MODELING OF FLOOD FLOWS IN NATURAL WATERSHEDS

A THESIS

submitted in fulfilment of the requirements for the award of the degree

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in

HYDROLOGY

by

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MAY, 1993

TO
MY WIFE
MINA

CANDIDATE'S DECLARATION

I hereby certify that the work which is being presented in the thesis entitled **"MODELING OF FLOOD FLOWS IN NATURAL WATERSHEDS"** in fulfillment of the requirements for the award of the Degree of **Doctor of Philosophy**, submitted in the Department of Hydrology of the University of Roorkee, Roorkee-247667, India is an authentic record of my own work carried out during a period from July 1988 to May 1993 under the supervision of **Dr. B.S. Mathur** and **Dr. Deepak Kashyap.**

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

(Mohammad Reza Najafi Shahri)

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

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The candidate has passed the Viva-Voce examination held on at . The thesis is recommended for award of Ph.D. Degree.

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SYNOPSIS

In this research work an attempt has been made to develop a model having capabilities of accounting for the spatial and temporal variations of rainfall as well as of physiographic characteristics which do prevail in most of the tropical countries.

A literature survey conducted during the course of this study (Chapter-II) revealed that the Kinematic Wave (KW) theory and the Dynamic Wave (DW) theory based hydrologic models currently being used for solving the St. Venants equations have the capabilities of taking into account the distributed nature of the input function as well as of the physiographic characteristics. Thus, these mathematical theories have been applied to develop the following two physiographic models (Chapter-III).

(i) Physiographic Model-I; consisting of tributary subwatersheds and a single consolidated main channel subwatershed

(ii) Physiographic Model-II consisting of tributary subwatersheds and distributed main channel subwatersheds.

The details of these models have been discussed (Section 3.5). The later model is an extension to the first model. The main tributaries are identified and the watershed under consideration is split up into its tributary subwatersheds which remain common to both the models. The remaining area is considered as a single main channel subwatershed in the first case whereas it is further split up into smaller units in the second physiographic model given above. Drainage characteristics of the areas happen to be the criteria adopted for the demarcation of the subwatersheds. In order to obtain the conceptual configuration, the surface runoffs coming from each of these subwatersheds (i.e. tributary subwatersheds and main channel subwatersheds) are folded onto the main

channel (Section 3.5). The final physiographic pattern so arrived at will remain unique for the watershed under consideration. The surface runoffs from the overlapping overland planes are superimposed to compute the lateral flows coming to the main channel. Flows are routed through the main channel to compute the outflow hydrographs at the outlet. For the proposed configuration each of the subwatersheds becomes the elementary unit from which the runoff responses are to be computed. Any changes in its landuse can be appropriately taken care of by suitably modifying the values of the 'physiographic parameters' and thus affecting the runoff process.

For the application of the proposed physiographic models the KW theory is applied for routing the flows on the overland planes. The Lax-Wendroff explicit scheme has been used for the mathematical formulation of the KW equations. The criteria adopted for the applicability of the KW theory is $Fr^2 K > 5$, where Fr is Froude number and K is KW number. The computed overland runoffs form the lateral flows to the channel.

For routing the flows through the channel, the DW theory has been preferred. The mathematical formulation of St. Venant equations have been sought through the four point implicit scheme.

The application of the proposed models have been discussed in depth and details for the watershed of Kolar river. However, in order to draw logical conclusions about the applicability of the proposed models, the applications have been repeated onto the watershed of the Railway Bridge No. 719 and the Kassilian watershed.

The 'Open Book Type' physiographic model has also been applied for comparing its performance with proposed models. The comparison of the computed hydrographs with the observed ones as well as the model efficiencies, do suggest that the physiographic model-II consisting of tributary subwatershed and the distributed main channel subwatersheds gave better results. At the same time the physiographic model-I is comparatively simpler and easy to apply. The performance of open book type physiographic model in general was not found to be satisfactory. The proposed models are advantageous in a sense that the distributed responses of the surface runoff coming from different parts of the channel can separately be estimated. The model can be further strengthened and improved in future by linking it by infiltration based ground water models.

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LIST OF SYMBOLS AND ABBREVIATIONS

Symbol	Description
A	Area of the watershed; Channel cross-sectional area
$A_o, A_1, \ldots, A_n$	Constant function
$a_o, a_1, \ldots, a_n$	Constant function
A_o	Cross section of flow area of the plane
B	Channel width
C	Wave celerity
$\bar{C}$	Average representative wave celerity
D	Hydraulic depth
d	Differential operator
f( )	function of
Fr	Froude number
g	Acceleration due to gravity
h	Depth of flow
$\bar{h}$	Average overland flow depth
h_o	Overland flow depth
i	Variable grid number along x-direction
j	Variable grid number along the; Variable number
K	Kinematic wave number
k	Iteration number
L	Length of the plane; Length of Channel
l	Total length of grid line segment (horizontal & vertical)
L_o	Length of the overland flow
m	Kinematic wave parameter of overland flow; Metre
min	Minute
n	Mannings roughness coefficient
Q	Discharge
q, q_o	Lateral flow
$\bar{q}$	Average flow
q_i	Rainfall intensity
R	Hydraulic radius
R_j	Channel reach
R_o	Hydraulic radius

S_o	Channel bed slope; Overland slope
$S_1, S_2, \ldots, S_n$	Slope of main channel in its different segments
S_f	Friction slope
T	Width of the free surface of a channel
t	Time instant
u	X-component of mean velocity
V	x-component of velocity of the lateral inflow
W	Width of the plane
x	Distance measured in downstream direction
y(t)	Transfer function
ψ	Function
ξ	Function
α	Kinematic wave parameter
Δh	The contours interval
Δx	Space step
Δt	Time step
∂	Partial derivative operator
θ	Angle, time weighting coefficient
ϕ	Infiltration index
∞	Infinity
$\rightarrow$	Tends to

ABBREVIATIONS

DRH	Direct runoff hydrograph
DW	Dynamic wave
FDM	Finite Difference Method
FEM	Finite Element Method
KW	Kinematic Wave
MSL	Mean Sea Level
MOC	Method of Characteristics
RE	Rainfall Excess
CA	Watershed Area

CHAPTER I

INTRODUCTION: OBJECTIVES AND SCOPE

Water is an important natural resource vitally needed for the existence of life on this planet. In the past, it has played an important role in the development of different civilizations and in future too it will continue to play the same role. However, in the context of the present day situation, due to population explosion in most parts of the tropics, water now is a scarce resource and needs careful planning for its conservation and use. Apart from its usefulness, water has posed many problems to the society by way of floods, droughts, erosion, sedimentation, quality aspects, etc.

With the developments in the hydrological sciences, the ever increasing problems posed by the erratic flows of water have drawn the attention of the hydrologists to develop suitable methodologies for getting the correct estimates of its flows. From the point of view of computations, the complexities of hydrological processes become manifold due to the combinations of varieties of characteristics associated with the drainage basins as well as with the meteorological phenomena. In most cases, the solution to the complex problems of a watershed are thus obtained through models which are the simplified representations of complicated natural systems.

Transformation of precipitation into the runoff is one of the complex problems which has yet to find a suitable solution. Though numerous watershed models have been developed by the researchers in the past yet their suitability to solve the problems of unevenness of rainfall distribution with respect to space and time remains in doubt. In most cases, the inaccurate runoff estimates have resulted in uneconomical designs resulting into wasteful expenditures and wrong policy decisions at different stages of planning and management of a water resources project (Balek 1992)

Keeping in view the above aspects the following objectives were set for this work.

1.1 OBJECTIVES

The main objectives defined for the present dissertation can be outlined as under.

(i) To develop suitable distributed parameter physiographic model(s) to account for the uneven spatial distributions of rainfall and to compute the runoff.

(ii) To test the suitability of the proposed model(s) by applying the same onto the natural watersheds of different sizes.

(iii) To verify the applicability of the proposed model onto large watersheds where data may be scanty and some information about the surface runoff depths may be available (i.e. in the absence of recording raingauges correct estimation of rainfall excess distribution can not be ascertained).

(iv) To compare the watershed response generated from a unit pulse of rainfall excess by the proposed model with those obtained by the conventional approach of the Unit Hydrograph Theory.

(v) To draw suitable conclusions from the experience of application of the proposed model(s) onto the different watersheds.

In order to achieve the above mentioned objectives, the following approach was adopted for this work.

1.2 THE APPROACH

The mechanics of the runoff process has been studied through the overland phase and the channel phase of the surface runoff. The hydrodynamic approaches are considered to be the appropriate methodologies for modeling of these two phases of the runoff process. To account for the temporal and spatial variations in the inputs as well as of the model parameters, the watershed under study is first divided into a number of subwatersheds keeping in view the drainage patterns and characteristics. For this purpose, the main tributaries are identified and their drainage areas are delineated to form the tributary subwatersheds. The remaining area in the vicinity of the main channel has been taken care of by the main channel subwatershed(s). The Kinematic Wave (KW) Theory is applied for the overland runoff computations from these "subwatersheds". The overland flows are "folded" onto the main channel and the same are considered as the lateral flows coming to the channel. The Dynamic Wave (DW) Theory is used to route the flows through the main channel to compute the watershed responses at the outlet.

The chapterwise planning of the present dissertation is as under.

1.3 THE PLANNING OF THE DISSERTATION REPORT

The next Chapter is titled as "**Review of Literature**". Here terminologies and concepts pertaining to the analytical approaches used in the past have been discussed. A description of the approaches/models developed by different researchers is presented in chronological order.

The **CHAPTER-III** is devoted towards "**The Model Development**". Mathematical formulations based on the KW theory and the Dynamic Wave(DW) concepts are explained and their solution techniques have been briefly discussed. The proposed physiographic model configurations are also detailed in this chapter.

IN CHAPTER-IV a brief narration is presented for the four natural watersheds on which the proposed model(s) were applied. The availability of data on these watersheds has also been discussed.

The **CHAPTER-V** deals with **"The Model Application"**. The proposed physiographic models have been applied onto the three watersheds (viz. Kolar, Railway Bridge No. 719 and Kassilian). The capabilities of the proposed model by way of computing distributed responses of the runoff in different reaches of the main channel have also been shown for the Kolar river watershed. Comparison between the responses obtained from the proposed model as well as from the Unit Hydrograph theory have been compared for a rainfall excess pulse of unit depth.

Lastly in **CHAPTER-VI**, based on the experience of the computations carried out, suitable conclusions have been drawn.

CHAPTER II

REVIEW OF LITERATURE

2.1 INTRODUCTION

In this chapter, some of the concepts, terminologies, basic equations of kinematic wave and dynamic wave theories with their solution techniques will be discussed. Some of the currently used watershed models alongwith their important features and applications have also been described.

2.2 THE HYDROLOGIC SYSTEMS APPROACH

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)' (Clark, 1973).' In hydrologic process, generally the input function is the hyetograph and the transfer (or system) function accounts for the 'transformation process' due to watershed action on it. The response (or output) function is the runoff hydrograph which is observed at the outlet. Through the following three sets of possible system behaviours, the nature of the hydrologic system can be defined completely (i.e. by taking one property from each of them (Dooge, 1973)).

a) linear or nonlinear

b) lumped or distributed

c) Time-invariant or time variant

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

2.2.1 Linear and Nonlinear Systems

A hydrologic system is said to be linear, if the principles of superposition and proportionality (or homogeneity) are satisfied. The principle of proportionality suggests that the system input x(t) and output y(t) should have the same scale ratio; i.e.

$$y(t) = f(ax)_t = af(x)_t \quad \ldots (2.1)$$

where as the principle of superposition states that if $y_1(t)$ and $y_2(t)$ are the system outputs corresponding to system inputs $x_1(t)$ and $x_2(t)$ respectively, then the output due to $[x_1(t) + x_2(t)]$ is $[y_1(t) + y_2(t)]$ (Dooge 1973).

On the other hand, a nonlinear system is represented by a nonlinear function. The extent of the nonlinearity depends upon the system itself. A hydrologic system may be defined by a general differential equation of the following type;

$$f(x) = a_n \frac{dx^n}{dt^n} + a_{n-1} \frac{dx^{n-1}}{dt^{n-1}} + \ldots + a_1 \frac{dx}{dt} + a_o x \quad \ldots \quad (2.2)$$

The system would be nonlinear if any of the coefficients $a_o, a_1, a_2, \ldots, a_n$ etc. happen to be the function of x, or the function x carries an exponent other than unity.

2.2.2 The Lumped and Distributed Systems

In a lumped hydrologic system, the spatial variability of inputs, transfer functions and outputs are not explicitly taken into account. For the lumped systems, average conditions (or values) of the input and of the parameters are applicable. Thus, they are usually represented by an ordinary differential equation or a set of linked ordinary differential equations.

The system is said to be distributed, if it accounts for spatial variations in the inputs, outputs and the system parameters. The distributed systems are often mathematically represented by a set of linked partial differential equations. A theoretical solution for such a system requires complete knowledge of the initial and boundary conditions.

2.2.3 Time-Invariant and Time-Variant Systems

In a time-invariant system, the input-output relationship does not change with time. The form of the output depends on the form of input and not on the time at which the input is applied to the system. Thus, the analysis becomes simpler as the temporal effect on the input is not taken into account. On the other hand, in a time-variant system, the input-output relationship is a dependent function of time. Therefore, the analysis of a variant system is more complex.

In watershed hydrology, the natural phenomena are mostly nonlinear as well as time and space distributed. Depending on the objectives of a study, for simplifications one may consider the parameters of the system to be linear. For these reasons, a distributed time variant hydrologic system is often assumed to be lumped and time invariant. Such approximations are generally made for the convenience of computations. Prior to the advent of computers, handling of the complicated mathematical formulations of nonlinear, distributed and time variant natural hydrologic systems used to be an impossible task. However, now with availability of main frame as well as personal computers, such approximations are not compelling. Depending on the degree of accuracy needed and subject to the availability of data, researchers can now handle the complicated natural systems. Various hydrologic system studies carried out by the researchers are described below.

2.3 APPROACHES IN HYDROLOGIC MODELING

There are many approaches which have been adopted for the hydrologic system representations. These methodologies facilitate the formulation of hydrologic models which try to account for the watershed behaviour through a system, simpler in structure and working.

These models can be broadly classified into the following two categories(Haan et al., 1982)

1. Material models
2. Formal (or mathematical) models

A brief description of the two classifications is given in the following sections.

2.3.1 Material Models

A material model is a physical representation of a complex system that is assumed to be simpler than the prototype and is also assumed to have properties similar to those of the prototype system. These models include iconic or "look alike" models and analog models. An iconic model is a simplified version of the real-world system. It requires the same materials as that of the real system. Some of the models developed at the Colorado State University are the examples of iconic models. A material model that does not involve a change in scale may still be valuable because experiments can be carried out more conveniently or can be repeated at will (Woolhiser and Schulz, 1973).

The analog models are mostly based on the analogy between the movement of water, and other phenomena e.g. flow of electric current or the flow of heat etc.. Electric devices based on this analogy have sometimes been used for the forecast of the transformation of flood waves. However, such models had been extensively used in groundwater flow computations.

2.3.2 Formal Models

The formal models are symbolic expression in logical terms, usually mathematical in nature, for representation of an idealized situation that has the important structural properties of the real system. These models have been preferred for their flexibility in physical representation of a complex system that is assumed to be simpler than the prototype system. Thus, formal models have mostly been developed as mathematical models. "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" (Clark, 1973).

With the above definition, a 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 or has been verified.

The conceptual models, rely on theory 'to interpret' the phenomenon rather than 'to represent' the physical process. These models have been evolved in surface hydrology which simulate the watershed behaviour through conceptual elements. In this category, conceptual entities like linear reservoirs, linear channels and nonlinear reservoirs along with their series and other configurations have been successfully tried by various researchers. Some of the conceptual models which are found to be more useful for application have been proposed by Clark (1945), Nash (1957, 1960), Singh (1962), Dooge (1959), Mathur (1972), Pedersen et al., (1980) etc.. Comprehensive models based on simulation theory have also been developed e.g. Stansford Watershed Model (SWM-IV) (Crawford et al., 1966), Storm Water Management Model Chen et al., (1971), etc.

Comprehensive simulation models, deterministic in nature, suffer from the limitation that the requirement of data is very large. Also, to define many of the boundary conditions, these need many non-point sources of stream flow for the quantity and quality, in urban as well as rural watersheds.

Complex watershed problems may be analyzed by using a suitable mathematical model. The model adopted need be much simpler than the actual system. Such mathematical models rely on mathematical statements to represent the system and are often more useful in hydrologic studies when compared with the physical models. The relationship between watershed response and its parameters can suitably be studied with the help of mathematical models.

A mathematical model is generally developed through a four step process involving (1) an examination of the physical 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).

Another subdivision of the mathematical models comprises of the models based on the dynamic wave theory and the kinematic wave theory. These models are comprehensive in a sense that they involve most of the influencing parameters which take part in constructing the watershed response due to precipitation. Since the present study is devoted to watershed modeling through dynamic wave (DW) theory as well as kinematic wave (KW) approximation, therefore, the literature relating to development of DW models, KW models, and their applications to the hydrologic problem are reviewed in more details in the forthcoming sections.

2.4 UNSTEADY FLOW EQUATIONS

The unsteady flow equations which incorporate inertia and other external forces, are called dynamic wave equations. They are commonly referred to as St. Venant equations, or the shallow water equations.

The St. Venant equations comprising of the continuity and the momentum equations for the gradually varied flow are given below.

Continuity

$$\underset{(i)}{B\frac{\partial h}{\partial t}} + \underset{(ii)}{uB\frac{\partial h}{\partial x}} + \underset{(iii)}{A\frac{\partial u}{\partial x}} + \underset{(iv)}{u\frac{\partial A}{\partial x}\Big|_{z=\text{cont.}}} = \underset{(v)}{q} \quad \ldots \quad (2.3)$$

Momentum

$$\underset{(vi)}{\frac{\partial u}{\partial t}} + \underset{(vii)}{u\frac{\partial u}{\partial x}} + \underset{(viii)}{g\frac{\partial h}{\partial x}} = \underset{(ix)}{g}\,(\underset{(x)}{S_o - S_f}) + \underset{(xi)}{\frac{q(v-u)}{h}} \quad \ldots \quad (2.4)$$

where

- h = depth of flow (m)
- u = flow velocity (m/sec)
- S_o = channel bed slope (dimensionless)
- g = acceleration due to gravity (m/sec^2)
- S_f = friction slope (dimensionless)
- x = distance along the direction of flow (m)
- A = water flow area (i.e. cross sectional area of water) (m^2)
- t = time (sec)
- B = channel width (m)
- v = lateral flow velocity in x-direction (i.e. direction of flow) (m/sec)
- q (or q_o) = net lateral inflow per unit width per unit length of the channel

In a hydraulic (or hydrologic) system, the term 'q' appearing in equations 2.3 and 2.4 is generally referred as lateral flow and constitutes the causative flows. A direct incorporation of this term in unsteady flow equations requires a change in its sign. Thus, q with a positive sign is regarded as inflow, whereas a negative sign of q refers to outflow.

The derivation of equations 2.3 and 2.4 are well documented in standard text books (Chow, 1959; Abbott, 1979).

The terms in the continuity and momentum equations are defined as follows;

(i) rate of change of storage due to rate of change of water surface elevation with time.

(ii) wedge storage term due to variations in water level with distance along the channel.

(iii) prism storage term due to variations in velocity with distance along the channel.

(iv) the change in area (per unit length of channel) below level surface that intersects the water surface at the centre of a short reach, (areal variation with distance along the channel).

(v) lateral inflow which gives the net spatial variation in rate of change of mass.

(vi) local acceleration (variation in velocity with respect to time).

(vii) convective acceleration (variation in velocity with distance along the channel).

(viii) hydrostatic pressure force term.

(ix) horizontal component of body force.

(x) friction slope effect.

(xi) acceleration effect due to lateral inflow.

The terms in the momentum equation have got their own role in determining the nature of flow. This can be illustrated by rearranging equation 2.4 in the following form; (Henderson, 1966)

$$S_f = \underbrace{\underbrace{\underbrace{S_o}_{\text{Steady Uniform Flow}} - \frac{\partial h}{\partial x} - \frac{u}{g}\frac{\partial u}{\partial x}}_{\text{Steady non-uniform flow}} - \frac{1}{g}\frac{\partial u}{\partial t}}_{\text{Unsteady non-uniform flow}} \quad \ldots (2.5)$$

To avoid the complexity, the additional terms shown in equation 2.4 are not included in the above equation to describe the flow.

In the derivation of St.Venant equations the following fundamental assumptions are made.

(i) The water density is homogeneous.

(ii) The acceleration of water particle is only in the direction of x.

(iii) The vertical acceleration is small, and consequently the pressure distribution is hydrostatic and there is no vertical velocity component.

(iv) The resistance to flow is approximated by the equations valied for open channel flow, such as Manning's equation.

(v) Lateral inflow and outflow rates are uniform with respect to a particular channel element.

(vi) The momentum influx of the lateral inflow and the momentum efflux of lateral outflow are negligible.

(vii) The average channel bed slope is so small that the following approximation is considered to be valid.

$$\sin\theta = S_o = \text{Slope of the bed and } \cos\theta = 1$$

2.5 THE DYNAMIC WAVE (DW) THEORY AND THE KINEMATIC WAVE (KW) THEORY

A water wave or "a surface wave is a temporal, spatially propagated change in water surface" (Yevjevich 1975a). The study of motion, in which the influences of mass and forces are included, is essentially dynamic in nature. On the other hand, in kinematic wave motion these influences (mass and forces) are not taken into account. Thus, dynamic wave is the study of a disturbance which propagates in shallow water under the influences of mass and forces. On the other hand "Kinematic wave is one in which discharge is a function of flow depth alone" (Henderson, 1966).

Generally, a dynamic wave characterizes the movement of long waves in shallow water, e.g. movement of a large flood wave in a wide river, where 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 weight of the fluid flowing down owing to gravity, is approximately balanced by the resistive forces of channel bed friction. This is equal to assuming that the energy line is parallel to the bed of the channel. In most cases, the friction force is accounted for by the Manning's equation. 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. 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, in many cases flood wave travel can better be defined through KW phenomenon rather than dynamic waves.

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 of flow. In case of overland flows, the hydraulic depth D is represented by mean depth of flow h.

Dominance of kinematic or dynamic wave is generally ascertained through the Froude's number. The Froude's number (Fr) is defined as the ratio of 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. $Fr = U/\sqrt{gD}$) where U is the mean velocity.

From the analysis, Lighthill and Whitham (1955) found analytically that when the average velocity of flow 'U' is greater than twice the speed of the wave relative to the water $\sqrt{gD}$, depth of flow will continue to increase and a surge or bore will be developed. They have further shown that when Froude's number is 2, kinematic waves dominate over dynamic waves and for $Fr<2$ the dynamic waves are damped. However, Gburek and Overton (1973) have reported that a subcritical kinematic stream flow was found to occur in a seven mile reach of a 162 $mile^2$ watershed tributary in Pennsylvania where Froude number was found to be less than 0.34.

Generally speaking, the KW approximation would be good when $Fr > 1$ (Supercritical) because the waves could not move upstream (since $U > \sqrt{gh}$ or $\sqrt{gD}$). Therefore, kinematic waves ultimately prevail the flow characteristics for overland flows as well as the channel flow when $Fr \leq 2$.

Thus KW theory is one of the most important methods for estimating storm water runoff rates and their volumes. Though, it is a simplified form of DW approach, yet it has proved itself to be an efficient method for simulating storm water runoffs from small watersheds (Overton and Meadows, 1976 and Ponce, 1991).

2.6 APPLICABILITY OF DYNAMIC WAVE AND KINEMATIC WAVE THEORY

In a natural channel, the flood wave is formed mainly due to direct runoff caused by the storm rainfall and may also have contributions from the snow melt as well as from the baseflow. Such flows are generally unsteady and dynamic in nature.

The kinematic wave theory finds its proper application in overland flow routing where backwater effects are insignificant (Wooding 1965, Woolhiser and Liggett, 1967 and Field, 1983). Woolhiser and Liggett (1967) have discussed the applicability of KW theory in terms of kinematic flow number K.

$$K = \frac{S_o L_o}{Fr^2 h} \quad .... \quad (2.7)$$

Where, L_o and S_o are the length and slope of overland plane respectively. They stated that the KW approximation of St. Venant equations should be preferred if $K > 20$ and $Fr \geq 0.5$. They also recommend that KW approximation is sufficient for the study of flow in a small watershed. Overton et al., (1976) have recommended the application of kinematic wave when Froude number is greater than 1. Kinematic wave models have been used and assessed by various researchers like Ponce et al., (1978), Bren et al., (1978) and Hromadka et al., (1988). In another work, Vieira (1983) has discussed the validity of various approximations for the St. Venant equations. The applicability of the KW approximations of St. Venant equations have been investigated by Morris and Woolhiser (1980). They suggest that the KW approximation may be used instead of the St. Venant equations provided $Fr^2K > 5$. Vieira has compared his study with the results obtained by Woolhiser and Liggett (1967), Morris (1979) and, Morris and Woolhiser (1980) for the applicability of DW and the various approximations of DW equations.

From these studies, one can conclude that KW approximation is now a well established method for surface runoff computations and is generally applicable where the watershed slopes are high. The models based on KW approximation are applied to a wide range of watersheds i.e. from mountainous to urban watersheds, of small geometry. The characteristics of flows in these two extreme type of watersheds suggest that the flow velocities may be high with Froude number greater than 1. As a result of it, there is no backwater effect and bore formation. In majority of catchments, the average slope of the drainage system is much less than that of the mountainous watersheds and the flows along the river channels modify to a great extent. To account for the physical conditions governing the flow movement, it is better to retain all the parameters involved in the St. Venant equations. The approximations caused by dropping some of the terms in these equations may result into serious distortions and hydraulic characteristic may not be correctly estimated.

2.7 SOLUTION TECHNIQUES FOR UNSTEADY FLOW EQUATIONS

St. Venant equations which describe the movement of flood defy a closed form solution unless many simplifying assumptions are introduced (Yevjevich, 1975).

The solution techniques for St. Venant equations are broadly divided into two categories.

i) Analytical solution techniques

ii) Numerical solution techniques

These two techniques will be discussed briefly in the following sections.

2.7.1 Analytical Solution Techniques

In practical applications, the analytical solution for St. Venant equations are limited and restricted to simplified cases. If all the terms of the unsteady flow equations are taken into account, the analytical solution becomes more difficult.

The diffusion wave approximation of DW equations has also been tried analytically. Analytical solutions to the linearized form of diffusion equation are given by Sutherland and Bornett (1972), Keefer and McQuivey (1974), Dooge et al., (1983), Tingsanchali and Manandhar (1985) and Gonwa et al., (1986). The work of Chalfen et al., (1986) is another example of analytical solution of the simplified form of the St. Venant equations.

2.7.2 Numerical Solution Techniques

With the advent of digital computers, researchers have the advantage of using the numerical techniques. A large number of such techniques are available for solving the St. Venant equations. Each of these offer particular advantages and disadvantages in terms of convergence, stability, consistency, accuracy and efficiency.

The numerical solution of unsteady state flow equations can be obtained by using the Method of Characteristics, Finite Difference Methods or the Finite Element Techniques. Among these three, the finite difference methods are the most popular and advantageous. Different computational schemes have been proposed by the researchers for the finite difference solution. However, as mentioned above, their relative advantages and disadvantages are to be ascertained in terms of the following properties.

Convergence

A finite difference scheme is convergent if the computed value of the unknowns in the governing flow equations approach the exact solutions of the equations as the grid intervals (i.e. Δx and Δt) are reduced. It may be mentioned that the full differential equations can not be solved exactly but for a few exceptional (i.e. mostly linearized) cases of the nonlinear differential equations. Therefore, it is impossible to compare numerical and exact analytical solutions of the differential equations. It does not mean that the convergence of a finite difference scheme can not be analyzed, but one can use the convergence conditions derived for linearized version of the full equations to estimate the behaviour of the scheme when applied to the full nonlinear equations. For the linear system problems, convergence is guaranteed if the Lax Theorem is satisfied. It may be stated that:

> *"Given a properly posed initial-value problem and a finite difference approximation to it that satisfies the consistency condition, stability is the necessary and sufficient condition for convergence"* (Abbott, 1980)

Stability and Approximation Error

Stability is an important aspect of a finite difference scheme. This can be estimated by prescribing the boundary and initial conditions correctly. This way the numerical solution obtained by using a numerical scheme are banded, i.e. a small error should remain small during the whole range of computation, and should never become great to be significant enough. In other words, the stability may be studied by fining the space and time intervals (Δx and Δt) and then examining the results of the computation for a continuous time domain i.e. $t \longrightarrow \infty$. If the errors are not magnified and remain restricted such that the results obtained are valid then, the finite difference method is said to be stable.

Since in the present work the finite difference methods have been preferred for solving the unsteady flow equations, the same have been discussed in greater depths in the forthcoming discussion. However, a brief review of the research work carried out to obtain solutions for St. Venant equations by using the method of characteristics and the finite element techniques has been reported in Section 2.10.

2.7.2.1 Finite Difference Method

The finite difference method is used by various researchers for water flow modeling. Following the finite difference method, the St. Venant partial differential equations are replaced by functions defined on finite number of grid points within the considered domain. The derivatives are then replaced by dividing differences. Thus, the differential equations are replaced by algebraic finite difference relationships.

In the finite difference based models, the following two approaches are commonly used for the solution of St. Venant's equations.

a) explicit finite difference schemes

b) implicit finite difference schemes

Both the schemes have been extensively used by researchers for the solution of unsteady flow differential equations. Some of the characteristics of these two solution schemes are outlined in the following sections.

(a) Explicit Finite Difference Methods

In explicit finite difference models, the spatial derivatives are written at the beginnings of a time step in terms of the known depths and/or velocities. Thus, the solution of the St. Venant equations advances point by point along one time line in the x-t solution domain, until all the unknowns associated with that time line have been computed. In this way, the solution procedure is carried out until for all time lines, the unknowns in the entire solution domain are evaluated.

The development of explicit models perhaps began with the work of Stoker (1953) who developed an explicit scheme which was later utilized by Isaacson et al., (1958). They later applied it for the flood prediction of the Ohio river. There are a number of explicit schemes which are now in use for the solution of St. Venant equations. Some of the most popular explicit schemes are as under.

a) the unstable scheme b) the diffusion scheme
c) lax-Wendroff scheme d) leap frog scheme
e) Dronker's explicit scheme, etc.

The explicit methods are simpler in application but suffer from the limitations that the solution may not be always stable. The stability is generally checked be the Courant's condition which is given below.

$$\Delta t \le \frac{\Delta x}{|v \pm c|} \quad \ldots \quad (2.6)$$

Thus, the above condition has to be satisfied in the entire solution domain. The condition limits the size of time step (Δt) and as a result it takes more computer time. Therefore at times it is found to be uneconomical. Liggett and Woolhiser (1967) have investigated the stability of the above mentioned first four explicit methods. Constantinides (1982) has used various schemes for solving the KW approximation to asses the suitability of the schemes for different prevailing conditions. It was suggested that the 'backward central explicit finite-difference scheme' yields better results. Huang et al., (1985) have studied the conditions under which the diffusive finite differencing scheme remains stable. They also studied the criteria for the applicability of diffusive explicit scheme alongwith Lax-Wendroff. Field and Williams (1983) have adopted the Lax-Wendroff scheme in their KW catchment model. A detailed description of various explicit schemes has also been given by Liggett and Cunge (1975).

(b) Implicit Finite Difference Methods

The implicit finite difference methods are those in which the unknowns (dependent variables) along a time line are simultaneously computed in terms of dependent variable along the same time line. In these models, the spatial derivatives are written at the end of a time step in terms of the unknown depths and/or velocities of flow.

Implicit models had to be developed because of the limitations on the size of time step required for the numerical stability of explicit methods. The implicit finite difference methods need not obey the Courant's stability condition as they are unconditionally stable.

In general, the implicit finite difference schemes may be classified into the following two broad categories.

a) four point implicit schemes

b) six point implicit schemes

The use of implicit models was probably first suggested by Isaacson et al., (1958). However, the strength of this methodology was realized later when the sufficiently fast computers were available for the construction of computational hydraulic models. Subsequently, it was Preissmann (1961) who was among the first to apply such a scheme for solving the unsteady open channel flow equations. In the Preissmann's method of solution, the unsteady flow equations are converted into a linear system which is solved by the double sweep algorithm. Liggett and Woolhiser (1967) have developed an implicit finite difference solution of unsteady flow equations which solved the resultant linear system by this algorithm. Abbott and Ionescu (1967) have followed a similar method of linearization of the unsteady nonlinear equations which was solved by using 'double sweep' algorithm.. The proposed method could also use variable mesh sizes in space and time. Baltzer and Lai (1968) employed an implicit finite difference solution of the partial differential equations in which the resultant nonlinear system of equations was solved by trial and error procedure. Amein (1968, 1970, 1975) developed an implicit finite difference solution based on the use of 'central differences' for representing the partial differential equation and solved the resultant system of simultaneous equations by generalized Newton's iteration method. Implicit finite difference methods have also been used in KW as well as diffusion wave approximations of unsteady flow equations. Field et al. (1987) used a four point implicit finite difference method in his kinematic catchment model. Akan (1981) used a four point implicit method for flow simulations in channel networks using diffusion wave approximation. Works on similar lines, using implicit methods have also been reported by Vasiliev, et al., (1965), Dronkers (1969), Kamphuis (1970), Quinn and Wylie (1972), Fread (1973), Chaudhry and Contractor (1973), Greco and Panattoni (1975), Amein and Chu (1975), Lai (1988), Fennema, et al., (1989), Tayfur, et al., (1993), etc.

2.8 KW THEORY BASED HYDROLOGIC MODELS

The KW theory has been introduced and utilized in describing flood movements in rivers by Lighthill and Whitham (1955). Iwagaki (1955) developed an approximate 'method of characteristics' for the steady flows in open channel and proposed the use of this method for practical purposes. In this analysis he assumed the lateral inflows to be approximately uniform. These two researchers laid the foundations for the KW theory and demonstrated the applicability and usefulness of this theory. The KW theory was probably first used by Henderson and Wooding (1964) for watershed hydrology. They applied KW theory for describing the hydrograph from a steady rain over a sloping plane, neglecting the slope of water surface relative to the slope of plane. Wooding (1965a, 1965b, 1966) further employed the KW theory in the development of a runoff model for a V-shaped watershed geometry. Though a good agreement was reported between the observed and the computed runoff hydrographs yet it was concluded that a better geometric description of stream network would be desirable.

Woolhiser and Liggett (1967) solved the overland flow equations, to obtain typical results for the rising limbs of the hydrographs. Solutions for these rising hydrograph for a wide range of KW number K, were presented. They delineated a criterion for choice between the DW and its KW approximation. Woolhiser (1969) developed a KW model for converging surface flow analysis. He pointed out that in actual watershed hydrograph, the steep rising portion is caused by the concentration of runoff. This portion may be reproduced by converging surface overland flow models. He numerically solved the characteristic equations of kinematic flow on a converging surface.

The work of these investigators provided a sound base for future research on application of KW theory (Eagleson, 1967, 1968, 1971, 1972, Overton and Brakensiek 1970; Li, 1974; Singh 1974). A V-shaped geometry, proposed by Wooding (1965a), was subsequently taken to represent the natural watershed.

Brakensiek (1967a) introduced the concept of kinematic cascade. He utilized this concept in the transformation of an upland watershed into a cascade of planes discharging into a single channel.

Wei and Larson (1971) attempted to incorporate the spatial and temporal variations of storm rainfall movement alongwith watershed physiographic shapes to see the effects on the runoff hydrographs.

Smith and Woolhiser (1971a, 1971b) combined the KW approximation for the unsteady overland flow, with a mathematical model of infiltration based partial differential equation for vertical, one-phase, unsaturated flow in soils.

Kibler and Woolhiser (1972) developed dimensionless equations for the kinematic cascade and derived general equations for a single element in the cascade. They developed a criterion to delineate the development of kinematic shock waves. Singh (1974) and Lane (1975), applied the kinematic cascade to predict surface runoff from agricultural and urban watersheds. Lane (1975) experimentally studied the influence of simplifications of watershed geometry in the simulation of surface runoff. Singh (1974, 1975a) utilized the converging section of geometry in developing a nonlinear KW model for watershed runoff. Singh (1975c) also derived general solutions to KW equations using space-time variant input.

Towards practical applications, Izzard (1943) conducted experiments for the study of overland flow hydraulic. Morgali and Linsley (1965), and Morgali (1970) utilized these data to verify their analytical and numerical results.

The other studies based on the laboratory investigation of KW theory are the work of researchers, Chow and Harbaugh (1965), Harbough and Chow (1967), Dass and Haggins (1970). Langford and Turner (1973) described an experimental test to establish the accuracy of KW theory for overland flow over a rough uneven surface. Muzik (1973, 1974a, 1974b) studied the response of a well-defined impervious surface to a specified input of rainfall. The mathematical model developed was tested by simulating experimental hydrographs of surface runoffs generated by uniformly varying rainfall. Singh (1974, 1975a, 1975b) studied the applicability of the KW theory on a large experimental rainfall-runoff facility. He further used the method of characteristics (1976) to solve the kinematic flow equations on a converging surface. The proposed methodology was tested on several natural agricultural watersheds. Through a regression analysis, the friction parameter was corrected through physically measurable characteristics of watershed physiography.

Field (1982) developed a KW model which included the effects of linear storage elements within the catchment. He used method of characteristics to solve the KW equation. In another attempt, Field et al., (1983), improved a previous study by including the effects of storages, in nonlinear form, to solve the kinematic wave equation using Lax-Wendroff explicit numerical technique. Further, Field et al., (1987) also developed a model based on KW theory, incorporating the lateral flows from storages (surface and subsurface) through nonlinear relations, solving the KW equation by using implicit Lax-Wendroff scheme.

Rose et al., (1983) developed an approximate kinematic overland flow routing equation which was applicable in some special cases of the steady-state water surface profile and nonconvergence and nondiverging flow planes. It was assumed that an overland flow plane acts as a single nonlinear storage plane. Moore (1985) introduced the concept of a water surface profile shape factor that related to the depth of flow. This concept extended the Rose's solution by evaluating a constant shape factor for the unsteady state water surface profiles over converging and diverging flow planes. Moore et al., (1987) generalized the water surface profile shape factor of his earlier work for various phases of the overland flow hydrograph (i.e. rising and recession parts). Analytical equations have been developed for predicting the water surface shape factor as a dependent variable.

Some of the KW models worth mentioning are those developed by Hjelmfelt (1981), Smith and Hebbert (1983), Akan (1985a, 1985b, 1988). Hromadka et al., (1988), Dawdy (1990), Goldman (1990), Merkel (1990), Unkrich and Woolhiser (1990), Woolhiser and Goodrich (1990), Hromadka et al., (1990) and Ponce (1991) have studied the causes of the computational errors in KW models. They have also highlighted the applicability of kinematic wave overland flow models.

2.9 DYNAMIC WAVE MODELS

Modeling of one dimensional dynamic wave is not an easy task because of the large number of properties involved in the numerical solutions. Many models have been reported in the past which use different methodologies. Some of the models, which have a relevance in the context of this study, are being summarized in the forthcoming discussions.

The basic analytical approach to the overland and channel flow phenomena was provided by de Saint Venant (1871) who derived the equations of continuity and momentum for unsteady gradually-varied flows. These equations adequately describe the surface flow and happen to be nonlinear partial differential equations of hyperbolic nature.

Massau (1889) transformed the St. Venant equations into a set of equivalent characteristic equations and presented a graphical method for their integration. His method was simplified and adapted to practical purposes by Graya (1946) who made the assumptions of straight-line characteristics to solve the unsteady flow equations for problems involving flow resistance in sloping channels with changing cross-sections. Nosek et al., (1947) used a modified Graya method to route the flood wave which resulted from the failure of the Saint Francis Dam, California USA.

Thomas (1937) was probably the first to outline finite difference methods for St. Venant equations for the study of movement of flood waves in rivers. Isaacson et al., (1958) formulated a model for routing the flood waves through the Ohio river using finite difference form of the unsteady flow equations. After the pioneering work of Stoker (1953) and his co-workers, many mathematical models of channel flow based on the St. Venant equations have come up. Morgali and Linsely(1965) used the DW equations to formulate their overland flow model.

Brakensiek (1966) developed a 'four point implicit model' for routing of flows. By testing his model over a watershed of 330 acre, he concluded that it is feasible to utilize the unsteady flow equations for flow simulations over small watersheds.

Several finite difference schemes were examined theoretically as well as empirically by Liggett and Woolhiser (1967) for convergence of the finite difference solutions. They pointed out that the method of characteristics and the implicit methods provide stable solutions over a wide range of parameters. Fletcher et al., (1967) used method of characteristic for the solution of unsteady flow equation for modeling of flow in an irregular river channel. He successfully applied this model for flow studies of the Emory River.

Wylie(1970) assessed the merits and demerits of four different methods of solutions based upon the method of characteristics. He suggested that the high friction may lead to instabilities in the solution of the problems when the explicit formulations are used. Amein et al., (1970) adopted the Preissmann Scheme to solve the unsteady flow equations. The four point finite difference scheme was adopted to convert the differential equations into their finite difference forms. He solved the system of finite difference equation using the Newton Raphson's iterative procedure. He used the data of an irregular river channel (Neuse River, USA) to test the feasibility of application of his model.

From 1960 to 1975, a number of models based on dynamic wave equations have been developed. Some of these are summarized in a study conducted by Miller et al., (1975).

A number of models for routing of flow through a network of river system have also been developed. The implicit junction flow models were developed by Abbott - Ionescu (1967), Preissmann (1961), Fread (1973), Kao (1980) and Joliffe (1982). Fread (1973) proposed an iterative procedure for considering flow interaction at channel junctions. Joliffe (1982) suggested that his implicit model for simulation of flow in channel networks required much smaller computer memory then an alternate model formulated without using the sparse matrix solution technique.

Modeling of unsteady shallow water flow on infiltrating surface by a conjunctive surface-subsurface flow system has been suggested by Akan et al., (1981a). He adopted the two dimensional unsteady flow equations for the surface flow modeling. The subsurface flow was modeled through an equation for two-dimensional motion of a single phase incompressible fluid in a nondeformable porous medium. The method of solution was accomplished implicitly for both the types of the flows.

Stepien (1984) used a hydrodynamic model for flood propagation for application to real-time forecasting system. He adopted the 'box-scheme' for his model formulation and verified his model by applying it for routing of flows below a dam on the Vistula River in Poland. In recent papers, good amount of research work on modeling of shallow water flows have been reported. Among these are the works of Baines et al., (1993), Jhonson et al., (1993), Tayfur et al., (1993), O'Brien et al. (1993) and Naot et al., (1993).

It may be remarked that in the present work the proposed model is developed by using the KW as well as the DW theories. Therefore, in the earlier sections the research work based on these theories was discussed in much details. However it would be pertinent to note that dynamic models have also been used in the field applications using the method of characteristics as well as the finite element techniques. Therefore, in the forthcoming sections a brief review is presented for these applications.

2.10 USE OF METHOD OF CHARACTERISTICS AND THE FINITE ELEMENT TECHNIQUES IN WATERSHED MODELING

In the Method of Characteristics (MOC), the two non-linear partial differential equations of unsteady flow are converted into a set of four ordinary differential equations. The resulting ordinary differential equations are then replaced by their finite difference forms and are solved for the unknown parameters. The details of the method of characteristics are explained by Abbott (1966). Generally, there are two broad approaches of the numerical solutions of St. Venant equations used in the method of characteristics;

(a) the rectangular grid, and

(b) the characteristic grid.

