT1,M1,L1,Y1,Y2,Y3,BZ
        READ(1,*) DX,DT,AM,M,L2,NR
        DO 100 KK=1,L2
        IF(KK.EQ.1) L1=4
        IF(KK.EQ.2) L1=2
        IF(KK.EQ.3) L1=2
        READ(1,*)( SO1(K),K=1,L1)
        READ(1,*)(AN1(K),K=1,L1)
        READ(1,*)(B(K),K=1,L1)
        READ(1,*)( N1(K),K=1,L1)
        READ(1,*)( AL1(K),K=1,L1)
        READ(1,*)( AN(K),K=1,L1)
        READ(1,*)( SO(K),K=1,L1)
        READ(1,*)( N(K),K=1,L1)
        READ(1,*)( AL(K),K=1,L1)
        READ(1,*)( AN2(K),K=1,L1)
        READ(1,*)( SO2(K),K=1,L1)
        READ(1,*)( N2(K),K=1,L1)
        READ(1,*)( AL2(K),K=1,L1)
        READ(1,*)((QR(K,J),J=1,NR),K=1,L1)
        WRITE(2,950)DX1,DT1,M1,L1,Y1,Y2,Y3,BZ
```

```
          WRITE(2,960)(DX,DT,AM,M,L2,NR)
  950  FORMAT(2X,'DX1=',F9.4,4X,'DT1=',F9.4,4X,
      1'M1=',I9,4X,'L1=',I9
      2,4X,'Y1=',F10.4,4X,'Y2='
      3,F9.4,4X,'Y3=',F9.4,4X,'BZ=',F9.4)
  960  FORMAT(2X,'DX=',F9.4,4X,'DT=',F9.4,4X,
      1'AM=',F9.4,4X,'M=',I9,4X,'L2=',I9
      2,4X,'NR=',I9)
C        CALL LAND (DX,DT,AN,SO,AM,M,N,NR,L1,AL,QR,QI)
          CALL ALAND
          DO 1001 K=1,L1
          DO 1001 J=2,M1
          QBR1(J,K)=Q(N(K),J,K)
 1001     CONTINUE
          CALL ALAND2
          DO 1002 K=1,L1
          DO 1002 J=2,M1
          QBR2(J,K)=Q(N2(K),J,K)
 1002     CONTINUE
          WRITE(2,955)
  955     FORMAT(6X,'CHANNEL FLOW ROUTING')
          CALL ASALP
C         REWIND2
          WRITE(2,953)
  953     FORMAT(9X,'TIME(HRS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL'
         1,9X,'QBR',14X,'ABR',12X,'FR1',16X,'AK')
          DXT1=DX1/DT1
          DTX1=DT1/DX1
C         SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
          DO 3J=1,M1
          A(1,J,1)=0.0
          Q(1,J,1)=0.0
    3     CONTINUE
          DO 12 K=1,L1
        DO 4I=1,N1(K)
        A(I,1,K)=0.0
        Q(I,1,K)=0.0
```

```
   4 CONTINUE
12     CONTINUE
       T=0.00
        DO 5J=2,M1
       T=T+DT1
C      TT=T/60.0
         TT=T/3600.0
       DO 8 K=1,L1
       DO 50 I=2,N1(K)
C        ABJ=0.0
C      DO 6 II=2,N1(K)-1
C      ABJ=ABJ+A(II,J-1,K)
C 6      CONTINUE
C       ABJ1=ABJ+(A(1,J-1,K)+A(N1(K),J-1,K))*0.5
C        QBR=Q(N(K),J,K)+Q(N2(K),J,K)
C      ABR1=DT1*QBR
C        ABR(J)=ABR1+ABJ1*(DX1/((N1(K)-1)*DX1))
C       IF (NR1.GT.M1)QR(J)=0.0
       ABR1=(A(I,J-1,K)+A(I-1,J,K))*0.5
       QBR=QBR1(J,K)+QBR2(J,K)
       CBR=ALP(K)*EC(K)*(ABR1**(EC(K)-1.0))
     A1=A(I,J-1,K)+A(I-1,J-1,K)
     A2=A(I,J-1,K)-A(I-1,J-1,K)
     A3=A(I-1,J,K)-A(I-1,J-1,K)
     A4=QBR*DT1+A(I,J-1,K)
     A5=ALP(K)*EC(K)*DTX1*(A1/2)**(EC(K)-1.0)*A2
     IF(CBR.LE.DXT1)GO TO 200
     IF(CBR.GT.DXT1) GOTO 300
 200 A(I,J,K)=A4-A5
     Q(I,J,K)=ALP(K)*(A(I,J,K))**EC(K)
       GO TO 50
       GO TO 5
  300 Q(I,J,K)=Q(I-1,J,K)+QBR*DX1-DXT1*A3
     A(I,J,K)=(Q(I,J,K)/ALP(K))**(1./EC(K))
 50      CONTINUE
       V=Q(N1(K),J,K)/A(N1(K),J,K)
       BS=SQRT(B(K)*B(K)+4.0*BZ*A(N1(K),J,K))
```

```
          YS=(-B(K)+BS)/(2.*BZ)
          Y=ABS(YS)
          TP=B(K)+2.*BZ*Y
          XP=9.81*A(N1(K),J,K)
C         XP=32.2*A(N1(K),J,K)
          FR1=SQRT(V*V*TP/XP)
          SK=SO1(K)*AL1(K)
          FK=Y*FR1*FR1
          AK=SK/FK
C         WRITE(2,954)TT,Q(N1(K),J,K),A(N1(K),J,K),V,QBR,ABR1,
C         1FR1,AK
  954    FORMAT(5X,8F15.7)
          A(1,J,K+1)=A(N1(K),J,K)
          Q(1,J,K+1)=Q(N1(K),J,K)
8         CONTINUE
          J1=J-1
          TYPE*,J1,Q(N1(L1),J,L1),V
          IF(KK.EQ.1) Q1(J)=Q(N1(L1),J,L1)
          IF(KK.EQ.2) Q2(J)=Q(N1(L1),J,L1)
          IF(KK.EQ.3) Q3(J)=Q(N1(L1),J,L1)
          QC(J)=Q1(J)+Q2(J)+Q3(J)
          IF(KK.EQ.3)TYPE*,Q1(J),Q2(J),Q3(J),QC(J)
          IF(KK.EQ.3)WRITE(2,910)TT,Q1(J),Q2(J),Q3(J),QC(J)
910       FORMAT(2X,5F12.5)
    5     CONTINUE
100       CONTINUE
C          QO=OBSERVED DISCHARGE IN CUMECS
          QO=29.800
          QA=0.0
          DO 20 J=2,M1
C         QA=QA+Q(N1(L1),J,L1)
          QA=QA+QC(J)
          IF(J.EQ.49) QA8=QA
          IF(J.EQ.43) QA9=QA
20         CONTINUE
          WRITE(2,240) QO,QA8,QA9,QA
240       FORMAT(5X,'QO=',F10.3,4X,'QA8 AFTER 8.0HRSN=',F10.3,4X
```

```
         1,'QA9 7.00HRSN=',F10.3,4X,'TOTAL  CAL DISCH.IN CUMECS='
         2,F10.3)
         TYPE*,QO,QA8,QA9,QA
         CLOSE(UNIT=1)
         STOP
         END
C        SUBROUTINE SALP (Y1,Y2,Y3,B,BZ,SO1,AN1,EC,ALP)
         SUBROUTINE ASALP
C:::::*********************************************************
C.        CAL. OF ALP. AND EC. (ALP.FOR)
         COMMON/AAA/L1
         COMMON/AA1/Y1,Y2,Y3,BZ
         COMMON/AA2/SO1(4),AN1(4),B(4)
         COMMON/AA3/EC(4),ALP(4)
C        TYPE*,Y1,Y2,Y3,BZ
         DO 18 K=1,L1
         A1=(B(K)+BZ*Y1)*Y1
         P1=B(K)+2.*Y1*SQRT(1+BZ*BZ)
         R1=A1/P1
         HR1=R1**(2./3.)
         TYPE*,SO1(K),AN1(K),B(K)
         AR1=1.*SQRT(SO1(K))/AN1(K)
C        AR1=1.486*SQRT(SO1(K))/AN1(K)
         Q=AR1*HR1*A1
C        TYPE*,A1,P1,R1,AR1,Q
         A2=(B(K)+BZ*Y2)*Y2
         P2=B(K)+2.*Y2*SQRT(1+BZ*BZ)
         R2=A2/P2
         HR2=R2**(2./3.)
         Q2=AR1*HR2*A2
C        TYPE*,Q,R1,A2,P2
         A3=(B(K)+BZ*Y3)*Y3
         P3=B(K)+2.*Y3*SQRT(1+BZ*BZ)
         R3=A3/P3
         HR3=R3**(2./3.)
         Q3=AR1*HR3*A3
         Q4=Q/Q2
```

```
          A4=A1/A2
          A5=A1/A3
          Q5=Q/Q3
          XX=ALOG(Q4)
          YY=ALOG(A4)
          XX2=ALOG(Q5)
          YY2=ALOG(A5)
          EC1=XX/YY
          EC2=XX2/YY2
          EC(K)=(EC1+EC2)*0.5
          AB1=A1**EC(K)
          AB2=A2**EC(K)
          AB3=A3**EC(K)
          ALP1=Q/AB1
          ALP2=Q2/AB2
          ALP3=Q3/AB3
C         TYPE*,Q2,AB2,EC,AB1,AB2,ALP1(K),ALP2(K)
          ALP(K)=(ALP1+ALP2+ALP3)/3.
          WRITE(2,751)EC(K), ALP(K)
 751      FORMAT(5X,'EC=',F14.9,4X,'ALP=',F10.6)
 18       CONTINUE
          RETURN
          END
C   ******************************************************
C          SUBROUTINE LAND(DX,DT,AN,SO,AM,M,N,NR,L1,AL,QR,Q)
          SUBROUTINE ALAND
C       CAL. OF OVER LAND FLOW BY KWT MODEL(HEC.FOR)
          COMMON/AAA/L1
          COMMON/BB1/DX,DT,AM,M,NR,QR(5,85)
          COMMON/BB2/SO(4),AN(4),N(4),AL(4)
          COMMON/BB3/Q(112,85,5),A(112,85,5)
          WRITE(2,956)
          WRITE(2,963)
C         TYPE*,DX,DT,AM,M,NR,L1
          DO 20 K=1,L1
          TYPE*,AN(K),SO(K),N(K),AL(K)
          DXT=DX/DT
```

```
         DTX=DT/DX
         ALP=1.*SQRT(SO(K))/AN(K)
C         ALP=1.486*SQRT(SO(K))/AN(K)
C        SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
       DO 3J=1,M
       A(1,J,K)=0.0
       Q(1,J,K)=0.0
    3  CONTINUE
       DO 4I=1,N(K)
       A(I,1,K)=0.0
       Q(I,1,K)=0.0
    4  CONTINUE
C        T=13.00*3600.00
         T=0.00
       DO 5J=2,M
         T=T+DT
C         TT=T/60.0
         TT=T/3600.0
         DO 50 I=2,N(K)
         ABJ=0.0
         DO 6 II=2,N(K)-1
         ABJ=ABJ+A(II,J-1,K)
    6    CONTINUE
         ABJ1=ABJ+(A(1,J-1,K)+A(N(K),J-1,K))*0.5
          ABRR=DT*QR(K,J)
         ABRK=(DX/((N(K)-1)*DX))*ABJ1+ABRR
C         IF (NR.GT.M)QR(J)=0.0
       QBR=QR(K,J)
       CBR=ALP*AM*(ABRK**(AM-1.0))
       A1=A(I,J-1,K)+A(I-1,J-1,K)
       A2=A(I,J-1,K)-A(I-1,J-1,K)
       A3=A(I-1,J,K)-A(I-1,J-1,K)
       A4=QBR*DT+A(I,J-1,K)
       A5=ALP*AM*DTX*(A1/2)**(AM-1.0)*A2
       IF(CBR.LE.DXT)GO TO 200
       IF(CBR.GT.DXT) GOTO 300
C        IF(CBR.EQ.DXT) GOTO 200
```

```
200     A(I,J,K)=(A4-A5)
      Q(I,J,K)=ALP*(A(I,J,K))**AM
        GO TO 50
        GO TO 5
  300 Q(I,J,K)=Q(I-1,J,K)+QBR*DX-DXT*A3
      A(I,J,K)=(Q(I,J,K)/ALP)**(1./AM)
 50      CONTINUE
        V=Q(N(K),J,K)/A(N(K),J,K)
C       Y1=A(N(K),J,K)/1.
        XP=9.81*A(N(K),J,K)
C       XP=32.2*A(N(K),J,K)
        FR2=V/SQRT(XP)
        SK1=SO(K)*AL(K)
        FK1=A(N(K),J,K)*FR2*FR2
        AK2=SK1/FK1
C       TYPE*,Q(N(K),J,K)
C         WRITE(2,964)TT,Q(N(K),J,K),A(N(K),J,K),V,QR(K,J)
C       1,ABRK,FR2,AK2
 963  FORMAT(9X,'TIME(HRS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL',
     19X,'QR',16X,'ABRK',16X,'FR2',12X,'AK2')
 964  FORMAT(2X,7F15.7,2X,F15.3)
   5  CONTINUE
956     FORMAT(6X,'OVER LAND FLOW ROUTING')
20      CONTINUE
       RETURN
       END
C   ***********************************************
        SUBROUTINE ALAND2
C     CAL. OF OVER LAND FLOW BY KWT MODEL(HEC.FOR)
        COMMON/AAA/L1
        COMMON/BB1/DX,DT,AM,M,NR,QR(5,85)
        COMMON/CC1/SO2(4),AN2(4),N2(4),AL2(4)
        COMMON/BB3/Q(112,85,5),A(112,85,5)
        WRITE(2,256)
        WRITE(2,263)
C       TYPE*,DX,DT,AM,M,NR,L1
        DO 20 K=1,L1
```

```
            TYPE*,AN2(K),SO2(K),N2(K),AL2(K)
            DXT=DX/DT
            DTX=DT/DX
            ALP=1.*SQRT(SO2(K))/AN2(K)
C            ALP=1.486*SQRT(SO2(K))/AN2(K)
C           SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
          DO 3J=1,M
          A(1,J,K)=0.0
          Q(1,J,K)=0.0
      3   CONTINUE
          DO 4I=1,N2(K)
          A(I,1,K)=0.0
          Q(I,1,K)=0.0
      4   CONTINUE
            T=0.00
          DO 5J=2,M
            T=T+DT
C            TT=T/60.0
            TT=T/3600.0
            DO 50 I=2,N2(K)
            ABJ=0.0
            DO 6 II=2,N2(K)-1
            ABJ=ABJ+A(II,J-1,K)
    6       CONTINUE
            ABJ1=ABJ+(A(1,J-1,K)+A(N2(K),J-1,K))*0.5
             ABRR=DT*QR(K,J)
            ABRK=(DX/((N2(K)-1)*DX))*ABJ1+ABRR
C          IF (NR.GT.M)QR(J)=0.0
          QBR=QR(K,J)
          CBR=ALP*AM*(ABRK**(AM-1.0))
          A1=A(I,J-1,K)+A(I-1,J-1,K)
          A2=A(I,J-1,K)-A(I-1,J-1,K)
          A3=A(I-1,J,K)-A(I-1,J-1,K)
          A4=QBR*DT+A(I,J-1,K)
          A5=ALP*AM*DTX*(A1/2)**(AM-1.0)*A2
          IF(CBR.LE.DXT)GO TO 200
          IF(CBR.GT.DXT) GOTO 300
```

```
C       IF(CBR.EQ.DXT) GOTO 200
200      A(I,J,K)=(A4-A5)
        Q(I,J,K)=ALP*(A(I,J,K))**AM
         GO TO 50
         GO TO 5
  300 Q(I,J,K)=Q(I-1,J,K)+QBR*DX-DXT*A3
        A(I,J,K)=(Q(I,J,K)/ALP)**(1./AM)
 50       CONTINUE
         V=Q(N2(K),J,K)/A(N2(K),J,K)
C        Y1=A(N(K),J,K)/1.
         XP=9.81*A(N2(K),J,K)
C        XP=32.2*A(N2(K),J,K)
         FR2=V/SQRT(XP)
         SK1=SO2(K)*AL2(K)
         FK1=A(N2(K),J,K)*FR2*FR2
         AK2=SK1/FK1
C        TYPE*,Q(N(K),J,K)
C         WRITE(2,264)TT,Q(N2(K),J,K),A(N2(K),J,K),V,QR(K,J)
C        1,ABRK,FR2,AK2
 263  FORMAT(9X,'TIME(HRS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL',
     19X,'QR',16X,'ABRK',16X,'FR2',12X,'AK2')
 264  FORMAT(2X,7F15.7,2X,F15.3)
   5  CONTINUE
256      FORMAT(6X,'OVER LAND FLOW ROUTING')
20       CONTINUE
        RETURN
        END
```

```
****************************************************************************
C**CAL. OF CHANNEL FLOW BY KINEMATIC WAVE (DISTR) THEORY(AKK.FOR)
C:::*************   SCHEME III  ******* (1ST) ***********************
C        BY MD. MUBARAK HOSSAIN
C        ---------------------------------
C        ----------------------------------
C        CHANNEL FLOW ROUTING:
C        --------------------------------
C        DX1=LENGTH STRIP(METRE)
C        DT1=TIME  STEP(SECOND)
C        AN1=CHANNEL BED ROUGHNESS
C        SO1=CHANNEL BED SLOPE
C        M1=NOS. OF TIME STEPS
C        N1=NOS. OF LENGH STRIPS
C        AL1=LENGH OF THE MAIN CHANNEL(METRE)
C        Y1,Y2,Y3=DEPTHS OF WATER ON CHANNEL
C        B=BED WIDTH(METRE)
C        BZ=CHANNEL SIDE SLOPE
C        FR1=FRUDE NO.(CHANNEL)
C        AK=KINEMATIC WAVE NO.
C        Q=DISCHARGE(M**3/S) AT THE CHANNEL
C        A=AREA OF CROSS-SECTION AT THE CHANNEL
C        ---------------------------------
C        OVERLAND FLOW ROUTING:
C        --------------------------------
C        DX=LENGTH STRIP(METRE)
C        DT=TIME  STEP(SECOND)
C        AN= ROUGHNESS
C        SO= SLOPE
C        M=NOS. OF TIME STEPS
C        N=NOS. OF LENGH STRIPS
C        AL=LENGH OF THE OVER LAND(METRE)
C        FR2=FRUDE NO.
C        AK2=KINEMATIC WAVE NO.
C        AN2= ROUGHNESS
C        SO2= SLOPE
C        N2=NOS. OF LENGH STRIPS
C        AL2=LENGH OF THE OVER LAND(METRE)
```

```
C      Q=DISCHARGE(M**2/S) ON THE OVERLAND
C      A=AREA OF CROSS-SECTION OF THE OVERLAND DEPTH
       QR=RAINFALL EXCESS INTENSITY(METRE/SEC.)
C      ---------------------------------------------------------
       DIMENSION N1(7),AL1(7),QBR1(81,7),QBR2(81,7)
       COMMON/AAA/L1
       COMMON/AA1/Y1,Y2,Y3,BZ
       COMMON/AA2/SO1(7),AN1(7),B(7)
       COMMON/AA3/EC(7),ALP(7)
       COMMON/BB1/DX,DT,AM,M,NR,QR(8,81),ABR(81)
       COMMON/BB2/SO(7),AN(7),N(7),AL(7)
       COMMON/BB3/Q(93,81,8),A(93,81,8)
       COMMON/CC1/SO2(7),AN2(7),N2(7),AL2(7)
       OPEN(UNIT=1, DEVICE='DSK',FILE='KKA.DAT')
       OPEN(UNIT=2, DEVICE='DSK',FILE='KKAT.OUT')
       READ(1,*)DX1,DT1,M1,L1,Y1,Y2,Y3,BZ
       READ(1,*) DX,DT,AM,M,L2,NR
C      DO 100 KK=1,L2
       READ(1,*)( SO1(K),K=1,L1)
       READ(1,*)(AN1(K),K=1,L1)
       READ(1,*)(B(K),K=1,L1)
       READ(1,*)( N1(K),K=1,L1)
       READ(1,*)( AL1(K),K=1,L1)
       READ(1,*)( AN(K),K=1,L1)
       READ(1,*)( SO(K),K=1,L1)
       READ(1,*)( N(K),K=1,L1)
       READ(1,*)( AL(K),K=1,L1)
       READ(1,*)( AN2(K),K=1,L1)
       READ(1,*)( SO2(K),K=1,L1)
       READ(1,*)( N2(K),K=1,L1)
       READ(1,*)( AL2(K),K=1,L1)
       READ(1,*)((QR(K,J),J=1,NR),K=1,L1)
        WRITE(2,950)DX1,DT1,M1,L1,Y1,Y2,Y3,BZ
        WRITE(2,960)(DX,DT,AM,M,L2,NR)
 950  FORMAT(2X,'DX1=',F9.4,4X,'DT1=',F9.4,4X,
     1'M1=',I9,4X,'L1=',I9
     2,4X,'Y1=',F10.4,4X,'Y2='
```