Some attractive qualities and features of MOC are quite valuable and therefore many investigators have been constantly improving this method. As a numerical technique this method was first applied for the numerical modeling of St. Venant equations by Amein (1966), and subsequently by Liggett and Woolhiser (1967). This method has also been used and improved by other researchers (Chang and Richards (1971), Vardy (1977), Wiggert and Sundiquist (1977) Wylie (1980), Goldberg and Wylie (1983)). Price (1974) and Trikha, (1977), have shown that the method of characteristics can be modified such that the Courant's instability condition is no longer a limitation for the method.

Schmitz and Edenhofer (1980, 1983), Edenhofer and Schmitz (1981) were probably the first to free the method of characteristics (with specified time interval scheme) from the Courant's constraints with their implicit method of characteristics.

The implicit schemes though independent of Courant's conditions have some other constraints of their own. Many investigators (viz. Chang and Richard, 1971; Vardy, 1977; Wylie, 1980 and Goldberg and Wylie, 1983) have tried to improve the method of characteristics for flow modeling purposes. Schmitz and Edenhofer (1980 and 1983) and, Edenhofer and Schmitz (1981), were perhaps the first to free rectangular grid based method from Courant's condition with their newly developed implicit method of characteristics (Braines et al., 1992). In a research work, Lai (1988) have combined different schemes and developed a comprehensive model with new advantageous features.

The Finite Element Method (FEM) is another class of techniques used for the solution of unsteady flow equations. Finite element methods approximates the differential equations by integral approaches.

The process strength of FEM lies in its ability to accommodate curved boundaries and non-homogeneous material properties. As in finite difference methods, the finite element methods also exhibit problems of numerical dispersion and dissipation. Numerical dispersion produces oscillations in the computed solution and are caused by phase difference between the computed and the observed solutions. Numerical dissipation produces 'smeared out solutions' and are caused by amplitude difference between true and computed solutions.

Finite element method has been applied successfully to the shallow water wave equations. In the analysis of a few surface water FEM models, Gray (1980) found that excessive numerical damping is the common shortcoming of these models. The models he considered included Tayler and Davis model (1975), Linch and Gray (1978), Kawahara et al., (1978) and Masuda model (1979), etc. Propagation characteristics of numerical schemes have been studied by Gray et al. (1976) using one dimensional convective diffusion equation.

Zienkiewicz, et al., (1975) have presented a comparison of finite difference method and finite element method. They found that in slow flows where convective terms are insignificant, FEM is superior technique. With high velocities, where connective terms become important, as well as in transient problems, the FDM maintains some apparent superiority. The FEMs are found as accurate as finite difference method. In an analysis Gray et al., (1976) found that while most finite difference schemes produce an accuracy of the solution which is the same at all nodes, the accuracy of finite element schemes differed from one type to the other.

Example of the models in which finite element method is used for the solution of complete St. Venant equations is the work of Cooley and Moin (1976). This method of solution is popular in two dimensional unsteady flow models. The two dimensional wave model developed by Akanbi and Katopodes (1988) also makes use of the finite element method. The other model which makes use of FEM is the one developed by Hicks et al., (1992).

CHAPTER III

MODEL DEVELOPMENT

3.1 INTRODUCTION

In the previous chapter, the basic concepts along with the solution procedures of unsteady flow equations were briefly discussed. So far, the practicing hydrologists have put reliance on models which are mainly based on the unit hydrograph (UH) theory. These models are by and large lumped in nature and ignored the distributed characteristics of the flow process over a drainage basin. Therefore, the unevenness of rainfall, as well as the variations in physiographic conditions which are normal features of the watersheds in most of the tropical countries could not be taken into account. The objective of this study is to investigate the characteristics of flood flows with reference to mechanics of flow over natural watersheds and to develop a suitable hydrologic model.

In this study, a hydrologic model is considered as a quantitative expression of a hydrologic process or phenomenon which one is observing, analyzing, and wants to use for prediction purposes. Mathematical modeling of hydrologic process provides a tool by means of which one can study and gain an understanding of hydraulic flow phenomenon, select and design sound engineering projects and predict extreme situations, so as to be able to provide advance warnings. The essential quality of a mathematical model lies in its predictive capacity. For model prediction to be accurate and useful, an attempt is made to develop a model based on hydraulic equations which represent the dominant flow characteristics. There are many factors which influence the runoff process. These factors affect the movement of water on the overland planes as well as in channels. Different facets of the hydrologic transformation process have been investigated by the researchers in order to get an insight into complexities involved. But till now, a few methodologies (or models) are available for the numerical applications to predict (i) the water surface profile along the drainage system and (ii) flow characteristics over the overland planes incorporating the distributed effect of various parameters of the watershed.

To achieve this objective, various techniques and available models were studied. It was decided that investigation of the watershed response be carried out in two phases. The first phase may deal with the flood water movement over the overland surface, and the second phase should refer to the flood wave modification in the stream channels.

An overview of the present day hydrologic models indicates that kinematic wave (KW) theory has been widely used for simulation of flows over the planes provided the criteria for the KW application are satisfied. However, the applications of KW theory have been mostly restricted to urban watersheds and to some extent to the natural watersheds having comparatively small drainage areas. On the other hand, dynamic wave (DW) models were employed for simulations of flow in rivers. The DW models are mathematical deterministic and physical process based models which account for the dynamics of processes associated with the flow of water on the watershed. The DW theory is considered to be best suited methodology for taking into account the prevailing flow conditions over the watershed and its channel flows.

Thus, the present study is aimed at developing a mathematical model utilizing the characteristics of dynamic wave theory and its KW approximation.

3.2 ELEMENTS USED IN THE PROPOSED MODEL

To meet the objectives stated in Chapter I, a suitable physiographic model need be developed. The model should properly take into account the mechanics of the movement of water on the plane as well as in the channel. The details of the proposed physiographic models are given in Section 3.5. However, the model will comprise of the following two basic components.

(i) The overland plane components

(ii) The main channel component

Generation of flow profiles over the planes and in the channel are shown in Figure 3.1. The channel flows are routed by using the DW theory. This may help in satisfactorily routing even the flows with low Froude numbers (i.e. the non-monsoon flows). The lateral flows coming to the main channel are contributed by the overland planes. Keeping the above structure of the model in view, the mathematical formulation of the DW channel flow model is discussed first, to be followed by the KW model used to compute the lateral flows from the overland planes.

3.3 THE DW CHANNEL FLOW MODEL

Since adequate data of river sections are generally not available, the St. Venant's equations (2.3 and 2.4) are solved assuming the channel to be wide rectangular. Thus, the equations 2.3 and 2.4 are simplified to the following form.

The continuity equation :

$$\frac{\partial h}{\partial t} + u\,\frac{\partial h}{\partial x} + h\,\frac{\partial u}{\partial x} = q \quad \ldots (3.1)$$

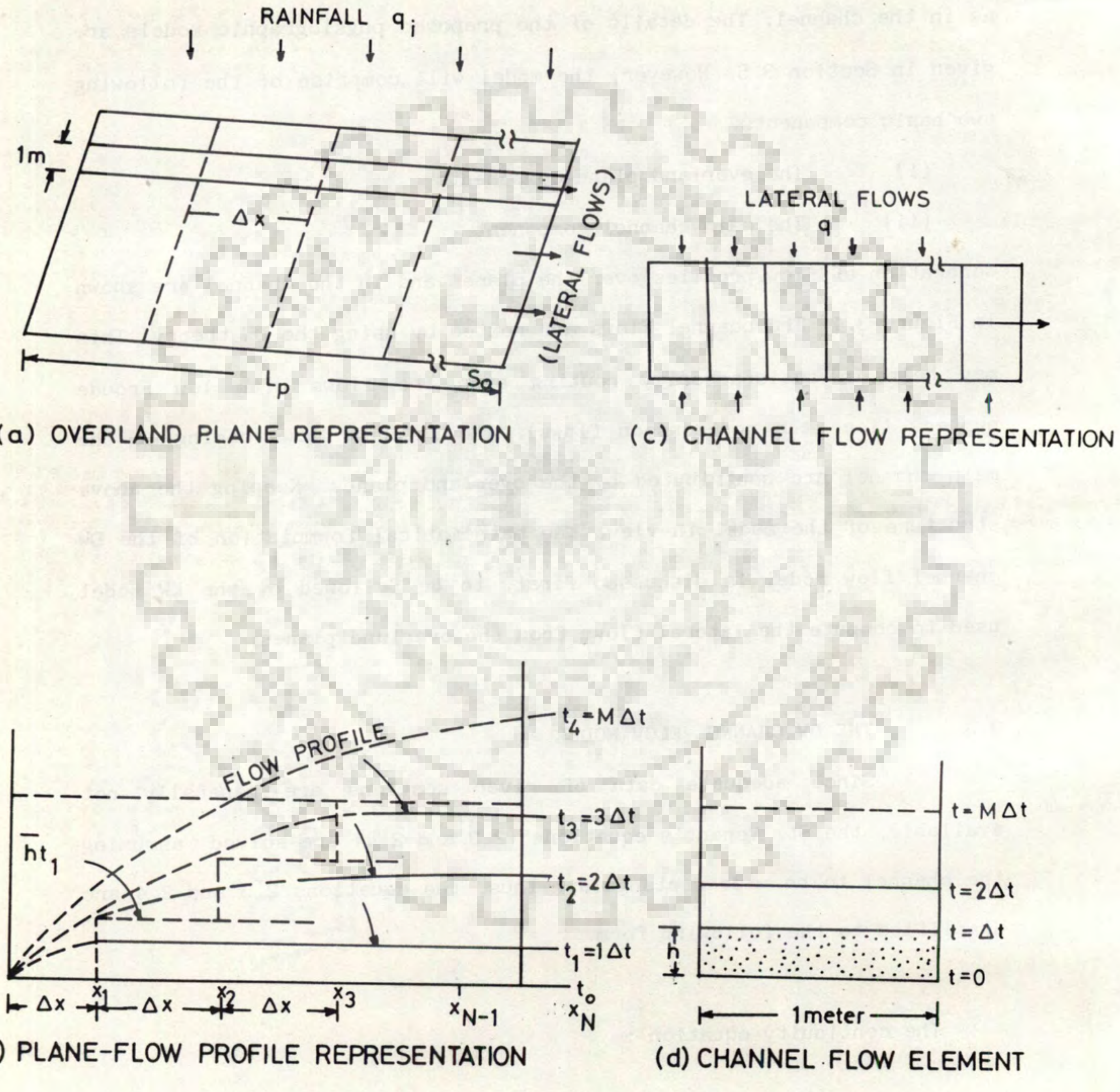


FIG. 3·1 - GENERATION OF FLOW PROFILES ON OVERLAND PLANES AND IN CHANNEL

The momentum equation is :

$$\frac{\partial u}{\partial t} + u\,\frac{\partial u}{\partial x} + g\,\frac{\partial h}{\partial x} = g(S_o - S_f) - \frac{qu}{h} \qquad \ldots (3.2)$$

In the above equation, it is further assumed that the lateral flow velocity component (V) is negligible. The friction slope (S_f) is approximated Manning's equation as under.

$$S_f = \frac{n^2\, u|\, u\,|}{h^{4/3}} \qquad \ldots (3.3)$$

Where n is the Mannings roughness coefficient. the term u^2 appearing in the Manning's equation is replaced by u | u | to account for the possibility of flow reversal.

Equations 3.1 and 3.2 are nonlinear partial differential equations, hyperbolic in nature. The finite difference scheme used for the solution of these equations is discussed in next section.

3.3.1 Finite Difference Scheme Used For The Solution of Continuity and Momentum Equations

Different types of numerical schemes which are used for solving the differential equations were discussed in Chapter II. The four point implicit method has been adopted for the solution of unsteady flow equations. Keeping in view the flow properties and the advantages of this method, equations 3.1 and 3.2 are converted into their finite difference forms in the following sections. As a first step towards the solution, the computational points on a time-distance grid (Figure 3.2) are discretized.

Numerical solution of these equations is sought over a distance (x)- time (t) rectangular domain. The distance domain is discretized by a finite number of nodal points. Similarly, the time domain

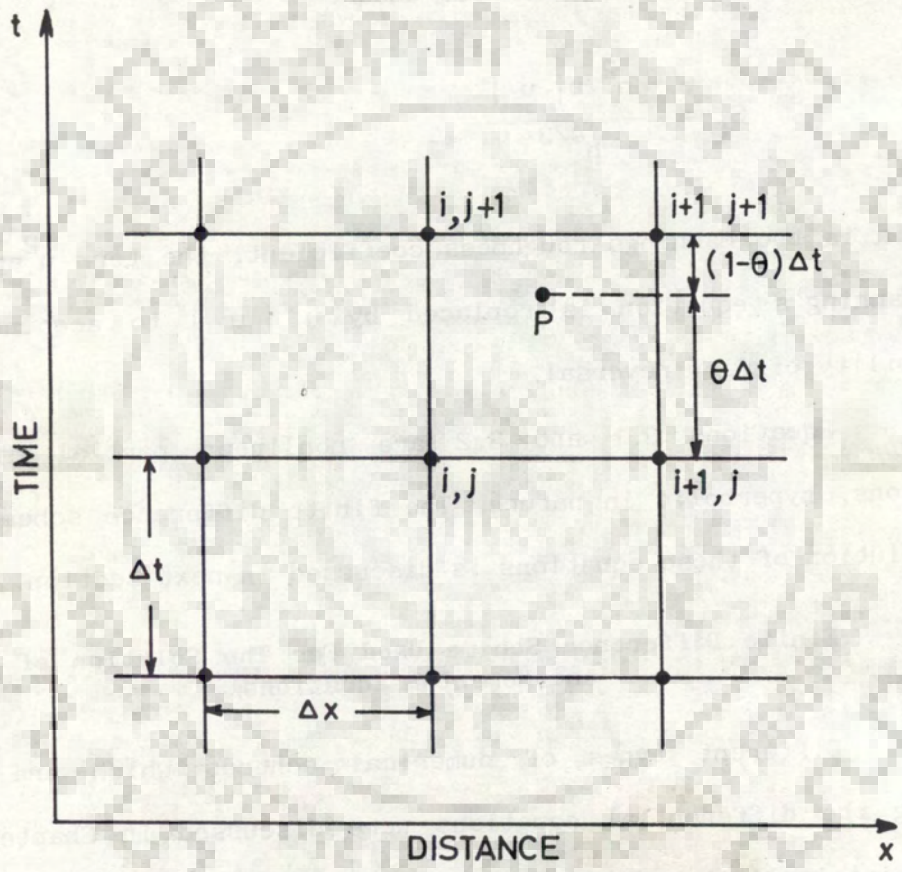


FIG. 3·2–FOUR POINT IMPLICIT FINITE DIFFERENCE SCHEME is discretized by a finite number of discrete times. Thus, the solution domain is discretized by a finite number of distance-time grid points (Figure 3.2). The grid point at i^{th} node and j^{th} time is designated as (i,j). The dependent variables u and h at the grid point (i,j) are discretized as u_i^j and h_i^j. Similarly, the lateral flow q at a grid point (i,j) is discretized as q_i^j. Using the four point difference method, the average values of a variable (f) and its derivatives at any interior grid point (i,j) are approximated as follows.

$$f = \frac{\theta}{2}\left(f_{i+1}^{j+1} + f_i^{j+1} \right) + \frac{1-\theta}{2}\left(f_{i+1}^{j} + f_i^{j}\right) \qquad \ldots(3.4)$$

$$\frac{\partial f}{\partial x} = \theta\left(\frac{f_{i+1}^{j+1} - f_i^{j+1}}{\Delta x}\right) + (1-\theta)\left(\frac{f_{i+1}^{j} - f_i^{j}}{\Delta x}\right) \qquad \ldots(3.5)$$

$$\frac{\partial f}{\partial t} = \frac{1}{2\Delta t}\left(f_{i+1}^{j+1} - f_{i+1}^{j} + f_i^{j+1} - f_i^{j} \right) \qquad \ldots(3.6)$$

Where θ is a time-weighting coefficient.

In equations 3.4 through 3.6 'f' may represent u, h and q so that u, h, q, $\frac{\partial u}{\partial x}$, $\frac{\partial u}{\partial t}$, $\frac{\partial h}{\partial x}$ and $\frac{\partial h}{\partial t}$ at (i,j) are approximated in terms of the corresponding values at for corner grid points (Figure 3.2).

3.3.2 The Finite Difference Form of Continuity And Momentum Equations

On substituting the equations 3.4, 3.5 and 3.6 for the appropriate terms in equations 3.1 and 3.2, the following two nonlinear algebraic equations are obtained.

$$\xi_i = h_{i+1}^{j+1} + h_i^{j+1} + 2\,\frac{\Delta t}{\Delta x}\Bigg[\,\theta^2\left\{u_{i+1}^{j+1}\,h_{i+1}^{j+1} - u_i^{j+1}\,h_i^{j+1}\right\}$$

$$+\theta(1-\theta)\left\{u_{i+1}^{j+1}\,h_{i+1}^{j} - u_i^{j+1} h_i^{j} + u_{i+1}^{j}\,h_{i+1}^{j+1} - u_i^{j}\,h_i^{j+1}\right\} + e_i\Bigg]$$

$$- 2\,\Delta t\,Q_i = 0 \qquad \ldots(3.7)$$

Where,

$$e_i = (1-\theta)^2\left\{u_{i+1}^{j}\,h_{i+1}^{j} - u_i^{j}\,h_i^{j}\right\}$$

$$Q_i = \frac{\theta}{2}\left[q_{i+1}^{j+1} + q_i^{j+1}\right] + \frac{1-\theta}{2}\left[q_{i+1}^{j} + q_i^{j}\right]$$

and

$$\psi_i = u_{i+1}^{j+1} + u_i^{j+1} + \frac{\Delta t}{\Delta x}\Bigg[\,\theta^2\left\{(u_{i+1}^{j+1})^2 - (u_i^{j+1})^2\right\}$$

$$+2\theta(1-\theta)\left\{u_{i+1}^{j+1}\,u_{i+1}^{j} - u_i^{j}\,u_i^{j+1}\right\} + (1-\theta)^2\left\{(u_{i+1}^{j})^2 - (u_i^{j})^2\right\}\Bigg]$$

$$+\,\theta\Bigg[\frac{2g\Delta t}{\Delta x}\left(h_{i+1}^{j+1} - h_i^{j+1}\right) + gn^2\Delta t\left\{\frac{u_{i+1}^{j+1}\,|\,u_{i+1}^{j+1}\,|}{(h_{i+1}^{j+1})^{4/3}}\right.$$

$$\left. + \frac{u_i^{j+1}\,|\,u_i^{j+1}|}{(h_i^{j+1})^{4/3}}\right\}$$

$$+\,\Delta t\left\{\frac{q_{i+1}^{j+1}\quad u_{i+1}^{j+1}}{h_{i+1}^{j+1}} + \frac{q_i^{j+1}\quad u_i^{j+1}}{h_i^{j+1}}\right\}\Bigg] + a_i = 0 \quad \ldots \quad (3.8)$$

Where

$$a_i = (1-\theta)\left[\frac{2g\Delta t}{\Delta x} (h^j_{i+1} - h^j_i) + gn^2\Delta t\left\{ \frac{u^j_{i+1} | u^j_{i+1} |}{(h^j_{i+1})^{4/3}} + \frac{u^j_i \quad u^j_i}{(h^j_i)^{4/3}} \right\} + \Delta t\left\{ \frac{q^j_{i+1} \; u^j_{i+1}}{h^j_{i+1}} + \frac{q^j_i \; u^j_i}{h^j_i} \right\} \right] - (u^j_{i+1} + u^j_i + 2gS_o\Delta t)$$

The equations 3.7 and 3.8 are two nonlinear algebraic equations in which all the variables with superscript 'j' are known and all the variables with superscript j+1 are unknowns. Thus, the unknowns are u^{j+1}_i, u^{j+1}_{i+1}, h^{j+1}_i and h^{j+1}_{i+1}. Since there are N nodal points on the $(j+1)^{th}$ row, there would be (N-1) rectangular grids and (N-1) interior nodal points. Therefore, the total number of equations are 2(N-1) with the following 2N unknowns.

$$u^{j+1}_i \quad , \; i=1,2,\ldots.,N$$

and

$$h^{j+1}_i \quad , \; i=1,2,\ldots.,N$$

The variables (h^{j+1}_i and u^{j+1}_i, i= 1,2,...,N) in equations 3.7 and 3.8 are replaced by a single variable (x_i, i=1,2,....,2N) to facilitate a systematic solution. Similarly, the functions (ξ_i, i=1,2,...,N) and (ψ_i ,i=1,2,...,N) are replaced by functions (F_i, i=1,2,...,2N). Thus, the variables and the equations are redefined as follows.

$$\left.\begin{array}{llll} x_1 &= h_1^{j+1} & x_2 &= u_1^{j+1} \\ \vdots & & \vdots & \\ x_{2i-1} &= h_i^{j+1} & x_{2i} &= u_i^{j+1} \\ x_{2i+1} &= h_{i+1}^{j+1} & x_{2i+2} &= u_{i+1}^{j+1} \\ \vdots & & \vdots & \\ x_{2N-1} &= h_N^{j+1} & x_{2N} &= u_n^{j+1} \end{array}\right\} \quad i = 1,2,3,\ldots N-1 \qquad \ldots (3.9)$$

$$F_{2i} = x_{2i+2} + x_{2i-1} + \frac{2\Delta t}{\Delta x}\Bigg[\theta^2\left\{x_{2i+2}x_{2i+1} - x_{2i}x_{2i-1}\right\}$$

$$+\theta(1-\theta)\left\{x_{2i+2}h_{i+1}^j - x_{2i}h_i^j + u_{i+1}^j x_{2i+1} - u_i^j x_{2i-1}\right\}$$

$$+(1-\theta)^2\left\{u_{i+1}^j h_{i+1}^j - u_i^j h_i^j\right\}\Bigg] - h_{i+1}^j - h_i^j - 2\Delta t\, Q_i = 0 \qquad \ldots (3.10)$$

$i = 1, 2, \ldots, N-1$

and

$$F_{2i+1} = x_{2i+2} + x_{2i} + \frac{\Delta t}{\Delta x}\Bigg[\theta^2\,(x_{2i+1}^2 - x_{2i}^2)$$

$$+2\theta(1-\theta)\left\{x_{2i+2}\,u_{i+1}^j - u_i^j\,x_{2i}\right\} + (1-\theta)^2\left\{(u_{i+1}^j)^2 - (u_i^j)^2\right\}\Bigg]$$

$$+\ \theta\left[\frac{2g\Delta t}{\Delta x}(x_{2i+1} - x_{2i-1}) + gn^2\Delta t\left\{\frac{x_{2i+2}\,|x_{2i+2}|}{x_{2i+1}^{4/3}}\right.\right.$$

$$+ \frac{x_{2i} \mid x_{2i} \mid}{x_{2i-1}^{4/3}} \Bigg\} + \Delta t \left\{ \frac{q_{i+1}^{j+1} \; x_{2i+2}}{x_{2i+1}} + \frac{q_i^{j+1} \; x_{2i}}{x_{2i-1}} \right\} \Bigg]$$

$$+ (1-\theta) \left[\frac{2g\Delta t}{\Delta x} (h_{i+1}^{j} - h_i^{j}) + gn^2 \Delta t \left\{ \frac{u_{i+1}^{j} \mid u_{i+1}^{j} \mid}{(h_{i+1}^{j})^{4/3}} + \frac{u_i^{j+1} \mid u_i^{j+1} \mid}{(h_i^{j+1})^{4/3}} \right\} \right.$$

$$\left. + \Delta t \left\{ \frac{q_{i+1}^{j} \; u_{i+1}^{j}}{h_{i+1}^{j}} + \frac{q_i^{j} \; u_i^{j}}{h_i^{j}} \right\} \right] - (u_{i+1}^{j} + u_i^{j} + 2gS_o \Delta t) = 0 \quad \ldots \; (3.11)$$

$$i = 1, 2, \ldots, N-1$$

The system of (2N-2) equations defined by 3.10 and 3.11 is indeterminate in nature since there are 2N unknowns. Two more equations are obtained by defining the upstream and downstream boundary conditions. One such equation is obtained as follows if the stage is known at the upstream boundary as a function of time.

$$F_1(x_1) = x_1 - h_1^{j+1} = 0 \quad \ldots \; (3.12)$$

where h_1^{j+1} is the ordinate of stage hydrograph at time j+1. The other equation is derived from the stage-velocity relationship at the downstream boundary and is of the following type :

$$F_{2N} = x_{2N} - (A_o + A_1 \, x_{2N-1} + A_2 \, x_{2N-1}^2 + A_3 \, x_{2N-1}^3) = 0 \quad \ldots \; (3.13)$$

This can be used as the second supplementary equation provided by the downstream boundary. Where A_o, A_1, A_2 and A_3 are constant coefficients. Thus, the equations 3.10 and 3.11 along with equations 3.12 and 3.13

lead to a determinate system of nonlinear equations.

Using the function $F_{2i}(x)$ as the one representing the continuity equation and function $F_{2i+1}(x)$ as representing the momentum equation, the 2N equations obtained in the previous section can be reassembled as under.

$$F_1(x_1) = 0 \qquad \text{... (3.14a)}$$

$$F_2(x_1, x_2, x_3, x_4) = 0 \qquad \text{... (3.14b)}$$

$$F_3(x_1, x_2, x_3, x_4) = 0 \qquad \text{... (3.14c)}$$

.

.

$$F_{2i}(x_{2i-1}, x_{2i}, x_{2i+1}, x_{2i+2}) = 0 \qquad \text{... (3.14d)}$$

$$F_{2i+1}(x_{2i-1}, x_{2i}, x_{2i+1}, x_{2i+2}) = 0 \qquad \text{... (3.14e)}$$

.

.

$$F_{2(N-1)}(x_{2(N-1)-1}, x_{2(N-1)}, x_{2(N-1)+1}, x_{2(N-1)+2}) = 0 \qquad \text{... (3.14f)}$$

$$F_{2N-1}(x_{2(N-1)-1}, x_{2(N-1)}, x_{2(N-1)+1}, x_{2(N-1)+2}) = 0 \qquad \text{... (3.14g)}$$

$$F_{2N}(x_{2N-1}, x_{2N}) = 0 \qquad \text{... (3.14h)}$$

Equations 3.14b, 3.14d and 3.14f represent the continuity equation. Equations 3.14c, 3.14e and 3.14g represent momentum equation. Equations 3.14a and 3.14h are the upstream and downstream boundary conditions respectively.

In the present study these equations are solved by Newton Raphson's algorithm (Hildebrand, 1956). In this algorithm, starting from the assigned trial values, the unknowns are sequentially modified systematically.

3.3.3 Solution Procedure for The Finite Difference Equations

Following the Newton Raphson's algorithm the values of the unknowns (stage and velocity) are obtained by systematic adjustment of the variables until the residuals are below a specified tolerance level. This method of solution can be illustrated by assuming that iteration has been carried out k times and it is required to improve the solution through the $(k+1)^{th}$ iteration. The superscripts k and k+1 are used to denote the values of the unknowns after the k^{th} and $(k+1)^{th}$ iterations respectively. The values of depth and velocity obtained in the k^{th} iteration as substituted into equation 3.14. Representing the residuals from equation 3.14 as Rc^k and R_m^k where subscripts 'c' and 'm' indicate residuals from functions F_{2i} (viz. continuity equation) and function F_{2i+1} (viz. momentum equation) respectively, the residuals are given by:

$$F_1(x_1^k, x_2^k) = R_{m,1}^k \qquad \text{... (3.15a)}$$

$$F_2(x_1^k, x_2^k, x_3^k, x_4^k) = R_{c,2}^k \qquad \text{... (3.15b)}$$

$$F_3(x_1^k, x_2^k, x_3^k, x_4^k) = R_{m,3}^k \qquad \text{... (3.15c)}$$

.

.

$$F_{2i}(x_{2i-1}^k, x_{2i}^k, x_{2i+1}^k, x_{2i+2}^k) = R_{c,2i}^k \qquad \text{... (3.15d)}$$

$$F_{2i+1}(x_{2i-1}^k, x_{2i}^k, x_{2i+1}^k, x_{2i+2}^k) = R_{m,2i+1}^k \qquad \text{... (3.15e)}$$

.

.

.

$$F_{2(N-1)}(x_{2(N-1)-1}^k, x_{2(N-1)}^k, x_{2(N-1)+1}^k, x_{2(N-1)+2}^k) = R_{c,2N-2}^k \qquad \text{... (3.15f)}$$

$$F_{2N-1}(x_{2(N-1)-1}^k, x_{2(N-1)}^k, x_{2(N-1)+1}^k, x_{2(N-1)+2}^k) = R_{m,2N-1}^k \qquad \text{... (3.15g)}$$

$$F_{2N}(x^k_{2N-1}, x^k_{2N}) = R^k_{c,2N} \quad \ldots \text{ (3.15h)}$$

According to the Newton Raphson's algorithm the unknowns at the end of $(k+1)^{th}$ iteration are given by the following equalities.

$$x^{k+1}_i = x^k_i + \delta x_i \text{ , } i=1,2,\ldots,2N \quad \ldots \text{ (3.16)}$$

The values of δx_i are obtained by solving the following set of linear equations.

$$\frac{\partial F_1}{\partial x_1}\delta x_1 + \frac{\partial F_1}{\partial x_2}\delta x_2 = - R^k_{m,1} \quad \ldots \text{(3.17a)}$$

$$\frac{\partial F_2}{\partial x_1}\delta x_1 + \frac{\partial F_2}{\partial x_2}\delta x_2 + \frac{\partial F_2}{\partial x_3}\delta x_3 + \frac{\partial F_2}{\partial x_4}\delta x_4 = - R^k_{c,2} \quad \ldots \text{(3.17b)}$$

$$\frac{\partial F_3}{\partial x_1}\delta x_1 + \frac{\partial F_3}{\partial x_2}\delta x_2 + \frac{\partial F_3}{\partial x_3}\delta x_3 + \frac{\partial F_3}{\partial x_4}\delta x_4 = - R^k_{m,3} \quad \ldots \text{(3.17c)}$$

.

.

.

$$\frac{\partial F_{2i}}{\partial x_{2i-1}}\delta x_{2i-1} + \frac{\partial F_{2i}}{\partial x_{2i}}\delta x_{2i} + \frac{\partial F_{2i}}{\partial x_{2i+1}}\delta x_{2i+1} +$$

$$\frac{\partial F_{2i}}{\partial x_{2i+2}}\delta x_{2i+2} = - R^k_{c,2i} \quad \ldots \text{ (3.17d)}$$

$$\frac{\partial F_{2i+1}}{\partial x_{2i-1}} \delta x_{2i-1} + \frac{\partial F_{2i+1}}{\partial x_{2i}} \delta x_{2i} + \frac{\partial F_{2i+1}}{\partial x_{2i+1}} \delta x_{2i+1} + \frac{\partial F_{2i+1}}{\partial x_{2i+2}} \delta x_{2i+2} = - R^{k}_{m,2i+1} \quad \ldots (3.17e)$$

.

.

.

$$\frac{\partial F_{2(N-1)}}{\partial x_{2(N-1)-1}} \delta x_{2(N-1)-1} + \frac{\partial F_{2(N-1)}}{\partial x_{2(N-1)}} \delta x_{2(N-1)} + \frac{\partial F_{2(N-1)}}{\partial x_{2(N-1)+1}} \delta x_{2(N-1)+1} + \frac{\partial F_{2(N-1)}}{\partial x_{2(N-1)+2}} \delta x_{2(N-1)+2} = - R^{k}_{c,2N-2} \quad \ldots (3.17f)$$

$$\frac{\partial F_{2N-1}}{\partial x_{2(N-1)-1}} \delta x_{2(N-1)-1} + \frac{\partial F_{2N-1}}{\partial x_{2(N-1)}} \delta x_{2(N-1)} + \frac{\partial F_{2N-1}}{\partial x_{2(N-1)+1}} \delta x_{2(N-1)+1} + \frac{\partial F_{2N-1}}{\partial x_{2N}} \delta x_{2N} = - R^{k}_{m,2N-1} \quad \ldots (3.17g)$$

$$\frac{\partial F_{2N}}{\partial x_{2N-1}} \delta x_{2N-1} + \frac{\partial F_{2N}}{\partial x_{2N}} \delta x_{2N} = - R_{c,2N} \quad \ldots (3.17h)$$

Equations 3.17 is a system of 2N linear equations involving 2N unknowns. For the solution of the system of equations (viz. equations 3.17(a) through (h)) the matrix inversion procedure is used. The solution of the system will provide values of the following.

$$\delta x_1^{k+1},\ \delta x_2^{k+1},\delta x_3^{k+1}\ ,\ldots,\ \delta x_{2N-1}^{k+1},\ \delta x_{2N}^{k+1}$$

On adding the values of δx_i, $i = 1,2,3,\ldots, 2N$ to their corresponding values of x_i^k, i = 1,2,3,....,2N, the values of the unknowns at the $(k+1)^{th}$ iteration cycle are obtained. The partial derivatives appearing in equation 3.17 are computed by differentiating equations 3.10 , 3.11, 3.12 and 3.13.

Thus,

$$\frac{\partial F_1}{\partial x_1} = 1 \qquad \ldots (3.18)$$

$$\frac{\partial F_{2i}}{\partial x_{2i-1}} = 1 - \frac{2\Delta t}{\Delta x}\left\{\theta^2 x_{2i} + \theta(1-\theta)u_i^j\right\} \qquad \ldots (3.19)$$

$$\frac{\partial F_{2i}}{\partial x_{2i}} = -\frac{2\Delta t}{\Delta x}\left\{\theta^2 x_{2i-1} + \theta(1-\theta)h_i^j\right\} \qquad \ldots (3.20)$$

$$\frac{\partial F_{2i}}{\partial x_{2i+1}} = 1 + \frac{2\Delta t}{\Delta x}\left\{\theta^2 x_{2i+2} + \theta(1-\theta)u_{i+1}^j\right\} \qquad \ldots (3.21)$$

$$\frac{\partial F_{2i}}{\partial x_{2i+2}} = \frac{2\Delta t}{\Delta x}\left\{\theta^2 x_{2i+1} + \theta(1-\theta)h_{i+1}^j\right\} \qquad \ldots (3.22)$$

$$\frac{\partial F_{2i+1}}{\partial x_{2i-1}} = \theta\left[-\left(\frac{2g\Delta t}{\Delta x} + \left(\frac{4}{3}\right) gn^2\Delta t \frac{x_{2i}|x_{2i}|}{x_{2i-1}^{7/3}} + \frac{\Delta t\ q_i\ x_{2i}}{x_{2i-1}^2}\right)\right] \qquad \ldots (3.23)$$

$$\frac{\partial F_{2i+1}}{\partial x_{2i}} = 1 - \frac{2\ \Delta t}{\Delta x}\left\{\theta^2 x_{2i} + \theta(1-\theta)u_i^j\right\} + \theta\left\{gn^2\Delta t + \frac{2x_{2i}}{x_{2i-1}^{4/3}} + \Delta t\ \frac{q_i^{j+1}}{x_{2i-1}}\right\}$$

$$\ldots (3.24)$$

$$\frac{\partial F_{2i+1}}{\partial x_{2i+1}} = \theta \left\{ \frac{2g\Delta t}{\Delta x} - \frac{4}{3} gn^2 \Delta t \frac{x_{2i+2}|x_{2i+2}|}{x_{2i+1}^{7/3}} - \Delta t \frac{q_{i+1}^{j+1} x_{2i+2}}{x_{2i+1}^{2}} \right.$$

$$\dots (3.25)$$

$$\frac{\partial F_{2i+1}}{\partial x_{2i+2}} = 1 + \frac{\Delta t}{\Delta x} \left\{ \theta^2 2x_{2i+2} + 2\theta(1-\theta) u_{i+1}^{j} \right\}$$

$$+ \theta \left\{ gn^2 \frac{2x_{2i+2}}{x_{2i+1}^{4/3}} + \Delta t \frac{q_{i+1}^{j+1}}{x_{2i+1}} \right\} \quad \dots (3.26)$$

$$\frac{\partial F_{2N}}{\partial x_{2N-1}} = -(A_1 + 2A_2 x_{2N-1} + 3A_3 x_{2N-1}^2) \quad \dots (3.27)$$

$$\frac{\partial F_{2N}}{\partial x_{2N}} = 1 \quad \dots (3.28)$$

By substituting equations 3.18 through 3.28 in equations 3.17(a) through 3.17(h), the values of the unknowns, δx_i are obtained thus computing the values of x_i^{k+1}, i = 1, 2, ..., 2N for a time line j+1.

Convergence

The iterative procedure outlined above is continued till the following convergence criteria are satisfied.

$$| x_{2i-1}^{(k+1)} - x_{2i-1}^{(k)} | \le \varepsilon_1$$

$$| x_{2i}^{(k+1)} - x_{2i}^{(k)} | \le \varepsilon_2$$

Where ε_1 and ε_2 are the tolerance levels in respect of depth and velocity respectively. when these convergence criteria are satisfied the values of $x_i^{(k+1)}$ will comprise of the values of u_i^{j+1} and h_i^{j+1}, i = 1,2,...., N.

3.3.4 Initial Conditions

Following the procedure outlined in the proceeding section, the space-distributed depths (h_i^{j+1}) and the velocities (u_i^{j+1}) at $(j+1)^{th}$ time level can be computed provided the corresponding distribution at the j^{th} time [i.e. (h_i^j) and (u_i^j)] are known/ assigned. Thus, the values of velocity 'u' and depth 'h' at all sections should be known at the beginning of a time step to advance the solution to the end of the time step. For solution at $t = t_o + \Delta t$, all the depth and velocity values should be known at $t = t_o$. Therefore, to initialize the solution, the space distribution of the depth and velocity at the beginning of the simulation [i.e. (h_i^1)] and (u_i^1), i = 1,2,..., N] must be assigned. These are termed as Initial Conditions. In a subsequent time step the solution arrived at in the preceding time step would mathematically serve as the initial conditions.

3.3.5 Dynamic Wave Celerity

In the dynamic channel flow phenomenon, the flow characteristics have two directions (viz. forward and backward relative to the medium). The flow in which the inertia and gravity forces are as large as friction force or more the effect of the wave propagates in upstream and downstream directions of flow. In such flows the wave velocity (or wave celerity) relative to the bank is expressed as

$$C = u \pm \sqrt{gh} \qquad \text{... (3.29)}$$

Equation 3.29 expresses the celerity of positive and negative waves developed in water moving along the channel with the mean velocity u.

The study of wave velocity through the equation 3.29 shows that the celerity of a wave consists of two velocity components. One component is u, i.e. mean velocity of water and the other is $\sqrt{gh}$, which is the velocity with which a disturbance in open channel flow tends to move over the surface. Thus, celerity of a wave can also be expressed as

$$C = \sqrt{gh} \qquad \ldots (3.30)$$

The wave celerity expressed by equation 3.30 is used as a criterion of flow regime. It defines the critical state (u=C) and for $u > C$ the flow is said to be supercritical and if $u < C$ the flow is said to be subcritical. In the present work the relation 3.30 is utilized in the analysis of channel flow phenomena. The Froude number 'Fr' is thus the measure of state of flow which is the ratio of mean flow velocity to the wave velocity i.e.

$$Fr = \frac{u}{\sqrt{gh}} \qquad \ldots (3.31)$$

As the channel flow simulation is carried out implicitly for all the computational nodes on the channel for a time step, the flow parameters i.e. depth and velocity of flow are computed. Thus, analysis of flow profile can be carried out longitudinally along the channel at the end of each computational time interval. Using equations 3.30 and 3.31 and the computed flow parameters (depth and velocity), the state of flow can be also analyzed for that particular time step. Utilizing the computed value of velocity and depth of flow at the outlet, discharge per unit width can be obtained.

3.4 KINEMATIC WAVE MODEL (OVERLAND FLOW MODEL)

The kinematic flow approximation has proved to be a useful tool in storm water modeling. Starting with formulation of kinematic wave theory by Lighthill and Whitham (1955), KW overland flow models have been increasingly employed in hydrological investigations. Woolhiser and Liggett (1967) found it as an accurate approximation of the St. Venant's equations for most of the overland flow cases. Keeping in mind the recommendations of these researchers, for this study, it has been decided to adopt the KW theory for the routing of flows in its overland plane.

3.4.1 The Governing Equations of Kinematic Wave Flow

In deriving the kinematic wave equations it is assumed that the overland flow is in the form of sheet flow. A unit width of the rectangular plane is considered for the runoff generation in computational schemes. The KW equations can, therefore, be written as follow.

$$\frac{\partial h_o}{\partial t} + \frac{\partial q_o}{\partial x} = q_i \qquad \dots \qquad (3.32)$$

Where q_i is the net lateral inflow (m/sec) in the form of rainfall excess intensity and q_o is outflow from a plane (per unit width), h_o is the corresponding overland flow depth. With the assumptions of KW theory which were discussed at length in the previous Chapter, the momentum equation (i.e equation 3.2) is simplified by equating the friction slope (S_f) to the bed slope (S_o) written under.

$$S_f = S_o \qquad \dots \qquad (3.33)$$

This indicates uniform, steady state nature of flow. Using the Manning's friction law as well as the equation 3.33, the depth-discharge relationship is expressed as under.

$$q_o = \alpha\, h_o^m \qquad \ldots \quad (3.34)$$

where α and m are 'kinematic routing parameters' which are directly related to conveyance of a particular surface (i.e. to the slope and roughness).

The overland flow on a plane is considered as a shallow flow. Therefore, the kinematic wave equation (3.34) along with the Manning's equation for discharge per unit width (q_o) is written as under.

$$q_o = \frac{1}{n} A_o R_0^{2/3} S_o^{1/2} \qquad \ldots \quad (3.35)$$

Where,

A_o is cross section of flow area of the plane

R_o is the hydraulic radius. The other terms are defined in the previous sections.

For a sheet flow on a rectangular plane of unit width, A_o and R_o are replaced by h_o i.e. the mean depth of flow. Substituting these values in equation 3.35, the discharge per unit width (q_o) can be written as:

$$q_o = \frac{1}{n} h_o^{5/3} S_o^{1/2} \qquad \ldots \quad (3.36)$$

Comparison of equations 3.34 and 3.36 reveals that KW parameter α and m may be expressed in terms of S_o and n_o as given under.

$$\alpha = \frac{\sqrt{S_o}}{n_o} \qquad \ldots \quad (3.37)$$

and

$$m = 5/3 \qquad \ldots \quad (3.38)$$

The parameters n_o and S_o being known, the values of α is worked out using equation 3.37.

3.4.2 The Final Form of KW Equations for the Overland Flow

Combining equations 3.32 and 3.34, the following complete form of KW equation for overland flow is arrived at.

$$\frac{\partial h_o}{\partial t} + \alpha m\, h_o^{m-1} \frac{\partial h_o}{\partial x} = q_i \qquad \ldots \quad (3.39)$$

In equation 3.39, h_o is a dependent variable and it is a function of x, t and rainfall excess intensity (q_i). Thus, h_o can be determined explicitly by using equation 3.39. From the computed values of h_o, the overland surface runoff per unit width (q_o) can be computed by using equation 3.34.

For the numerical solution of equation 3.39, an appropriate numerical technique need be employed.