```
          3,F9.4,4X,'Y3=',F9.4,4X,'BZ=',F9.4)
  960  FORMAT(2X,'DX=',F9.4,4X,'DT=',F9.4,4X,
          1'AM=',F9.4,4X,'M=',I9,4X,'L2=',I9
          2,4X,'NR=',I9)
C        CALL LAND (DX,DT,AN,SO,AM,M,N,NR,L1,AL,QR,QI)
         CALL ALAND
         DO 1001 K=1,L1
         DO 1001 J=2,M1
         QBR1(J,K)=Q(N(K),J,K)
1001     CONTINUE
         CALL ALAND2
         DO 1002 K=1,L1
         DO 1002 J=2,M1
         QBR2(J,K)=Q(N2(K),J,K)
1002     CONTINUE
         CALL ASALP
         WRITE(2,955)
955      FORMAT(6X,'CHANNEL FLOW ROUTING')
C        REWIND2
         WRITE(2,953)
953      FORMAT(9X,'TIME(HRS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL'
         1,9X,'QBR',14X,'ABR',12X,'FR1',16X,'AK')
C953     FORMAT(9X,'TIME(HRS)',3(6X,'DISCHARGE'))
C        CALL SALP (Y1,Y2,Y3,B,BZ,SO1,AN1,EC,ALP)
         DXT1=DX1/DT1
         DTX1=DT1/DX1
C        SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
         DO 3J=1,M1
         A(1,J,1)=0.0
         Q(1,J,1)=0.0
    3    CONTINUE
         DO 12 K=1,L1
         DO 4I=1,N1(K)
         A(I,1,K)=0.0
         Q(I,1,K)=0.0
    4    CONTINUE
12       CONTINUE
```

```
         T=0.00
          DO 5J=2,M1
         T=T+DT1
C        TT=T/60.0
         TT=T/3600.0
         DO 8 K=1,L1
         DO 50 I=2,N1(K)
         ABR1=(A(I,J-1,K)+A(I-1,J,K))*0.5
         QBR=QBR1(J,K)+QBR2(J,K)
         CBR=ALP(K)*EC(K)*(ABR1**(EC(K)-1.0))
         CRR=CBR*DTX1
         A2=QBR*DT1
         A(I,J,K)=(A2+A(I,J-1,K)+CRR*A(I-1,J,K))/(1+CRR)
         Q(I,J,K)=ALP(K)*(A(I,J,K))**EC(K)
  50      CONTINUE
         V=Q(N1(K),J,K)/A(N1(K),J,K)
         BS=SQRT(B(K)*B(K)+4.0*BZ*A(N1(K),J,K))
         YS=(-B(K)+BS)/(2.*BZ)
         Y=ABS(YS)
         TP=B(K)+2.*BZ*Y
         XP=9.81*A(N1(K),J,K)
C        XP=32.2*A(N1(K),J,K)
         FR1=SQRT(V*V*TP/XP)
         SK=SO1(K)*AL1(K)
         FK=Y*FR1*FR1
         AK=SK/FK
C        WRITE(2,954)TT,Q(N1(L1),J,L1),A(N1(L1),J,L1),V,QBR,ABR1,
C  954    FORMAT(5X,3F15.7)
         A(1,J,K+1)=A(N1(K),J,K)
         Q(1,J,K+1)=Q(N1(K),J,K)
8        CONTINUE
         WRITE(2,954)TT,Q(N1(L1),J,L1),A(N1(L1),J,L1),V,QBR,ABR1
         1,FR1,AK
   954   FORMAT(5X,8F15.7)
         J1=J-1
         TYPE*,J1,Q(N1(L1),J,L1)
C        IF(KK.EQ.1) Q(J)=Q(N1(L1),J,L1)
```

```
C         IF(KK.EQ.2) Q2(J)=Q(N1(L1),J,L1)
C         QC(J)=Q(J)+Q2(J)
C         TYPE*,Q(J),Q2(J),QC(J)
C         WRITE(2,910)TT, Q(J),Q2(J),QC(J)
C910      FORMAT(5X,4F15.7)
    5     CONTINUE
C100      CONTINUE
C         QO=OBSERVED DISCHARGE IN CUMECS
          QO=85.218
          QA=0.0
          DO 20 J=2,M1
C         QA=QA+QC(J)
          QA=QA+Q(N1(L1),J,L1)
          IF(J.EQ.51) QA8=QA
          IF(J.EQ.63) QA9=QA
20        CONTINUE
          WRITE(2,240) QO,QA8,QA9,QA
240       FORMAT(5X,'QO=',F10.3,4X,'QA8 AFTER 50HRSS=',F10.3,4X
          1,'QA9 62HRSN=',F10.3,4X,'TOTAL  CAL DISCH.IN CUMECS='
          2,F10.3)
          TYPE*,QO,QA8,QA9,QA
          CLOSE(UNIT=1)
          STOP
          END
C         SUBROUTINE SALP (Y1,Y2,Y3,B,BZ,SO1,AN1,EC,ALP)
          SUBROUTINE ASALP
C:::::*************************************************
C         CAL. OF ALP. AND EC. (ALP.FOR)
          COMMON/AAA/L1
          COMMON/AA1/Y1,Y2,Y3,BZ
          COMMON/AA2/SO1(7),AN1(7),B(7)
          COMMON/AA3/EC(7),ALP(7)
C         TYPE*,Y1,Y2,Y3,BZ
          DO 18 K=1,L1
          A1=(B(K)+BZ*Y1)*Y1
          P1=B(K)+2.*Y1*SQRT(1+BZ*BZ)
          R1=A1/P1
```

```
      HR1=R1**(2./3.)
      TYPE*,SO1(K),AN1(K),B(K)
      AR1=1.*SQRT(SO1(K))/AN1(K)
C     AR1=1.486*SQRT(SO1(K))/AN1(K)
      Q=AR1*HR1*A1
C     TYPE*,A1,P1,R1,AR1,Q
      A2=(B(K)+BZ*Y2)*Y2
      P2=B(K)+2.*Y2*SQRT(1+BZ*BZ)
      R2=A2/P2
      HR2=R2**(2./3.)
      Q2=AR1*HR2*A2
C     TYPE*,Q,R1,A2,P2
      A3=(B(K)+BZ*Y3)*Y3
      P3=B(K)+2.*Y3*SQRT(1+BZ*BZ)
      R3=A3/P3
      HR3=R3**(2./3.)
      Q3=AR1*HR3*A3
      Q4=Q/Q2
      A4=A1/A2
      A5=A1/A3
      Q5=Q/Q3
      XX=ALOG(Q4)
      YY=ALOG(A4)
      XX2=ALOG(Q5)
      YY2=ALOG(A5)
      EC1=XX/YY
      EC2=XX2/YY2
      EC(K)=(EC1+EC2)*0.5
      AB1=A1**EC(K)
      AB2=A2**EC(K)
      AB3=A3**EC(K)
      ALP1=Q/AB1
      ALP2=Q2/AB2
      ALP3=Q3/AB3
C     TYPE*,Q2,AB2,EC,AB1,AB2,ALP1(K),ALP2(K)
      ALP(K)=(ALP1+ALP2+ALP3)/3.
      WRITE(2,751)EC(K), ALP(K)
```

```
 751     FORMAT(5X,'EC=',F14.9,4X,'ALP=',F10.6)
 18      CONTINUE
         RETURN
         END
C   ************************************************
C         SUBROUTINE LAND(DX,DT,AN,SO,AM,M,N,NR,L1,AL,QR,Q)
         SUBROUTINE ALAND
C      CAL. OF OVER LAND FLOW BY KWT MODEL(HEC.FOR)
         COMMON/AAA/L1
         COMMON/BB1/DX,DT,AM,M,NR,QR(8,81),ABR(81)
         COMMON/BB2/SO(7),AN(7),N(7),AL(7)
         COMMON/BB3/Q(93,81,8),A(93,81,8)
         WRITE(2,956)
         WRITE(2,963)
C        TYPE*,DX,DT,AM,M,NR,L1
         DO 20 K=1,L1
         TYPE*,AN(K),SO(K),N(K),AL(K)
         DXT=DX/DT
         DTX=DT/DX
         ALP=1.*SQRT(SO(K))/AN(K)
C         ALP=1.486*SQRT(SO(K))/AN(K)
C        SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
        DO 3J=1,M
        A(1,J,K)=0.0
        Q(1,J,K)=0.0
   3    CONTINUE
        DO 4I=1,N(K)
        A(I,1,K)=0.0
        Q(I,1,K)=0.0
   4    CONTINUE
         T=0.00
        DO 5J=2,M
         T=T+DT
C        TT=T/60.0
         TT=T/3600.0
         DO 50 I=2,N(K)
         ABJ=0.0
```

```
          DO 6 II=2,N(K)-1
          ABJ=ABJ+A(II,J-1,K)
    6     CONTINUE
          ABJ1=ABJ+(A(1,J-1,K)+A(N(K),J-1,K))*0.5
           ABRR=DT*QR(K,J)
          ABR(J)=(DX/((N(K)-1)*DX))*ABJ1+ABRR
C        IF (NR.GT.M)QR(J)=0.0
         QBR=QR(K,J)
         CBR=ALP*AM*(ABR(J)**(AM-1.0))
          CRR=CBR*DTX
          A2=QBR*DT
          A(I,J,K)=(A2+A(I,J-1,K)+CRR*A(I-1,J,K))/(1+CRR)
          Q(I,J,K)=ALP*(A(I,J,K))**AM
   50      CONTINUE
          V=Q(N(K),J,K)/A(N(K),J,K)
          XP=9.81*A(N(K),J,K)
C         XP=32.2*A(N(K),J,K)
          FR2=V/SQRT(XP)
          SK1=SO(K)*AL(K)
          FK1=A(N(K),J,K)*FR2*FR2
          AK2=SK1/FK1
C         TYPE*,Q(N(K),J,K)
C        WRITE(2,964)TT,Q(N(K),J,K),A(N(K),J,K),V,QR(K,J)
C      1,ABR(J),FR2,AK2
  963  FORMAT(9X,'TIME(HRS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL',
      19X,'QR',16X,'ABR',16X,'FR2',12X,'AK2')
  964  FORMAT(2X,7E15.7,2X,F15.3)
    5  CONTINUE
956       FORMAT(6X,'OVER LAND FLOW ROUTING')
20        CONTINUE
         RETURN
         END
C    ************************************************
          SUBROUTINE ALAND2
C      CAL. OF OVER LAND FLOW BY KWT MODEL(HEC.FOR)
          COMMON/AAA/L1
          COMMON/BB1/DX,DT,AM,M,NR,QR(8,81),ABR(81)
```

```
              COMMON/CC1/SO2(7),AN2(7),N2(7),AL2(7)
C             COMMON/CC2/Q(93,81,8)
              COMMON/BB3/Q(93,81,8),A(93,81,8)
              WRITE(2,256)
              WRITE(2,263)
C             TYPE*,DX,DT,AM,M,NR,L1
              DO 20 K=1,L1
              TYPE*,AN2(K),SO2(K),N2(K),AL2(K)
              DXT=DX/DT
              DTX=DT/DX
              ALP=1.*SQRT(SO2(K))/AN2(K)
C              ALP=1.486*SQRT(SO2(K))/AN2(K)
C             SPECIFY THE INITIAL AND BOUANDARY CONDITIONS
           DO 3J=1,M
           A(1,J,K)=0.0
           Q(1,J,K)=0.0
      3    CONTINUE
           DO 4I=1,N2(K)
           A(I,1,K)=0.0
           Q(I,1,K)=0.0
      4    CONTINUE
             T=0.00
           DO 5J=2,M
             T=T+DT
C            TT=T/60.0
             TT=T/3600.0
             DO 50 I=2,N2(K)
             ABJ=0.0
             DO 6 II=2,N2(K)-1
             ABJ=ABJ+A(II,J-1,K)
 6           CONTINUE
             ABJ1=ABJ+(A(1,J-1,K)+A(N2(K),J-1,K))*0.5
              ABRR=DT*QR(K,J)
             ABR(J)=(DX/((N2(K)-1)*DX))*ABJ1+ABRR
C           IF (NR.GT.M)QR(J)=0.0
           QBR=QR(K,J)
           CBR=ALP*AM*(ABR(J)**(AM-1.0))
```