3.4.3 Numerical Techniques for Solving the KW Equations

Different numerical techniques employed for solving KW equations have been discussed in Section 2.7 (Chapter-II). In this study, the second order Lax-Wendroff explicit scheme (Lax et al., 1960) has been preferred for solving the KW equations characterizing the flow of water in its overland phase. Therefore, application of this technique in the context of this work has been discussed in further details.

As shown in Figure 3.3, the flow domain is characterized by x i.e. for spatial coordinates and t i.e. for time coordinates. In this plane, the lines parallel to x-axis are the 'time-lines', where as the lines parallel to t-axis are the 'space-lines'. The solution grid are defined with steps of Δx and Δt in the x-t directions respectively. The Δx and Δt values are being kept constant in the present case. Thus, the grid consists of the set of lines parallel to t-axis and are given by

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

where,

$$x_i = i\Delta x$$

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

$$t = t_j, \qquad j = 0,1,2,3,.....$$

where

$$t_j = j\Delta t$$

Therefore, the point ($i\Delta t$, $j\Delta t$) is called the grid point (i,j) and is surrounded in the neighbourhood by other grid points. Solution of the governing equations through the finite difference scheme thus will be obtained at each of these grid points. Computations advance along the downstream direction for a time step Δt, until all water depths are computed at all the grid points in the entire longitudinal length (Figure 3.3). At all points on the time line, the equation 3.34 can be used for flow simulation.

Likewise the computations can be extended for another time step Δt and the process to continue.

The rainfall excess intensity (q_i) is assumed to be constant within a time step Δt, but may change from one time step to the next time step. This helps in accounting for the variation in rates of rainfall excess intensities occurring within a storm event, temporally as well as spatially.

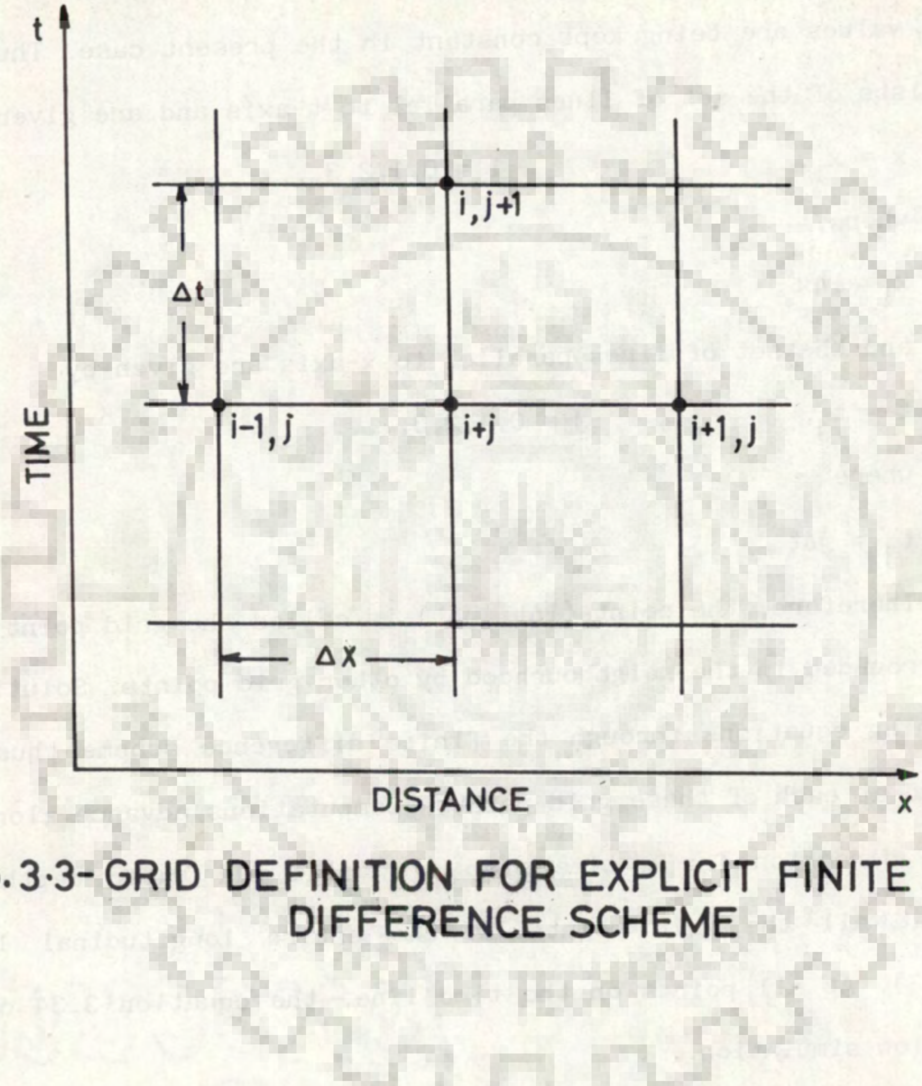


FIG. 3·3- GRID DEFINITION FOR EXPLICIT FINITE DIFFERENCE SCHEME

3.4.4 Approximation of the Derivatives of KW Equations Through Finite Differences

The flow depth at any point x, at a given instant t, is written as h(x,t). Considering h(x,t) and its derivative $\frac{\partial h}{\partial x}$ in equation 3.39 and using the Taylor's series expansion, knowing the value of $h_o(x,t)$ at time t the value of $h(x,t+\Delta t)$ can be found out.

Expanding h(x,t) through Taylor's series, the terms $h(x,t+\Delta t)$ is written as:

$$h(x,t+\Delta t) = h(x,t) + \frac{\partial h}{\partial t}\Delta t + \frac{\partial^2 h}{\partial t^2}\frac{\Delta t^2}{2!} + HOT \quad \ldots (3.40)$$

where HOT denotes 'Higher Order Terms'.

When $h(x,t) = h_o(x,t)$, then equation 3.39 can be written as:

$$\frac{\partial h}{\partial t} = -\alpha \frac{\partial h^m}{\partial x} + q_i \quad \ldots (3.41)$$

Differentiating equation 3.41 with respect to t, and multiplying by $\Delta t^2/2$ then

$$\frac{\Delta t^2}{2}\frac{\partial^2 h}{\partial t^2} = \frac{\Delta t^2}{2}\left[-\alpha \frac{\partial}{\partial x}(mh^{m-1}\frac{\partial h}{\partial t}) + \frac{\partial q_i}{\partial t}\right] \quad \ldots (3.42)$$

Substituting equation 3.41 in 3.42, the following is obtained.

$$\frac{\Delta t^2}{2}\frac{\partial^2 h}{\partial t^2} = \frac{\Delta t^2}{2}\left[\alpha \frac{\partial}{\partial x}\left\{mh^{m-1}(\alpha \frac{\partial h^m}{\partial x} - q_i)\right\} + \frac{\partial q_i}{\partial t}\right] \quad \ldots (3.43)$$

Substituting equations 3.41 and 3.43 in equation 3.40, the following is obtained

$$h(x,t+\Delta t) = h(x,t)-\Delta t\left\{ \alpha \frac{\partial h^m}{\partial x} - q_i\right\}$$

$$+ \frac{\Delta t^2}{2} \left[m\alpha \frac{\partial}{\partial x}\left\{h^{m-1}(\alpha \frac{\partial h^m}{\partial x} - q_i)\right\} + \frac{\partial q_i}{\partial t} \right] \quad \ldots (3.44)$$

Following the notations of Figure 3.3, equation 3.44 can be written in the finite difference form as under. (from here onwards 'i' indicates the location of a point in x-direction)

$$h_i^{j+1} = h_i^j - \Delta t \left\{ \alpha \left[\frac{{h_{i+1}^j}^m - {h_{i-1}^j}^m}{2\Delta x}\right] - q_i^j \right\}$$

$$+ \frac{m\alpha}{4} \frac{\Delta t^2}{\Delta x}\left[\left[{h_{i+1}^j}^{m-1} + {h_i^j}^{m-1}\right] \left\{\alpha \left[\frac{{h_{i+1}^j}^m - {h_i^j}^m}{\Delta x}\right] - \left[\frac{q_{i+1}^j - q_i^j}{2}\right]\right\}\right.$$

$$- \left[{h_i^j}^{m-1} + {h_{i-1}^j}^{m-1}\right] \left\{\alpha \left[\frac{{h_i^j}^m - {h_{i-1}^j}^m}{\Delta x}\right] - \left[\frac{q_i^j + q_{i-1}^j}{2}\right]\right\}$$

$$\left. + \frac{2\Delta x}{m\alpha\Delta t} \left\{q_i^{j+1} - q_i^j\right\}\right] \quad \ldots (3.45)$$

Equation 3.45 can provide the computed overland flow depth for points i = 2,3,4,...., N-1 of the total N number of grid points on a time line. The solution at the other two nodes are provided by the boundary conditions. The upstream boundary condition is:

$$h(0,t) = 0 \qquad \ldots (3.46)$$

The depth of flow at the outlet (i=N) can be obtained using the first order scheme:

$$h_N^{j+1} = h_N^j + \Delta t \left[\alpha \left\{ \frac{(h_{N-1}^j)^m - (h_N^j)^m}{\Delta x} \right\} + q_N^j \right] \ldots (3.47)$$

Equations 3.45, 3.46, 3.47 along with initial conditions can simulate the overland flow over a plane. It is assumed that initially the surface of the plane is dry, then

$$h(x,0) = 0 \qquad \ldots (3.48)$$

The stability and convergence criteria for this scheme is well documented (Yevjevich et al., (1970), Price (1974), Yevjevich et al., (1975), Liggett et al., (1975), Singh (1976), and Field et al., (1983)).

3.4.5 Kinematic Wave Celerity

The kinematic wave celerity 'C' value will depend on 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 mechanics 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 a number of rectangular planes and a single channel in between.

As discussed earlier, for the mechanics of flows the KW celerity in its general form can be expressed as:

$$C = \frac{\partial q_o}{\partial A} \qquad \text{... (3.49)}$$

The KW celerity relationship for a unit width of the plane is obtained by making a substitution of equation 3.34 in equation 3.49 which gives the following.

$$C = \alpha m h^{m-1} \qquad \text{... (3.50)}$$

Thus, the average flow celerity may be expressed as

$$\bar{C} = \alpha \; m \; (\bar{h}\;)^{m-1} \qquad \text{... (3.51)}$$

Where $\bar{h}$ is the average flow depth over the overland plane.

Equation 3.51 gives the average flow celerity and can be used in the estimation of wave celerity over the planes. In KW theory, it is normally assumed that the flow propagates in one direction and it possesses only one wave velocity at each point. As a result of this assumption, the KW celerity also has only one component that too in the positive direction of flow.

For overland flow studies, in equation 3.50, h is the overland flow depth. Thus,at the end of each time step Δt, the computed flow depth will be available at all nodes. The wave celerity for a node is computed by assigning the corresponding value of h at that node in equation, 3.50. Applying the same equation for all the nodes the wave celerity can be computed for all the nodes along a time line. This can be a useful guide in the study of flow behaviour along the overland plane of a subwatershed.

The other wave characteristics related to flow parameters (viz applicability of KW approximation, stability and convergence of the scheme) which may not be a function of flow parameters at a single node but dependent on flow characteristics over the plane, can be studied through average celerity expressed by equation 3.51, (i.e. through the arithmetic mean of flow depth at 'N' number of nodes along the plane strip).

The physiographic parameters are assumed to be uniform over a plane. Thus, all the strips of an overland plane have the same values of α and of the outflow q_o.

The rainfall excess (q_i) generated over the overland plane is the cause of the surface runoff. This surface runoff contributes to the channel in the form of gradually varied flows. The overland flow elements are assumed to be perpendicular to the direction of channel flow. Outflows from the planes (q_o) are taken as lateral flows to the channel.

For computational purposes, the surface planes are divided into the elements of unit width called the 'overland strips'. Further, these strips of unit width and length L_o are divided into smaller segments of length Δx in 'N' number. Therefore, each strip consists of a length $L_o = N\Delta x$. By taking rainfall excess intensity q_i (i.e. gross rainfall-losses) as input within a time step Δt, overland flow simulation is performed per unit width to compute flow depths. The computation is carried out from one nodal point to another nodal point upto the last point at L_o (i.e. $L_o = N\Delta x$). The flow depth (h_o) and the corresponding discharge (q_o) so obtained have been used for the computation of the flow parameter and estimation of lateral flow to the channel.

3.5 THE PHYSIOGRAPHIC MODELS USED

In the past, open book type physiographic models have been used by various researchers (Singh 1976, Hossain 1989 etc.) for the application of KW models for distributed hydrologic systems. Such models have been found very useful in urban watersheds as well as in small and medium sized natural watersheds. As shown in Figure 3.4, the watershed is conceptually represented by two rectangular planes having the length equal to that of the main channel, and the width such that total area equals the watershed area. If the channel is located in the central part of the watershed, the two widths of the rectangular planes may be equal(Figure 3.4a). However, in cases where the main channel moves along one side of the watershed, the width of the left bank and the right bank rectangular planes may be different (Figure 3.4b). In such models the KW theory was used for routing the flows on the overland as well as in the channel.

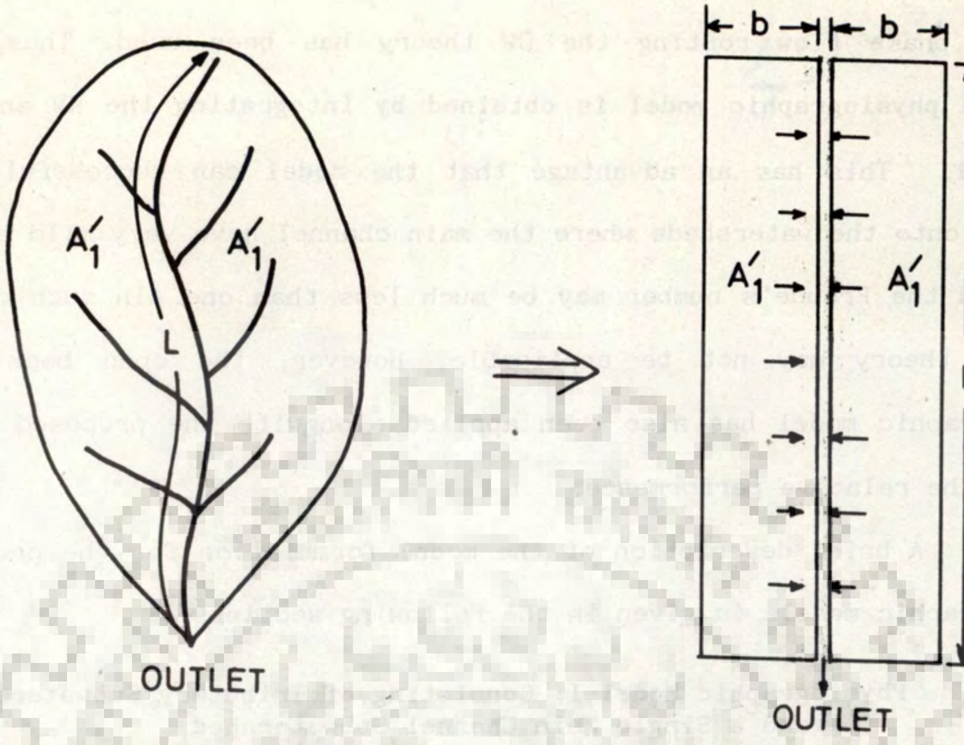
3.5.1 Proposed Physiographic Models

Since, within a natural watershed, the soil type and the landuse may vary from one part to another part alongwith the channel configurations, slopes etc., the open book type models are likely to distort the actual physiographic effects on the runoff process. This will be more true when the watersheds are not small in size. Therefore, for a comprehensive coverage of these distributed aspects, the following two models have been developed.

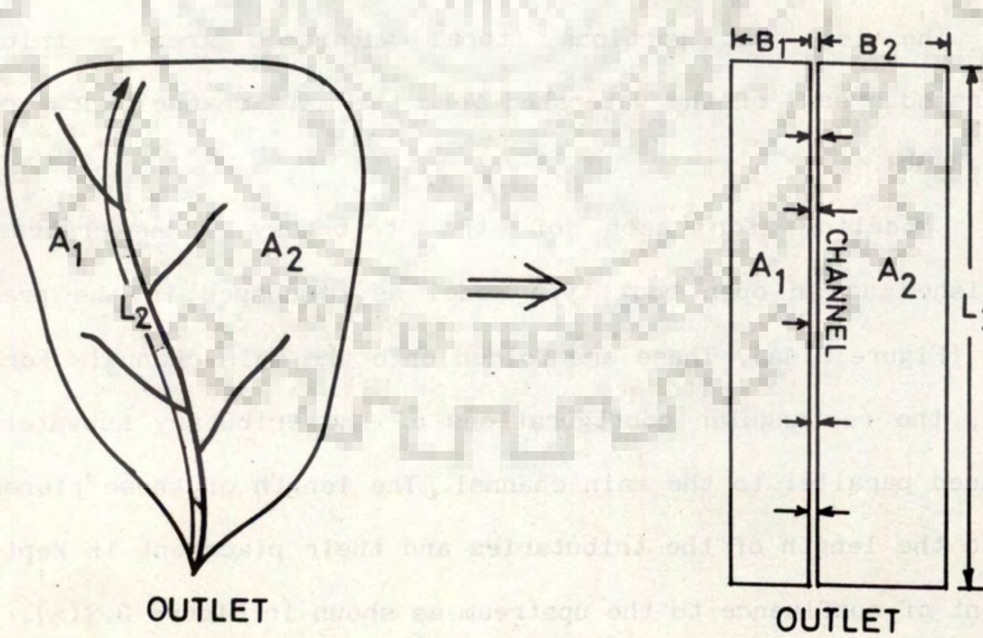
(i) Physiographic model-I; Consisting of tributary subwatersheds and a unified main channel subwatershed.

(ii) Physiographic model-II; Consisting of tributary subwatersheds and the distributed main channel subwatersheds.

For the overland surface runoff computations the KW theory is used to compute the lateral flows coming to the main channel. For the



(a) SYSTEM WITH EQUAL OVERLAND PLANES



(b) SYSTEM WITH UNEQUAL OVERLAND PLANES

FIG. 3·4 – OPEN BOOK TYPE PHYSIOGRAPHIC MODELS channel phase flow routing the DW theory has been used. Thus, the **proposed physiographic model is obtained by integrating the KW and the DW model.** This has an advantage that the model can successfully be applied onto the watersheds where the main channel have very mild slopes i.e. and the Froude's number may be much less than one. In such cases, the KW theory may not be applicable. However, the open book type physiographic model has also been applied alongwith the proposed model to see the relative performance.

A brief description of the model formulation for the proposed physiographic models is given in the following sections.

3.5.1.1 Physiographic Model-I; Consisting of Tributary Subwatersheds and a Single Main Channel Subwatershed

As shown in Figure 3.5, in the proposed model the main tributaries (T_j) are first identified. Each of the tributary subwatersheds when discretized have the areas (A_j) as shown in Figure 3.6(a). The left out portions (total watershed area - tributary subwatershed areas) of the watershed form the main channel subwatershed (Figure 3.6b).

Modeling for each of the tributary subwatersheds is accomplished as an open book type model as discussed in the previous section (Figure 3.4a). These are folded onto the main channel. For this purpose, the rectangular configurations of the tributary subwatersheds are placed parallel to the main channel. The length of these planes are equal to the length of the tributaries and their placement is kept from the point of confluence to the upstream as shown in Figure 3.7(a). Here, the main channel subwatershed (Figure 3.6b) is formed by a single rectangular plane with half of its width located on the two sides of the main channel.

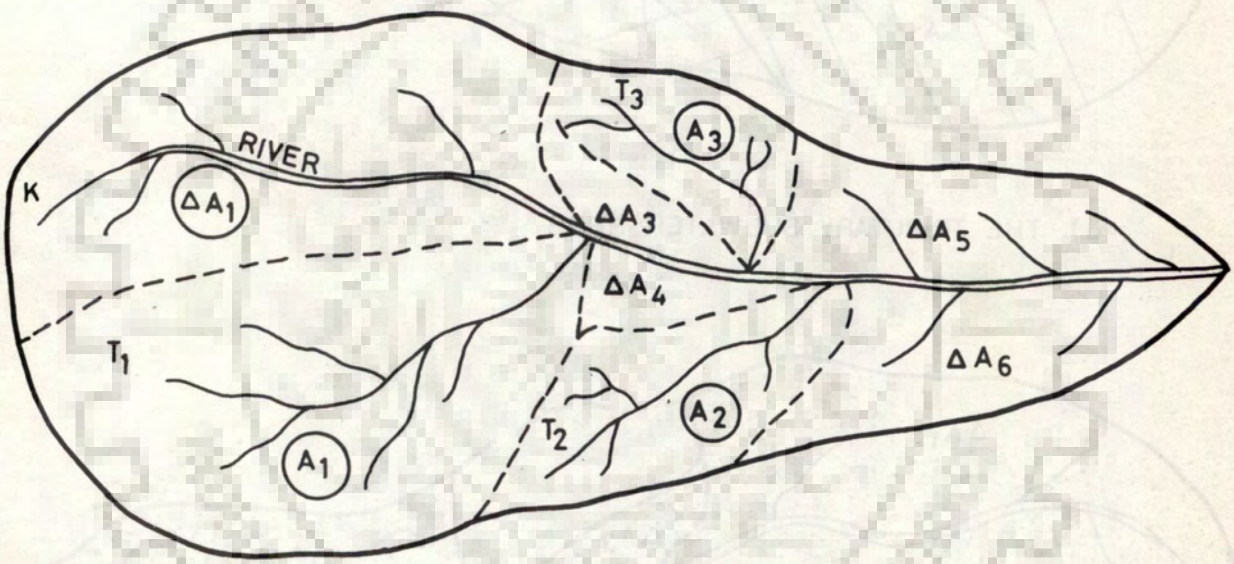
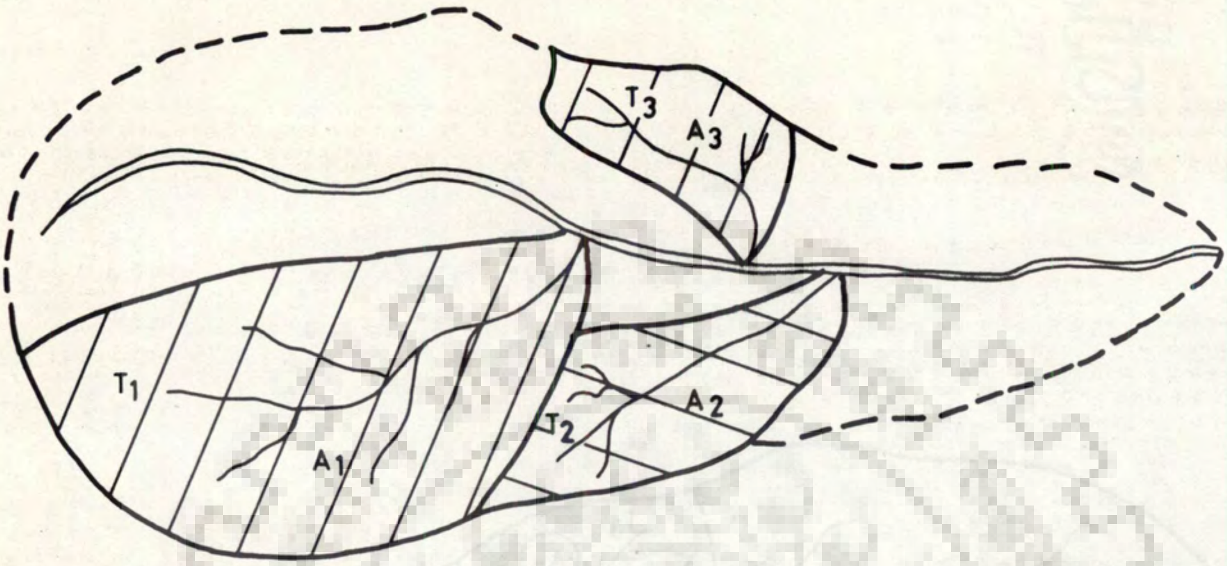
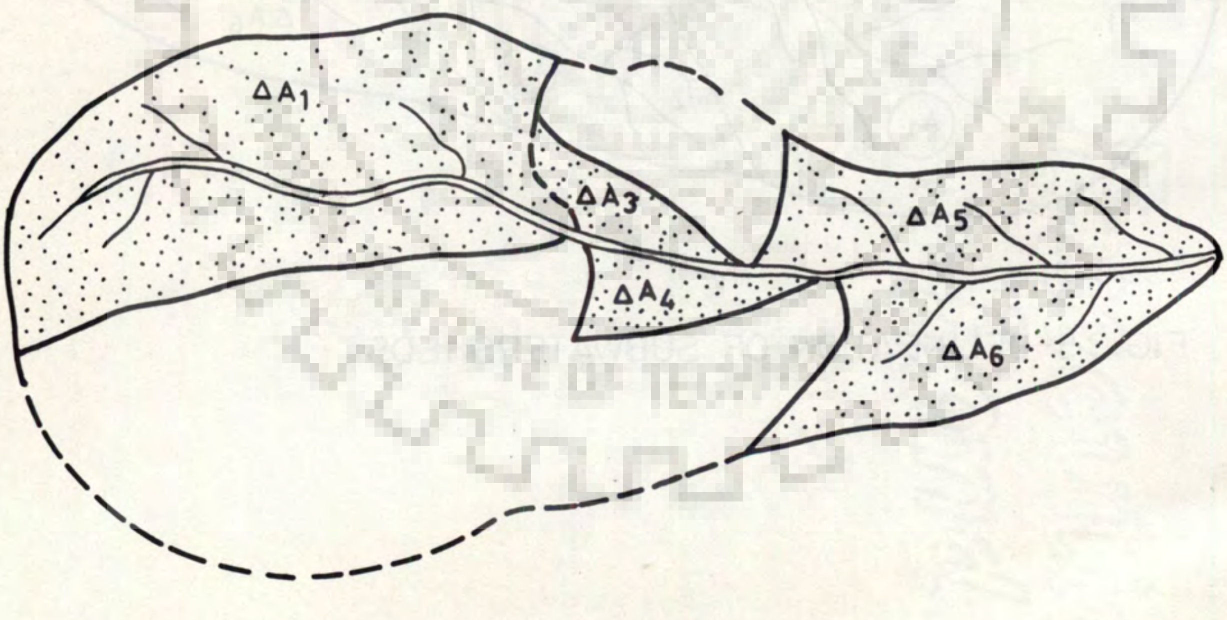


FIG. 3·5-DELINEATION OF SUBWATERSHEDS

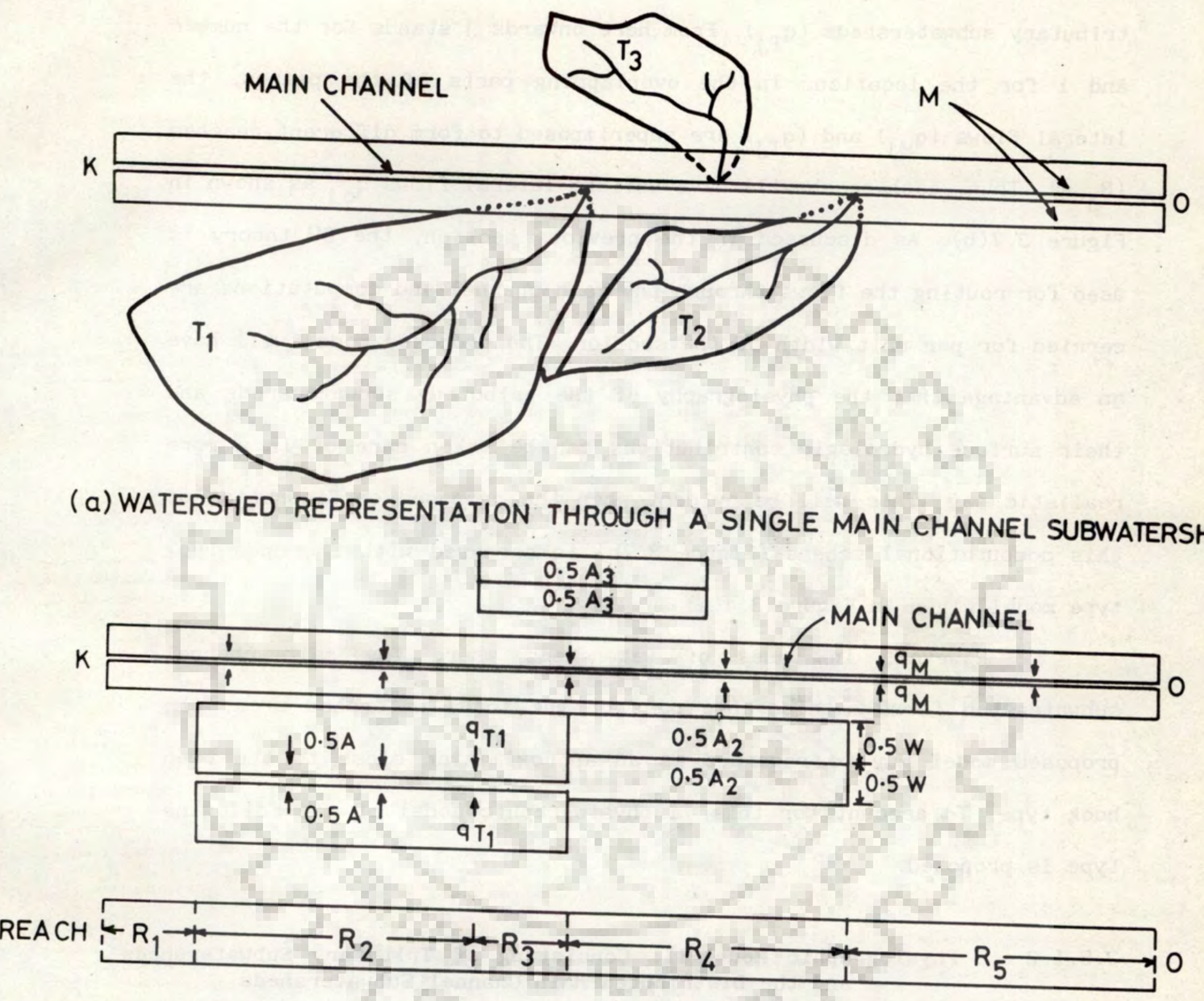


a) THE TRIBUTARY SUBWATERSHEDS



b) THE MAIN CHANNEL SUBWATERSHEDS

FIG. 3·6 - DISCRETIZATION OF WATERSHED INTO ITS SUBWATERSHEDS



(b) CONCEPTUAL REPRESENTATION OF PHYSIOGRAPHIC MODEL-I

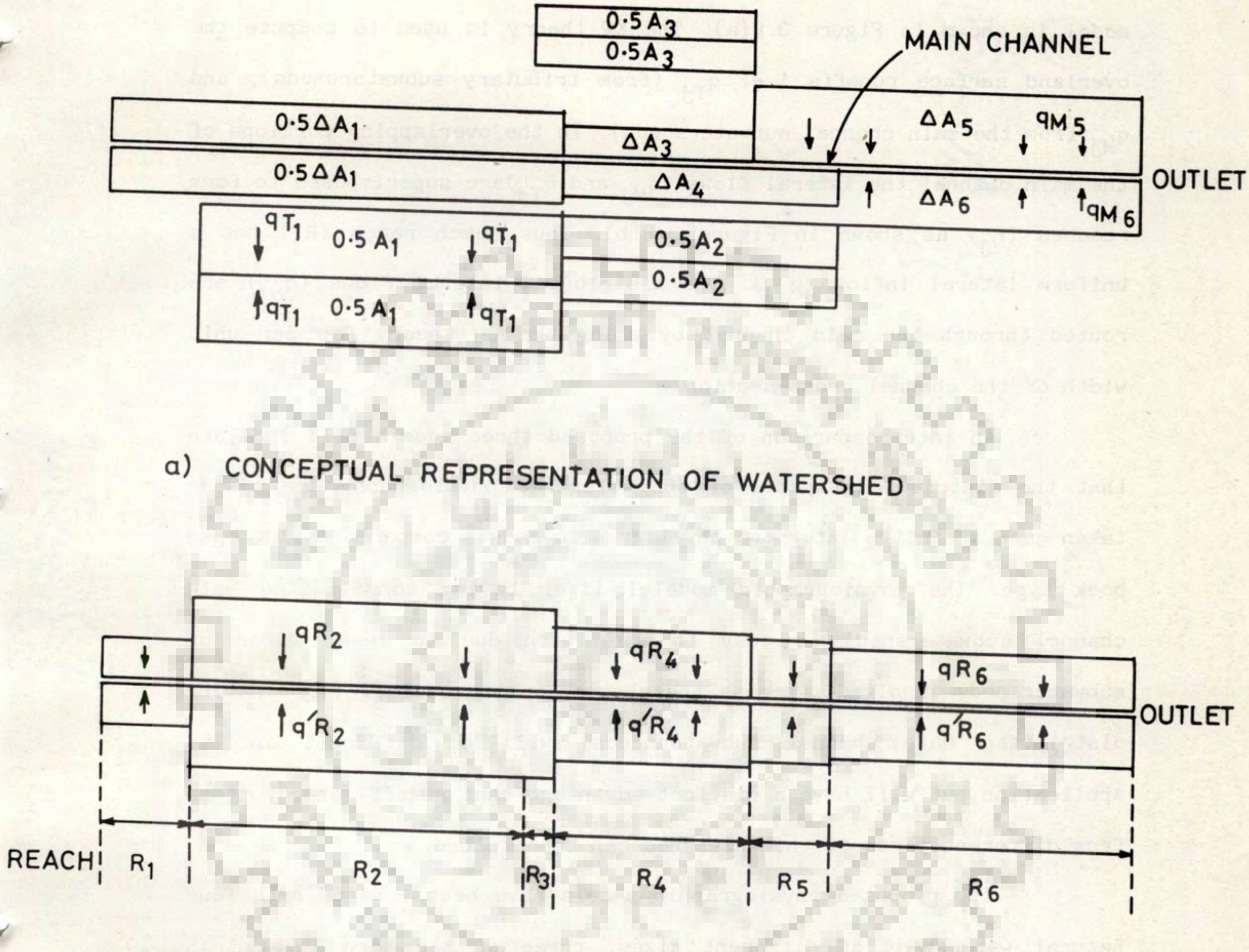
FIG. 3·7-PHYSIOGRAPHIC MODEL-I: CONSISTING OF TRIBUTARY SUBWATERSHEDS AND A SINGLE MAIN CHANNEL SUBWATERSHED

Kinematic Wave theory is used for computing the overland flows of the main channel subwatershed (i.e. q_M), as well as from the tributary subwatersheds (q_{Tj}). From here onwards j stands for the number and i for the location. In the overlapping parts of the planes, the lateral flows (q_{Mj}) and (q_{Tj}) are superimposed to form different reaches (R_j,s). Thus, each reach will have uniform lateral flows q_{oj} as shown in Figure 3.7(b). As discussed in the previous section, the DW theory is used for routing the flows through the main channel and computations are carried for per unit width of the section. The proposed model will have an advantage that the physiography of the tributary subwatersheds and their surface hydrologic contributions can be taken care of in a more realistic way. This will be evident when the proposed configuration of this computational scheme (Figure 3.7b) is compared with the open book type model given in Figure 3.4(a).

However, in cases of watersheds where the main channel subwatershed forms considerable part of the total watershed area, the proposed model may be regarded as an approximation close to the open book type. To account for this, a physiographic model of the following type is proposed.

3.5.1.2 Physiographic Model-II; Consisting of Tributary Subwatersheds and the Distributed Main Channel Subwatersheds

In the proposed model, the different main channel subwatersheds shown as ΔA_j (Figure 3.6b) are not coalesced to form a single rectangular plane. Rather these are modeled on similar lines as the tributary subwatersheds of the previous case. In this model, each of the main channel subwatersheds is modeled on the lines of the tributary subwatershed. These are then folded onto the main channel like the tributary subwatershed following the same procedure as discussed in the earlier section. The tributary subwatersheds as well as the main channel



a) CONCEPTUAL REPRESENTATION OF WATERSHED

(b) DISTRIBUTED LATERAL FLOW CONTRIBUTION TO MAIN CHANNEL

FIG. 3·8 – PHYSIOGRAPHIC MODEL–II: CONSISTING OF TRIBUTARY SUBWATERSHEDS AND DISTRIBUTED MAIN CHANNEL SUBWATERSHEDS

subwatersheds are modeled like open book models and the conceptualized model is shown in Figure 3.8(a). The KW theory is used to compute the overland surface runoffs i.e. q_{Tj} (from tributary subwatersheds), and q_{Mj}(from the main channel subwatersheds). In the overlapping portions of the main channel the lateral flows (q_{Mj} and q_{Tj})are superimposed to form reaches (R_j) as shown in Figure 3.8(b). Thus, each reach (R_j) has a uniform lateral inflow (q_{oj}). The distributed lateral flows (q_{oj}) are routed through the main channel by using the DW theory for per unit width of the channel cross-section.

An intercomparison of the proposed three models will indicate that the distributed aspects of the watershed physiography are better taken care of in the later two physiographic models compared to the open book type. The physiographic model-I (i.e. having consolidated main channel subwatershed) is easy to work with due to less number of subwatersheds involved, whereas the physiographic model-II (i.e. having distributed main channel subwatersheds) will be difficult in its application but will have a distinct advantage that runoff contributions from different parts of the watershed can be obtained.

The proposed physiographic models have been applied onto four natural watersheds of different sizes, three of them are located in India and one is in Iran. The availability of data on these watersheds is discussed in the next chapter.

3.6 COMPUTER PROGRAMMES

For the physiographic models discussed in Section 3.5, the KW application for the overland flow computations was carried out. The computer programme (*WAVE-K.FOR)* for these applications was developed in FORTRAN-IV for the PC-AT 386. The details of the programme is given in Appendix-I-A.

Based on DW theory and the implicit computational scheme discussed in Section 3.3, for the channel flow computations, a separate computer programme (*WAVE-D.FOR)* was developed in FORTRAN-IV. The details of the main programme and the subroutines are given in Appendix-I-B.

For developing stage velocity values (i.e corresponding to various discharge values), a separate computer programme was developed. The details of the programme is given in Appendix-I-C.

CHAPTER IV

WATERSHEDS UNDER STUDY AND AVAILABILITY OF DATA

4.1 INTRODUCTION

As discussed in the previous chapter, in order to carry out the studies of the one dimensional flood wave movement in a watershed on the proposed lines, one would require detailed data pertaining to rainfall, river flood flows and watershed physiography. For each of these factors the requirement of data is given as under.

A. Morphometric Data

(i) Details of cross-sections preferably at shorter intervals on the main river and the tributaries.

(ii) Details of longitudinal section of the main river and its tributaries.

(iii) Channel bed slopes at shorter intervals.

(iv) Roughness of the main channel as well as of tributaries and overland planes.

(v) Topographic map indicating spot levels, contours, land use and soil type details.

(vi) Details of urbanization and its development (if any), etc.

B . Meteorological and Hydrological Data Required.

(i) Data of self recording raingauges for the storms under study, preferably at more than one station on the watershed.

(ii) Data of non-recording rainguages available at different stations on the watershed.

(iii) Information pertaining to infiltration capacities at different places under different land uses and soil types.

(iv) River stages at different sections of the main river.

(v) Measured discharges at shorter time intervals.

(vi) Stage-discharge relationships at different sections on the main river.

(vii) Stage-velocity relationship at different sections on the main river.

(viii) Inflow hydrograph data of tributaries at shorter time intervals.

(ix) Backwater data upstream of hydraulic structure (if any).

(x) Submergence of flood planes.

(xi) Baseflow estimates.

4.2 WATERSHEDS UNDER STUDY

Mostly due to budgetary constraints, such an elaborate data base as stated in the earlier section remains non-existing in most of the developing countries. For that matter India and Iran are no exceptions. Detailed data of the type stated above and needed for the model described earlier were available on a few small watersheds only. With a difficulties the data could be procured for the following small and medium watersheds.

i) Watershed of Railway Bridge No. 719 (India)

ii) Kassilian River watershed (Iran)

iii) Kolar River watershed (Narmada sub-basin in India)

Apart from these small watersheds, the short interval hydrological and meteorological data were available only at the Barakar river of Damodar Valley Corporation (DVC) of India. The river flows were available at shorter intervals of three hours for the major flood events. The river cross-sections and river stages are also available at different sections which were approximately 40 km apart. Therefore the proposed model is tried in this study for the watershed of Barakar river stretch between the two gauging stations (viz. Barkisuriya and Nandadhi) which is 2893 km^2 in areal extent. Thus in all, the proposed theory has been applied to four watersheds.

In the end, it may be remarked that there was no other choice in the selection of watersheds or for the data of the storm events which were made available. The details of availability of data on the previously mentioned four watersheds are described in the following sections and summarized in Table 4.1.

4.3 AVAILABILITY OF DATA AT THE WATERSHED OF RAILWAY BRIDGE NO. 719 (INDIA)

The Bridge No. 719 is situated on the Jalarpet - Bangalore section of the Indian Railways (Southern Section). The index map giving details of the watershed of this Bridge is given in Figure 4.1. Some important features, general information and physiographic details as extracted from the topographic map and the available records of this bridge is given in column No. (3) of Table 4.1.

4.3.1 The Storm Rainfall Data

No recording raingauge was installed on this watershed. Since the watershed is small, one non-recording raingauge was installed for registering the storm rainfall data.

For the available storm events, the rainfall readings were usually taken at an interval of 30 minutes. This period was reduced to smaller time steps in some cases. The original record was made available in FPS units which was converted to metric units and is given in Appendix-II-A.

4.3.2 The Runoff Data

The stage-discharge and stage-velocity relationships were established for the main channel at the outlet, (i.e. at the bridge site). The flood discharge data were made available at an interval of 30

TABLE 4.1 General Information and Salient Features of Watersheds Under Study

Sl. No	Particulars	Bridge No. 719	Kassilian	Kolar	Barakar
1	2	3	4	5	
i	Geographical Location (i) longitude(app.)	78° 16' East	53° 11' to 53° 17'	77° 01' to 77° 29'	85° 0' to 87° 0'
	(ii) Latitude(app.)	12° 52' North	35° 59'36 07'	22° 40' to 23° 08'	23° 45' to 25° 0'
ii	Terrain	Semi Hilly	Hilly	Semi-hilly	Semi-hilly with plane areas
iii	Shape of Watershed	Normal	Oblong	Near oblong (Ramp Shape)	Oblong
iv	Climate	Humid	Humid	Humid	Humid
v	Type of Soil	Rocky Red Earth	Rocky Cotton Black Soil	Rocky Cotton Black Soil	Rocky Forest/Agriculture
vi	Land Use	Dry Cultivation	Semi Forest and Rice Cultivation	Deciduous Forest Partly Cultivated	
vii	No.of Raingauge station(s)	1	1	4	3
viii	No. of Discharge Location	1	1	1	3
ix	Relief (m)	136	1400	200	200
x	Watershed Area	14.376 sq. KM	67 sq.KM	870.85 sq. KM	2893 sq. KM
xi	Length of the Main Channel (Longest)	6.65 KM	14.86 KM	92.5 KM	115 KM
xii	Toatl Average Annual Rainfall(mm)	500-1000	500-700	500-1000	500-1000

minutes. Enquiries revealed that during the working hours, the current meter was employed for the measurement of flow velocities and then discharges were computed accordingly. However, during odd hours, only the channel stages were observed and the runoff values were read off from stage-discharge relationship established for the bridge site. The runoff data of these six storm events are also given in Appendix-II-A.