```
            CRR=CBR*DTX
            A2=QBR*DT
            A(I,J,K)=(A2+A(I,J-1,K)+CRR*A(I-1,J,K))/(1+CRR)
            Q(I,J,K)=ALP*(A(I,J,K))**AM
   50        CONTINUE
            V=Q(N2(K),J,K)/A(N2(K),J,K)
            XP=9.81*A(N2(K),J,K)
C           XP=32.2*A(N2(K),J,K)
            FR2=V/SQRT(XP)
            SK1=SO2(K)*AL2(K)
            FK1=A(N2(K),J,K)*FR2*FR2
            AK2=SK1/FK1
C           TYPE*,Q(N(K),J,K)
C          WRITE(2,264)TT,Q(N2(K),J,K),A(N2(K),J,K),V,QR(K,J)
C        1,ABR(J),FR2,AK2
   263  FORMAT(9X,'TIME(HRS)',6X,'DISCHARGE',6X,'AREA',12X,'VEL',
       19X,'QR',16X,'ABR',16X,'FR2',12X,'AK2')
   264  FORMAT(2X,7F15.7,2X,F15.3)
     5  CONTINUE
256         FORMAT(6X,'OVER LAND FLOW ROUTING')
20          CONTINUE
           RETURN
           END
```

APPENDIX --II(A)

BRIDGE NO. 319

RAINFALL RUNOFF DISTRIBUTION

(1) STORM EVENT, DATED: 14.10.1962

RUNOFF OBSERVATION		RAINFALL OBSERVATION			
TIME (MIN)	DISCHARGE (M**3/S)	TIME (MIN)	RAIFALL (MM)	Ø-INDEX (MM/HR)	RAINFALL EXCESS (MM)
(1)	(2)	(3)	(4)	(5)	(6)
0.0	0.000				
10.0	0.113	0.0-20.0	19.81	19.035	13.465
20.0	0.991	20.0-40.0	13.72		7.375
30.0	8.659				
40.0	9.405				
50.0	5.512				
60.0	2.321				
70.0	0.623				
80.0	0.425				
90.0	0.170				
100.0	0.142				
110.0	0.113				
120.0	0.000				

(2) STORM EVENT, DATED:4.11.1962

(1)	(2)	(3)	(4)	(5)	(6)
0.0	0.000				
10.0	0.255	0.0-30.0	21.40	11.60	15.60
20.0	4.953	30.0-60.0	1.78		
30.0	7.301				
40.0	4.953				
50.0	1.981				
60.0	0.623				
70.0	0.509				
80.0	0.425				
90.0	0.255				
100.0	0.057				
110.0	0.000				

(3) STORM EVENT, DATED:17.7.1963

(1)	(2)	(3)	(4)	(5)	(6)
0.0	0.000				
10.0	1.274	10.0	16.51	62.16	6.15
20.0	3.339	20.0	5.59		
30.0	1.274				
40.0	0.934				
50.0	0.621				
60.0	0.453				
70.0	0.198				
80.0	0.169				
90.0	0.141				
100.0	0.000				

(4) STORM EVENT, DATED:7.10.1963

(1)	(2)	(3)	(4)	(5)	(6)
0.0	0.000	0.0-30.0	20.32	29.46	5.59
10.0	0.453	30.0-60.0	11.68		
20.0	0.566				
30.0	1.075				
40.0	1.500				
50.0	1.075				
60.0	0.566				
70.0	0.481				
80.0	0.453				
90.0	0.423				
100.0	0.311				
110.0	0.283				
120.0	0.198				
130.0	0.142				
140.0	0.113				
150.0	0.000				

(5) STORM EVENT, DATED:5.8.1964

(1)	(2)	(3)	(4)	(5)	(6)
0.0	0.000	0.0-30.0	10.92	1.056	10.392
10.0	0.283	30.0-50.0	4.32		3.968
20.0	0.481				
30.0	0.708				
40.0	2.858				
50.0	3.651				
60.0	3.170				
70.0	2.689				
80.0	1.726				
90.0	1.019				
100.0	0.906				
110.0	0.849				
120.0	0.538				
130.0	0.340				
140.0	0.255				
150.0	0.142				
160.0	0.000				

(6) STORM EVENT, DATED:7/8.9.1964

(1)	(2)	(3)	(4)	(5)	(6)
0.0	0.000	0.0-30.0	17.27	0.404	17.10
10.0	0.170	30.0-60.0	7.62		7.42
20.0	0.425	60.0-90.0	2.03		1.83
30.0	3.311	90.0-120.0	1.02		0.82
40.0	3.651	120.0-150.0	1.02		0.82
50.0	3.962	150.0-180.0	1.02		0.82
60.0	4.471				
70.0	3.962				
80.0	3.311				
90.0	2.689				
100.0	2.264				
110.0	2.010				
120.0	1.726				
130.0	1.528				
140.0	1.302				
150.0	1.217				
160.0	1.019				
170.0	0.708				
180.0	0.566				
190.0	0.368				
200.0	0.283				
210.0	0.226				
220.0	0.142				
230.0	0.000				

(7) STORM EVENT, DATED:8/9.9.1964

(1)	(2)	(3)	(4)	(5)	(6)
0.0	0.000				
10.0	0.849	0.0-30.0	3.56	0.392	3.37
20.0	2.123	30.0-60.0	20.83		20.63
30.0	2.547	90.0	10.41		10.22
40.0	3.396	120.0	2.03		1.834
50.0	3.651	150.0	1.27		1.074
60.0	3.651	180.0	1.52		1.324
70.0	5.094	210.0	1.27		1.074
80.0	6.281				
90.0	3.368				
100.0	3.056				
110.0	3.000				
120.0	2.689				
130.0	2.264				
140.0	2.123				
150.0	2.009				
160.0	1.755				
170.0	1.585				
180.0	1.245				
190.0	0.906				
200.0	0.736				
210.0	0.566				
220.0	0.425				
230.0	0.312				
240.0	0.198				
250.0	0.142				
260.0	0.085				
270.0	0.000				

(8) STORM EVENT, DATED:9.7.1988

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000				
10.0	0.150	30.0	6.60	0.33	6.435
20.0	0.600	60.0	5.60		5.435
30.0	1.416	90.0	3.50		3.335
40.0	2.120	120.0	2.40		2.235
50.0	2.650	150.0	1.80		1.635
60.0	2.775				
70.0	2.620				
80.0	2.300				
90.0	1.954				
100.0	1.680				
110.0	1.400				
120.0	1.132				
130.0	1.020				
140.0	0.879				
150.0	0.795				
160.0	0.650				
170.0	0.535				
180.0	0.425				
190.0	0.350				
200.0	0.250				
210.0	0.170				
220.0	0.125				
230.0	0.073				
240.0	0.000				

(9) STORM EVENT, DATED:8.8.1988

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000				
30.0	0.396	0.0-30.0	13.93	16.625	5.62
60.0	0.906	30.0-60.0	9.16		0.85
90.0	0.566				
120.0	0.425				
150.0	0.340				
180.0	0.198				
210.0	0.113				
240.0	0.000				

(10) STORM EVENT, DATED:11.9.1988

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000	0.0-30.0	17.00	3.17	15.42
30.0	0.113	60.0	16.7		15.12
60.0	4.190	90.0	7.1		5.52
90.0	5.891	120.0	3.90		2.31
120.0	2.718				
150.0	2.067				
180.0	1.220				
210.0	0.878				
240.0	0.283				
270.0	0.113				
300.0	0.000				

APPENDIX --II(B)

BRIDGE NO. 317

RAINFALL RUNOFF DISTRIBUTION

(1) STORM EVENT, DATED: 25.10.1961

RUNOFF OBSERVATION		RAINFALL OBSERVATION			
TIME (MIN)	DISCHARGE (M**3/S)	TIME (MIN)	RAINFALL (MM)	Ø-INDEX (MM/HR)	RAINFALL EXCESS (MM)
(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000				
30.0	2.038	0.0-30.0	4.88	5.92	1.92
60.0	7.810	30.0-60.0	22.40		19.44
90.0	12.820	60.0-90.0	19.90		16.94
120.0	2.010	90.0-120.0	1.63		
150.0	1.472				
180.0	0.991				
210.0	0.877				
240.0	0.594				
270.0	0.509				
300.0	0.425				
330.0	0.226				
360.0	0.000				

(2) STORM EVENT, DATED:24.9.1964

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000	0.0-30.0	13.72	4.21	11.61
10.0	0.708	30.0-60.0	10.67		8.56
20.0	2.321	60.0-90.0	10.92		8.81
30.0	3.764	90.0-120.0	20.57		18.464
40.0	3.679	120.0-150.0	11.18		9.074
50.0	3.622	150.0-180.0	13.97		11.864
60.0	3.113	180.0-210.0	6.35		4.244
70.0	2.434	210.0-240.0	2.54		0.434
80.0	2.830				
90.0	3.764				
100.0	3.990				
110.0	4.896				
120.0	7.342				
130.0	15.100				
140.0	14.716				
150.0	12.452				
160.0	9.056				
170.0	7.641				
180.0	6.509				
190.0	7.641				
200.0	10.471				
210.0	7.641				
220.0	6.509				
230.0	5.688				
240.0	3.764				
250.0	3.169				
260.0	2.830				
270.0	2.321				
280.0	2.122				
290.0	1.896				
300.0	1.415				
310.0	1.188				
320.0	1.160				
330.0	0.991				
340.0	0.934				
350.0	0.792				
360.0	0.764				
370.0	0.708				
380.0	0.283				
390.0	0.142				
400.0	0.000				

(3) STORM EVENT, DATED:14.10.1962

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000	0.0-30.0	19.81	19.32	10.15
30.0	0.226	30.0-50.0	13.72		7.28
60.0	5.773	50.0-90.0	1.27		
90.0	1.613	90.0-120.0	1.52		
120.0	0.849	120.0-150.0	2.03		
150.0	0.708	150.0-180.0	0.73		
180.0	0.623	180.0-210.0	0.76		
210.0	0.509				
240.0	0.509				
270.0	0.481				
300.0	0.453				
330.0	0.425				
360.0	0.396				
390.0	0.368				
420.0	0.283				
450.0	0.226				
480.0	0.113				
510.0	0.000				

(4) STORM EVENT, DATED: 18.7.1963

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000				
30.0	0.142	30.0	6.35	9.9	1.40
60.0	0.283	60.0	6.86		1.91
90.0	9.905	80.0	17.53		14.23
120.0	1.358				
150.0	0.764				
180.0	0.283				
210.0	0.283				
240.0	0.255				
270.0	0.226				
300.0	0.142				
330.0	0.000				

(5) STORM EVENT, DATED:26.7.1964

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000				
10.0	0.142	0.0-30.0	16.00	5.944	13.03
20.0	2.066				
30.0	6.481				
40.0	5.688				
50.0	5.462				
60.0	2.066				
70.0	1.925				
80.0	1.783				
90.0	1.160				
100.0	0.878				
110.0	0.594				
120.0	0.481				
130.0	0.396				
140.0	0.368				
150.0	0.311				
160.0	0.255				
170.0	0.198				
180.0	0.142				
190.0	0.000				

(6) STORM EVENT, DATED:7/8.9.1964

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000				
10.0	2.462	0.0-20.0	17.27	5.436	15.458
20.0	3.028	20.0-50.0	7.62		4.902
30.0	3.170	50.0-90.0	2.03		
40.0	4.245	90.0-120.0	1.02		
50.0	4.245	120.0-150.0	1.02		
60.0	4.047				
70.0	3.849				
80.0	3.622				
90.0	3.424				
100.0	3.198				
110.0	2.830				
120.0	2.462				
130.0	2.264				
140.0	1.274				
150.0	1.160				
160.0	0.708				
170.0	0.566				
180.0	0.425				
190.0	0.283				
200.0	0.142				
210.0	0.085				
220.0	0.000				

(7) STORM EVENT, DATED:8.9.1964

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000	0.0-30.0	20.83	2.14	19.76
30.0	0.934	30.0-60.0	10.67		9.60
60.0	4.245	60.0-90.0	2.03		0.96
90.0	5.098	90.0-120.0	1.27		0.20
120.0	2.830	120.0-150.0	1.52		0.45
150.0	2.689	150.0-180.0	1.27		0.20
180.0	2.462	180.0-210.0	0.51		
210.0	2.066	210.0-240.0	0.51		
240.0	1.213	240.0-270.0	0.51		
270.0	0.934				
300.0	0.849				
330.0	0.481				
360.0	0.283				
390.0	0.142				
420.0	0.000				

(8) STORM EVENT, DATED:15.9.1964

(1)	(2)	(3)	(4)	(5)	(6)
00.0	0.000				
30.0	0.302	0.0-30.0	6.35	0.565	6.07
60.0	3.424	30.0-60.0	8.89		8.61
90.0	2.830	60.0-90.0	3.04		2.76
120.0	2.462	90.0-120.0	2.79		2.51
150.0	2.264	120.0-150.0	0.25		
180.0	1.443	150.0-180.0	0.25		
210.0	1.160				
240.0	0.934				
270.0	0.566				
300.0	0.125				
330.0	0.000				

APPENDIX --II(C)

BRIDGE NO. 719

RAINFALL RUNOFF DISTRIBUTION

(1) STORM EVENT, DATED: 24/25.7.1964

RUNOFF OBSERVATION		RAINFALL OBSERVATION			
TIME (HR)	DISCHARGE (M**3/S)	TIME (HR)	RAIFALL (MM)	Ø-INDEX (MM/HR)	RAINFALL EXCESS (MM)
(1)	(2)	(3)	(4)	(5)	(6)
0.0	1.133	0.0-0.5	5.08	2.07	4.045
0.5	4.560	0.5-1.0	10.16		9.125
1.0	8.100	1.0-1.5	3.05		2.015
1.5	9.770	1.5-2.0	4.06		3.025
2.0	12.461	2.0-2.5	1.02		
2.5	20.107	2.5-3.0	1.02		
3.0	25.233	3.0-3.5	0.76		
3.5	21.523	3.5-4.0	0.76		
4.0	16.142				
4.5	12.461				
5.0	9.062				
5.5	5.806				
6.0	4.420				
6.5	2.410				
7.0	1.303				
7.5	1.246				
8.0	1.133				

(2) STORM EVENT, DATED: 26/27.7.1964

(1)	(2)	(3)	(4)	(5)	(6)
0.0	1.133				
0.5	2.050	0.0-0.5	8.13	2.296	6.982
1.0	2.690	0.5-1.0	6.60		5.452
1.5	4.280	1.0-1.5	2.03		0.882
2.0	6.542	1.5-2.0	4.06		2.912
2.5	8.383	2.0-2.5	6.10		4.952
3.0	10.195	2.5-3.0	10.16		9.012
3.5	12.400				
4.0	14.533				
4.5	18.258				
5.0	24.922				
5.5	33.842				
6.0	26.904				
6.5	22.146				
7.0	18.691				
7.5	14.273				
8.0	11.328				
8.5	8.638				
9.0	6.514				
9.5	5.098				
10.0	3.398				
10.5	2.577				
11.0	1.602				
11.5	1.218				
12.0	1.133				

(3) STORM EVENT, DATED: 3.9.1964

(1)	(2)	(3)	(4)	(5)	(6)
0.0	1.133				
0.5	2.133	0.0-0.5	4.57	6.570	1.283
1.0	3.965	0.5-1.0	6.86		3.573
1.5	5.097	1.0-1.5	7.37		4.084
2.0	6.796				
2.5	8.496				
3.0	11.610				
3.5	10.760				
4.0	8.496				
4.5	6.853				
5.0	5.806				
5.5	4.432				
6.0	3.257				
6.5	2.648				
7.0	2.407				
7.5	1.700				
8.0	1.133				

(4) STORM EVENT DATED: 11.8.1965

(1)	(2)	(3)	(4)	(5)	(6)
0.0	1.000	0.0-0.5	25.40	15.75	17.525
0.5	2.010	0.5-1.0	18.29		10.415
1.0	4.210	1.0-1.5	1.52		
1.5	10.200	1.5-2.0	0.76		
2.0	21.240				
2.5	29.740				
3.0	41.022				
3.5	35.400				
4.0	31.152				
4.5	21.810				
5.0	13.480				
5.5	10.223				
6.0	6.090				
6.5	3.400				
7.0	1.982				
7.5	1.000				

(5) STORM EVENT, DATED: 18.9.1965

(1)	(2)	(3)	(4)	(5)	(6)
0.0	1.133				
0.5	1.700	0.0-0.5	20.80	30.36	5.62
1.0	5.660	0.5-1.0	2.03		
1.5	8.550	1.0-1.5	1.02		
2.0	10.480				
2.5	6.510				
3.0	5.660				
3.5	5.150				
4.0	3.960				
4.5	2.970				
5.0	2.550				
5.5	2.040				
6.0	1.700				
6.5	1.440				
7.0	1.133				

(6) STORM EVENT, DATED: 6.7.1966

(1)	(2)	(3)	(4)	(5)	(6)
0.0	1.190	0.0-0.5	10.92	9.64	6.1
0.5	2.832	0.5-1.0	4.57		
1.0	7.08	1.0-1.5	4.06		
1.5	8.496	1.5-2.0	2.03		
2.0	11.555	2.0-2.5	4.06		
2.5	8.836	2.5-3.0	2.03		
3.0	5.947	3.0-3.5	1.02		
3.5	4.701				
4.0	3.257				
4.5	2.832				
5.0	2.407				
5.5	2.010				
6.0	1.669				
6.5	1.190				

(7) STORM EVENT DATED: 16.9.1966

(1)	(2)	(3)	(4)	(5)	(6)
0.0	1.416	0.0-0.5	43.18	45.56	20.400
0.5	2.266	0.5-1.0	2.03		
1.0	7.640				
1.5	35.400				
2.0	33.270				
2.5	29.156				
3.0	19.820				
3.5	15.290				
4.0	10.190				
4.5	7.650				
5.0	6.230				
5.5	4.672				
6.0	3.250				
6.5	2.265				
7.0	1.416				

(8) STORM EVENT DATED: 20.9.1966

(1)	(2)	(3)	(4)	(5)	(6)

KACHWA WATERSHED

Rainfall runoff distribution

(1) **Dated 27.7.77**

Runoff observations				Rainfall observations	
Time hrs.	Discharge rate $m^3/sec.$	Time hrs.	Discharge rate $m^3/sec.$	Time interval hrs.	Rainfall intensity mm/hr
1.	0.107	27.	0.263	0.0 - 0.25	20.0
2.	0.249	28.	0.243	0.25 - 0.50	30.0
3.	0.431	29.	0.236	0.50 - 1.00	15.0
4.	0.642	30.	0.217	1.00 - 1.75	25.0
5.	0.756	31.	0.198	1.75 - 4.00	8.0
6.	0.832	32.	0.177	4.00 - 5.00	2.1
7.	0.851	33.	0.171		
8.	0.805	34.	0.167		
9.	0.784	35.	0.156		
10.	0.777	36.	0.151		
11.	0.738	37.	0.140		
12.	0.696	38.	0.135		
13.	0.666	39.	0.130		
14.	0.609	40.	0.125		
15.	0.566	41.	0.114		
16.	0.503	42.	0.104		
17.	0.466	43.	0.098		
18.	0.431	44.	0.093		
19.	0.406	45.	0.082		
20.	0.386	46.	0.077		
21.	0.365	47.	0.072		
22.	0.346	48.	0.067		
23.	0.325	49.	0.064		
24.	0.305	50.	0.062		
25.	0.285	51.	0.062		
26.	0.275	52.	0.056		

Rainfall runoff distribution

(2) **Dated 4.8.77**

Runoff Observations				Rainfall observations	
Time hrs.	Discharge rate m^3/sec.	Time hrs.	Discharge rate m^3/sec.	Time interval hrs.	Rainfall intensity mm/hr.
1.	0.754	26.	0.329	0.0 – 0.25	36.0
2.	0.624	27.	0.314	0.25 – 0.75	40.0
3.	0.790	28.	0.298	0.75 – 1.00	48.0
4.	0.972	29.	0.279	1.00 – 3.00	3.7
5.	1.019	30.	0.261		
6.	0.981	31.	0.245		
7.	0.984	32.	0.218		
8.	0.993	33.	0.203		
9.	0.983	34.	0.198		
10.	0.931	35.	0.196		
11.	0.850	36.	0.195		
12.	0.808	37.	0.186		
13.	0.728	38.	0.166		
14.	0.632	39.	0.146		
15.	0.578	40.	0.136		
16.	0.511	41.	0.129		
17.	0.483	42.	0.121		
18.	0.469	43.	0.115		
19.	0.442	44.	0.110		
20.	0.423	45.	0.106		
21.	0.401	46.	0.102		
22.	0.388	47.	0.099		
23.	0.368	48.	0.096		
24.	0.341	49.	0.094		
25.	0.334	50.	0.094		

Rainfall runoff distribution

(3) Dated 2/3.8.78

Runoff Observations				Rainfall observations	
Time hrs	Discharge rate m^3/sec.	Time hrs.	Discharge rate m^3/sec.	Time interval hrs.	Rainfall intensity mm/hr.
1.	0.185	32.	0.446	0.00 - 0.50	11.25
2.	0.418	33.	0.430	0.50 - 1.25	26.00
3.	0.693	34.	0.420	1.25 - 2.60	21.40
4.	0.841	35.	0.402	2.60 - 3.50	29.50
5.	1.053	36.	0.387	3.50 - 8.50	3.10
6.	1.144	37.	0.369		
7.	1.178	38.	0.341		
8.	1.169	39.	0.323		
9.	1.146	40.	0.314		
10.	1.159	41.	0.305		
11.	1.139	42.	0.282		
12.	1.095	43.	0.255		
13.	1.059	44.	0.228		
14.	0.997	45.	0.219		
15.	0.954	46.	0.199		
16.	0.905	47.	0.175		
17.	0.853	48.	0.160		
18.	0.801	49.	0.155		
19.	0.784	50.	0.148		
20.	0.763	51.	0.141		
21.	0.739	52.	0.138		
22.	0.724	53.	0.132		
23.	0.710	54.	0.123		
24.	0.669	55.	0.114		
25.	0.666	56.	0.104		
26.	0.629	57.	0.094		
27.	0.606	58.	0.087		
28.	0.602	59.	0.069		
29.	0.581	60.	0.062		
30.	0.542	61.	0.061		
31.	0.498	62.	0.054		

Rainfall runoff distribution

(4) Dated 8/9.8.78

Runoff observations				Rainfall observations	
Time hrs.	Discharge rate m^3/sec.	Time hrs.	Discharge rate m^3/sec.	Time interval hrs.	Rainfall intensity mm/hr.
1.	0.213	31.	0.372	0.0 - 0.25	7.30
2.	0.483	32.	0.322	0.25 - 0.75	18.00
3.	0.603	33.	0.301	0.75 - 2.25	21.00
4.	0.738	34.	0.292	2.25 - 4.60	13.20
5.	0.869	35.	0.281		
6.	0.876	36.	0.271		
7.	0.881	37.	0.265		
8.	0.884	38.	0.262		
9.	0.888	39.	0.241		
10.	0.918	40.	0.221		
11.	0.857	41.	0.211		
12.	0.815	42.	0.190		
13.	0.768	43.	0.179		
14.	0.728	44.	0.161		
15.	0.706	45.	0.152		
16.	0.664	46.	0.144		
17.	0.642	47.	0.136		
18.	0.613	48.	0.120		
19.	0.605	49.	0.105		
20.	0.585	50.	0.100		
21.	0.559	51.	0.095		
22.	0.542	52.	0.091		
23.	0.526	53.	0.088		
24.	0.501	54.	0.083		
25.	0.481	55.	0.080		
26.	0.474	56.	0.073		
27.	0.459	57.	0.067		
28.	0.457	58.	0.060		
29.	0.423	59.	0.049		
30.	0.403	60.	0.045		

Rainfall runoff distribution

(5) **Dated 2.8.79**

Runoff observations				Rainfall observations	
Time hrs.	Discharge rate m^3/sec.	Time hrs.	Discharge rate m^3/sec.	Time interval hrs.	Rainfall intensity mm/hr
1.	0.090	31.	0.220	0.00 - 0.25	26.80
2.	0.199	32.	0.190	0.25 - 0.75	31.20
3.	0.240	33.	0.178	0.75 - 1.00	48.10
4.	0.290	34.	0.172	1.00 - 1.22	10.40