4.4 AVAILABILITY OF DATA AT THE KASSILIAN WATERSHED (IRAN)

Kassilian watershed is situated in the northern part of the Alborz Ranges (IRAN). It is a part of Kassilian - Talar river catchment which finally joins the Caspean Sea. The index map of the watershed is given in Figure 4.2. Some of the important features and general information about the watershed are given in column (4) of the Table-4.1.

4.4.1 The Storm Rainfall Data

One recording raingauge is located in the central part of the watershed. Data for the four storm events could be obtained for the study. The rainfall data have been read from the available charts at 15 minutes time interval. However in Appendix-II-B the same have been reported at 1/2 hour interval.

4.4.2 The Runoff Data

There is only one discharge gauging site at the outlet of the watershed. The charts of runoff data for the storm events under consideration have been procured for the analysis. The runoff data recorded at the site happen to be in the form of river stages. These were converted into discharges at the outlet by using the stage-discharge table. The discharge data of these five storm events are also given in Appendix-II-B.

4.5 AVAILABILITY OF DATA AT THE KOLAR RIVER WATERSHED (INDIA)

The Kolar river originates in the Vindhyachal mountain range at an elevation of 550 m above mean sea level (msl)in the Sehore district of the province of Madhya Pradesh in India. The river during its 92.5 km long course, first flows towards the east and then towards the south before joining the river Narmada.

The drainage network map as shown in Figure 4.3, was obtained from the topographic map drawn to a scale of 1:50,000. Topographically, the Kolar watershed can be divided into two distinct zones. The elevation of the upper four-fifth part of the watershed ranges from 680 m to 350 m . It is predominantly covered by deciduous forest. The lower part of the basin consists of flat bottomed valley narrowing towards the outlet and having elevations ranging from 350M to 300M. Some important physiographic features of the watershed are given in column (5) of Table 4.1.

The cross-section of the river is available only at its outlet which is a bridge. The same is given in Figure 4.4.

4.5.1 The Storm Rainfall Data

Discrete hourly rainfall data of five storm events of the watershed were made available for the study. There are four raingauge stations in the watershed located at Brijeshnagar, Birpur, Jhal iapur and Rehti. The location of these raingauge stations are shown in Figure-4.3. The rainfall data of these storm events are given in Appendix-II-C.

4.5.2 The Runoff Data

There is only one discharge measuring site at the outlet of the watershed. The hourly runoff data of this gauging site were made available corresponding to the four storm events for which the rainfall data were available. The runoff data of these five storm events are also given in Appendix-II-C.

4.6 AVAILABILITY OF DATA AT THE BARAKAR RIVER WATERSHED (INDIA)

The Barakar river originates at Chota Nagpur plateau and flows through narrow reaches in its upper portion (Figure 4.5). The river is gauged at five sites. At the upstream gauging point Tilaya Dam is located which receives drainage water from approximately 969 km^2 of the watershed which is marked as 'A' in Figure 4.5. Small flows are released from the Tilaya Dam to the down stream part of the river. The drainage area contributing to this river stretch has been found to be 1697 km^2 and is marked hatched and shown as 'B' in Figure 4.5. For the present study, the river flows between the stretch Barkisuriya and Nandadhi have been considered. The river stretch between Barkisuriya to Nandadhi is 85 km in length. The intermediate watershed area between these two gauging sites was measured as 2893 km^2 and is shown as dotted portion indicated by 'C' in Figure 4.5. For this stretch of the river, the drainage pattern and other features are shown given in Figure 4.6. Hereafter, in this work the areas 'A' and 'B' together will be referred to as the Barakar river watershed. In Figure 4.7 the river cross-section available at Barkisuriya and Nandadhi are shown.

4.6.1 The Rainfall Data

On this watershed, five recording raingauges and 13 non-recording raingauges reportedly do exist. However, short durationed data of recording raingauges was not available for any of three storm events for which the river discharges were available.

4.6.2 Available Discharge Data

The river stages at Barkisuriya and Nandadhi could be procured (Ghose 1986) for the three storm events registered during the years

1975, 1985, and 1989. Corresponding discharges were read from the G-D curves and the same are given in Appendix-II-D.

The data of the above mentioned four watersheds were used for the application of the model discussed in Chapter-III.

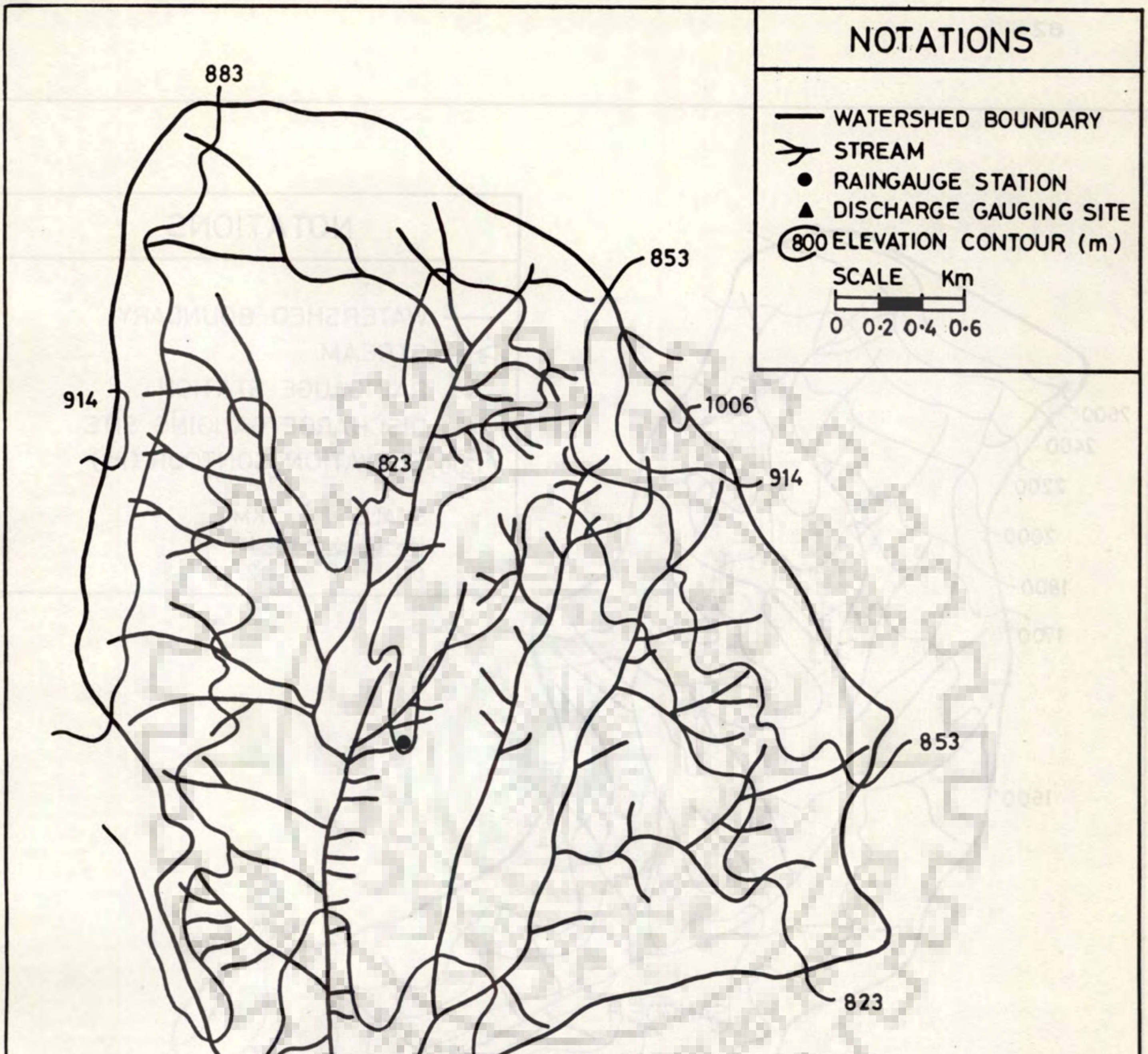


FIG. 4·1-THE INDEX MAP OF THE WATERSHED OF RAILWAY BRIDGE NO. 719 (SOUTHERN RAILWAY, INDIA)

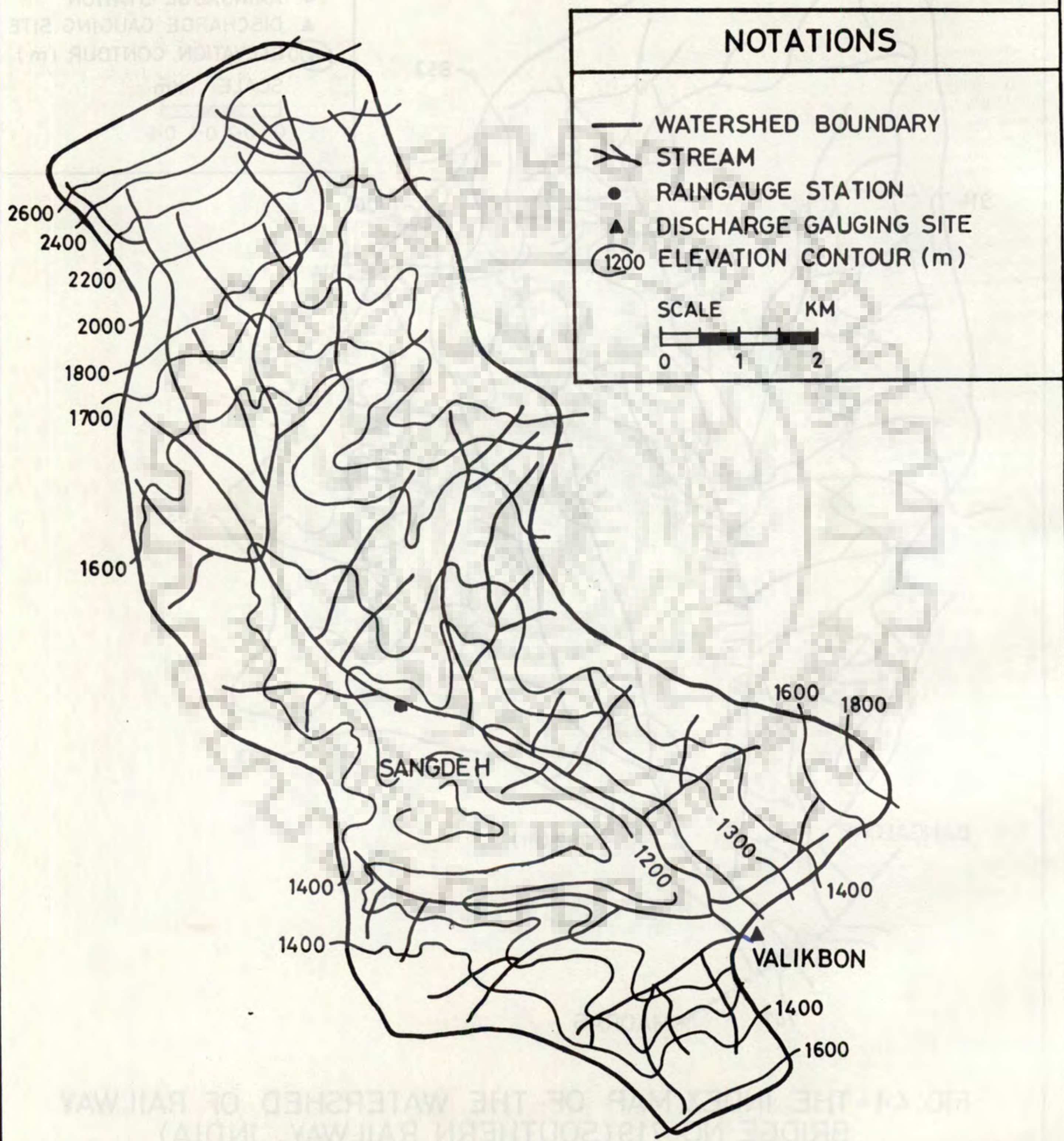


FIG. 4·2–THE INDEX MAP OF THE KASSILIAN RIVER WATERSHED

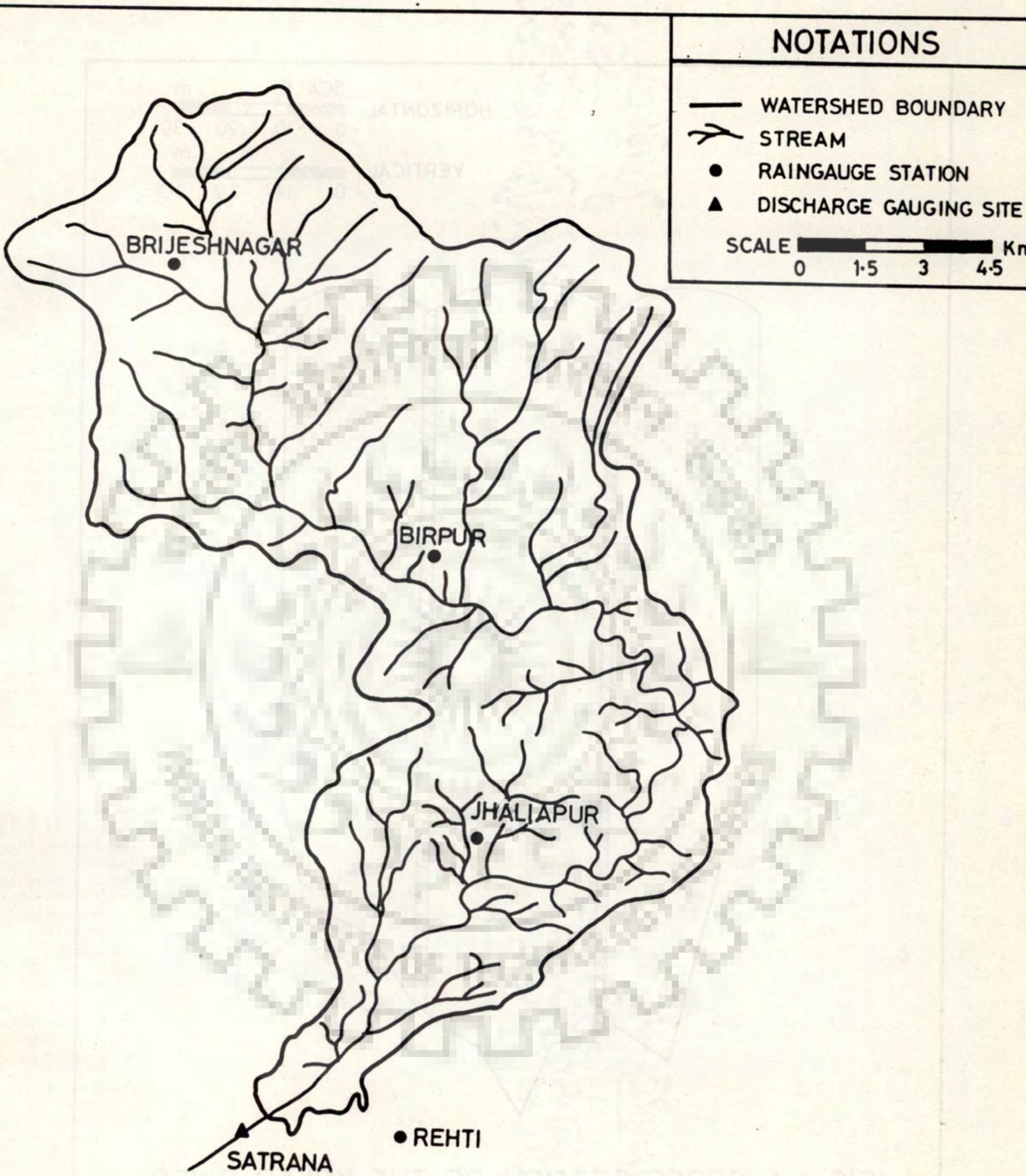


FIG. 4·3-THE DRAINAGE MAP OF THE KOLAR RIVER WATERSHED

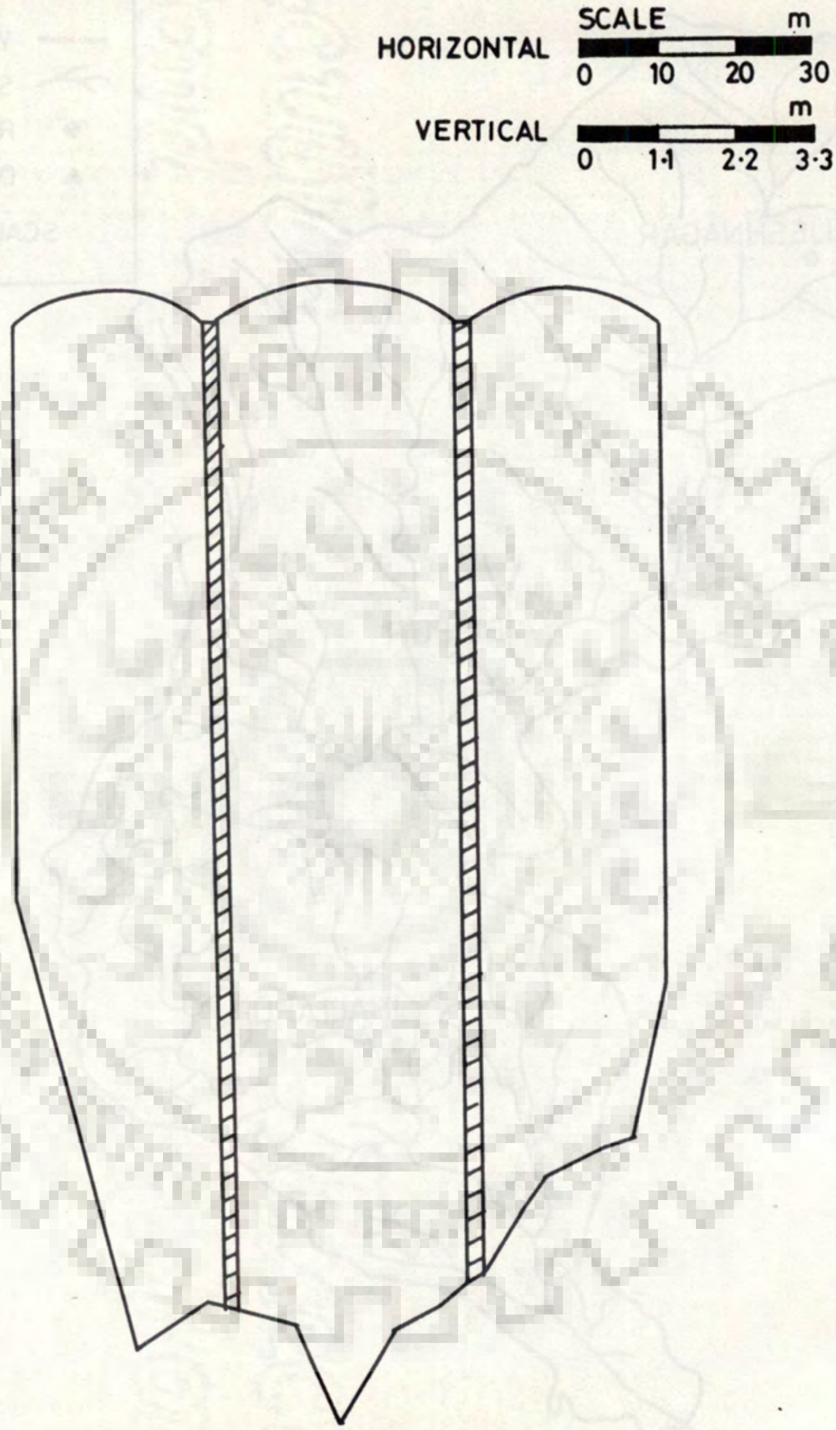


FIG. 4·4 – CROSS-SECTION OF THE KOLAR RIVER AT THE OUTLET LOCATED AT SATRANA

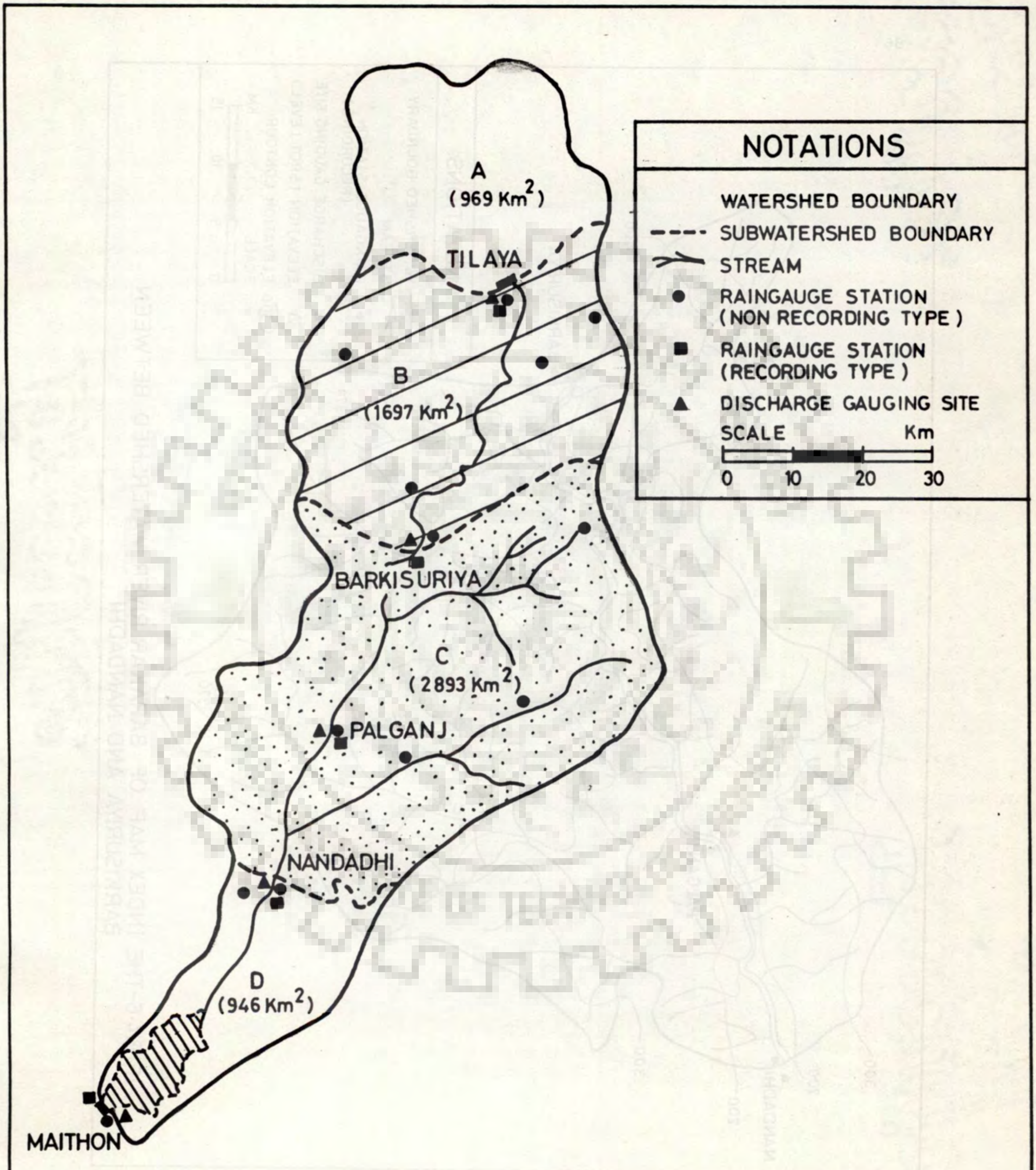


FIG. 4·5–THE DRAINAGE MAP OF BARAKAR RIVER

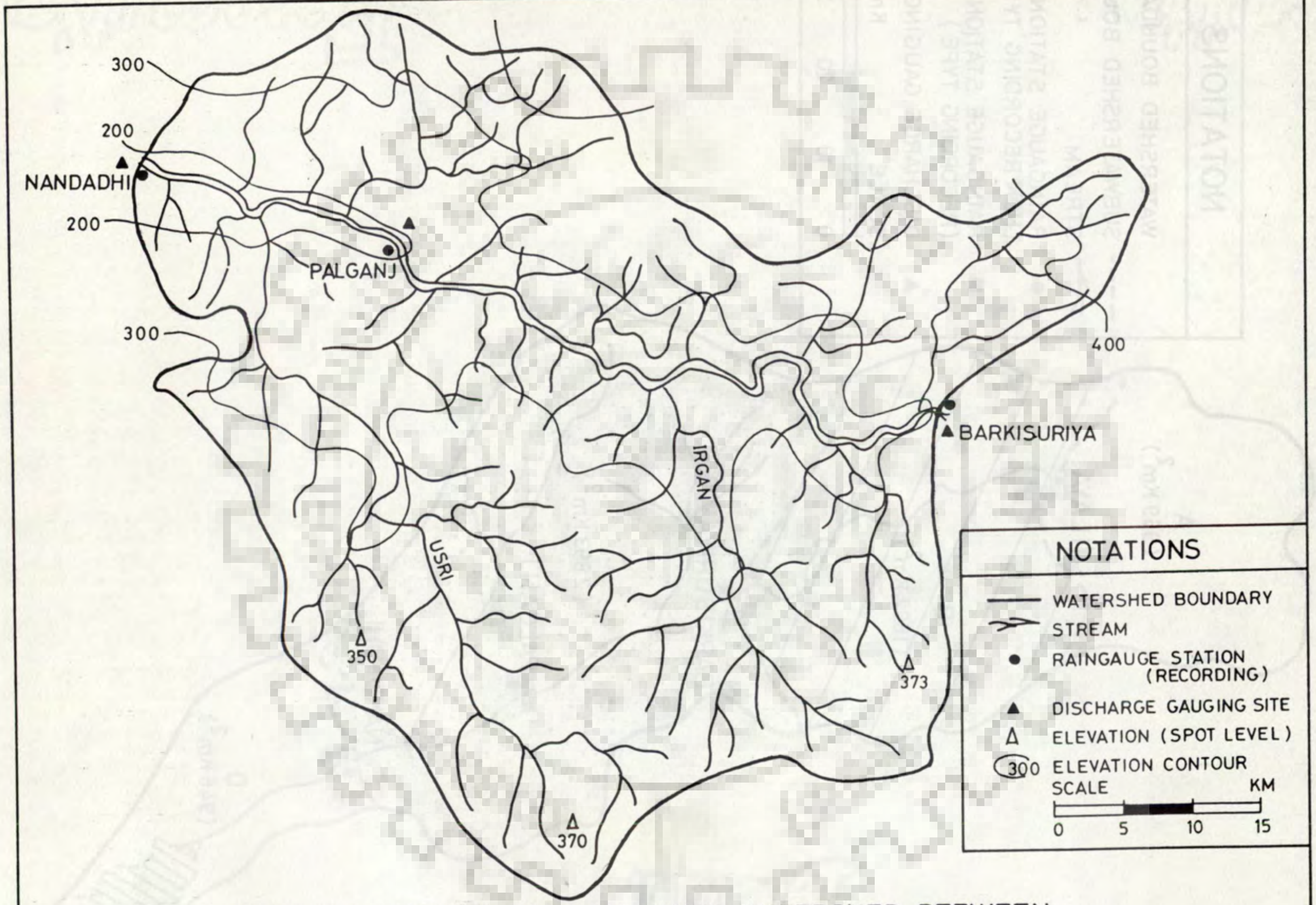
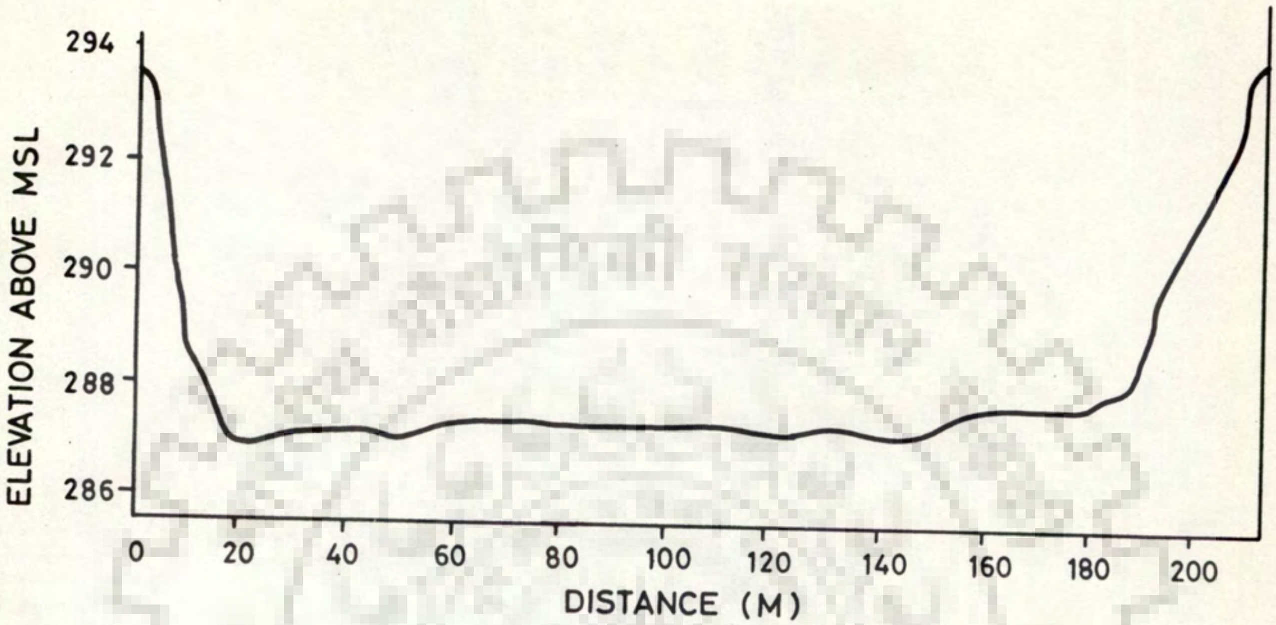
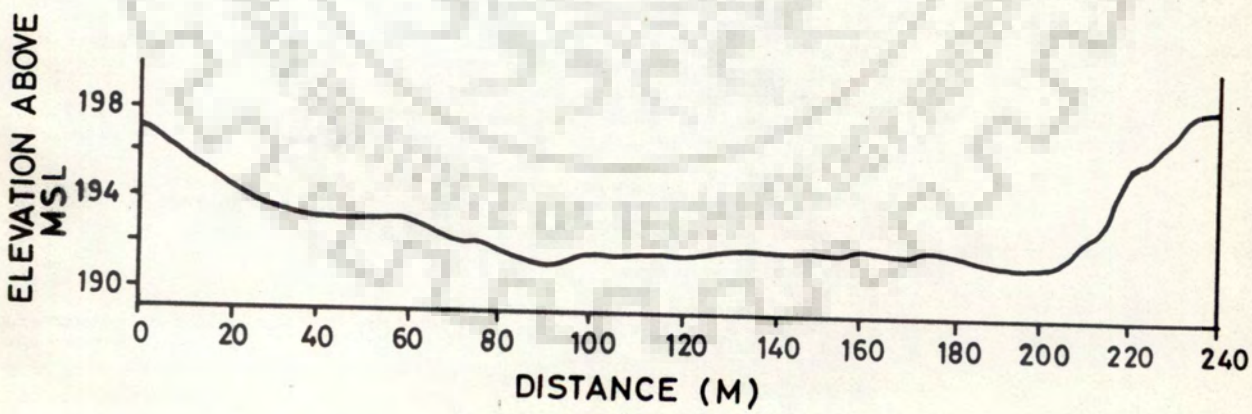


FIG.4·6-THE INDEX MAP OF BARAKAR RIVER WATERSHED BETWEEN BARKISURIYA AND NANDADHI



(a) BARKISURIYA



(b) NANDADHI

FIG. 4·7–THE BARAKAR RIVER CROSS-SECTIONS

CHAPTER V

MODEL APPLICATION

5.1 INTRODUCTION

In Chapter III, the following two physiographic models were developed for application onto the natural watersheds.

i) Physiographic Model-I; Consisting of tributary subwatersheds and a single consolidated main channel subwatershed.

ii) Physiographic Model-II; Consisting of tributary subwatersheds and distributed main channel subwatersheds.

It may be stated that the Physiographic Model-II is an extension of Physiographic Model-I as it covers the watershed physiography more comprehensively. In this chapter, the details of application of these physiographic models onto the following natural watersheds are discussed.

(i) Railway Bridge No. 719

(ii) Kassilian river

(iii) Kolar river

(iv) Barakar River

The availability of data on these watersheds was discussed in the previous chapter. In order to draw logical conclusions about the validity of the proposed physiographic models the same have been tested onto the three of the watersheds mentioned above (viz. Railway Bridge No. 719, Kassilian river and Kolar river). Since out of these three, the Kolar river watershed has the largest drainage area i.e. 870 km^2, the proposed models have been applied onto it in all details in the forthcoming section. Subsequently, in Section 5.5 the application of these models onto the watersheds of Railway Bridge No. 719 and the Kassilian have been discussed in brief. The runoff mechanism of the proposed model (viz. Physiographic Model-II) has further been tested for its applicability to the larger watershed (viz. Barakar river with 2893 km^2 area).

5.2 APPLICATION OF PHYSIOGRAPHIC MODEL-I ONTO KOLAR WATERSHED

The Kolar river watershed (latitude $35^{o}59'$N and longitude $53^{o}11'$ East) is a part of the Narmada river basin, situated in central India. The availability of data and the physiographic characteristics of the watershed were described in Section 4.5. The drainage map of the watershed is given in Figure 5.1. Four raingauges viz. Brijeshnagar, Birpur, Jhaliapur and Rehti are installed on the watershed and their locations are shown in this figure. The outlet is located at Satrana where the discharge measurements are carried out.

Twelve number of main tributaries have been identified and these are marked as T1 to T12 in Figure 5.1. The discretized drainage areas of these tributaries thus forming the tributaries subwatersheds are shown in Figure 5.2. In order to have an idea of the shapes and the drainage patterns of the tributary subwatersheds, the same are plotted separately in Figures 5.3 (a) and (b). The tributary subwatersheds T_1 to T_6 are located on the left bank where as the subwatersheds T_7 to T_{12} are situated on the right bank of the Kolar river. The total drainage area of the tributary subwatersheds works out to be nearly 60 percent of the total Kolar river watershed. A close look at these subwatersheds reveals that these are of different shapes i.e. they vary from oblong to fan shape. The physiographic parameters of these subwatersheds viz. drainage area (km^2), length of tributary (km), overland slope (m/m) and width of equivalent rectangular plane are given in Table 5.1. The left out portion (total watershed area - the tributary subwatershed areas) thus forms the main channel subwatershed and is shown in Figure 5.4(a). This forms nearly 40 percent of the total watershed area.

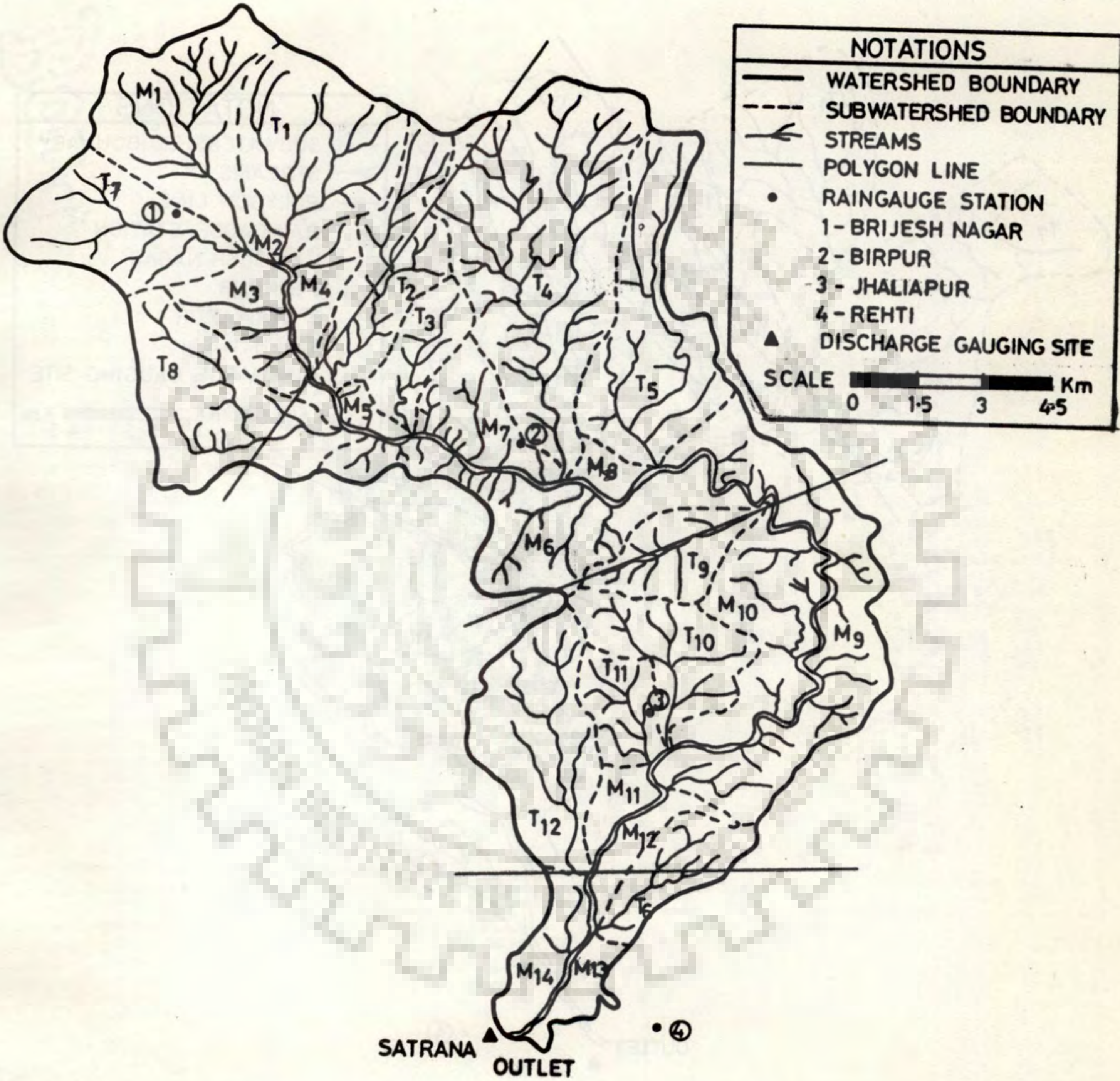


FIG. 5·1—THE KOLAR RIVER WATERSHED

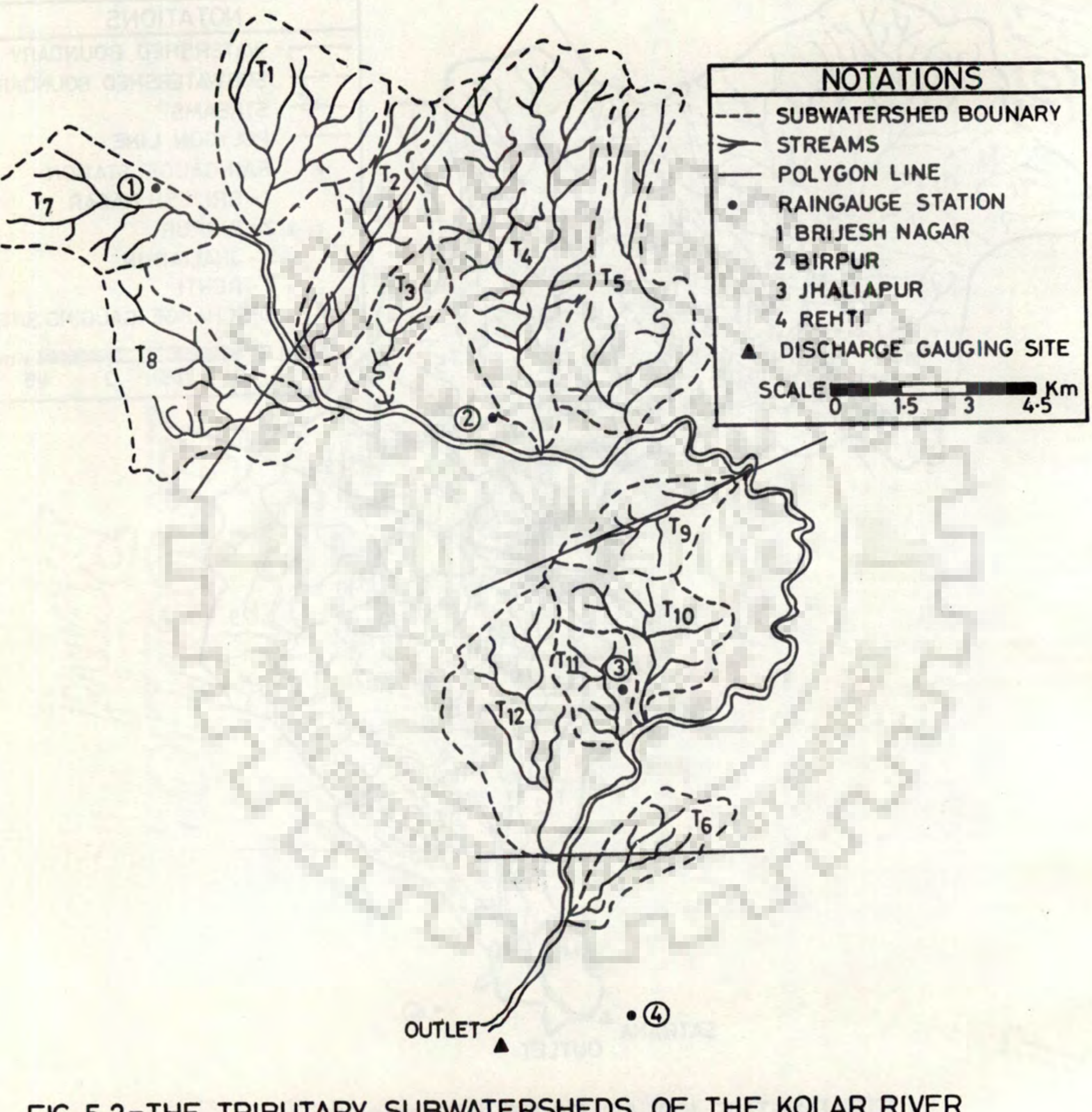


FIG. 5·2–THE TRIBUTARY SUBWATERSHEDS OF THE KOLAR RIVER DRAINAGE BASIN

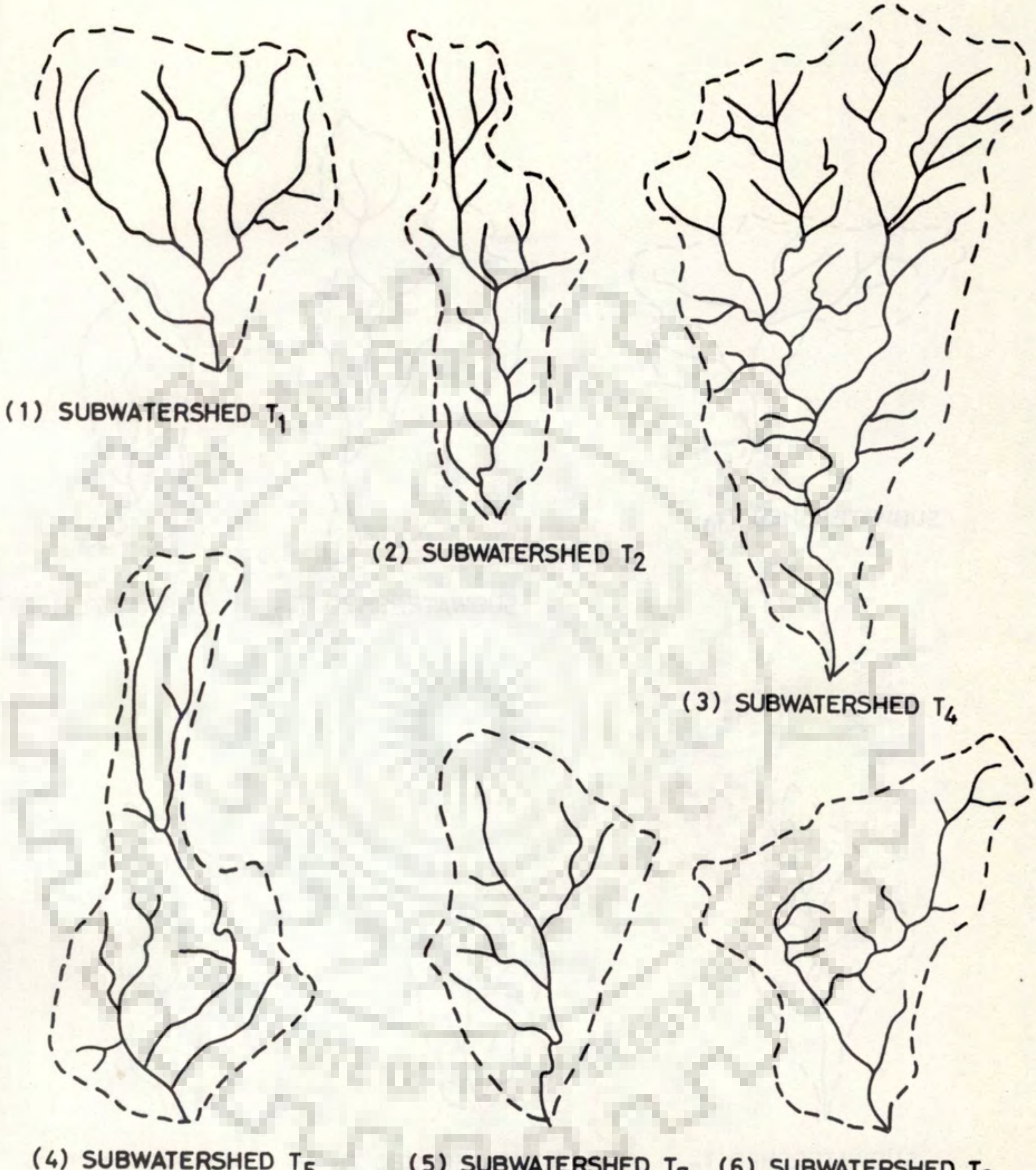


FIG. 5·3(a)-DISCRETIZED TRIBUTARY SUBWATERSHEDS OF KOLAR RIVER WATERSH

(CONTD)

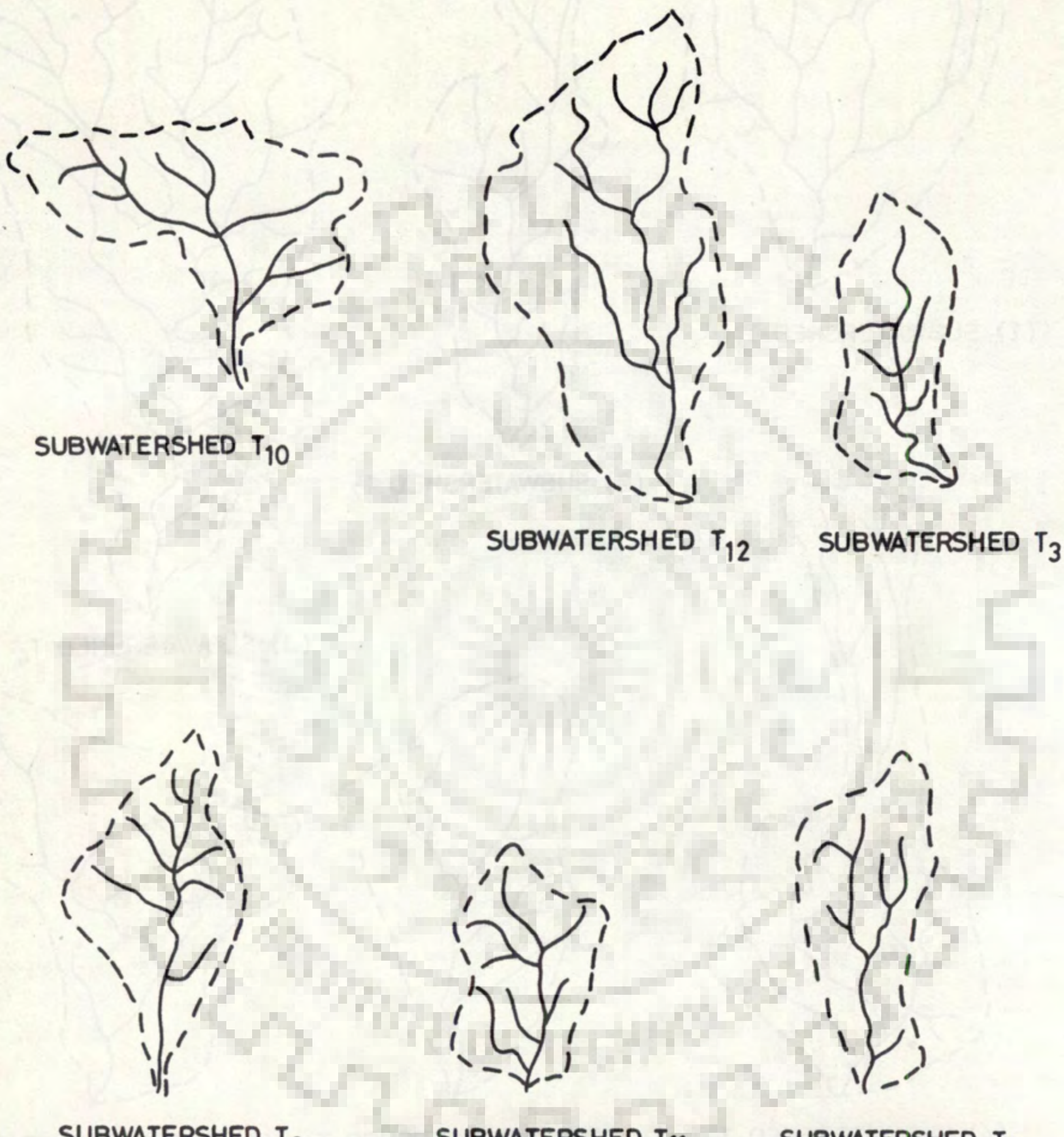
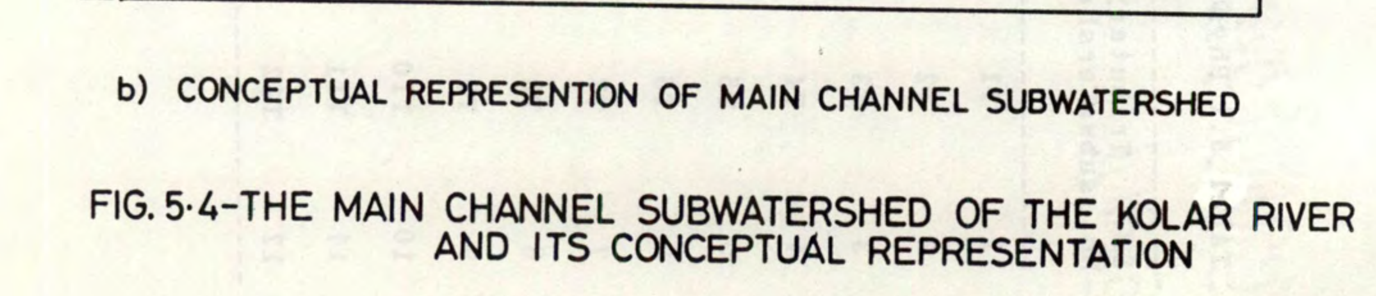


FIG. 5.3 (b) – DISCRETIZED TRIBUTARY SUBWATERSHEDS OF KOLAR RIVER WATERSHED

Table 5.1 Physiographic Parameters of Tributary Subwatersheds of Kolar Watershed

Sl. No.	Tributary subwatershed	Drainage Area (km**2)	Length of Ttributary (km)	Overland Slope	Overland width (0.5*W) (m)	α	Δ X (m)
1	T1	54.800	11.0	0.006364	2491	1.330	249.10
2	T2	39.500	15.5	0.005161	1274	1.197	212.33
3	T3	18.500	10.0	0.004000	925	1.054	185.00
4	T4	113.000	22.5	0.044400	2511	3.512	251.10
5	T5	58.750	20.5	0.005854	1433	1.275	204.71
6	T6	21.400	8.0	0.003125	1338	0.932	223.00
7	T7	42.950	12.5	0.003600	1718	1.000	245.43
8	T8	54.300	16.0	0.002938	1697	0.903	212.33
9	T9	22.375	9.5	0.006316	1178	1.325	235.60
10	T10	32.250	10.0	0.016000	1613	2.108	201.63
11	T11	16.750	6.5	0.012310	1288	1.849	214.66
12	T12	49.375	15.0	0.011330	1446	1.774	205.75

a) THE MAIN CHANNEL SUBWATERSHED



b) CONCEPTUAL REPRESENTION OF MAIN CHANNEL SUBWATERSHED

FIG. 5·4–THE MAIN CHANNEL SUBWATERSHED OF THE KOLAR RIVER AND ITS CONCEPTUAL REPRESENTATION

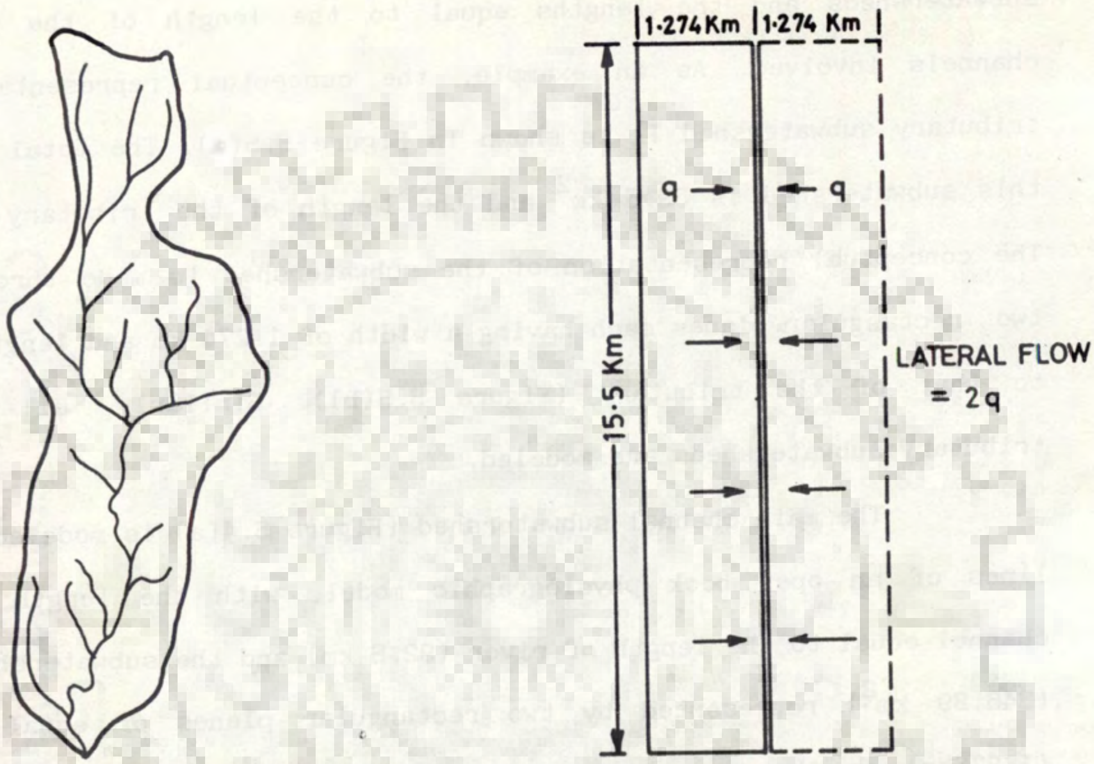
5.2.1 The Model Physiographic Configuration

As discussed in Section 3.5, the tributary subwatersheds as well as the main channel subwatershed are conceptually represented through rectangular planes having the same areas as that of the subwatersheds and the lengths equal to the length of the drainage channels involved. As an example, the conceptual representation of tributary subwatershed T_2 is shown in Figure 5.5(b). The total area of this subwatershed is 39.5 km^2 and the length of the tributary 15.5km. The conceptual representation of the subwatershed is made through the two rectangular planes each having a width of 1.274 km and length equal to that of the tributary (Figure 5.5(b)). Similarly, all the 12 tributary subwatersheds are modeled.

The main channel subwatershed (Figure 5.4(a) is modeled on the lines of an open book physiographic model, with the length of the channel equal to the length of river (92.5 km) and the subwatershed area (346.89 km^2) represented by two rectangular planes of equal widths (1.875km) on the two sides of the main channel as shown in Figure 5.4(b). To arrive at the final conceptual configuration, the tributary subwatersheds are folded onto the main channel of the Kolar river.

5.2.2 Folding of the Subwatersheds onto the Main Channel

As mentioned in Section 3.5.1.1, the conceptual rectangular configurations of the tributary subwatersheds are placed parallel to the main channel. These rectangular planes start from the point of confluence of the tributaries and extend towards upstream side, each having the length equal to its tributary. The conceptual configuration so arrived at is shown in Figure 5.6. The runoff mechanism is now adequately covered by using the KW and DW theories which were discussed in Sections 3.4 and 3.3 respectively.



a) TRIBUTARY SUBWATERSHED T_2 b) CONCEPTUAL REPRESENTATION

FIG. 5·5 - CONCEPTUAL REPRESENTATION OF TRIBUTARY SUBWATERSHED T_2

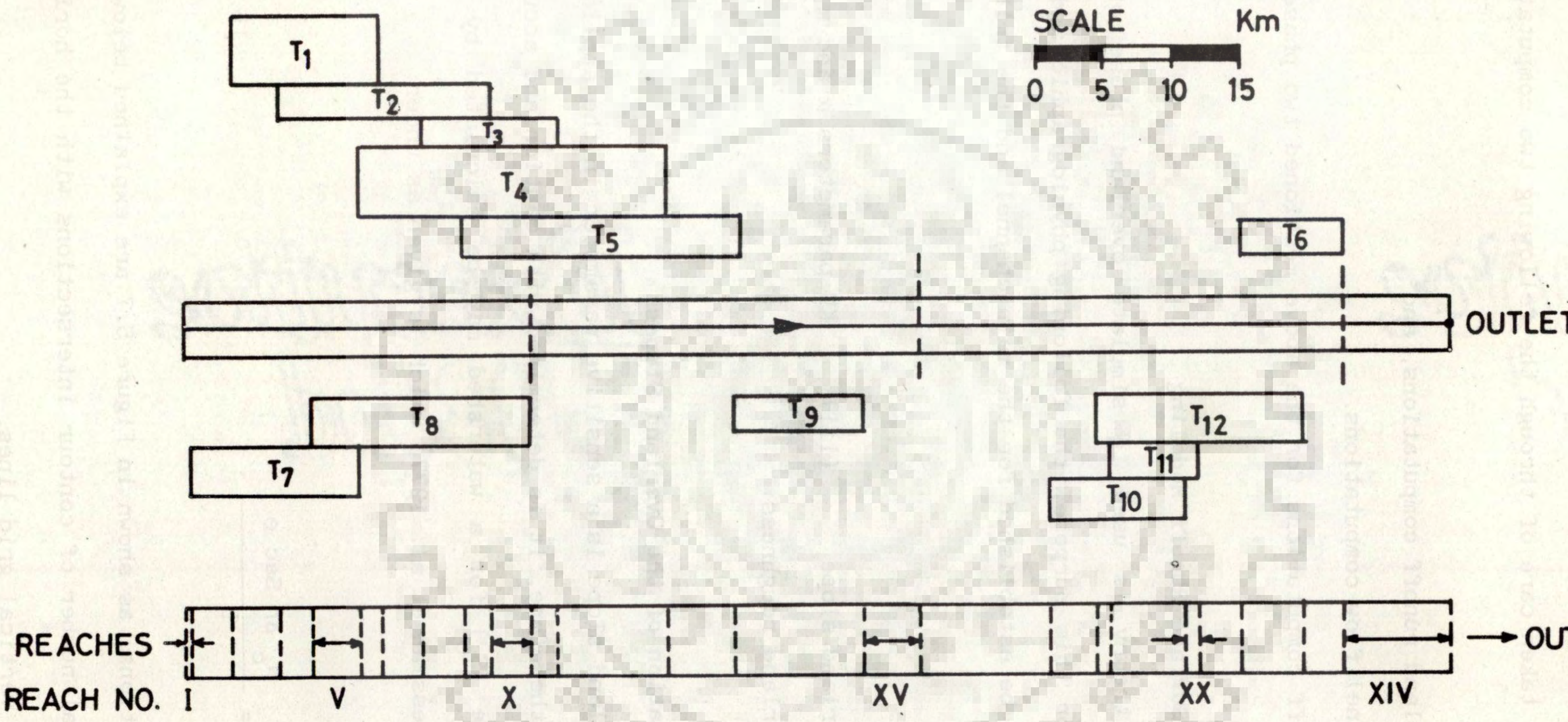


FIG. 5·6 – CONCEPTUAL CONFIGURATION OF KOLAR RIVER WATERSHED FOR PHYSIOGRAPHIC MODEL-I

5.2.3 The Runoff Mechanism

For the proposed model, the runoff mechanism of the Kolar river watershed is taken care of through the following two computational phases.

(i) The overland runoff computations, and

(ii) The channel flow computations.

The runoff computations for the above mentioned two phases are discussed below.

(a) Overland Surface Runoff Modeling

The KW theory is used to simulate overland phase of the surface runoff. For this purpose, the following physiographic and flow parameters have to be established for the main channel and the tributary subwatersheds.

(i) overland slope (iii) KW parameters α and m

(ii) overland roughness

(i) Determination of the Overland Slopes

The overland slope is a sensitive parameter in the application of the KW theory, therefore, it is determined with caution and accuracy. The overland slope (S_o) for a watershed may be calculated by using Horton formula (Viessman et al. 1977) which is given as under:

$$S_o = \frac{N_c \, \Delta h \, \text{Sec} \, \theta}{L} \qquad \ldots \quad (5.1)$$

The notations, as shown in Figure 5.7 are explained below.

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

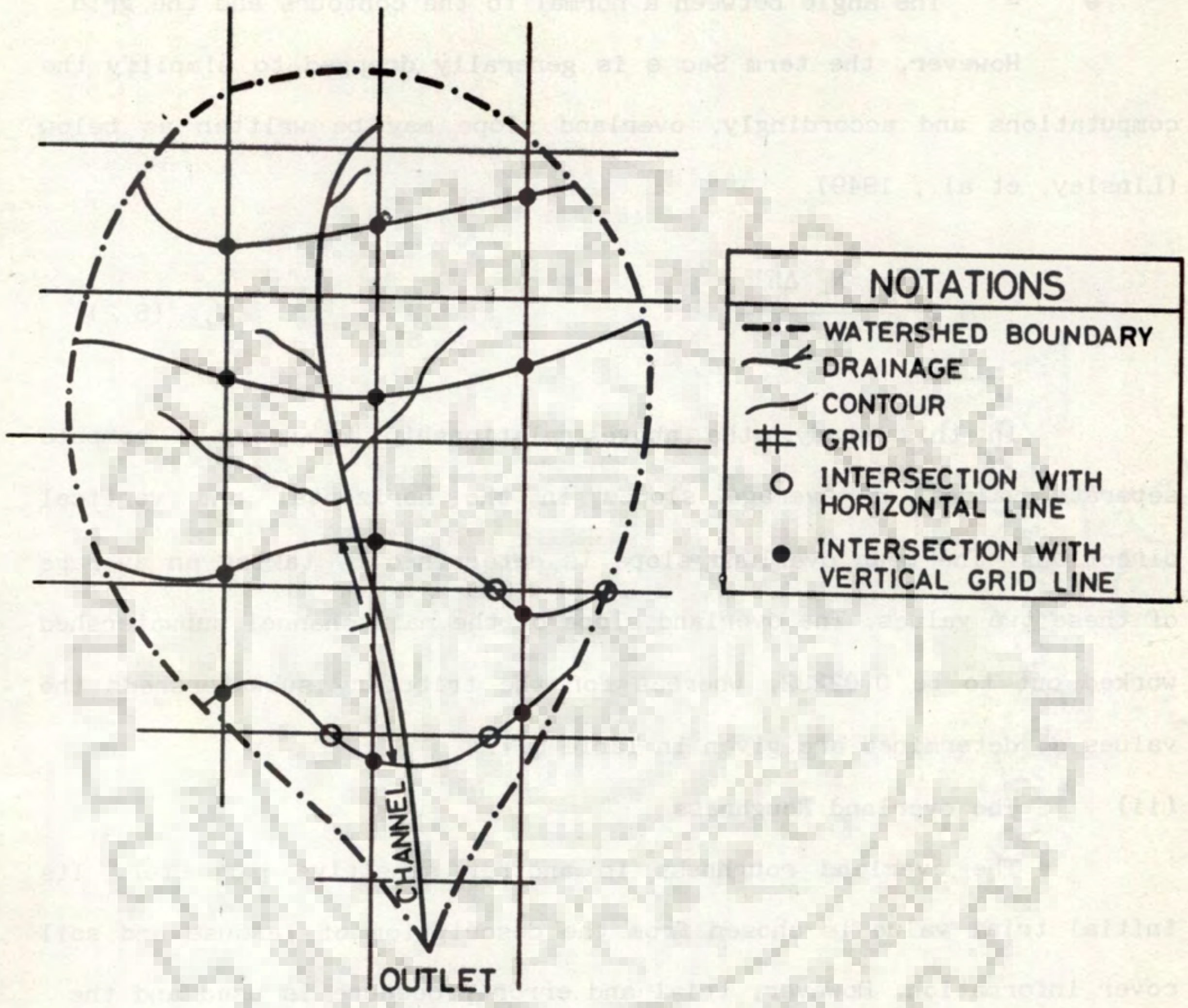


FIG. 5·7—DETERMINATION OF MEAN OVERLAND SLOPES
(AFTER VIESSMAN et al. 1977)

Δh = The contour interval (m)

L = Total length of grid line segments in meters (horizontal and vertical)

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

However, the term Sec θ is generally dropped to simplify the computations and accordingly, overland slope may be written as below (Linsley, et al., 1949).

$$S_o = \frac{N_c \Delta h}{L} \quad \ldots \quad (5.2)$$

In this study, the above relationship is used to compute separate values of average slopes in the horizontal and vertical directions. The mean overland slope is determined by taking an average of these two values. The overland slope of the main channel subwatershed worked out to be 0.02232, whereas for the tributary subwatersheds the values so determined are given in Table 5.1.

(ii) The overland Roughness

The overland roughness is another sensitive parameter. Its initial trial value is chosen from the description of landuse and soil cover information. However, trial and error procedure is used and the roughness values in the vicinity of the initial trial were tried. Thus, the value of that overland roughness which reproduced acceptable matches of the computed and the observed hydrographs was adopted. For this watershed the value of overland roughness worked out to be 0.06.

(iii) KW Parameters (α and m)

The value of KW parameter m for wide rectangular channel worked out to be 5/3. The values of parameter α for all the 12 tributaries have been computed using equation 3.37 and the worked out values of this parameter are given in Table 5.1 . For the main channel subwatershed the value of α worked out to be 2.489.

Having established the KW parameters for all the subwatersheds involved, the rainfall excess has been worked out.

(iv) Rainfall excess computations

As discussed in Chapter IV, the storm rainfall and the runoff data of the five storm events were available on the Kolar watershed (Appendix -II-C). The rainfall excess functions have been computed individually for the four raingauge stations located in the watershed. The areas of influence of these stations have been considered to be their respective Thiessen polygons whose details are given in Table 5.2.

Table 5.2 Thiessen Weights of Kolar Watershed Raingauge Stations

Station	Area (km^2)	Thiessen weights
Brijeshnagar	264.875	0.3042
Birpur	330.325	0.3793
Jhaliapur	235.650	0.2706
Rehti	40.000	0.0459

The time distribution of rainfall excess for each storm event is obtained raingauge-wise by using the runoff factor for the storm and ϕ-index. The runoff factor for the storm is computed as a ratio of depths of the direct runoff and the gross average rainfall. The Thiessen weights given in Table 5.2 are used to compute the gross average depth for the storm and a straight line distribution is assumed for the separation of base flow to obtain the Direct Runoff Hydrograph (DRH). The time distributions of the rainfall excess functions so arrived at for the four raingauge stations for the five storm events are given in Appendix -II-C.

In Table 5.3, the areal coverage of the tributary subwatersheds within different thiessen polygons is given. In case of tributary subwatersheds which get divided between the two polygons, the computer subroutine given in Appendix -I-A(2) computes the weighted average rainfall excess. This is to mention that the main channel subwatershed gets divided into four parts corresponding to the raingauges at Brijeshnagar, Birpur, Jhaliapur and Rehti. The rainfall excess functions as computed are fed on to the KW routing computer programme to compute the overland runoff.

Table 5.3 Areal Distribution of Tributary Subwatersheds within the Four Thiessen polygons

Sub-watershed	Distribution of Area within Polygons (km^2) (percentage)				
	Brijeshnagar	Birpur	Jhaliapur	Rehti	Total
T1	54.8(100)	-	-	-	54.8
T2	23.305(59)	16.195(41)	-	-	39.5
T3	-	18.5(100)	-	-	18.5
T4	4.25(3.8)	108.75(96.2)	-	-	113.0
T5	-	58.75(100)	-	-	58.75
T6	-	-	10.7(50)	10.7(50)	21.4
T7	42.95(100)	-	-	-	42.95
T8	45.3(83)	9(17)	-	-	54.3
T9	-	11.19(50)	11.19(50)	-	22.375
T10	-	-	32.25(100)	-	32.25
T11	-	-	16.75(100)	-	16.75
T12	-	-	49.175(99)	0.2(1)	49.375

(v) Overland Flow Computations

The computer programme given in Appendix -I-A(1) solves the KW equations 3.36 and 3.45. The overland surface runoffs from different tributary subwatersheds as well as from the main channel subwatershed (Figure 5.6) have been computed using equations 3.36 and 3.45. The computations are carried out for the strips of unit widths (i.e. 1 m width) of the overland planes. The time step for the KW overland flow routing has been taken as 300 seconds. Each strip is further divided into smaller space steps Δx_j. The values of space steps (Δx_j) for the 12 tributary subwatersheds are given in Table 5.1, where as the value of space step for the main channel subwatershed is taken as 208.33 m. In the region where the overland planes (Figure 5.6) of different subwatersheds overlap, the computer subroutine given in Appendix-I-A(3), superimposes the outflows. At a time, in different stretches of the main channel different lateral flows (q_{oj}) are received. These stretches of the main channel having the same lateral flow (q_{oj}) are identified as its 'reaches'. Thus, the main channel gets divided into 24 reaches. Table 5.4, gives the superposition of the overland runoffs coming from different overlapping planes of tributary subwatersheds to form the reaches. The minimum and the maximum lengths of the reach worked out to be 0.5 Km and 9.5 km respectively. The function $q_{o,j}$ thus forms the distributed input to the channel reach through which the flows are routed by using the concepts of DW theory.

(b) Channel Flow Computations

All along the main channel, the differential inputs q_{oj} coming from the overland planes form the input to the channel at a particular time. The flows are routed through the channel by using the concepts of DW theory as discussed in Chapter III. A computer programme has been developed for solving the DW equations (viz. equations 3.10 and 3.11) and the same is given in Appendix-I-B.

The proposed model assumes the channel cross-section to be rectangular. The equivalent rectangular section for the Kolar river is shown in Figure 5.8. The width of the main channel works out to be 80 m. The computations are carried out for unit width of the channel. The channel bed slope and the roughness are the two sensitive parameters.

Table 5.4 Superimposed Lateral Flows to Kolar River (Physiographic Model-I)

Reach No.	Lateral Flows to the Reach q_o(Subwatershed No.)	Reach length (km)	Space Step (m)
1	$q_o(1)$	0.5	500
2	$q_o(1) + q_o(11)$	3.0	1500
3	$q_o(1) + q_o(5) + q_o(11)$	3.5	1750
4	$q_o(1) + q_o(5) + q_o(6) + q_o(11)$	2.5	2500
5	$q_o(1) + q_o(5) + q_o(6) + q_o(11) + q_o(12)$	3.5	1750
6	$q_o(1) + q_o(5) + q_o(6) + q_o(8) + q_o(12)$	1.5	1500
7	$q_o(1) + q_o(6) + q_o(8) + q_o(12)$	3.0	3000
8	$q_o(1) + q_o(6) + q_o(7) + q_o(8) + q_o(12)$	3.0	3000
9	$q_o(1) + q_o(6) + q_o(7) + q_o(8) + q_o(9) + q_o(12)$	2.0	2000
10	$q_o(1) + q_o(7) + q_o(8) + q_o(9) + q_o(12)$	3.0	3000
11	$q_o(2) + q_o(7) + q_o(8) + q_o(9)$	2.0	2000
12	$q_o(2) + q_o(8) + q_o(9)$	8.0	2666.6
13	$q_o(2) + q_o(9)$	5.0	2500
14	$q_o(2) + q_o(13)$	9.5	2375
15	$q_o(2)$	4.0	2000
16	$q_o(3)$	9.5	2375
17	$q_o(3) + q_o(14)$	3.5	1750
18	$q_o(3) + q_o(14)\ q_o(16)$	1.0	1000
19	$q_o(3) + q_o(14)\ q_o(15)\ q_o(16)$	5.5	2750
20	$q_o(3) + q_o(15)\ q_o(16)$	1.0	1000
21	$q_o(3) + q_o(16)$	3.0	3000
22	$q_o(3) + q_o(10)\ q_o(16)$	4.5	2250
23	$q_o(3) + q_o(10)$	3.0	3000
24	$q_o(4)$	7.5	2500

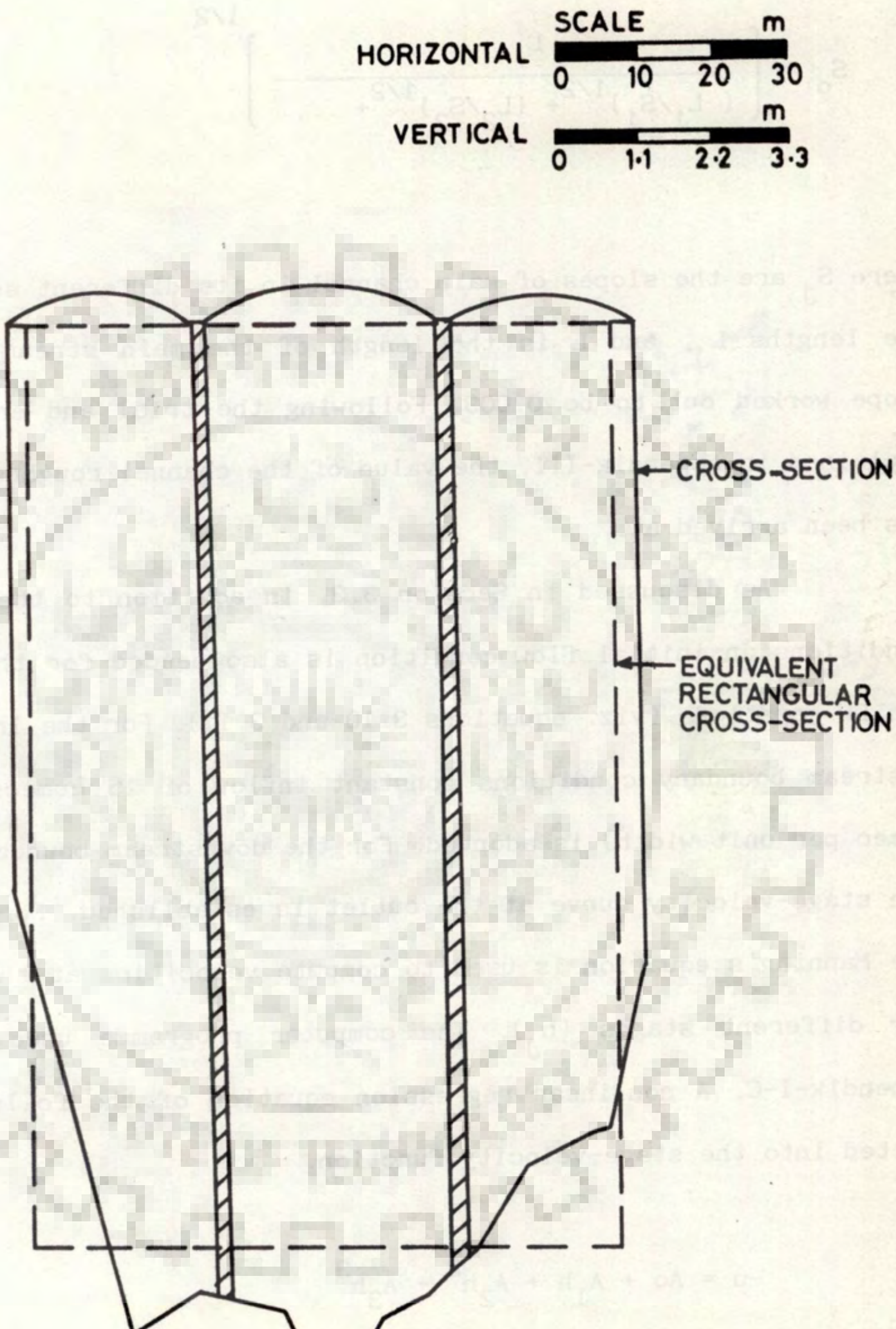


FIG. 5·8—EQUIVALENT RECTANGULAR CROSS-SECTION OF KOLAR RIVER

The channel bed slope (S_o) values has been arrived at by using the following relationship.

$$S_o = \left[\frac{L}{(L_1/S_1)^{1/2} + (L_2/S_2)^{1/2} + \ldots} \right]^{1/2} \quad \ldots. \quad (5.3)$$

where S_j are the slopes of main channel in its different segments having the lengths L_j, and L is the length of the main stream. The channel slope worked out to be 0.003. Following the trial and error procedure explained in Appendix-III, the value of the channel roughness n (=0.026) has been arrived at.

As discussed in Section 3.3, in addition to the two boundary conditions an initial flow condition is also needed for the solution of the DW equations (viz. equations 3.10 and 3.11). For the initial and the upstream boundary conditions constant inflow of 15 cumecs (i.e. 0.187 cumec per unit width) is adopted. For the downstream boundary condition, the stage-velocity curve at the outlet is established. For this purpose the Manning's equation is used to compute velocities at the outlet (u_j) for different stages (h_j). The computer programme used is given in Appendix-I-C. A nonlinear regression equation of the following type is fitted into the stage-velocity function.

$$u = Ao + A_1 h + A_2 h^2 + A_3 h^3 \quad \ldots. \quad (5.4)$$

The values of regression coefficients worked out to be as under.

Ao=0.4027035 A_1=1.829082 A_2=-0.2362494 A_3=0.01625036

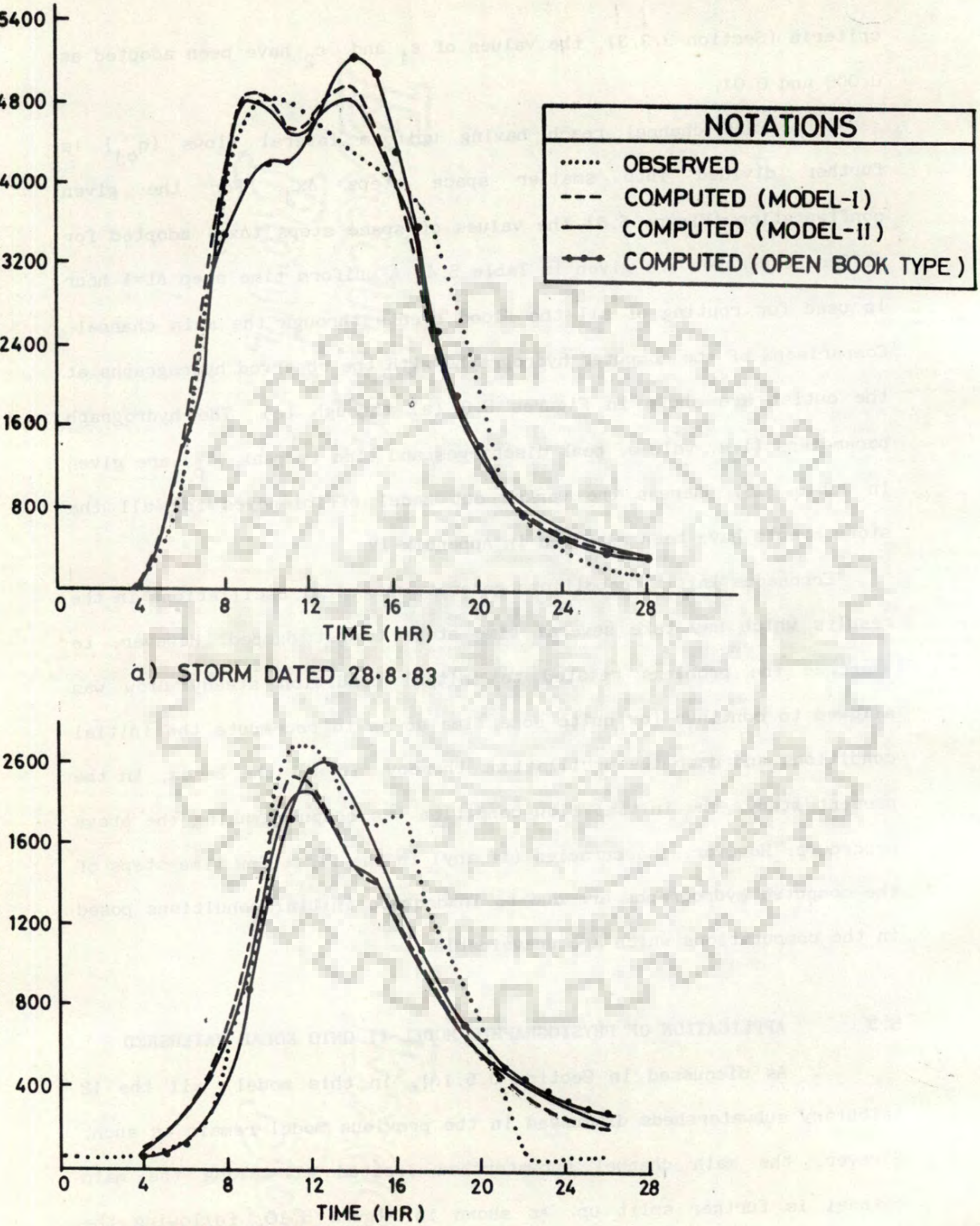
As recommended by various researchers (Mukumba 1978) the time weighting coefficient θ is assigned a value of 0.67. For the convergence criteria (Section 3.3.3), the values of ε_1 and ε_2 have been adopted as 0.006 and 0.01.

Each channel reach having uniform lateral flows (q_{oj}) is further divided into smaller space steps Δx_j. For the given configuration (Figure 5.6) the values of space steps (Δx_j) adopted for different reaches are given in Table 5.4 .A uniform time step Δt=1 hour is used for routing of all the flood events through the main channel. Comparisons of the computed hydrographs with the observed hydrographs at the outlet are shown in Figures 5.9 (a) through (e). The hydrograph parameters (i.e. volume, peak discharges and time to peak (t_p) are given in Table 5.5, whereas the worked out model efficiencies for all the storm events have been reported in Appendix-IV

Erroneous initial conditions generate numerical oscillations in the results which may take several time steps to get damped. However, to overcome the problems related to initial condition steady flow was allowed to continue for quite some time so as to recompute the initial conditions and damp the oscillations thereby. In all the cases, in the present study, the initial flow condition was computed using the above procedure. However, inaccuracies (if any) in the first few time steps of the computed hydrographs are due to inadequate initial conditions posed in the computations which are unavoidable.