5.	0.340	35.	0.166		
6.	0.357	36.	0.160		
7.	0.373	37.	0.156		
8.	0.396	38.	0.155		
9.	0.407	39.	0.147		
10.	0.417	40.	0.131		
11.	0.413	41.	0.124		
12.	0.396	42.	0.112		
13.	0.379	43.	0.099		
14.	0.367	44.	0.083		
15.	0.362	45.	0.074		
16.	0.350	46.	0.071		
17.	0.344	47.	0.066		
18.	0.337	48.	0.063		
19.	0.334	49.	0.059		
20.	0.326	50.	0.057		
21.	0.317	51.	0.054		
22.	0.309	52.	0.051		
23.	0.302	53.	0.049		
24.	0.289	54.	0.047		
25.	0.277	55.	0.043		
26.	0.273	56.	0.038		
27.	0.269	57.	0.035		
28.	0.259	58.	0.031		
29.	0.250	59.	0.029		
30.	0.238	60.	0.026		

Appendix - II -(E)

Kachwa Watershed :

Computation of Excitation Function at Hourly Time Step

(1) Storm event, dated : 27.7.1977

Time (hr)	1	0-1	1-2	2-3	3-4	4-5
Rainfall depth (mm)	2	19.0	20.8	8.0	8.0	2.1
Rainfall excess depth (mm)	3	2.2	4.0	-	-	-
ϕ-Index(mm/hr)	4		16.8			

(2) Storm event, dated : 4.8.1977

	0-1	1-2	2-3	3-4
1	0-1	1-2	2-3	3-4
2	41.0	3.8	3.6	-
3	7.7	-	-	-
4		33.3		

(3) Storm event, dated : 2/3.8.1978

1	0-1	1-2	2-3	3-4	4-5	5-6	6-7	7-8	8-9
2	18.6	22.4	24.7	16.1	3.2	3.2	3.2	3.3	1.4
3	0.46	4.26	6.56	-	-	-	-	-	-
4				18.14					

(4) Storm event, dated : 8/9.8. 1978

1	0-1	1-2	2-3	3-4	4-5
2	15.7	21.3	15.5	13.3	7.6
3	1.06	6.66	0.86	-	-
4				14.64	

(5) Storm event, dated : 2.8.1979

1	0-1	1-2	2-3
2	34.33	2.27	-
3	4.2	-	-
4		30.13	

APPENDIX - III-A

Sensitivity of (i) Channel Bed Width, (ii) Channel Side Slope and (iii) Channel Depths (Bridge No. 317, Storm dated 25.10.1961)

(i) Effect of Channel Bed Width :

Sl. No.	Channel Bed Width (m)	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3)x10^2
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.0	-	12.873	90.0	529.935
2.	2.0	50.0	12.875	90.0	529.794
3.	4.0	100.0	12.864	90.0	529.608
4.	6.0	150.0	12.848	90.0	529.440
5.	8.0	200.0	12.831	90.0	529.290

(ii) Effect of Channel Side Slope :

(1)	Channel side slope (2)	(3)	(4)	(5)	(6)
1.	1.0	-	12.886	90.0	529.536
2.	2.0	-	12.871	90.0	529.593
3.	2.5	-	12.864	90.0	529.608
4.	3.0	-	12.856	90.0	529.620
5.	3.5	-	12.8494	90.0	529.629

(iii) Effect of Channel Depths:

(1)	Channel depths (m) (2)	(3)	(4)	(5)	(6)
1.	0.1, 0.2 and 0.3	-	12.881	90.0	529.461
2.	0.25, 0.5 and 0.7	-	12.864	90.0	529.608
3.	1.0, 1.5 and 2.0	-	12.862	90.0	529.830
4.	2.0, 3.5 and 4.0	-	12.868	90.0	529.899

APPENDIX - III-B

Sensitivity of (i) Channel Bed Width, (ii) Channel Side Slope and (iii) Channel Depths (Bridge no. 719, Storm dated 24/25.7.1964

(i) Effect of Channel Bed Width :

Sl. No.	Channel Bed Width (m)	Percentage (%)	Peak ($m^3/s.$)	Time to peak (min)	Volume $(m^3)x10^3$
(1)	(2)	(3)	(4)	(5)	(6)
1.	7.5	50.0	24.41	130.0	239.01
2.	15.0	100.0	24.05	130.0	237.24
3.	22.5	150.0	23.58	130.0	235.78

(ii) Effect of Channel Side Slope :

(1)	Channel Side Slope (2)	(3)	(4)	(5)	(6)
1.	1.0	50.0	24.13	130.0	237.19
2.	2.0	100.0	24.05	130.0	237.24
3.	3.0	150.0	23.54	140.0	237.26

(iii) Effect of Channel Depths :

(1)	Channel Depths (m) (2)	(3)	(4)	(5)	(6)
1.	0.5, 0.9 and 0.15	-	24.107	130.0	237.03
2.	0.5, 0.9 and 0.15	-	24.05	130.0	237.24
3.	1.5, 2.9 and 3.15	-	24.14	130.0	239.05

APPENDIX - III-C

Sensitivity of (i) Channel Bed Width, (ii) Channel Side Slope and (iii) Channel Depths (Kachwa Watershed, Storm dated 4.8.1977)

(i) Effect of Channel Bed Width :

Sl. No.	Channel Bed Width (m)	Percentage (%)	Peak (m^3/s)	Time to peak (hr.)	Volume $(m^3)\times10^3$
(1)	(2)	(3)	(4)	(5)	(6)
1	0.0	-	0.994	9.0	77.159
2	1.0	50.0	0.995	9.0	77.043
3	2.0	100.0	0.993	9.0	76.802
4	3.0	150.0	0.990	9.0	76.554

(ii) Effect of Channel Side Slope :

(1)	Channel side slope (2)	(3)	(4)	(5)	(6)
1	1.0	50.0	1.000	8.0	76.871
2	1.5	75.0	0.997	8.0	76.838
3	2.0	100.0	0.993	9.0	76.802
4	2.5	125.0	0.990	9.0	76.759
5	3.0	150.0	0.987	9.0	76.716

(iii) Effect of Channel Depths

(1)	Channel depths (m) (2)	(3)	(4)	(5)	(6)
1	0.25, 0.4 and 0.5	-	0.993	9.0	76.802
2	0.45, 0.6 and 0.8	-	0.992	9.0	76.910
3	0.8, 1.00 and 1.60	-	0.993	9.0	77.000
4	1.25, 1.50 and 2.05	-	0.994	9.0	77.097

APPENDIX - III-D

Sensitivity of (i) Channel Slope and (ii) Channel Roughness (Bridge No. 317, Storm dated 25.10.1961).

(i) Effect of Channel Slope :

Sl. No.	Channel Slope	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3)x10^2
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.0525	50.0	12.799	90.0	529.377
2.	0.084	80.0	12.845	90.0	529.539
3.	0.105	100.0	12.864	90.0	529.608
4.	0.126	120.0	12.877	90.0	529.659
5.	0.1575	150.0	12.893	90.0	529.716

(ii) Effect of Channel Roughness

	Channel Roughness				
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.020	50.0	12.945	90.0	529.923
2.	0.032	80.0	12.895	90.0	529.728
3.	0.040	100.0	12.864	90.0	529.608
4.	0.048	120.0	12.832	90.0	529.494
5.	0.060	150.0	12.785	90.0	529.329

APPENDIX - III-E

Sensitivity of (i) Channel Slope and (ii) Channel Roughness (Bridge No. 719, Storm dated 24/25.7.1964).

(i) Effect of Channel Slope :

Sl. No.	Channel Slope	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3)x10^3
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.0105	50.0	23.17	140.0	235.14
2.	0.0168	80.0	23.82	130.0	236.63
3.	0.0210	100.0	24.05	130.0	237.24
4.	0.0252	120.0	24.20	130.0	237.69
5.	0.00315	150.0	24.34	130.0	238.21

(ii) Effect of Channel Roughness:

(1)	Channel Roughness (2)	(3)	(4)	(5)	(6)
1.	0.0225	50.0	24.93	120.0	239.97
2.	0.036	80.0	24.36	130.0	238.29
3.	0.045	100.0	24.05	130.0	237.24
4.	0.054	120.0	23.64	130.0	236.22
5.	0.0675	150.0	23.04	140.0	234.72

APPENDIX - III-F

Sensitivity of (i) Channel Slope and (ii) Channel Roughness (Kachwa Watershed, Storm dated 4.8.1977)

(i) Effect of Channel Slope :

Sl. No.	Channel Slope	Percentage (%)	Peak (m^3/s)	Time to peak (hr.)	Volume (m^3)x10^3
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.000235	50.0	0.967	10.0	76.136
2.	0.0003525	75.0	0.984	9.0	76.546
3.	0.00047	100.0	0.993	9.0	76.802
4.	0.0005875	125.0	1.000	8.0	76.982
5.	0.00076	150.0	1.006	8.0	77.122

(ii) Effect of Channel Roughness :

(1)	Channel Roughness (2)	(3)	(4)	(5)	(6)
1.	0.026	50.0	1.025	7.0	77.778
2.	0.039	75.0	1.010	8.0	77.245
3.	0.052	100.0	0.993	9.0	76.802
4.	0.065	125.0	0.976	9.0	76.392
5.	0.078	150.0	0.960	10.0	75.996

APPENDIX - III-G

Sensitivity of (i) Overland Slope and (ii) Overland Roughness
(Bridge No. 317, Storm dated 25.10.1961)

(i) Effect of Overland Slope :

Sl. No.	Overland Slope	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3)x10^2
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.0405	50.0	11.354	90.0	524.187
2.	0.0648	80.0	12.508	90.0	528.153
3.	0.0810	100.0	12.864	90.0	529.608
4.	0.0972	120.0	13.075	90.0	529.637
5.	0.1215	150.0	13.255	90.0	531.744

(ii) Effect of Overland Roughness :

(1)	Overland Roughness (2)	(3)	(4)	(5)	(6)
1.	0.096	50.0	13.550	80.0	534.906
2.	0.144	75.0	13.346	90.0	532.461
3.	0.192	100.0	12.864	90.0	529.608
4.	0.240	125.0	12.032	90.0	526.437
5.	0.288	150.0	10.977	90.0	522.969

APPENDIX - III-H

Sensitivity of (i) Overland Slope and (ii) Overland Roughness (Bridge No. 719, Storm dated 24/25.7.1964)

(i) Effect of Overland Slope :

Sl. No.	Overland Slope	Percentage (%)	Peak (m^3/s)	Time to peak (min)	Volume (m^3)x10^3
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.0355 0.0325	50.0	19.70	140.0	228.15
2.	0.0568 0.052	80.0	22.64	130.0	234.72
3.	0.071 0.065	100.0	24.05	130.0	237.24
4.	0.0852 0.078	120.0	25.05	130.0	239.05
5.	0.1065 0.0975	150.0	26.332	120.0	241.02

(ii) Effect of Overland Roughness :

(1)	Overland Roughness (2)	(3)	(4)	(5)	(6)
1.	0.0665	50.0	30.925	100.0	247.45
2.	0.1064	80.0	26.97	120.0	241.94
3.	0.138	100.0	24.05	130.0	237.24
4.	0.159	120.0	22.18	130.0	233.886
5.	0.200	150.0	19.37	140.0	227.34

APPENDIX - III-I

Sensitivity of (i) Overland Slope and (ii) Overland Roughness
(Kachwa Watershed, Storm dated 4.8.1977)

(i) Effect of Overland Slope :

Sl. No.	Overland Slope	Percentage (%)	Peak (m^3/s)	Time to peak (hr.)	Volume (m^3)x10^3