5.3 APPLICATION OF PHYSIOGRAPHIC MODEL-II ONTO KOLAR WATERSHED

As discussed in Section 3.5.1.1, in this model all the 12 tributary subwatersheds discussed in the previous model remain as such. However, the main channel subwatershed located all along the main channel is further split up. As shown in Figure 5.10, following the water divide, the main channel subwatershed is further divided into smaller units and 14 spatially distributed main channel subwatersheds



b) STORM DATED

FIG. 5·9 a & b-COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF KOLAR WATERSHED

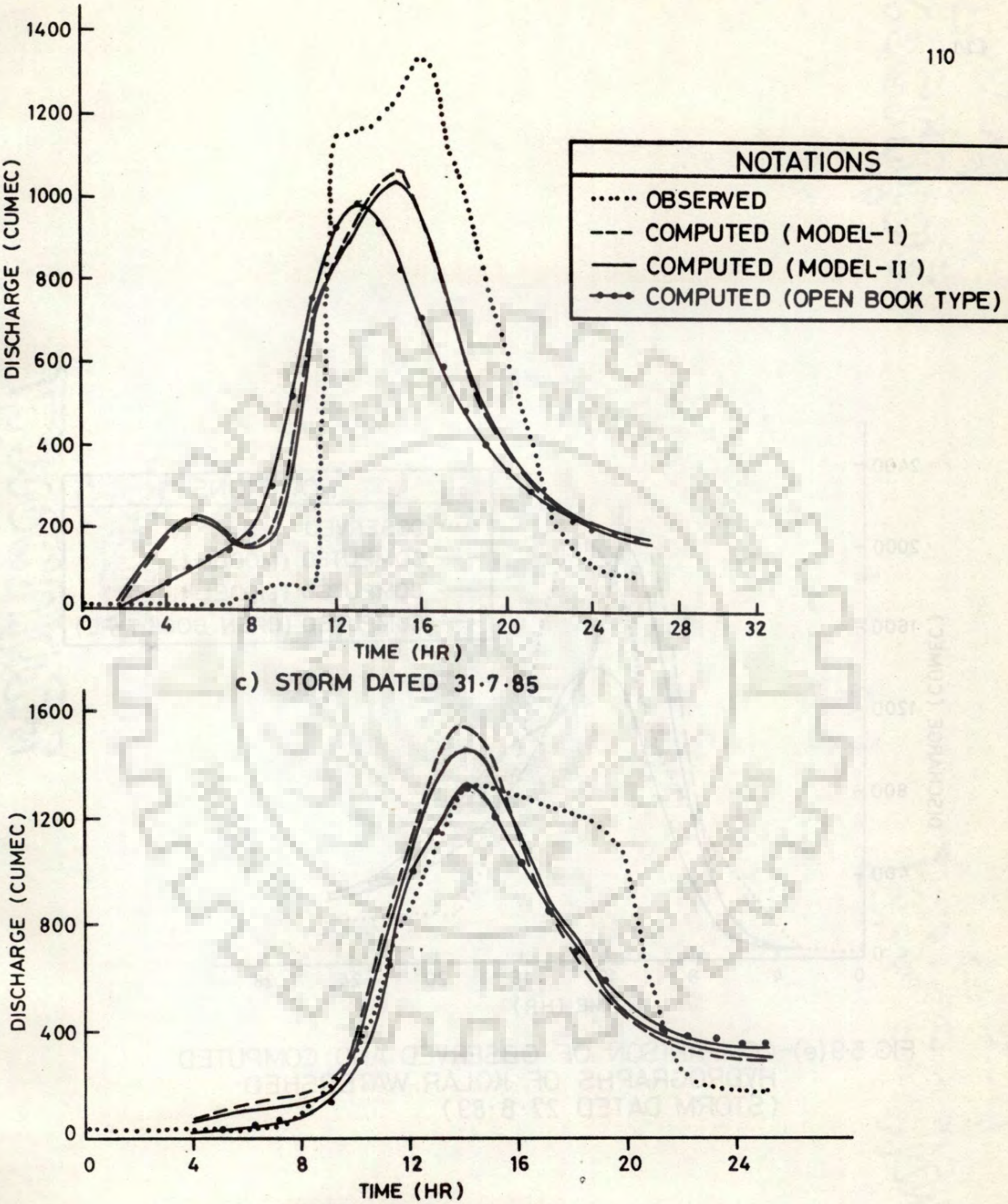


FIG. 5·9 c & d—COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF KOLAR WATERSHED

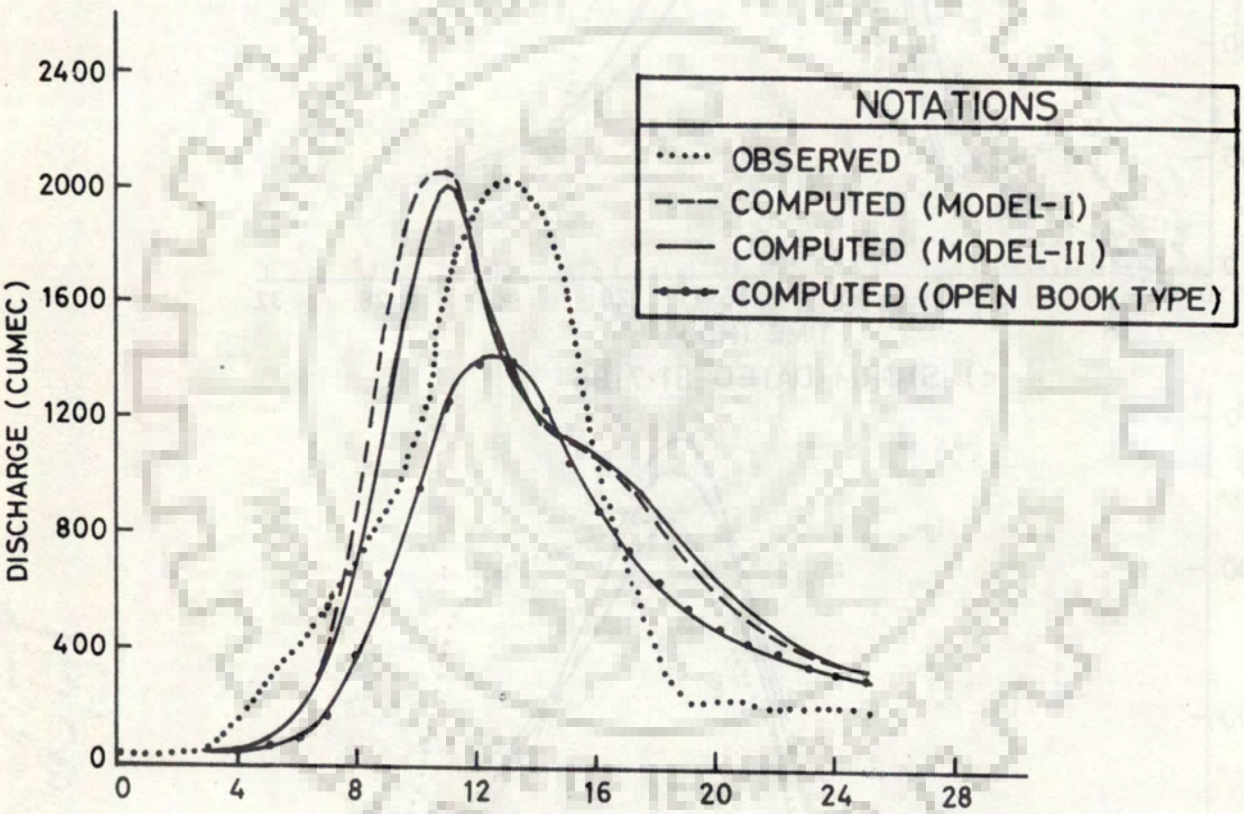


FIG. 5·9(e)-COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF KOLAR WATERSHED (STORM DATED 27·8·89)

Table 5.5 Comparison of Parameters of the Computed Hydrographs (Using Different Physiographic Models) with the observed Hydrographs of Kolar River

Sl. No.	Storm Dated	Parameters of Observed Hydrograph			Prameters of computed Hydrograph (Physiographic Model-I)			Error in Prediction		Parameters of computed Hydrograph (Physiographic Model-II)			Error in Prediction		Parameters of computed Hydrograph (Open Book Type)			Error in Prediction	
		DRH Volume MCM	DRH Peak cumec	Tp HR	DRH Volume MCM	DRH Peak cumec	Tp HR	Absolute	Relative %	DRH Vol MCM	DRH Peak cumec	Tp HR	Absolute	Relative %	DRH Volum MCM	DRH Peak cumec	Tp HR	Absolute	Relative %
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20
1	28.8.83	211.143	4830	10	217.21	4890	10	6.07	2.87	209.94	5388	10	0.68	0.32	217.22	4966	-	6.08	2.88
2	10.8.84	66.864	2027	12	66.32	1859	12	0.544	0.81	65.849	1776	12	1.015	1.518	65.52	1926	13	1.344	2.01
3	31.7.85	36.901	1346	16	36.89	1017	15	0.011	0.03	36.844	1006	15	0.057	3.17	36.33	983	14	0.571	1.57
4	15.8.86	40.866	1242	14	39.59	1375	14	1.276	3.12	39.567	1396	14	1.299	3.2	40.31	1289	14	0.556	1.36
5	27.8.87	50.725	1898	13	52.81	1876	11	2.09	4.11	53.68	1915	11	2.955	5.8	44.63	1389	12	6.095	12.02

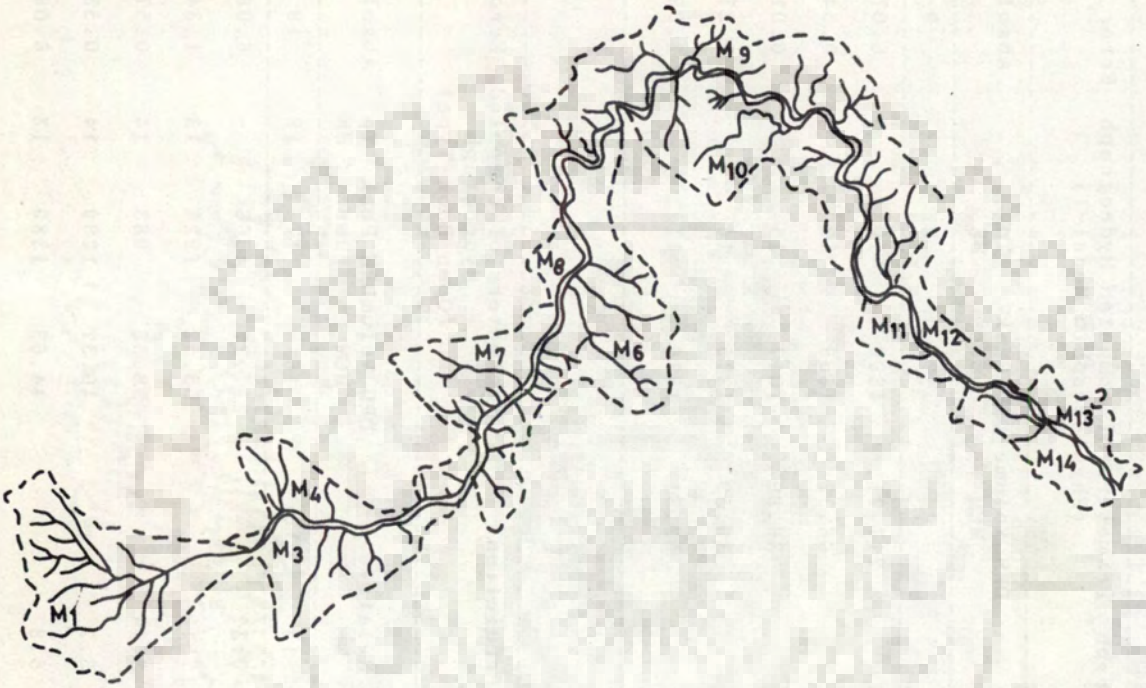


FIG. 5·10 -THE DISTRIBUTED MAIN CHANNEL SUBWATERSHEDS OF THE KOLAR RIVER are chalked out. Nine of these subwatersheds are located on the left bank of the river whereas the rest are on the right bank. The physiographic parameters of these subwatersheds are measured from topographical map drawn to a scale 1:50,000. The values of these parameters (i.e. drainage area, length of longest stream and overland slopes) are given in the first three columns of Table 5.6.

5.3.1 The Physiographic Configuration

Modeling of the tributary subwatersheds has already been explained in the previous section (Section 5.2.1). Since these subwatersheds are maintained as such, therefore, the same remain valid for this model too. However, as discussed earlier in this model, instead of one consolidated main channel subwatershed, 14 distributed subwatersheds have been chalked out. These subwatersheds are located along the main channel and each of them had a few small drainage channels. These subwatersheds are conceptually represented through rectangular planes having the areas equal to the subwatersheds. The lengths of the planes are kept equal to the stretches of the subwatersheds all along the main channel. The widths of the planes are worked out accordingly. In order to illustrate the same, the main channel subwatershed M10 with a drainage area of 42.8 km^2 is shown in Figure 5.11(a). Geographically its location covers a length of 23.5 km along the main channel. Thus as shown in Figure 5.11(b), it is conceptually represented by an equivalent rectangular plane having the same length (23.5 km) and a width of 1.82 km so as to keep its area 42.8 km^2. Following the same procedure, all the 14 subwatersheds are modeled and placed all along the length of the main channel.

Following the procedure discussed in the earlier section, the 12 tributary subwatersheds and the 14 main channel subwatersheds are folded onto the main channel to arrive at a conceptual configuration of

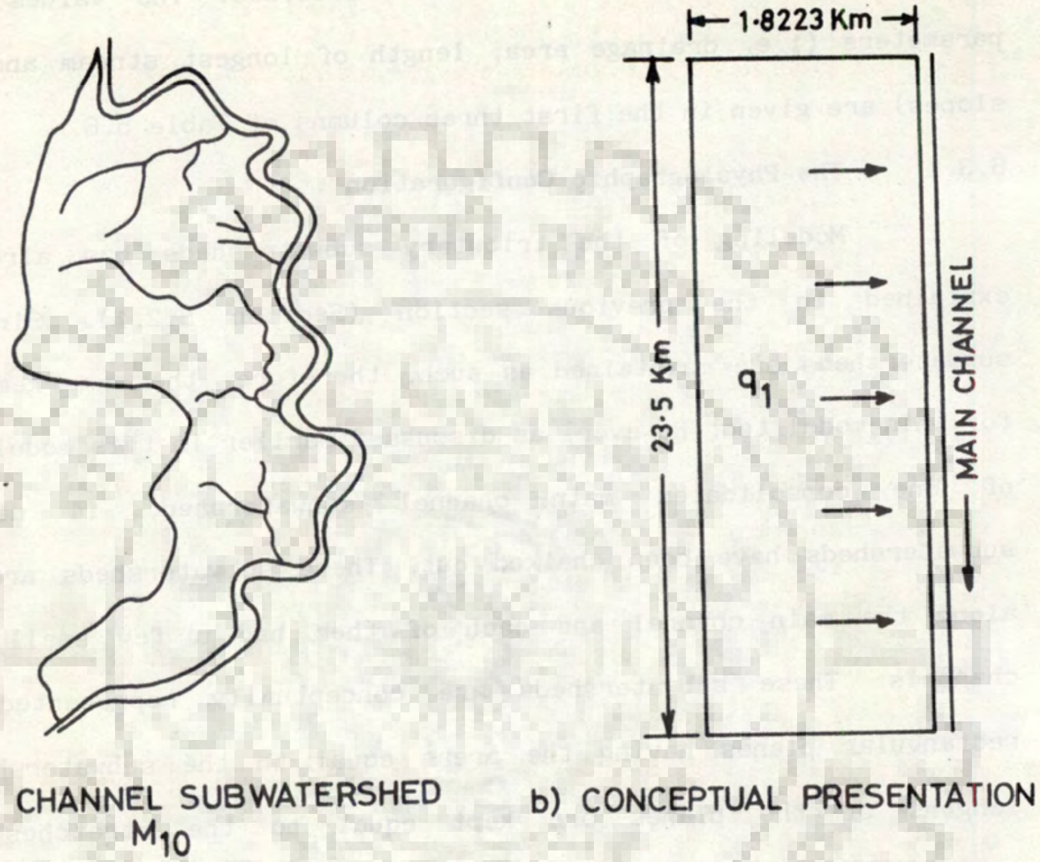


FIG. 5.11 - CONCEPTUAL REPRESENTATION OF MAIN CHANNEL SUBWATERSHED M_{10}

Table 5.6 Physiographic Parameters of the Main-channel Subwatersheds of Kolar Watershed (Physiographic Model-II)

Sl. No.	Main channel subwatershed	Drainage Area (km**2)	Length of longest stream (km)	Overland Slope	Overland width (W) (m)	α	ΔX
1	M1	53.750	13.0	0.0035	2067*	0.981	206.70
2	M2	0.600	1.5	0.0500	400.0	3.727	133.33
3	M3	26.700	12.5	0.0094	2136.0	1.612	213.60
4	M4	15.750	8.0	0.0102	1969	1.679	218.77
5	M5	4.950	5.0	0.0202	990.0	2.368	247.50
6	M6	54.800	24.5	0.0089	2236.7	1.576	223.67
7	M7	21.750	8.0	0.0147	2718.8	2.021	247.16
8	M8	6.125	5.5	0.0359	1113.6	3.159	222.72
9	M9	73.325	35.0	0.0477	2095.0	3.640	209.50
10	M10	42.825	23.5	0.0439	1822.3	3.492	202.48
11	M11	9.100	7.5	0.0165	1213.3	2.140	242.66
12	M12	10.550	9.0	0.0085	1172.2	1.539	234.44
13	M13	9.550	7.5	0.0158	1273.3	2.094	212.21
14	M14	17.125	10.5	0.0129	1630.9	1.892	203.75

*(0.5W)

the proposed model which is given in Figure 5.12.

5.3.2 The Runoff Mechanism

As mentioned in Section 5.2.3, like the previous model the runoff mechanism of this model will also consist of the following two computational phases.

(i) The overland runoff computations, and

(ii) The channel flow computations.

The KW modeling is used for the surface runoff computations of all the 26 subwatersheds. Application of this theory onto the tributary subwatersheds has already been discussed in detail while describing the Physiographic Model-I (Section 5.2). The application of KW theory for the computation of surface runoff from the 14 main channel subwatersheds will therefore be taken up in detail in this section.

The 14 main channel subwatersheds lie within the four thiessen polygons. The areal coverage of these subwatersheds under the four thiessen polygons are given in Table 5.7. As per procedure discussed earlier (Section 5.2.3(a)) the rainfall excess functions for the five storm events has been computed. The computer subroutine given in Appendix-I-A(4) also computes the weighted average rainfall excess functions over the planes influenced by more than one thiessen polygons. The rainfall excess values so computed are fed to the main computer programme to carry out the overland flow computations.

(a) Overland Flow Computations

The overland runoffs are computed by solving the KW equations 3.36 and 3.45 (Appendix-I-A(1)). For the physiographic configuration shown in Figure 5.12, the computations are carried out for the unit width of the overland planes. For all the planes, the time step is taken as 300 seconds. The space step Δx_j adopted for the 14 main channel subwatersheds are given in column 8 of Table 5.6 . The programme is run

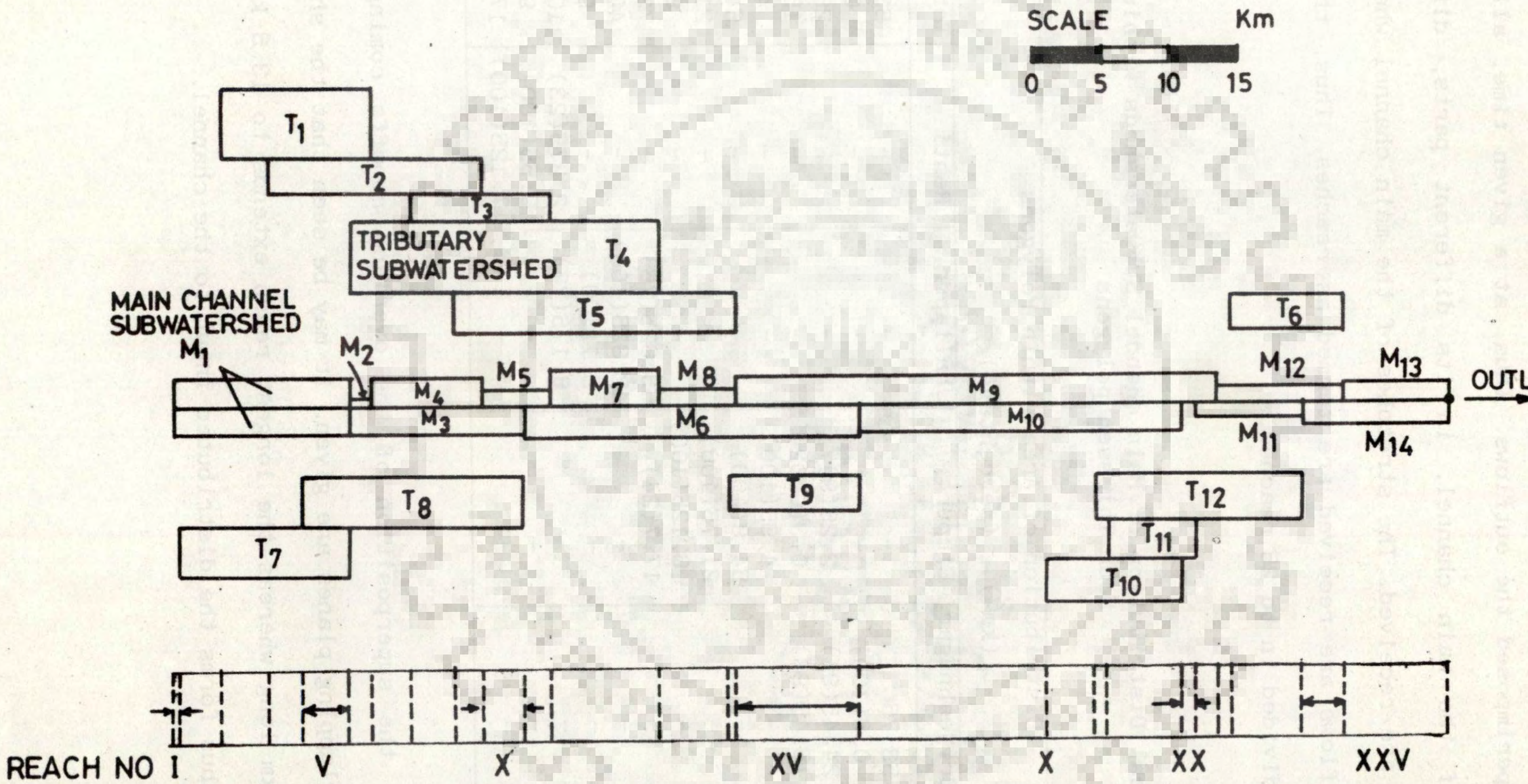


FIG. 5·12 – CONCEPTUAL CONFIGURATION OF KOLAR RIVER WATERSHED FOR PHYSIOGRAPHIC MODEL-II to compute lateral flows (q_{oj}) from all the 26 planes shown in Figure 5.12. In regions where overland planes overlapped, the computer subroutine superimposed the outflows. Thus, at a given time, all along the length of the main channel, in its different parts, different lateral flows are received. The stretches of the main channel where the same lateral flows are received are termed as reaches. Thus, the main channel gets divided in to 25 reaches.

Table 5.7 Areal Distribution of Main channel Subwatersheds within the Four Thiessen polygons

Sub-watershed	Distribution of Area within Polygons (km^2) (percentage)				
	Brijeshnagar	Birpur	Jhaliapur	Rehti	Total
M1	53.75	-	-	-	53.75
M2	0.6(100)	-	-	-	0.6
M3	24.45(91)	2.25(9)	-		26.7
M4	15.45(98)	0.3(0.02)	-	-	15.75
M5	-	4.95(100)	-	-	4.95
M6	-	54.8(100)	-	-	54.8
M7	-	21.75(100)	-	-	21.75
M8	-	6.125(100)	-	-	6.125
M9	-	17.7(24)	55.625(76)	-	73.325
M10	-	-	42.825(100)	-	42.825
M11	-	-	9.1(100)	-	9.1
M12	-	-	8.175(77)	2.375(23)	10.55
M13	-	-	-	9.55(100)	9.55
M14	-	-	-	17.125(100)	17.125

In Table 5.8, the superposition of the overland runoffs coming from different overlapping planes are given. It may be seen that the shortest reach is 0.5 km long whereas the longest reach extends to 13.5 km. The functicn q_{oj} thus forms the distributed input to the channel.

Table 5.8 Superimposed Lateral Flows to Kolar River (Physiographic Model-II)

Reach No.	Lateral Flows to the Reach q_o(Subwatershed No.)	Reach length (km)	Space Step (m)
1	$q_o(1)$	0.5	500
2	$q_o(1) + q_o(21)$	3.0	300
3	$q_o(1) + q_o(15) + q_o(21)$	3.5	3500
4	$q_o(1) + q_o(15) + q_o(16) + q_o(21)$	2.5	2500
5	$q_o(1) + q_o(15) + q_o(16) + q_o(21) + q_o(22)$	3.5	3500
6	$q_o(2)+q_o(3)+q_o(15)+q_o(16)+q_o(18)+q_o(22)$	1.5	1500
7	$q_o(3)+q_o(4)+q_o(15)+ q_o(16)+q_o(18)+ q_o(22)$	3.0	3000
8	$q_o(3)+q_o(4) + q_o(16)+q_o(17)+q_o(18)+q_o(22)$	3.0	3000
9	$q_o(3) + q_o(4)+q_o(17)+q_o(18)+q_o(19)+q_o(22)$	3.0	3000
10	$q_o(3) + q_o(5)+q_o(17)+q_o(18)+q_o(19)+q_o(22)$	3.0	3000
11	$q_o(5) + q_o(6) + q_o(17) + q_o(18) + q_o(19)$	2.0	2000
12	$q_o(6) + q_o(7) + q_o(18) + q_o(19)$	8.0	2666.67
13	$q_o(6) + q_o(8) + q_o(19)$	5.0	2500
14	$q_o(6) + q_o(8) + q_o(19) + q_o(23)$	0.5	500
15	$q_o(6) + q_o(9) + q_o(23)$	9.0	3000
16	$q_o(9) + q_o(10)$	13.5	3375
17	$q_o(9) + q_o(10) + q_o(24)$	3.5	3500
18	$q_o(9) + q_o(10) + q_o(24) + q_o(26)$	1.0	1000
19	$q_o(9) + q_o(10) + q_o(24) + q_o(25) + q_o(26)$	5.5	2750
20	$q_o(9) + q_o(25) + q_o(26)$	1.0	1000
21	$q_o(9) + q_o(11) + q_o(26)$	1.5	1500
22	$q_o(11) + q_o(12) + q_o(26)$	1.0	1000
23	$q_o(11) + q_o(12) + q_o(20) + q_o(26)$	5.0	2500
24	$q_o(12) + q_o(14) + q_o(20)$	3.0	3000
25	$q_o(13) + q_o(14)$	7.5	2500

(b) Channel Flow computations

As discussed in the application of the Physiographic Model-I, the DW theory is used for routing of flows through the main channel. The channel parameters viz. the cross-section (Figure 5.8), slope, the roughness have already been worked out (So=0.003 and n=0.026). The time weighting coefficient θ is assigned a value of 0.67. For the physiographic representation given in Figure 5.12, the space step (Δx_j) adopted for the 25 channel reaches are given in Table 5.8. The time step Δt is kept as one hour. The computer programme (Appendix-I-B) is used for the channel flow computations. For the initial and the upstream boundary conditions, a uniform flow value of 0.187 cumecs per unit width (i.e. total channel discharge 15 cumecs) is adopted . For the downstream boundary condition, the stage-velocity curve is developed and is fitted with the equation 5.4. In this equation, the values of the regression coefficient Ao through A_3 worked out to be as under.

A_o=0.4027035 A_1=1.829082 A_2=-0.2362494 A_3=0.01625036

The channel flow computations for the differential lateral flows (q_{oj}) are carried out for all the five storm events of this watershed. The data file for the storm dated 28.8.83 is given in appendix-I-D. The outputs obtained in the form of hydrographs are plotted and the same are compared with the observed hydrographs at the outlet as well as with the computed hydrographs obtained from the Physiographic Model-I (Figure 5.9 (a) through (e)).

It may be seen that inspite of complexities involved in this model, the results have not offset the accuracies and are quite satisfactory. Comparison of the significant parameters of the computed and the observed hydrographs are given in Table 5.5. The model efficiencies for all the storm events have been reported in Appendix-IV.

The conventionally used open book type physiographic model was also developed on similar lines and the same is given in the forthcoming section to study the comparative performance with the proposed models.

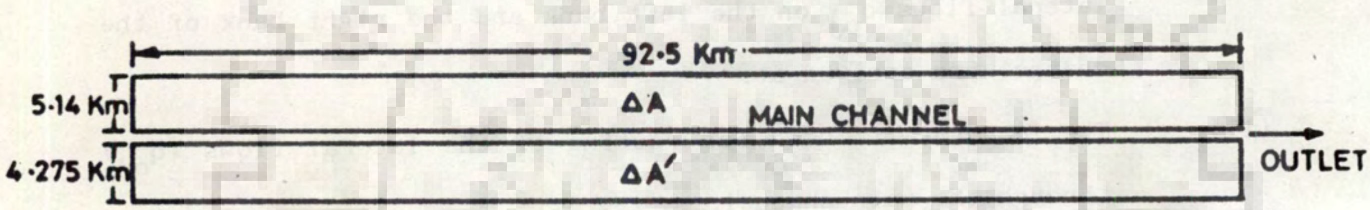
5.4 APPLICATION OF OPEN BOOK TYPE PHYSIOGRAPHIC MODEL ONTO KOLAR WATERSHED

The proposed runoff mechanism for the open book type physiographic model differs from the conventional type in the following ways.

(i) KW approach has been used for modeling of the surface flows over the two rectangular planes (Figure 3.4(b)) and thus the lateral flows q_{oj} on the left bank and the right bank of the channel are arrived at.

(ii) The DW model is used for routing of the lateral flows (q_{oj}) through the main channel.

While applying this model onto the Kolar watershed, the left bank and the right bank subwatershed areas worked out to be 475.425 km^2 and 395.425 km^2 respectively. The length of main channel is 92.5 km and therefore, the width of the two rectangular planes happen to be 5.14 km and 4.275 km respectively for the left and for the right bank. The open book type physiographic representation of Kolar watershed is shown in Figure 5.13. The uniform overland slopes of these two rectangular planes were computed by taking the weighted average of the slopes of the subwatersheds. Its value works out to be 0.01784. The overland roughness of the two planes were adopted to be the same as in the previously discussed case (i.e. n = 0.06). The space step (Δx) was taken as 513.97 m for the left bank and 427.5 m for the right bank plane. The time step (Δt) for the overland computations were taken as 1200 seconds. However, for the sever storm recorded on 28.8.1983 it was adopted as 900 seconds.



FIG·5·13–OPEN BOOK TYPE PHYSIOGRAPHIC REPRESENTATION OF KOLAR WATERSHED

For the five storm events, using the Thiessen polygon approach, average hyetographs were worked out. The ϕ-index approach was adopted to compute the rainfall excess function. These values are given in Appendix-II-C(6). The values of KW routing parameters α in this case is different as the values of the slopes are different for the two overland planes. The value of α for the left bank and the right bank planes works out to be 2.226 and 2.243 respectively. These are different from the earlier physiographic model due to differences in overland slope values. The value of parameter m is taken to be the same (i.e. m = 5/3.

Using the kinematic wave computer programme (Appendix-I-A(1)), the lateral flows were computed for one meter wide strips on the overland as discussed in section 3.4. The lateral flows on the two banks coming from the two planes are different and they are added to obtain the total lateral flow (q_{oj}) to the channel.

The DW model computer programme (Appendix-I-B) is used to route the flows through the channel. The time step Δt is taken as 3600 seconds, whereas, the Δx is adopted as 2642.8 m for the total 36 nodes on the main channel. The channel roughness and the channel slopes were computed earlier in Section 5.2.3(b), their values being n=0.026 and S_o=0.003.

The computed and observed values of runoffs for the five storm events are plotted on the Figures 5.9 (a) to (e) for making comparisons. The hydrograph parameters (i.e. volume, peak discharges and time to peak are given in Table 5.5. The model efficiencies for all the storm events are given in Appendix-IV.

To arrive at logical conclusions about the usefulness of the two proposed physiographic models (Sections 5.2 and 5.3), the same have also been applied onto two more natural watersheds (i.e Railway Bridge

No. 719 and Kassilian watersheds). These applications have been discussed in the forthcoming sections.

5.5 APPLICATION OF THE PROPOSED PHYSIOGRAPHIC MODELS ONTO THE WATERSHEDS OF RAILWAY BRIDGE NO. 719 AND THE KASSILIAN RIVER

In the previous section, application of the following two physiographic models onto the Kolar river watershed were discussed in details.

(i) Physiographic Model-I: Consisting of tributary subwatersheds and a single consolidated main channel subwatershed.

(ii) Physiographic Model-II: Consisting of tributary subwatersheds and the distributed main channel subwatersheds.

As seen in the earlier section, compared to these two categories of the proposed models, the open book type physiographic model used in the past by the researchers did not produce satisfactory responses for the Kolar river watershed. However, to arrive at logical conclusions all the three above mentioned categories of the physiographic models have also been applied onto the watersheds of Railway Bridge No. 719 and the Kassilian river where the necessary data needed for the analysis was available to some extent.

The drainage patterns of the two watersheds are given in Figures 4.1 and 4.2 and the data availability on them was discussed in Chapter - IV. Application of the proposed two models on these watersheds is discussed in the following sections.

5.5.1 Application of Physiographic Model-I onto the Railway Bridge No. 719 and the Kassilian Watersheds

In order to apply the proposed physiographic models onto these two watersheds the drainage areas are divided into tributary subwatersheds and the main channel watershed(s). As shown in Figures 5.14 (a) and (b), total number of tributary subwatersheds identified for

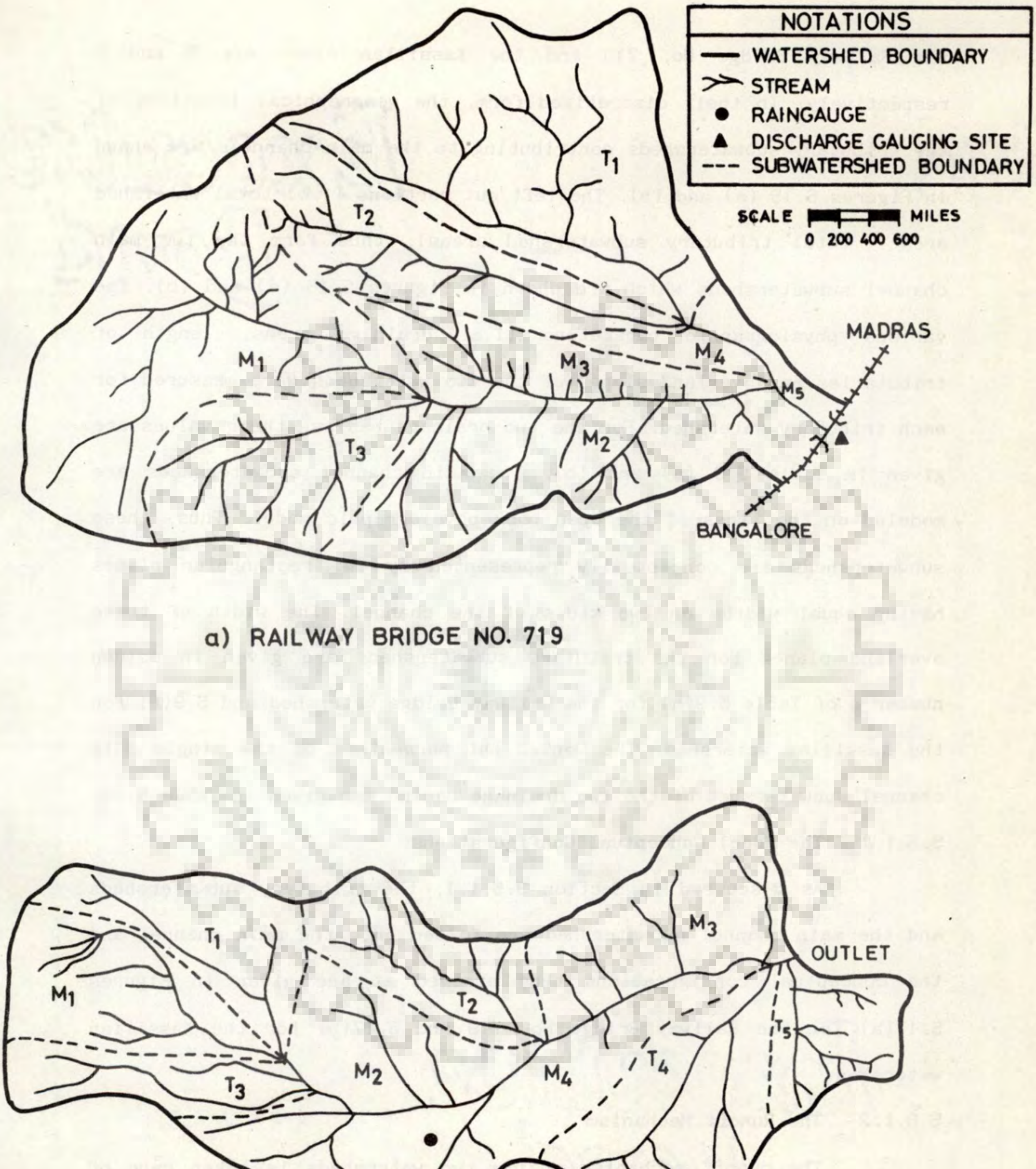


FIG. 5.14 - THE WATERSHED OF RAILWAY BRIDGE NO. 719 AND KASSILIAN RIVER AND THEIR DISTRIBUTED SYSTEMS

the Railway Bridge No. 719 and the Kassilian river are 3 and 5 respectively. In their discretized form, the geographical locations of the tributary subwatersheds contributing to the main channels are shown in Figures 5.15 (a) and (b). The left out portions (i.e. total watershed area - total tributary subwatershed areas), thus form the two main channel subwatersheds which are shown in Figures 5.16 (a) and (b). The various physiographic parameters (i.e. drainage areas, length of tributaries and overland slope) of the two watersheds were measured for each tributary watershed. For the two drainage basins, their values are given in Table 5.9 (a) and (b). The main channel subwatersheds are modeled on the lines of the open book physiographic model. Thus, these subwatersheds are conceptually represented by two rectangular planes having equal widths on two sides of the channel. The width of these overland planes for the tributary subwatersheds are given in column number 6 of Table 5.9(a) for the Railway Bridge watershed and 5.9(b) for the Kassilian watershed. The worked out parameters of the single main channel subwatershed of the two drainage basins are given in Table 5.10.

5.5.1.1 The Model Conceptual Configurations

As discussed in Section 3.5.1.1, the tributary subwatersheds and the main channel subwatershed are folded onto the main channel and the conceptual configurations thus arrived at are given in Figures 5.17(a) for the Railway Bridge No. 719 and 5.17(b) for the Kassilian watershed.

5.5.1.2 The Runoff Mechanism

The runoff mechanism of the two watersheds is taken care of through the following two computational phases.

(i) The overland runoff computations, and

(ii) The channel flow computations.

These two are described in brief as under.

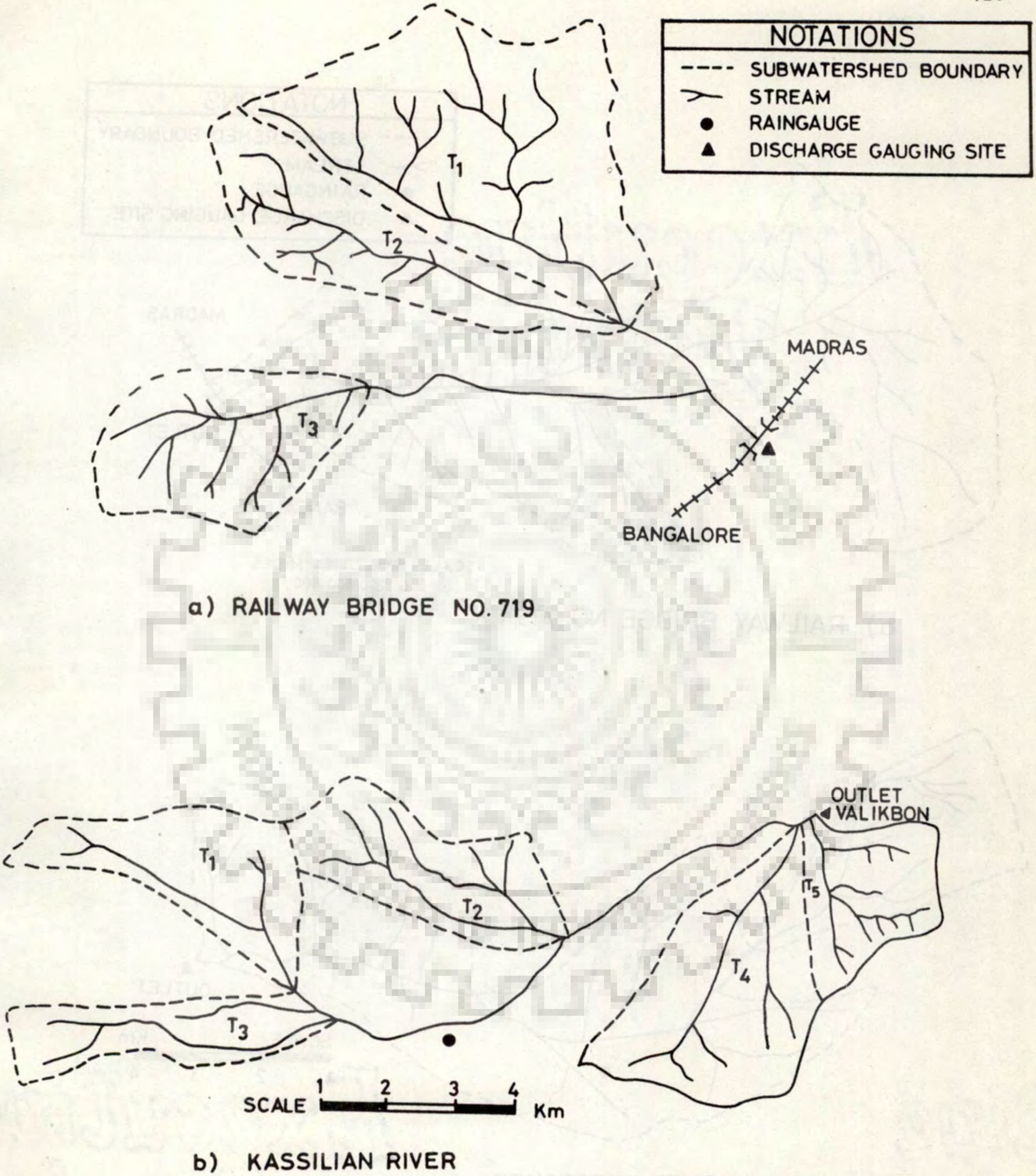
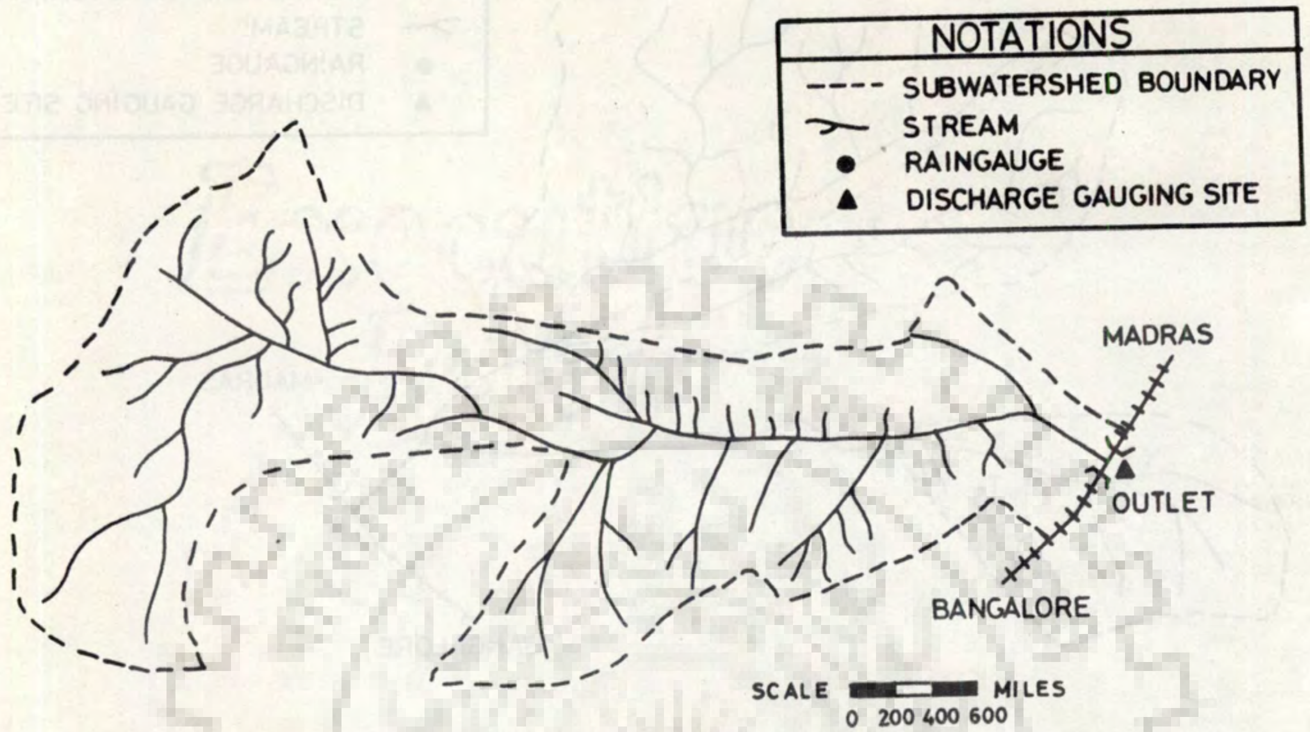
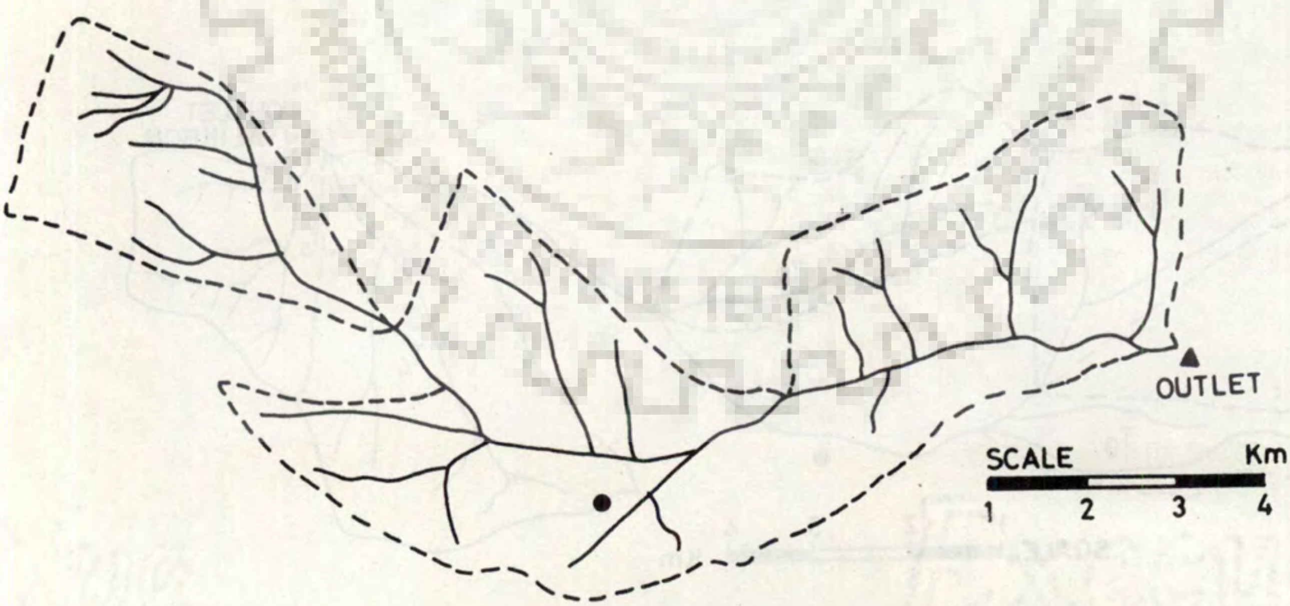