(1)	(2)	(3)	(4)	(5)	(6)
1.	0.0006	50.0	0.729	11.0	74.656
2.	0.0009	75.0	0.877	9.0	76.320
3.	0.0012	100.0	0.993	9.0	76.802
4.	0.0015	125.0	1.091	8.0	76.849
5.	0.0018	150.0	1.169	8.0	76.705

(ii) Effect of Overland Roughness :

(1)	Overland Roughness (2)	(3)	(4)	(5)	(6)
1.	0.05950	50.0	1.616	6.0	73.854
2.	0.08925	75.0	1.2475	7.0	76.446
3.	0.11900	100.0	0.993	9.0	76.802
4.	0.14875	125.0	0.8175	10.0	75.823
5.	0.17850	150.0	0.688	12.0	73.915

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AN1= 0.0350 .01= 0.0792 M1= 49 N1= 111

DX1= 15.0000 DT1= 150.0000 B= 3.0000 B2= 2.5000 N= 49 N= 63 NR= 49 AC=

.2500 Y2= 0.5500 X3= 0.7500 AN= 0.0520 S0= 0.0920 AN= 1.6667

DX= 4.0000 DT= 150.0000

248.0000

OVER LAND FLOW ROUTING

TIME(MINS)	DISCHARGE	AREA	VEL	QR	ADC	FR2	AK2
2.5000000	0.0000895	0.0012930	0.0692277	0.0000086	17645.7860000	0.6146760	46703.4020000
5.0000000	0.0002842	0.0025859	0.1098892	0.0000086	8823.2799000	0.6899470	18535.2550000
7.5000000	0.0005575	0.0038746	0.1438919	0.0000086	5888.5599000	0.7380512	10810.2580000
10.0000000	0.0008899	0.0051294	0.1734851	0.0000086	4448.0601000	0.7733806	7436.7659000
12.5000000	0.0012430	0.0062684	0.1982985	0.0000086	3639.8639000	0.7996645	5692.0602000
15.0000000	0.0015621	0.0071893	0.2172744	0.0000086	3173.5971000	0.8181449	4741.2328000
17.5000000	0.0019053	0.0078414	0.2302214	0.0000086	2909.6880000	0.8300698	4222.9625000
20.0000000	0.0019640	0.0082482	0.2381169	0.0000086	2766.1746000	0.8370970	3947.5530000
22.5000000	0.0020637	0.0084968	0.2428769	0.0000088	2685.2566000	0.8412495	3794.3393000
25.0000000	0.0021180	0.0086302	0.2454133	0.0000088	2643.7344000	0.8434373	3716.3121000
27.5000000	0.0021462	0.0086990	0.2467148	0.0000088	2622.6432000	0.8445534	3677.2073000
30.0000000	0.0021607	0.0087342	0.2473799	0.0000088	2612.2720000	0.8451220	3657.4596000
32.5000000	0.0017040	0.0075744	0.2249646	0.0000000	3012.2657000	0.8252902	4422.6246000
35.0000000	0.0013375	0.0065501	0.2041960	0.0000000	3483.3219000	0.8055459	5368.0187000
37.5000000	0.0010490	0.0056617	0.1852873	0.0000000	4029.9139000	0.7862112	6519.5452000
40.0000000	0.0008248	0.0049010	0.1682956	0.0000000	4655.3732000	0.7675309	7902.4683000
42.5000000	0.0006518	0.0042552	0.1531664	0.0000000	5361.8944000	0.7496670	9540.7262000
45.0000000	0.0005185	0.0037095	0.1397743	0.0000000	6150.6655000	0.7327135	11456.5470000
47.5000000	0.0004158	0.0032492	0.1279575	0.0000000	7022.0524000	0.7167104	13670.2570000
50.0000000	0.0003362	0.0028607	0.1175421	0.0000000	7975.7682000	0.7016580	16200.2450000
52.5000000	0.0002744	0.0025320	0.1083574	0.0000000	9011.0380000	0.6875299	19063.0190000
55.0000000	0.0002259	0.0022530	0.1002448	0.0000000	10126.7300000	0.6742832	22273.3020000
57.5000000	0.0001875	0.0020153	0.0930621	0.0000000	11321.4620000	0.6618658	25844.1760000
60.0000000	0.0001570	0.0018117	0.0866841	0.0000000	12593.6770000	0.6502216	29787.2030000
62.5000000	0.0001326	0.0016365	0.0810022	0.0000000	13941.7090000	0.6392941	34112.5830000
65.0000000	0.0001127	0.0014850	0.0759232	0.0000000	15363.8210000	0.6290281	38829.2590000
67.5000000	0.0000966	0.0013534	0.0713673	0.0000000	16859.2300000	0.6193714	43945.0250000
70.0000000	0.0000833	0.0012384	0.0672664	0.0000000	18423.1320000	0.6102752	49466.5940000
72.5000000	0.0000723	0.0011376	0.0635625	0.0000000	20056.7050000	0.6016948	55399.6520000
75.0000000	0.0000631	0.0010487	0.0602060	0.0000000	21757.1030000	0.5935891	617[illegible].8930000
77.5000000	0.0000554	0.0009700	0.0571547	0.0000000	23522.4530000	0.5859208	695[illegible].0150000
80.0000000	0.0000489	0.0009009	0.0543724	0.0000000	25350.8400000	0.5786560	7570[illegible].6790000
82.5000000	0.0000434	0.0008376	0.0518281	0.0000000	27240.2870000	0.5717644	83325.4010000
85.0000000	0.0000387	0.0007817	0.0494952	0.0000000	29188.7220000	0.5652187	91365.4600000
87.5000000	0.0000346	0.0007314	0.0473507	0.0000000	31193.9220000	0.5589941	99826.7170000
90.0000000	0.0000311	0.0006861	0.0453748	0.0000000	33253.4950000	0.5530689	108712.3300000
92.5000000	0.0000281	0.0006452	0.0435504	0.0000000	35364.7720000	0.5474236	118011.3500000
95.0000000	0.0000255	0.0006080	0.0418627	0.0000000	37524.7590000	0.5420413	127718.3200000
97.5000000	0.0000231	0.0005743	0.0402990	0.0000000	39729.9690000	0.5369067	137822.6300000
100.0000000	0.0000211	0.0005435	0.0388481	0.0000000	41976.3250000	0.5320074	148309.5100000
102.5000000	0.0000193	0.0005155	0.0375006	0.0000000	44258.9340000	0.5273330	159158.9600000
105.0000000	0.0000178	0.0004899	0.0362485	0.0000000	46571.8190000	0.5228749	170344.3000000
107.5000000	0.0000164	0.0004665	0.0350850	0.0000000	48907.5150000	0.5186274	181829.9800000
110.0000000	0.0000151	0.0004451	0.0340045	0.0000000	51257.1140000	0.5145874	[illegible]569.0400000
112.5000000	0.0000141	0.0004256	0.0330026	0.0000000	53608.7650000	0.5107544	205499.8400000
115.0000000	0.0000131	0.0004078	0.0320762	0.0000000	55947.9600000	0.5071316	217541.8800000
117.5000000	0.0000122	0.0003916	0.0312232	0.0000000	58256.3310000	0.5037257	229591.0200000
120.0000000	0.0000114	0.0003761	0.0303915	0.0000000	60663.9880000	0.5003371	242329.0400000

CANNEL FLOW ROUTING

EC= 1.4235[illegible]7000 ALP= 2.950607

TIME(MINS)	DISCH	AREA	VEL	QBR	ADC1	FR1	AK
2.5000000	0.0159246	0.0268534	0.5930175	0.0001790	14707.3490000	2.0159783	3618.7842000
5.0000000	0.1252199	0.1120456	1.1175174	0.0005683	3604.6382000	1.9011762	997.2796000
7.5000000	0.4619402	0.2767758	1.6690054	0.0011151	1518.0644000	1.8759992	431.3451100
10.0000000	1.1543206	0.5219824	2.2114280	0.0017798	847.5606700	1.8977277	235.3439200
12.5000000	2.1952262	0.8147633	2.6943116	0.0024860	572.6705900	1.9393162	152.2676400
15.0000000	3.4039396	1.1040736	3.0830731	0.0031241	442.4766600	1.9821145	112.6245100
17.5000000	4.5491067	1.3495130	3.3701837	0.0036105	374.8917500	2.0172172	92.1298890
20.0000000	5.4713010	1.5338237	3.5670991	0.0039281	337.9573900	2.0425149	81.0086630
22.5000000	6.1380687	1.6611180	3.6955043	0.0041273	317.0691900	2.0594268	74.7586240
25.0000000	6.5773609	1.7424757	3.7747217	0.0042359	305.2555400	2.0699975	71.2399990
27.5000000	6.8450302	1.7912960	3.8212726	0.0042923	298.6602500	2.0762527	69.2814580
30.0000000	6.9994087	1.8191360	3.8475497	0.0043213	295.0444500	2.0797968	68.2096200
32.5000000	6.4752668	1.7236947	3.7566206	0.0034079	307.8878400	2.0675736	72.0228950
35.0000000	5.6297579	1.5644696	3.5985296	0.0026750	332.6319200	2.0466271	79.4120610
37.5000000	4.7191113	1.3844625	3.4086233	0.0020981	367.1600900	2.0220874	89.7957170
40.0000000	3.8802417	1.2089220	3.2096708	0.0016496	410.3497700	1.9972939	102.8656200
42.5000000	3.1644588	1.0496650	3.0147322	0.0013035	461.6033000	1.9741442	118.4434800
45.0000000	2.5769958	0.9104775	2.8303784	0.0010370	520.6496400	1.9535461	136.4263400
47.5000000	2.1038127	0.7911079	2.6593276	0.0008315	587.4276700	1.9358089	156.7578900
50.0000000	1.7255801	0.6896214	2.5022136	0.0006725	662.0022800	1.9209141	179.4087400
52.5000000	1.4236794	0.6036039	2.3586319	0.0005487	744.5023700	1.9086795	204.3619600
55.0000000	1.1822134	0.5306859	2.2277084	0.0004517	835.0790900	1.8988522	231.6036000
57.5000000	0.9883077	0.4687494	2.1083926	0.0003751	933.8804500	1.8911572	261.1175300
60.0000000	0.8317938	0.4159792	1.9996043	0.0003141	1041.0381000	1.8853241	292.8832800
62.5000000	0.7047320	0.3709522	1.9003040	0.0002651	1156.6619000	1.8811000	326.8757200
65.0000000	0.6009511	0.3321044	1.8095248	0.0002255	1280.8389000	1.8782553	363.0657300
67.5000000	0.5156573	0.2985919	1.7263854	0.0001932	1413.6340000	1.8765859	401.4210200
70.0000000	0.4451202	0.2697548	1.6500917	0.0001666	1555.0926000	1.8759128	441.9070700
72.5000000	0.3864291	0.2445850	1.5799341	0.0001446	1705.2414000	1.8760810	484.4876100
75.0000000	0.3372984	0.2225979	1.5152812	0.0001263	1864.0916000	1.8769567	529.1256000
77.5000000	0.2959337	0.2033197	1.4555734	0.0001109	2031.6394000	1.8784251	575.7830300