FIG. 5·15 – TRIBUTARY DRAINAGE PATTERN OF THE RAILWAY BRIDGE NO. 719 WATERSHED AND THE KASSILIAN WATERSHED



a) RAILWAY BRIDGE NO. 719



b) KASSILIAN RIVER WATERSHED

FIG. 5·16 – THE SINGLE MAIN CHANNEL SUBWATERSHEDS OF RAILWAY BRIDGE NO. 719 AND KASSILIAN WATERSHEDS

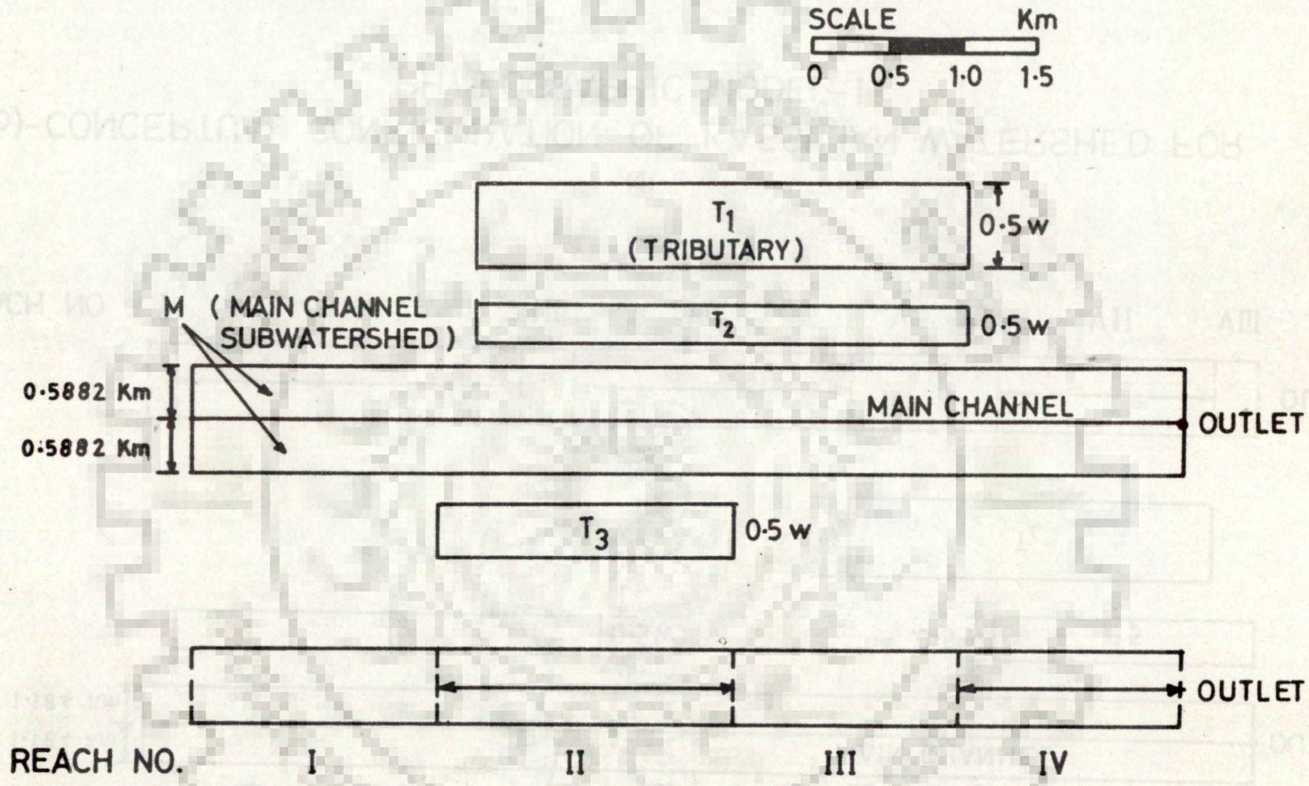


FIG. 5·17(a) - CONCEPTUAL CONFIGURATION OF THE RAILWAY BRIDGE NO. 719 WATERSHED FOR PHYSIOGRAPHIC MODEL-I

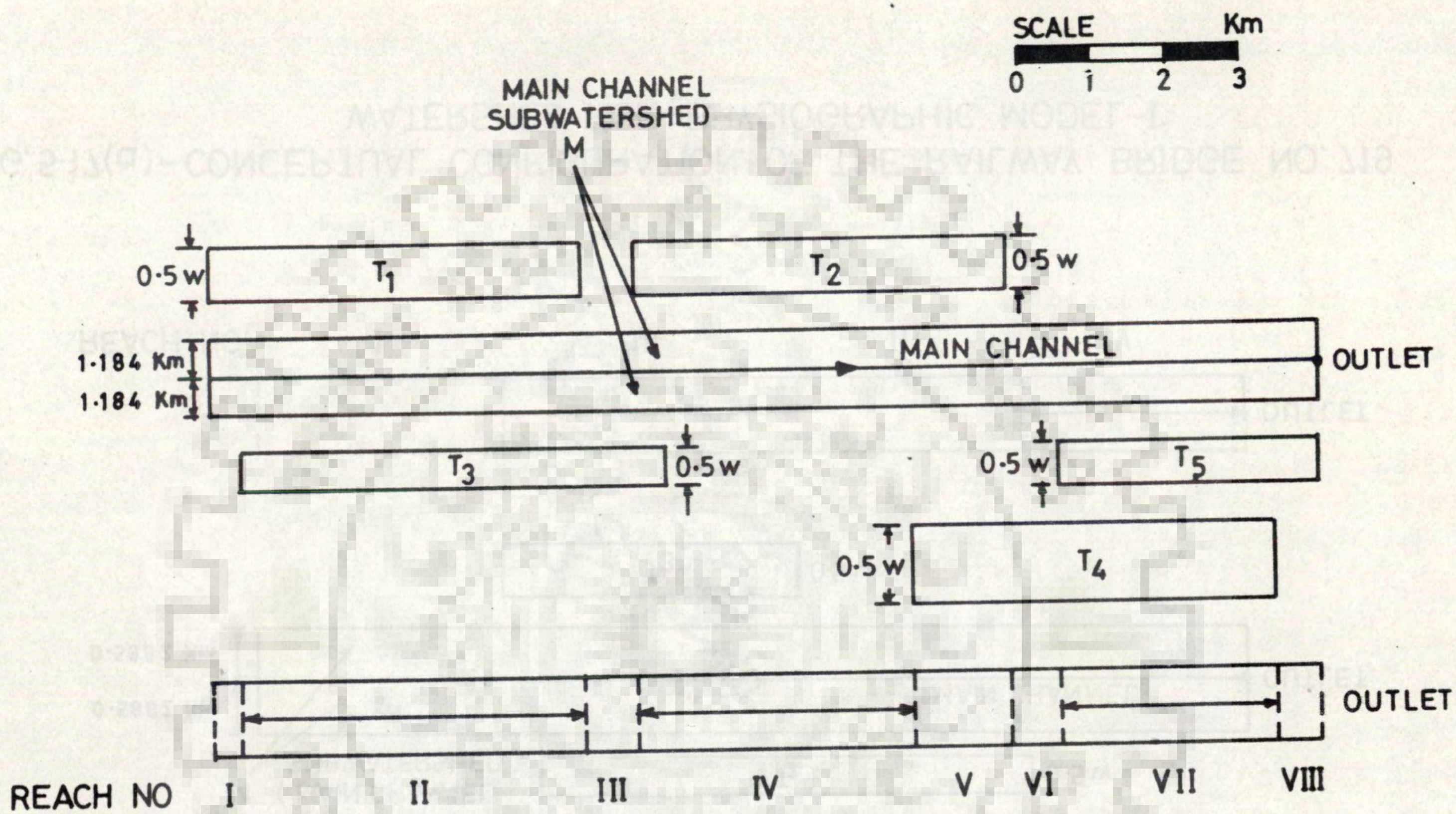


FIG. 5·17(b)-CONCEPTUAL CONFIGURATION OF KASSILIAN WATERSHED FOR PHYSIOGRAPHIC MODEL-I

Table 5.9 Physiographic Parameters of Tributary Subwatersheds

(a) Railway Bridge No.719 Watershed

Sl. No.	Tributary subwatershed	Drainage Area (km**2)	Length of Tributary (km)	Overland Slope	Overland width (0.5*W) (m)	α	ΔX (m)
1	T1	3.541	3.325	0.0481	532.5	0.681	133.13
2	T2	1.726	3.325	0.0340	259.5	0.572	86.50
3	T3	1.286	2.0	0.0680	321.5	0.810	107.17

(b) Kassilian Watershed

Sl. No.	Tributary subwatershed	Drainage Area (km**2)	Length of Ttributary (km)	Overland Slope	Overland width (0.5*W) (m)	α	ΔX (m)
1	T1	7.000	5.0	0.1400	700.0	1.740	233.33
2	T2	6.600	5.0	0.1000	660.0	1.471	220.00
3	T3	4.500	5.7	0.1490	395.0	1.795	131.00
4	T4	9.500	5.0	0.0900	950.0	1.395	316.67
5	T5	4.200	3.5	0.1140	600.0	1.570	200.00

(a) The Overland Flow Computations

The KW theory is used to compute the overland surface runoffs from the rectangular planes which conceptually represent the subwatersheds. For the Railway Bridge No.719, a comprehensive analysis

Table 5.10 The Worked out Parameters of the Single Main Channel Subwatersheds

Parameter / Watershed	Area (km^2)	length of channel (km)	Overland Slope	Width of each side plane(m)	α	Δx (m)
Railway Bridge No. 719	7.823	6.65	0.0473	588.2	0.675	19.07
Kassilian	35.2	14.86	0.1282	1184	1.665	197.39

pertaining to the effects of roughness on the computed discharges was carried out by Hossain (1989) using the KW theory. As recommended by him, an average overland roughness value of n(=0.322) is adopted. For the Kassilian watershed, from the available description of the watershed landuse and soil type, the overland roughness value n(=0.215) as recommended in the standard text books on the subject (Chow 1959) is adopted.

The overland planes were divided into 1 meter wide strips. These strips were further divided into the space steps (Δx_j) for the application of KW equations (equations 3.36 and 3.45). The values adopted for the space step (Δx_j) on the various overland planes at the two watersheds are given in the last column of Tables 5.9 (a) and (b). A uniform time step of 300 seconds was adopted for the overland flow routing. The KW parameter m was taken as 5/3 and the values of the parameter α arrived at different watersheds are given in column 7 of

Table 5.9 (a) and (b). The values of these parameters (i.e. Δx_j and α) for the two main channel subwatersheds are given in Table 5.10.

For these two watersheds, the rainfall excess functions for the given storm events have been computed by following the procedure similar to the one adopted for the Kolar watershed. These watersheds being small, only one raingauge is located on them. Therefore, the rainfall function is uniformly distributed with respect to space. The ϕ-index approach is used to compute the rainfall excess distribution in time. The worked out values of rainfall excess for the six Storm events on the watershed of Railway Bridge are given in Appendix-II-A, where as these values for the four storm events at the Kassilian watershed are given in Appendix-II-B.

The KW equations are solved (Appendix-I-A(1)) to compute surface runoffs from the tributary watersheds as well as the main channel subwatershed. In the region where the two types of the overland planes (i.e. tributary and main channel) do overlap, the computer subroutines given in Appendix-I-A(6) for the watershed of Railway Bridge No. 719 , and in Appendix-I-A(8)for the Kassilian watershed superimposes their outflows. For the two watersheds, the superposition of the lateral flows from different planes coming to the reaches are given in Tables 5.11 (a) and (b). Therefor, at a given time all along different stretches of the main channel, different lateral flows (q_{oj}) are received. The stretches of the main channel having the same lateral flows (q_{oj}) are identified as its reaches. The length of reaches in case of these two watersheds are given in Tables 5.11 (a) and (b). The function q_{oj} thus forms the distributed input to the channel reach through which the flows are routed by using the DW theory.

Table 5.11 Superimposed Lateral Flows to Main Channel of the Railway Bridge and Kassilian Watersheds (Physiographic Model-I)

(a) Railway Bridge No.719 Watershed

Reach No.	Lateral Flows to the Reach q_o(Subwatershed No.)	Reach length (km)	Space Step (m)
1	$q_o(1)$	1.65	825
2	$q_o(1) + q_o(2) + q_o(3) + q_o(4)$	2.00	1000
3	$q_o(1) + q_o(2) + q_o(3)$	1.575	787.5
4	$q_o(1)$	1.420	712.5

(b) Kassilian Watershed

Reach No.	Lateral Flows to the Reach q_o(Subwatershed No.)	Reach length (km)	Space Step (m)
1	$q_o(1) + q_o(2)$	0.36	360
2	$q_o(1) + q_o(2) + q_o(4)$	4.64	663
3	$q_o(1) + q_o(4)$	0.66	660
4	$q_o(1) + q_o(3) + q_o(4)$	0.4	400
5	$q_o(1) + q_o(3)$	3.2	800
6	$q_o(1) + q_o(3) + q_o(5)$	1.4	700
7	$q_o(1) + q_o(5)$	0.7	700
8	$q_o(1) + q_o(5) + q_o(6)$	2.9	725
9	$q_o(1) + q_o(6)$	0.6	600

(b) Channel Flow Computations

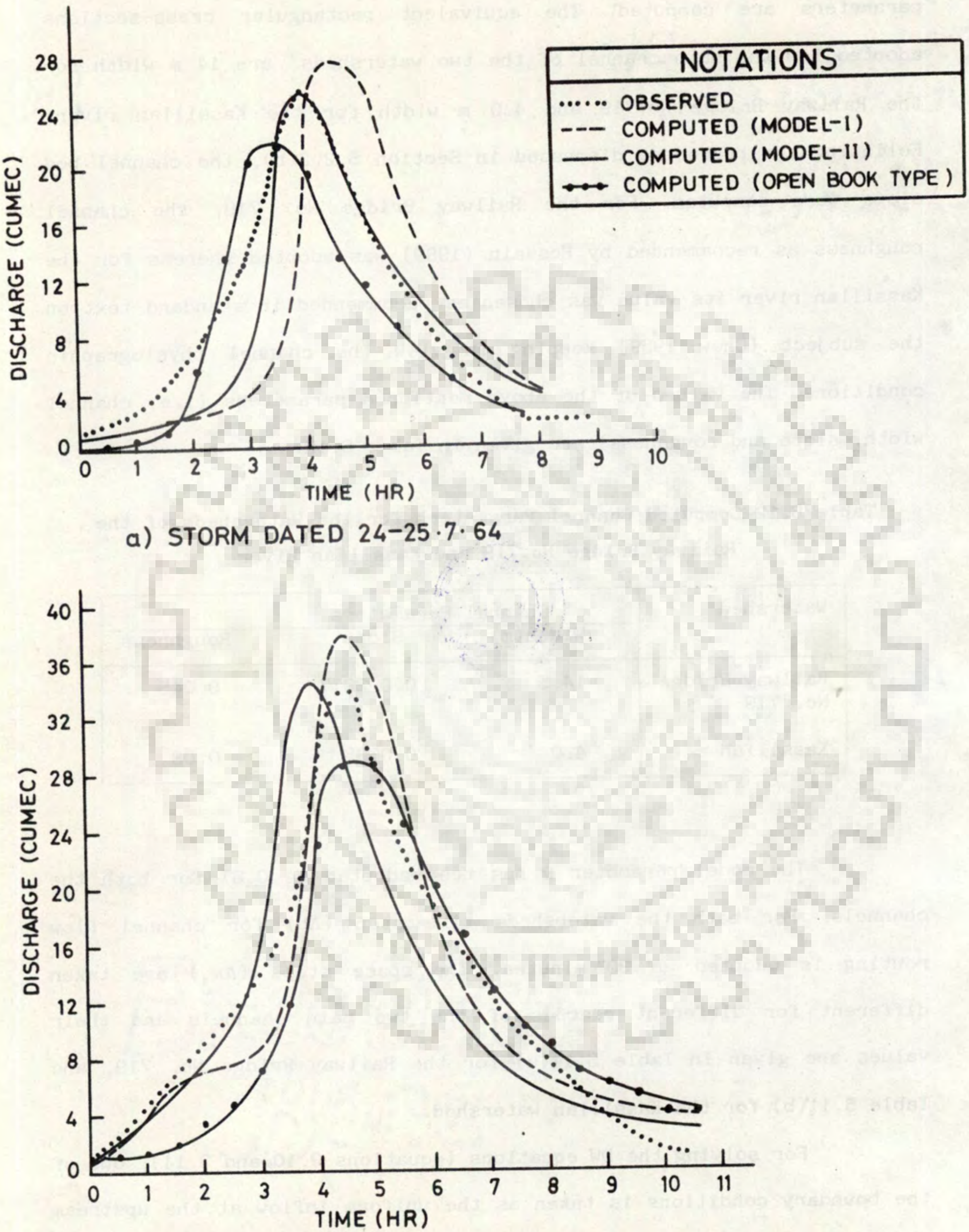
All along the main channel, the differential inputs coming from the overland planes (i.e. q_{oj}) to the different reaches of the main channel form the inputs to the channel at a particular time. For the application of the DW theory, firstly the channel physiographic parameters are computed. The equivalent rectangular cross-sections adopted for the main channel of the two watersheds are 14 m width for the Railway Bridge No.719 and 4.0 m width for the Kassilian river. Following the procedure discussed in Section 5.2.3(b), the channel bed slope were computed. For the Railway Bridge No. 719, the channel roughness as recommended by Hossain (1989) was adopted whereas for the Kassilian river its value was chosen as recommended in standard text on the subject (Chow 1959) keeping in view the channel physiographic conditions. The values of the above mentioned parameters (i.e. channel width, slope and roughness) are given in Table 5.12.

Table 5.12 Computed Channel Parameters for the Watersheds of the Railway Bridge No.719 and Kassilian River

Watershed	Channel Parameters		
	Width(m)	Slope	Roughness
Railway Bridge No. 719	14.5	0.022917	0.045
Kassilian	4.0	0.0635	0.08

The model parameter θ has been adopted as 0.67 for both the channels. For both the watersheds time step (Δt) for channel flow routing is adopted as 30 minutes. The space steps (Δx_j) are taken different for different reaches of the two main channels and their values are given in Table 5.11(a) for the Railway Bridge No. 719, and Table 5.11(b) for the Kassilian watershed.

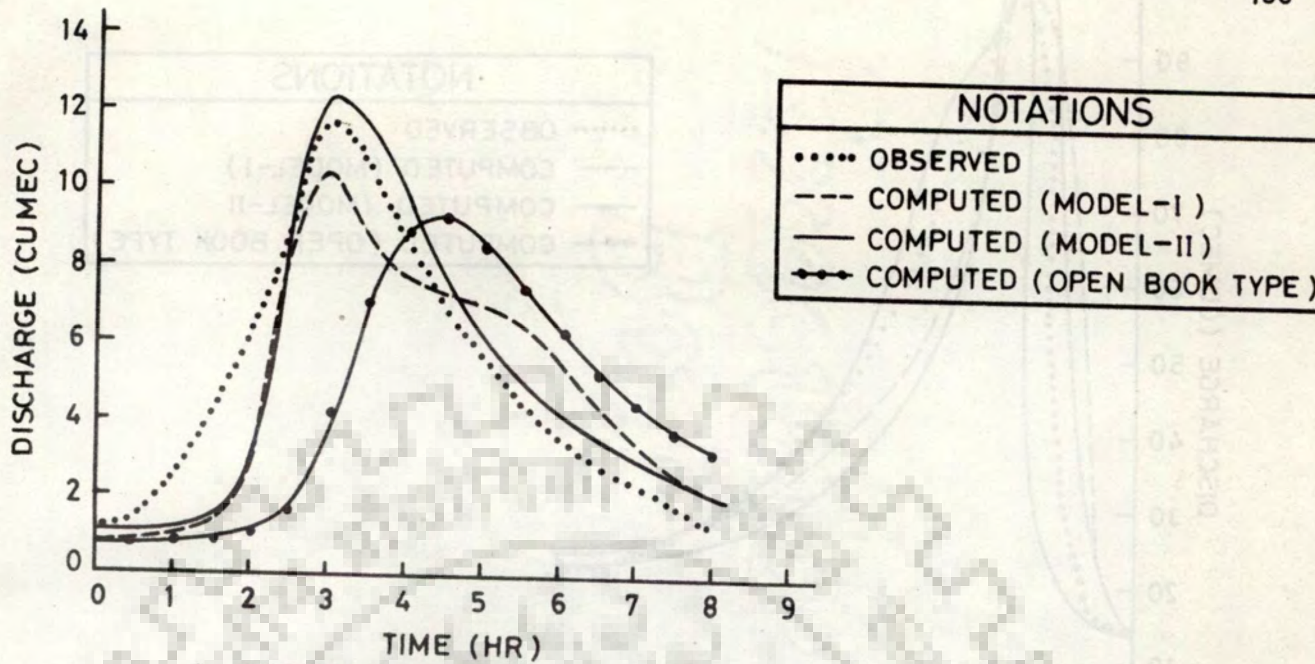
For solving the DW equations (equations 3.10 and 3.11), one of the boundary conditions is taken as the uniform inflow at the upstream end of the channel. It is adopted as 3 cumecs for the Railway Bridge watershed and 0.5 cumec for the Kassilian watershed. The downstream boundary conditions are in the form stage-velocity curves. These have



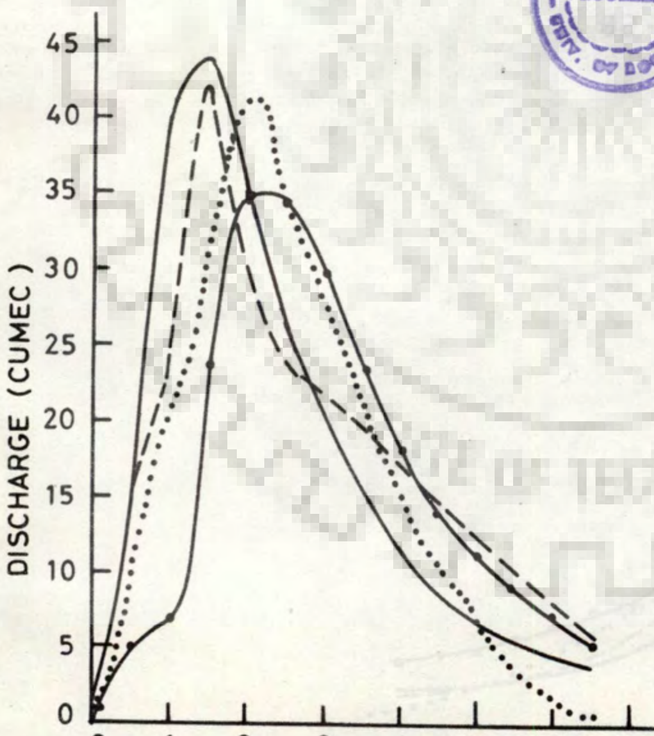
a) STORM DATED 24-25·7·64

b) STORM DATED 26-27·7·64

FIG. 5·18 a & b-COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF THE RAILWAY BRIDGE NO. 719

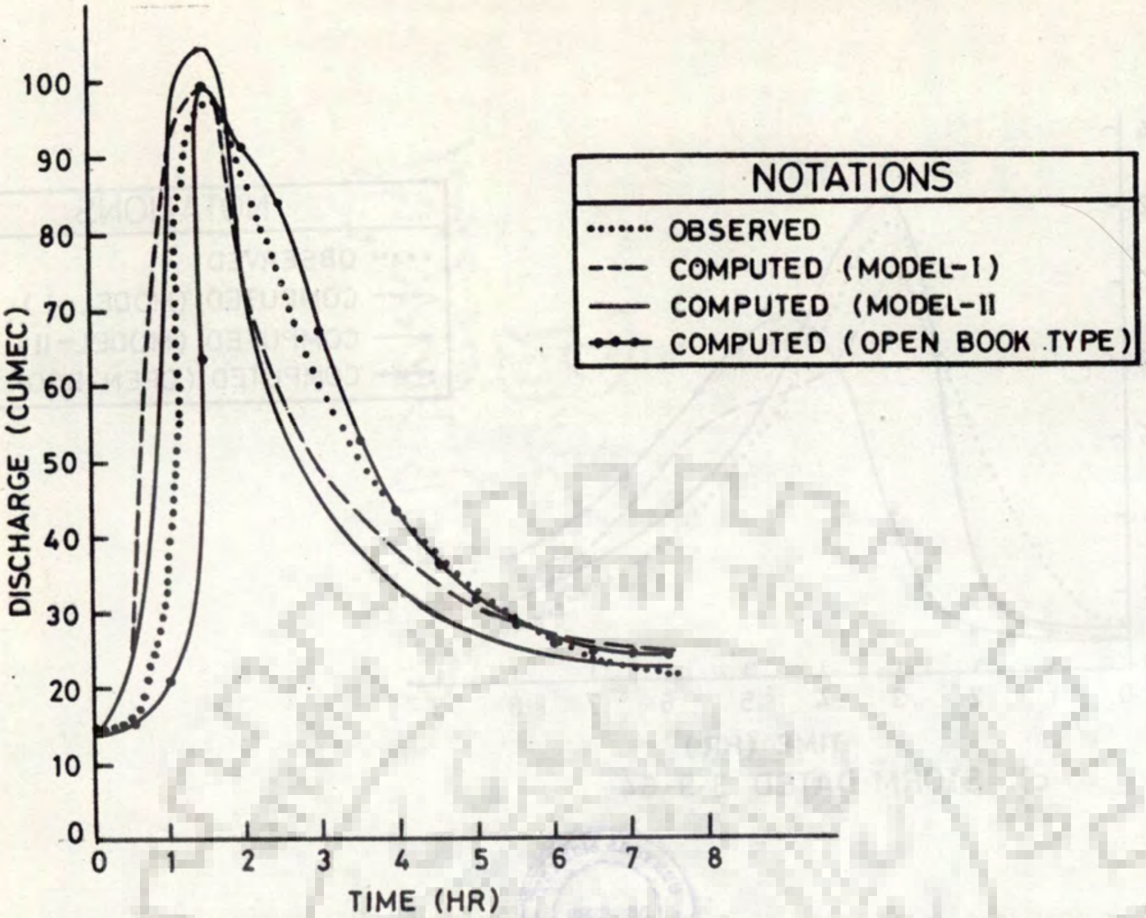


c) STORM DATED 3·9·64

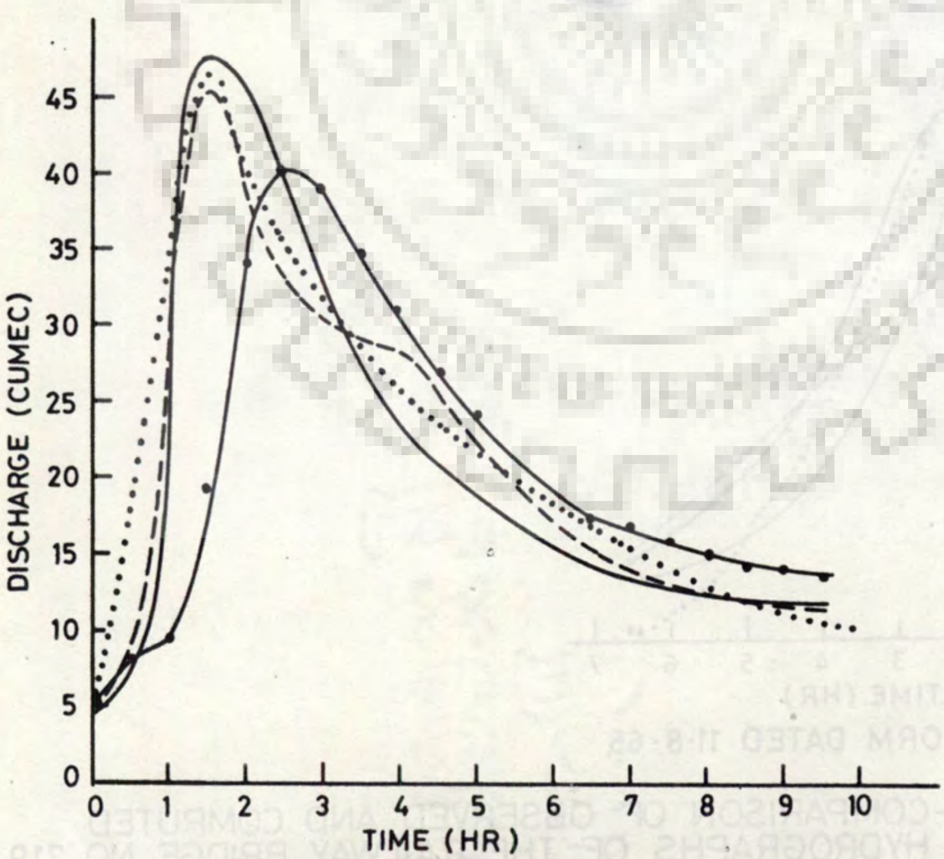


d) STORM DATED 11·8·65

FIG. 5·18 c & d–COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF THE RAILWAY BRIDGE NO. 719



e) STORM DATED 16·9·66



f) STORM DATE 19·9·66

FIG. 5·18 e & f–COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF THE RAILWAY BRIDGE NO. 719

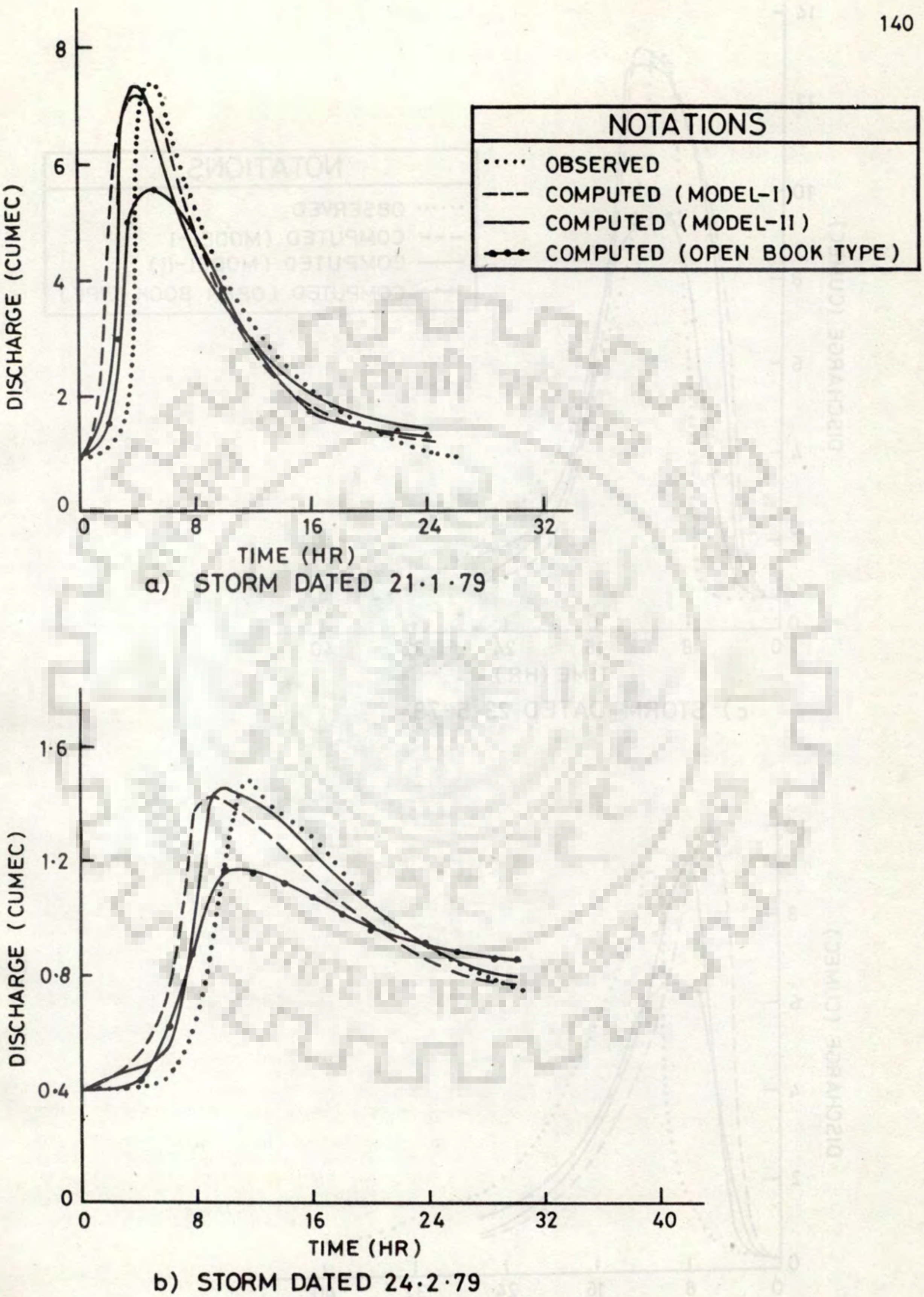
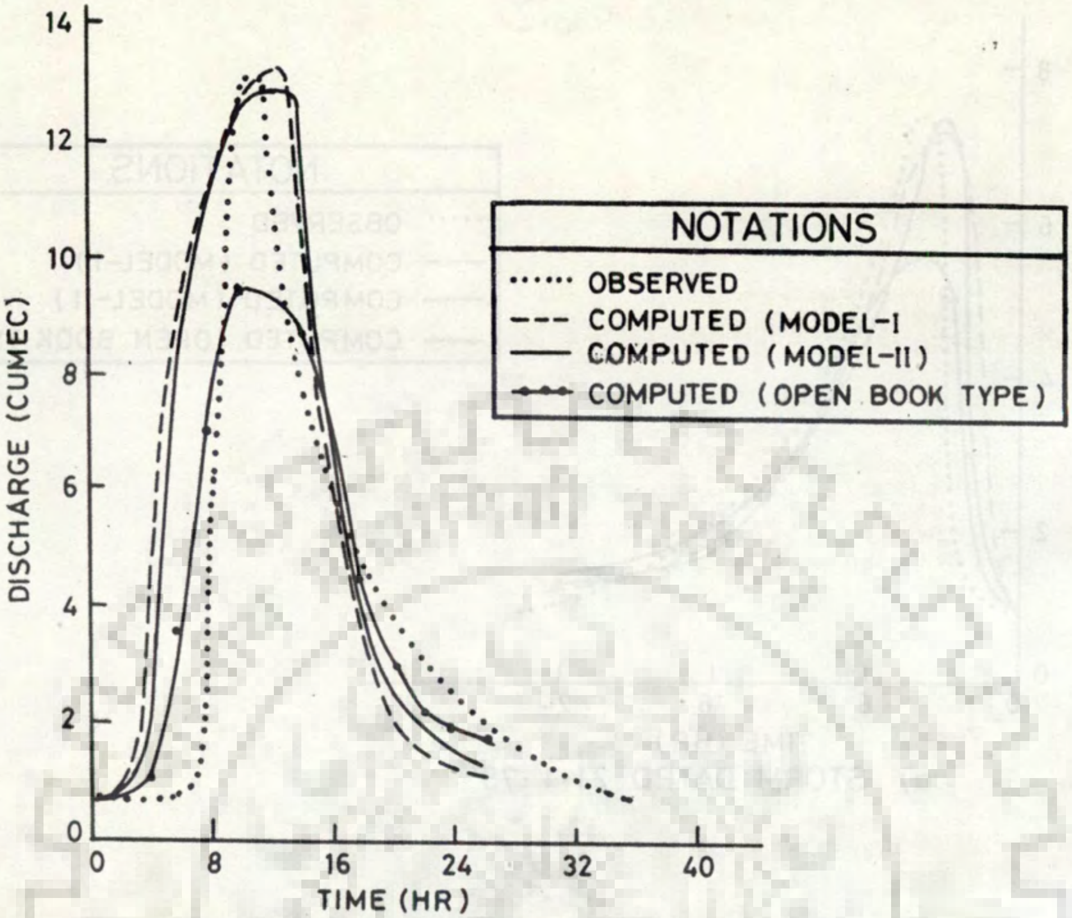
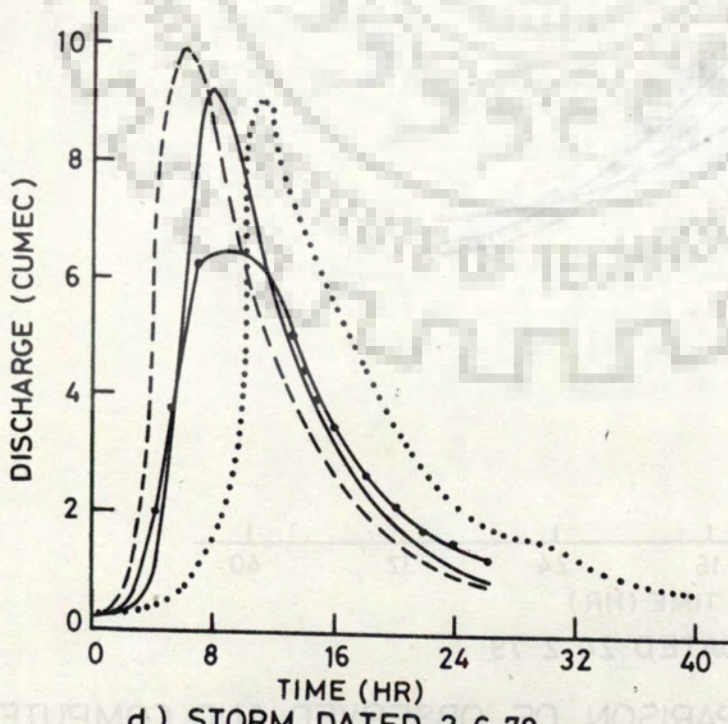


FIG. 5·19 a & b – COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF KASSILIAN WATERSHED



c) STORM DATED 23·5·79



d) STORM DATED 2·6·79

FIG. 5·19 c & d–COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF KASSILIAN WATERSHED been computed by following the procedure discussed in section 5.2.3(b). The coefficients of the nonlinear regression equation (equation 5.4) arrived at for the two watersheds are given in Table 5.13.

Table 5.13 Coefficients of Stage-Velocity Curve at the Railway Bridge No. 719 and Kassilian River Outlet

Watershed	Coefficients of Stage-Velocity Curve			
	Ao	A_1	A_2	A_3
Railway Bridge No. 719	0.2528638	4.891208	-3.220212	1.159854
Kassilian	0.5284337	2.656156	-0.8976812	0.1152283

The DW equations 3.10 and 3.11 are solved (Appendix-I-B) to compute the discharges at the outlet. The computed discharges are compared with the observed hydrographs for the six storm events recorded on watershed of Railway Bridge No, 719. This comparison is shown in Figures 5.18(a) through (e). Similar comparisons for the 4 storm events of Kassilian watershed are shown in Figure 5.19(a) through (d). Comparison of the significant parameters of the computed and the observed hydrographs are given in Tables 5.14 (a) and 5.14 (b). The model efficiencies for all the storm events have been reported in Appendix-IV.

5.5.2 Application of Physiographic Model-II onto the Railway Bridge No. 719 and Kassilian Watersheds

In the application of this model, the "tributary subwatersheds" (Figures 5.15 (a) and (b)) are maintained for hydrologic modeling. The computed values of parameters for the tributary subwatersheds as mentioned in the previous section are given in Tables 5.9 (a) and (b). For the proposed model, the main channel subwatershed (i.e. shown in Figures 5.16 (a) and (b)) located all along the main

Table 5.14(a) Comparison of parameters of the Computed Hydrographs (Using Different Physiographic Models) with the observed Hydrographs of the Railway Bridge No. 719 watershed

Sl. No.	Storm Dated	Parameters of Observed Hydrograph			Prameters of computed Hydrograph (Physiographic Model-I)			Error in Prediction	
		DRH Volume m**3	DRH Peak cumec	Tp HR	DRH Volume m3	DRH Peak cumec	Tp HR	Absolute	Relative %
1	2	3	4	5	6	7	8	9	10
1	24-25.7.64	255893	24.09	4.0	280068	27.06	4.5	24175	9.45
2	26-27.7.64	411585	32.64	4.5	444802	37.58	4.5	33217	8.07
3	3.9.64	121477	10.48	3.0	125454	9.23	3.0	3977	3.3
4	11.8.65	414029	39.96	2.0	443540	29.82	2.0	29511	7.1
5	16.9.66	661296	80.55	1.5	71238	82.44	1.5	590058	8.9
6	19.9.66	458594	38.42	1.5	481661	37.64	1.5	23067	5.03

Prameters of computed Hydrograph (Physiographic Model-II)			Error in Prediction		Prameters of computed Hydrograph (Open Book Type)			Error in Prediction	
DRH Volume cumec	DRH Peak cumec	Tp HR	Absolute	Relative %	DRH Volume cumec	DRH Peak cumec	Tp HR	Absolute	Relative %
11	12	13	14	15	16	17	18	19	20
257042	24.59	4.0	1149	0.4	241077	20.14	5.0	14816	5.8
406667	33.5	4.0	4918	1.19	37678	28.08	4.5	34804	8.46
126614	11.44	3.0	5167	4.22	114912	8.04	4.5	6564	5.4
415265	43.07	1.5	1236	0.29	403825	33.71	2.0	10204	2.46
665391	88.26	1.5	4095	0.60	659735	73.97	2.0	1561	0.23
454459	40.25	1.5	4135	0.90	434391	32.29	2.5	24203	5.27

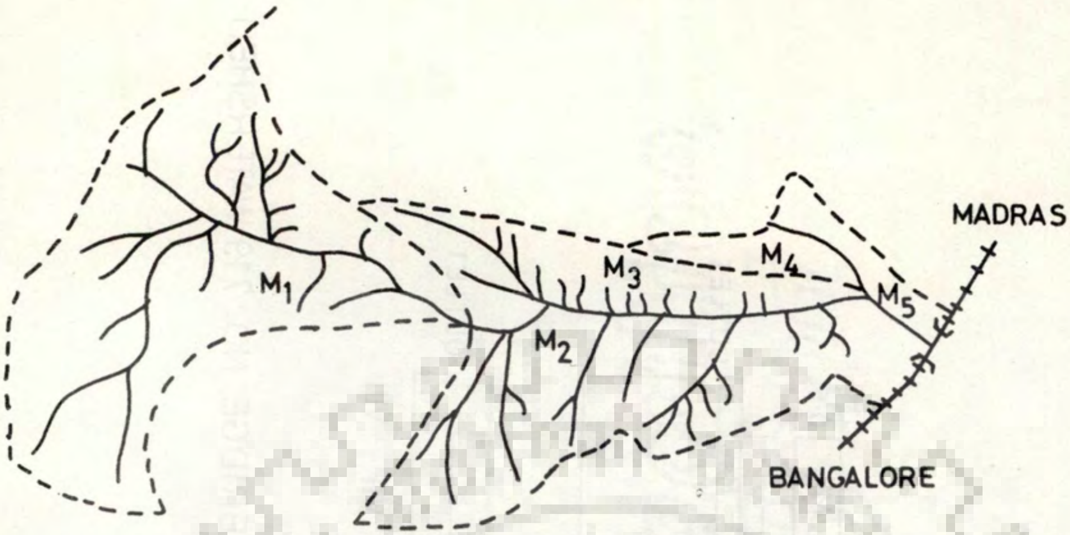
Table 5.14(b) Comparison of parameters of the Computed Hydrographs (Using Different Physiographic Models) with the observed Hydrographs of the Kassilian Watershed

Sl. No.	Storm Dated	Parameters of Observed Hydrograph			Prameters of computed Hydrograph (Physiographic Model-I)			Error in Prediction	
		DRH Volume m**3	DRH Peak cumec	Tp HR	DRH Volume m**3	DRH Peak cumec	Tp HR	Absolute	Relative %
1	2	3	4	5	6	7	8	9	10
1	21.1.79	175540	6.54	5	187324	6.31	4	11784	6.70
2	24.2.79	71891	1.10	12	63148	1.02	9	8743	12.16
3	23.5.79	389605	12.25	11	423684	12.56	13	34079	8.75
4	2.6.79	312220	8.74	11	328180	10.08	9	15960	5.11

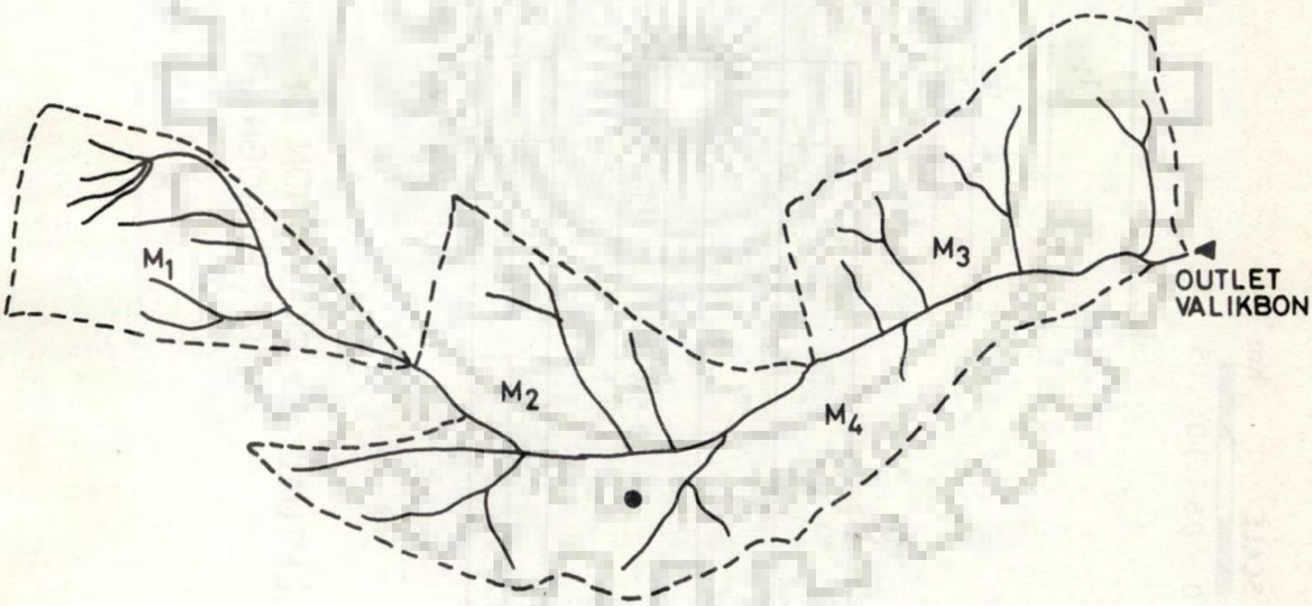
Prameters of computed Hydrograph (Physiographic Model-II)			Error in Prediction		Prameters of computed Hydrograph (Open Book Type)			Error in Prediction	
DRH Vol m**3	DRH Peak cumec	Tp HR	Absolute	Relative %	DRH Volume m**3	DRH Peak cumec	Tp HR	Absolute	Relative %
11	12	13	14	15	16	17	18	19	20
171853	6.33	4	3687	2.1	164132	4.70	5	11408	6.49
70142	1.06	10	1749	2.4	60048	1.17	8	11843	16.47
381210	10.26	9	8395	2.2	365368	10.37	12	24237	6.22
305832	9.00	8	6388	2.0	299148	11.35	9	13072	4.18

channel is further split up. As shown in Figures 5.20 (a) and (b), following the water divide the main channel subwatershed is further divided into smaller units. Both the categories of subwatersheds are now conceptually represented through equivalent rectangular planes. The physiographic conceptual model representations of the two watersheds so arrived are shown in Figure 5.21 (a) and (b). The values of overland slopes were worked out for the main channel subwatersheds and the same are given in Tables 5.15 (a) and (b) along with the other physiographic parameter values.

In order to compute the overland surface runoffs forming the lateral flows (q_{oj}) to the main channel, the KW model is applied. On the two watersheds under consideration, the rainfall excess functions remain the same as worked out in Section 5.5.1.2, which describes the Physiographic Model-I. For the two watersheds, the KW routing parameter m was taken as 5/3 and the values of the parameter α_j worked out for different subwatersheds are given in Table 5.15 (a) and (b), and 5.15 (a) and (b). In this computer programme the subroutines given in Appendices -I-A(7) and I-A(9) are called which superimpose the lateral flows (q_{oj}) from different overlapping planes over the two watersheds. For the application of this programme, the space steps (Δx_i) adopted for for the two watersheds are given in Tables 5.9 (a) and (b), and Tables 5.15 (a) and (b). A uniform time step (Δt) of 300 seconds is used for both the watersheds. In regions where the overland planes overlap, computer subroutines given in Appendix-I-A(7) (for Railway Bridge No. 719) and in Appendix-I-A(9) (for the Kassilian watershed) superimpose the outflows. Thus, at a given time all along the length of the main channel in its different part different lateral flows (q_{0j}) are received. The stretches of the main channel where the same lateral flows are received are termed as reaches. Tables 5.16 (a) and (b) give the



a) WATERSHED OF RAILWAY BRIDGE NO. 719



b) KASSILIAN WATERSHED

FIG. 5·20 a & b–THE DISTRIBUTED MAIN CHANNEL SUBWATERSHEDS OF RAILWAY BRIDGE NO. 719 AND KASSILIAN WATERSHEDS

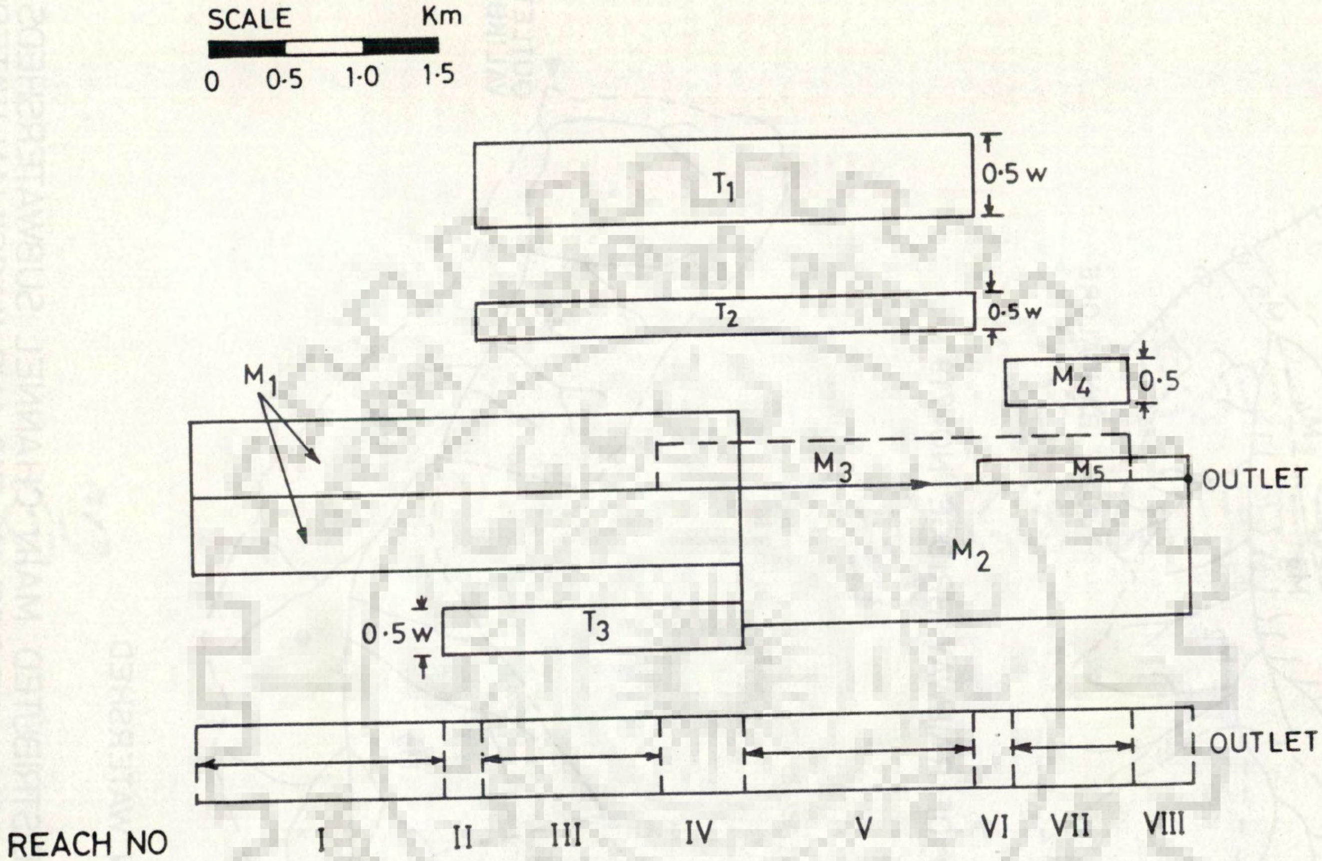


FIG. 5·21(a)- CONCEPTUAL CONFIGURATION OF THE RAILWAY BRIDGE NO. 719 WATERSHED FOR PHYSIOGRAPHIC MODEL-II

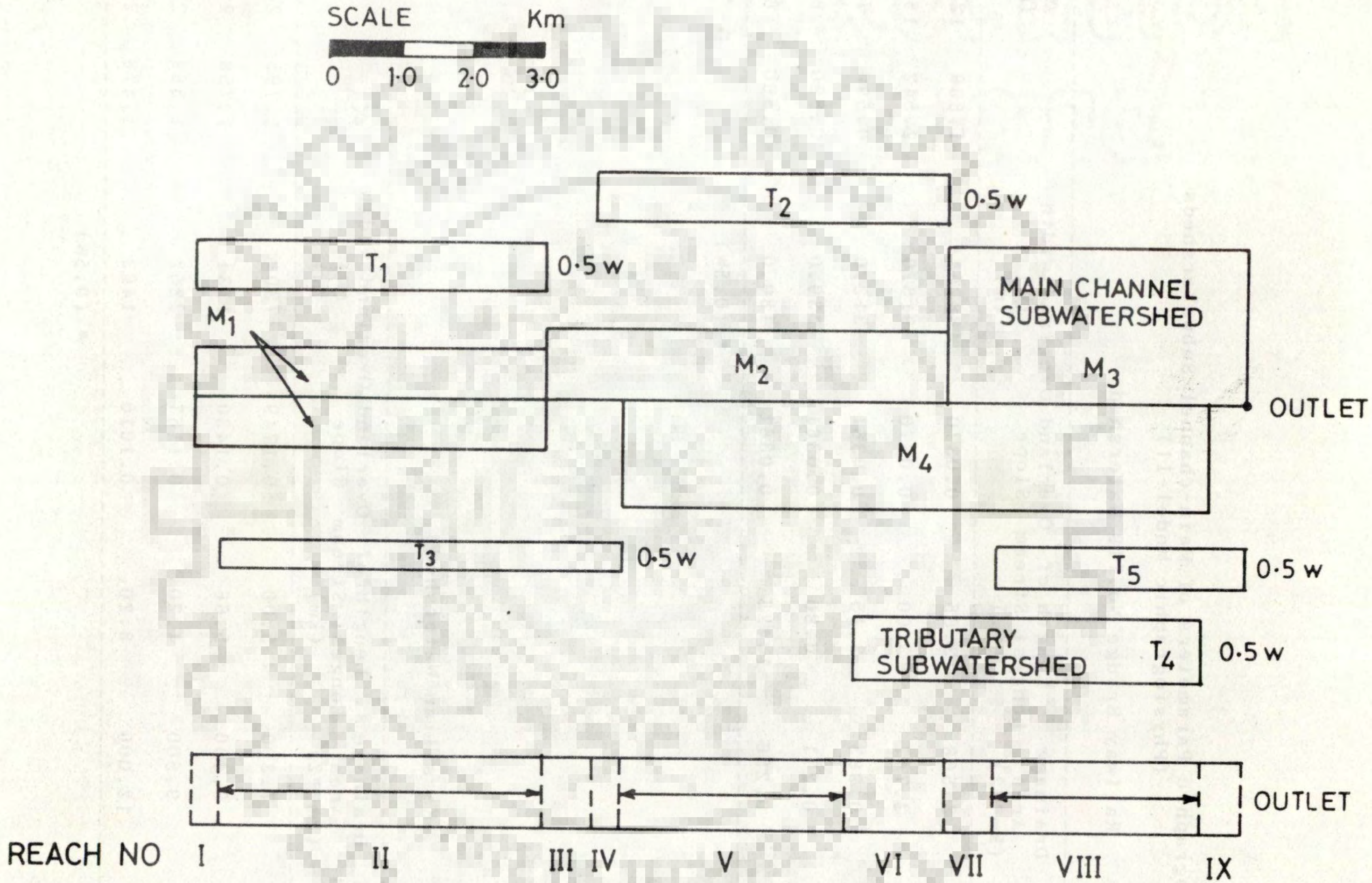


FIG. 5·21 (b)- CONCEPTUAL CONFIGURATION OF THE KASSILIAN WATERSHED FOR PHYSIOGRAPHIC MODEL-II

Table 5.15 Physiographic Parameters of Main-channel Subwatersheds (Physiographic Model-II)

(a) Railway Bridge No.719 Watershed

Sl. No.	Main-channel subwatershed	Drainage Area (km**2)	Length of Longest Stream (km)	Overland Slope	Overland width (W) (m)	α	ΔX (m)
1	M1	3.566	3.65	0.0370	488.5*	1.889	122.12
2	M2	2.747	3.00	0.0670	916.0	2.542	152.67
3	M3	0.983	3.15	0.0361	312.0	0.590	78.00
4	M4	0.271	0.80	0.0361	339.0	0.590	84.75
5	M5	0.256	1.425	0.0361	180.0	0.590	60.00

* (0.5W)

(b) Kassilian Watershed

Sl. No.	Main-channel subwatershed	Drainage Area (km**2)	Length of Longest Stream (km)	Overland Slope	Overland width (W) (m)	α	ΔX (m)
1	M1	7.300	5.00	0.1749	730*	1.795	243.33
2	M2	6.400	5.66	0.1430	1131	1.758	226.20
3	M3	9.500	4.20	0.1167	2262	1.589	251.33
4	M4	12.000	8.20	0.1010	1463	1.478	244.00

* (0.5W)

superposition of the overland flow components and the formation of 8 reaches in the main channel of the watershed of Railway Bridge No. 719 and 9 reaches in the watershed of Kassilian river. The distributed overland runoffs (q_{0j}) coming to the channel are routed through it usirg the DW equations (equations 3.10 and 3.11).

The DW model has been applied to route the flows through the main river. The model parameters were taken to be the same as mentioned in the previous configurations i.e. θ =0.67 and Δt=30 minutes. The channel roughness and its slope is adopted to be the same as mentioned in the previous section and given in Table 5.12. For the upstream boundary condition a constant flow of 3 cumecs for the watershed of Railway Bridge No. 719 and 0.5 cumecs for the Kassilian watershed have been adopted. For the downstream boundary condition the stage_velocity relationships established and the regression coefficients are given in Table 5.13 for the outlets of the two watersheds. As shown in Figure 5.21(a), the main channel of Railway Bridge watershed was divided into eight reaches. The space steps adopted for different reaches are given in Table 5.16(a). Similarly, for the Kassilian river watershed the river stretch was divided into nine reaches. Each reach has a uniform lateral flow (q_{oj}) and the Δx adopted for these reaches are given in Table 5.16(b) The computed hydrographs are obtained by using the computer programme given in Appendix-I-B. These computed discharge values are at the outlet are plotted in Figures 5.18 (a) through (f) for the six storm events of the Railway Bridge watershed and in Figures 5.19 (a) through (d) for the four storm events of Kassilian watershed and are compared with the observed hydrographs. Comparison of the significant parameters of the computed and the observed hydrographs are given in Tables 5.14 (a0 and (b). The model efficiencies for various storm events of the two watersheds are given in Appendix-IV.

Table 5.16 Superimposed Lateral Flows to Main Channel (Physiographic Model-II)

(a) The watershed of Railway Bridge No. 719

Reach No.	Lateral Flows to the Reach q_o(Subwatershed No.)	Reach length (km)	Space Step (m)
1	$q_o(1)$	1.65	825
2	$q_o(1) + q_o(8)$	0.225	225
3	$q_o(1) + q_o(6) + q_o(7) + q_o(8)$	1.24	620
4	$q_o(1) + q_o(3) + q_o(6) + q_o(8)$	0.535	535
5	$q_o(2) + q_o(3) + q_o(6) + q_o(7)$	1.575	787.5
6	$q_o(2) + q_o(3) + q_o(5)$	0.24	240
7	$q_o(2) + q_o(3) + q_o(4) + q_o(5)$	0.80	800
8	$q_o(2) + q_o(5)$	0.385	385

(b) Kassilian Watershed

Reach No.	Lateral Flows to the Reach q_o(Subwatershed No.)	Reach length (km)	Space Step (m)
1	$q_o(1) + q_o(5)$	0.36	360
2	$q_o(1) + q_o(5) + q_o(7)$	4.64	663
3	$q_o(2) + q_o(7)$	0.66	660
4	$q_o(2) + q_o(6) + q_o(7)$	0.4	400
5	$q_o(2) + q_o(4) + q_o(6)$	3.2	640
6	$q_o(2) + q_o(4) + q_o(6) + q_o(8)$	1.4	700
7	$q_o(3) + q_o(4) + q_o(8)$	0.7	700
8	$q_o(3) + q_o(4) + q_o(8) + q_o(9)$	2.9	725
9	$q_o(3) + q_o(9)$	0.6	600

5.5.3 Application of Open Book Type Physiographic Model

Application of the open book type physiographic model for the Kolar river watershed is discussed in Section 5.4. Following the similar lines, this model has been applied onto these two watersheds (viz. Railway Bridge No. 719 and Kassilian). The drainage areas on the two sides of the main channel have been conceptually represented through rectangular planes. Length of the planes is kept equal to the main channel length but their widths are so worked out. The values of the plane width for the two watersheds are given in Table 5.17. The conceptual representations of these two watersheds are shown in Figures 5.22 (a) and (b). The worked out physiographic parameters, flow parameters and the space steps Δx_i adopted for the overland and channel routing phases are given in Table 5.17.

The KW theory is used for the overland runoff computations which forms the lateral flows to the channel. The total lateral flows coming to the main channel are thus the sum of q_{oj} coming from the left bank and from the right bank planes. The DW model (Appendix-I-B) has been used to route the flows through the main channel. The discharges computed at the outlets of the two watersheds are marked on Figures 5.18 (a) through (f) and Figures 5.19 (a) through (d), for the watersheds of the Railway Bridge and and the Kassilian respectively. Comparison of the significant parameters of the computed and the observed hydrographs are given in Tables 5.14 (a) and (b). The model efficiencies for various storm events have been reported in Appendix-IV.

5.6 GENERAL REMARKS

In Sections 5.2 and 5.3, the application of the proposed two physiographic models onto the watersheds of Kolar river was discussed in details whereas, the application of these models onto the watersheds of

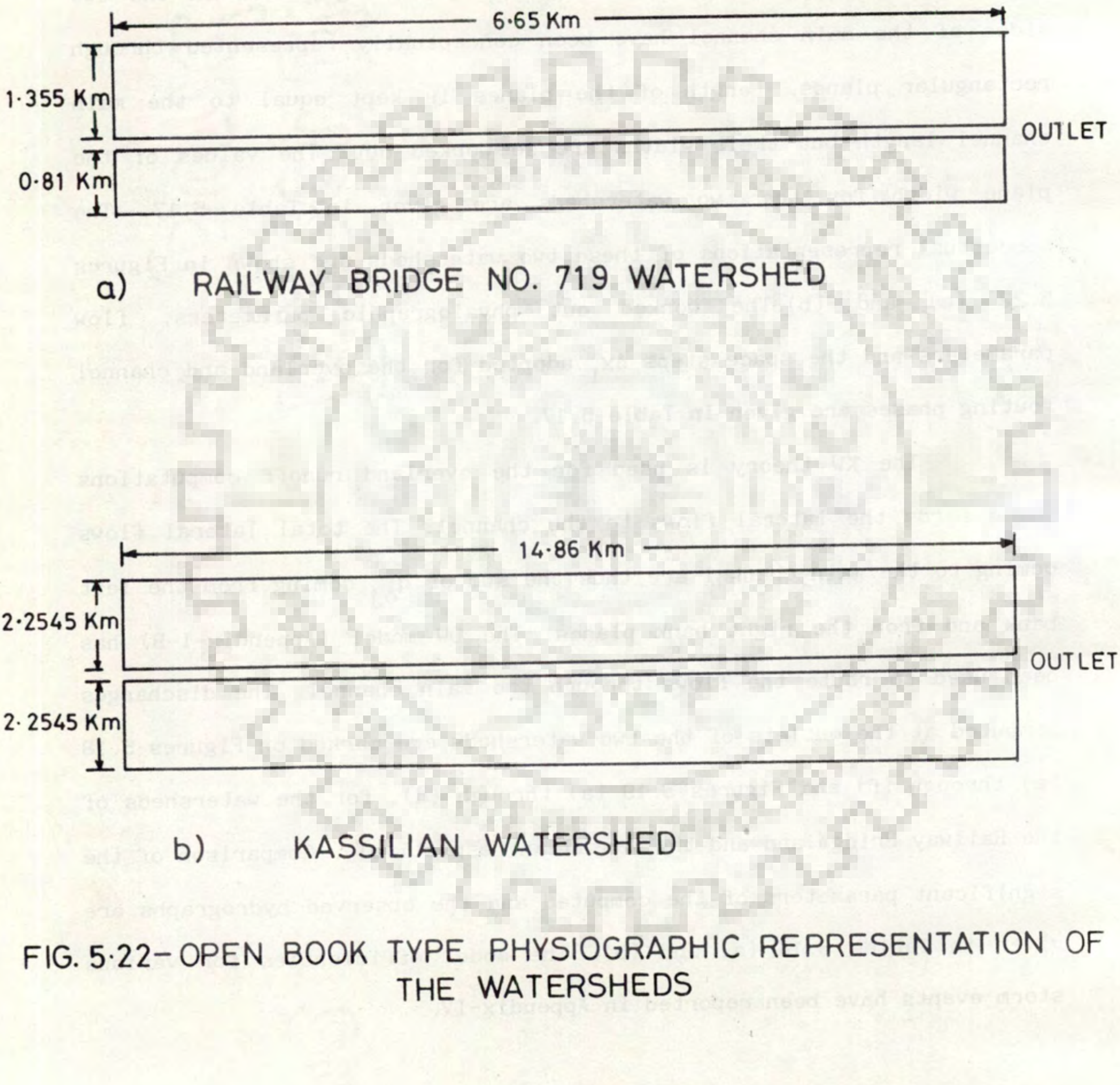


FIG. 5·22 - OPEN BOOK TYPE PHYSIOGRAPHIC REPRESENTATION OF THE WATERSHEDS

Table 5.17 Physiographic and Model Parameters for the watersheds of Railway Bridge No. 719 and for the Kassilian River for Open Book Type Physiographic Model Applications

Watershed		Railway Bridge No. 719		Kassilian	
Parameter		Left Bank Plane	Right Bank Plane	Left Bank Plane	Right Bank Plane
Area(km**2)		9.012	5.364	33.5	33.5
Plane Width (km)		0.62688	0.37312	2.2545	2.2545
Overland Slope		0.04063	0.0598	0.1029	0.1029
Overland Roughness		0.322	0.322	0.215	0.215
Plane ΔX(m)		156.72	124.373	225.45	225.45
α		0.62599	0.75944	1.492	1.492
Δt(sec)		300		300	
Parameters for Channel Flow Computations	ΔX	552.923(m)		1061.429 (m)	
	Δt	1800 (sec)		1200 (sec)	

the Railway bridge No. 719 and Kassilian river were briefly described in Section 5.5. The computed responses of the models were compared with the observed discharges at the outlets. These comparisons are shown in Figures 5.9, 5.19 and 5.20. Since in the past, researchers have used the open book type physiographic models therefore, this model was also applied onto all these three watersheds. The computed discharges from the open book type physiographic model were also plotted onto the figures mentioned above for the sake of comparison. The model efficiencies worked out on the basis of criteria given by Nash etal., (1970) for all the three models given above are given in Appendix-IV. The model efficiencies as well as the comparison of computed responses with the observed hydrographs suggest that the proposed Physiographic Model-I and II have given better results as compared to those obtained from the open book type physiographic models.

The Physiographic Model-I is comparatively simpler in application whereas the Physiographic Model-II consisting of the tributary and distributed main channel subwatersheds needs much labour to work with in terms of data preparation and computer time requirment. However, the Physiographic Model-II has got the added advantage that one can have an idea about the spatially distributed input volumes coming to the river system from various subwatersheds. This aspect has further been dealt in detail for the Kolar watershed in the forthcoming section.

5.7 WATER BALANCE STUDY OF THE KOLAR RIVER USING THE PROPOSED PHYSIOGRAPHIC MODEL-II

In Sections 5.2 and 5.5.1, the application of Physiographic Model-I consisting of the tributary subwatersheds and a single main channel subwatershed onto the three watersheds of Kolar river, Railway Bridge No. 719, and of the Kassilian river were discussed. Subsequently, on these three watersheds, the applications of the proposed

Physiographic Model-II consisting of tributary subwatersheds and the distributed main channel subwatersheds were described in Section 5.3 and Section 5.5.2. The Physiographic Model-II had the advantage that its subwatersheds comprehensively accounted for the distributed nature of the watershed physiography. This property enables us to compute contributions of the surface runoff thus forming the flood. This could be achieved by carrying out a hydrologic inventory into the water balance aspects of a watershed. For an illustration water balance inventory, the kolar river watershed which has the largest drainage area out of the three watersheds mentioned and on which the proposed models applications have been discussed in much greater depths and details (Sections 5.2 and 5.3) happen to be the natural choice. The following two events were taken up for this study.

(i) Runoff produced by a pulse of rainfall excess having a uniform depth of 1 cm over a period of one hour on the entire watershed.

(ii) The severe most storm registered over the watershed.

For carrying out the water balance study the principle of conservation of mass has been applied for the overland phase of runoff computations as well as the channel phase of the runoff. The following two relationships were thus formed.

(i) Total outflow at the outlet of the plane = Rainfall Excess Volume - Overland Storages

(ii) Total outflow volume at the channel outlet = Lateral inflow volume coming from the overland planes - Channel storages

5.7.1 Distributed Runoff Responses for Unit Pulse of Rainfall Excess

In Figure 5.12, the conceptual physiographic representation of the physiographic Model-II for the Kolar river is given. The overland computations for the unit pulse of rainfall excess were carried out (Appendix-I-A(1)). Values of physiographic parameters and KW routing parameters adopted for the analysis have already been reported in Section 5.3. The same are adopted for this study. The computed hydrographs from various subwatersheds (26 in number) which formed the lateral flows to the channel are shown in Figure 5.23(a) and (b) for the tributary subwatersheds, and in Figure 5.24(a), (b) and Figure 5.25 for the main channel subwatersheds. In Table 5.18 the volume of surface runoff from these 26 subwatersheds entering into the channel are given. The total overland surface runoff works out to be 8.382 MCM, thus involving volume error of less than four percent.

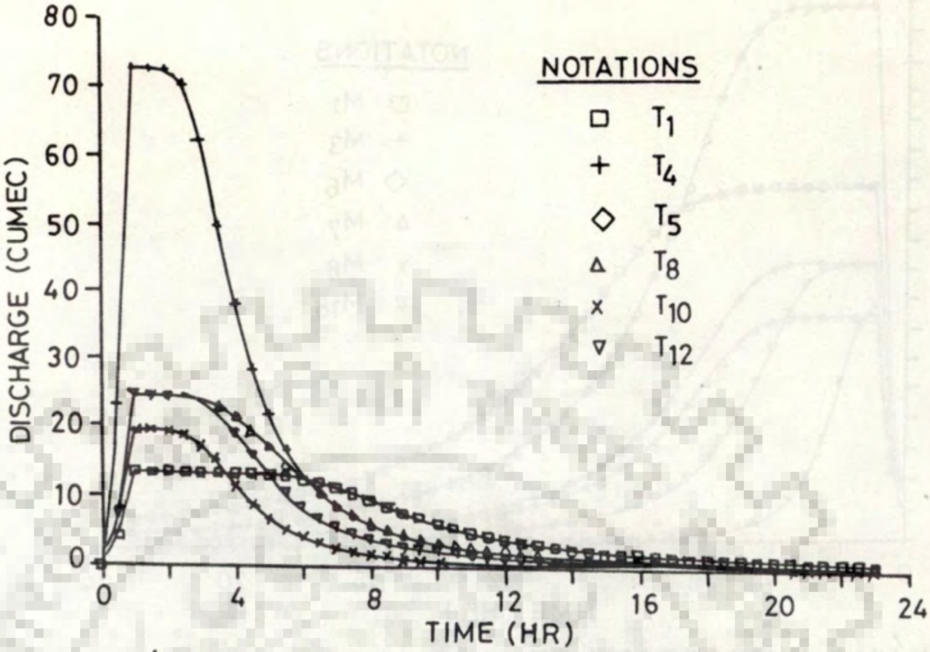
Table 5.18 Outflow Volumes from Kolar River Subwatersheds (1cm Rainfall/one hour Duration), (Physiographic Model-II)

(a) Tributary Subwatersheds

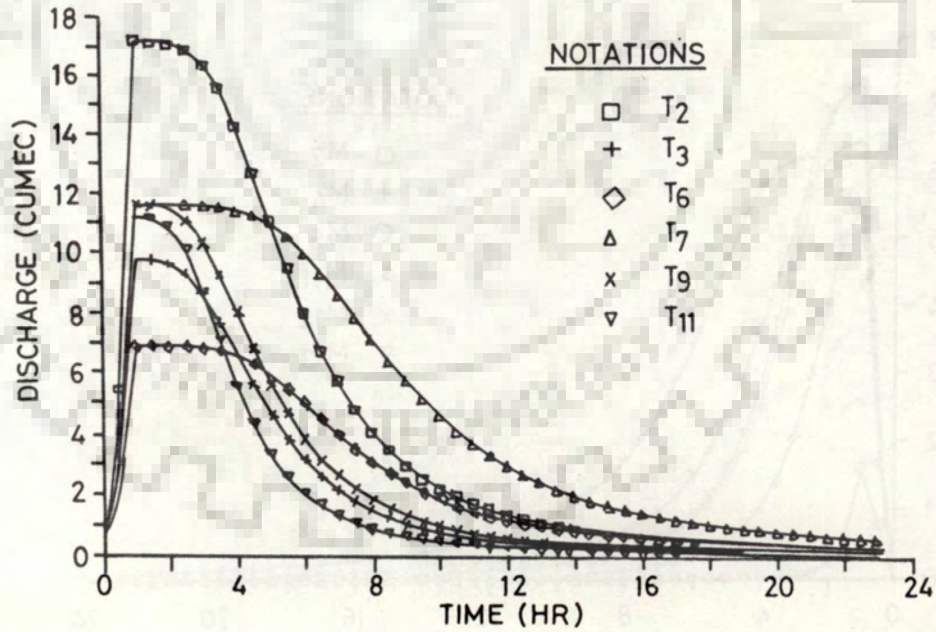
Sub-watershed	Runoff Volume (MCM)	
	1cm*CA	Model computed
T1	0.548	0.5103
T2	0.395	0.3827
T3	0.185	0.1805
T4	1.130	1.1064
T5	0.588	0.5680
T6	0.214	0.2037
T7	0.430	0.4029
T8	0.543	0.5055
T9	0.224	0.2183
T10	0.323	0.3157
T11	0.168	0.1644
T12	0.494	0.4807

(b) Main Channel Subwatersheds

Sub-watershed	Runoff Volume (MCM)	
	1cm*CA	Model computed
M1	0.5375	0.4934
M2	0.0060	0.0059
M3	0.2670	0.2559
M4	0.1575	0.1519
M5	0.0495	0.0489
M6	0.5480	0.5225
M7	0.2175	0.2081
M8	0.0612	0.0606
M9	0.7333	0.7202
M10	0.4282	0.4213
M11	0.0910	0.0898
M12	0.1055	0.1034
M13	0.0955	0.0940
M14	0.1712	0.1670

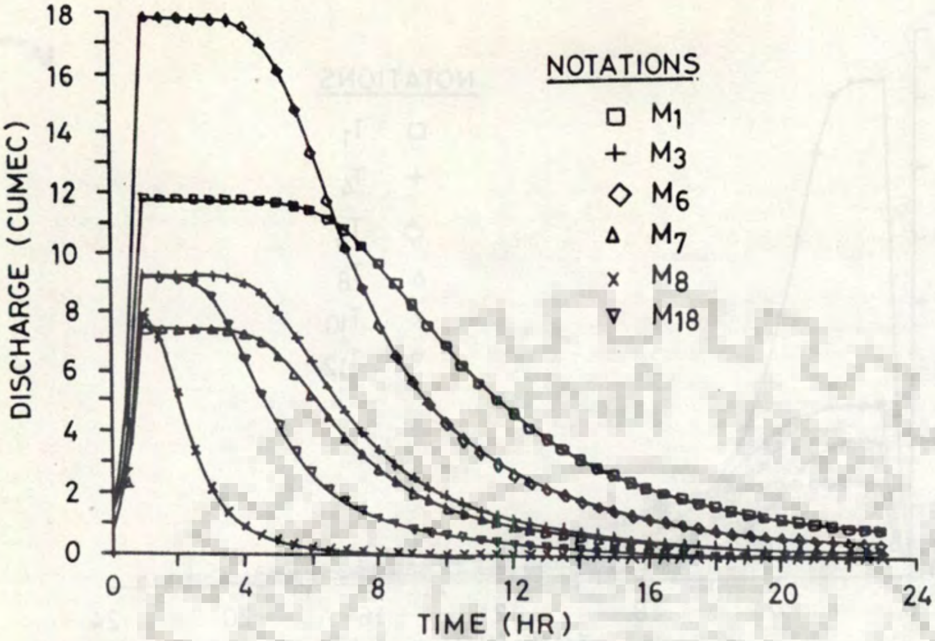


a) 'UNIT HYDROGRAPH' OF $T_1, T_4, T_5, T_8, T_{10}$ AND T_{12}

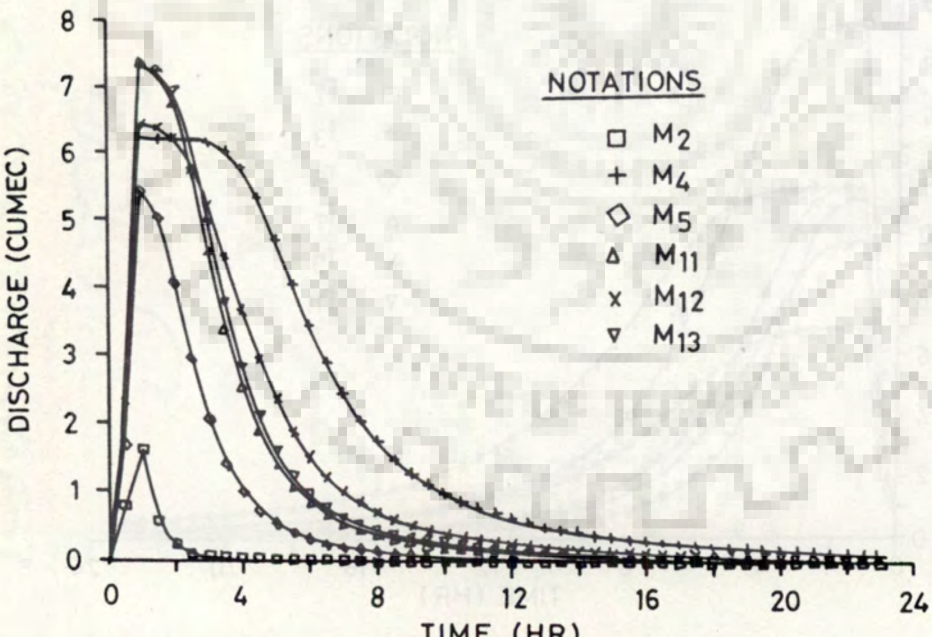


b) 'UNIT HYDROGRAPH' OF T_1 T_3 T_6 T_7 T_9 AND T_{11}

FIG. 5·23 a & b–UNIT HYDROGRAPHS FOR TRIBUTARY SUBWATERSHEDS OF KOLAR WATERSHED



a) 'UNIT HYDROGRAPH' OF M_1 M_3 M_6 M_7 M_8 AND M_{14} SUBWATERSHEDS



b) 'UNIT HYDROGRAPH' OF M_2 M_4 M_5 M_{11} M_{12} AND M_{13} SUBWATERSHEDS

FIG. 5·24 a & b–UNIT HYDROGRAPHS FOR MAIN CHANNEL SUBWATERSHEDS OF KOLAR WATERSHED

FIG. 5·25–UNIT HYDROGRAPHS FOR MAIN CHANNEL SUBWATERSHED M_9 AND M_{10}, OF KOLAR WATERSHED

As shown in Figure 5.12, and mentioned in Table 5.8, 25 reaches of the main channel were identified in the proposed model. Each of these has a uniform lateral flow to the channel which may differ from reach to reach. The reach wise input coming from the overland planes were also computed. A plot of "percent of contributing area upto the end of a reach starting from the upstream end of the main channel (plotted on the X-axis)" versus "percent of total volume contribution (i.e. shown on the Y-axis)" is shown in Figure 5.26. This plot enables us to have an idea that as one proceeds towards the downstream, how much percentage of volume are being contributed by the areas involved upto end of the reach. A similar plot in terms of "percent of channel length from the upstream end of the channel (i.e. shown on X-axis)" versus "the percent of runoff volume received" is shown in Figure 5.27. It is interesting to note that 50% of the runoff volume is received from the subwatersheds contributing to 1/3 length of the channel from the upstream. For a division of total channel length into five equal parts, the contributing subwatersheds (in percentages) are shown in Table 5.19.

Table 5.19 Contributing Subwatersheds along the Length (%) of Kolar River from the Upstream

Main channel length from upstream (%)	Lateral flow contribution (%)	Contributing aubwatersheds
0 - 20	28	M1(100) M2(100) M3(44) M4(50) T1(100) T2(73) T3(10) T4(24) T7(100) T8(56)
20 - 40	29	M3(56) M4(50) M5(100) M6(45) M7(100) M8(27) T2(27) T3(90) T4(76) T5(81) T8(44
40 - 60	14	M6(55) M8(73) M9(41) M10(23) T5(19) T9(100)
60 - 80	16	M9(59) M10(77) T10(100) T11(92) T12(44)
80 - 100	13	M11(100) M12(100) M13(100) M14(100) T6(100) T11(8) T12(56)

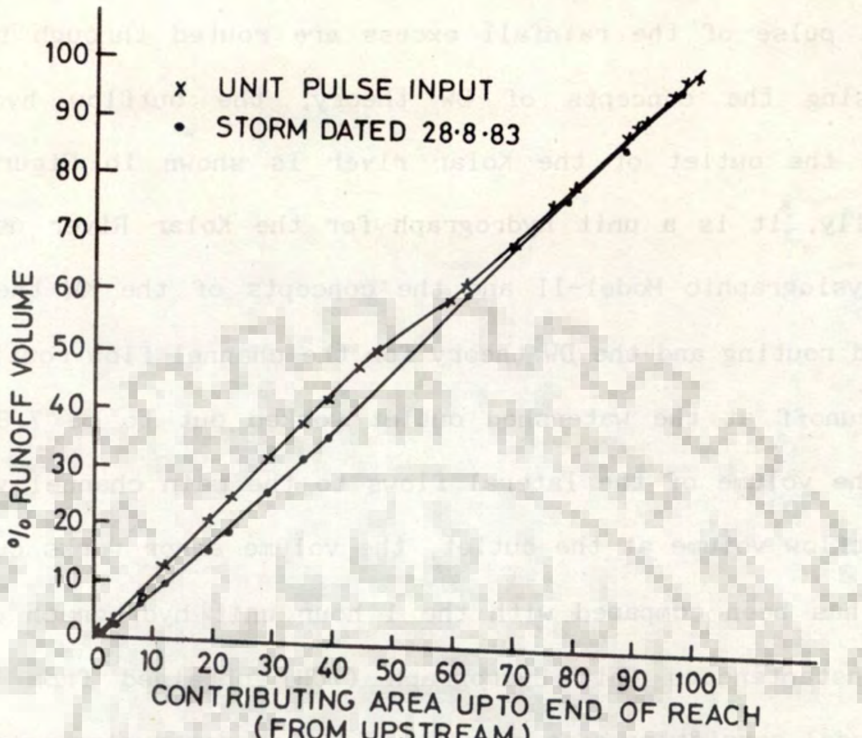


FIG. 5·26 RUNOFF VOLUME (%) Vs. PERCENTAGE CONTRIBUTING AREAS

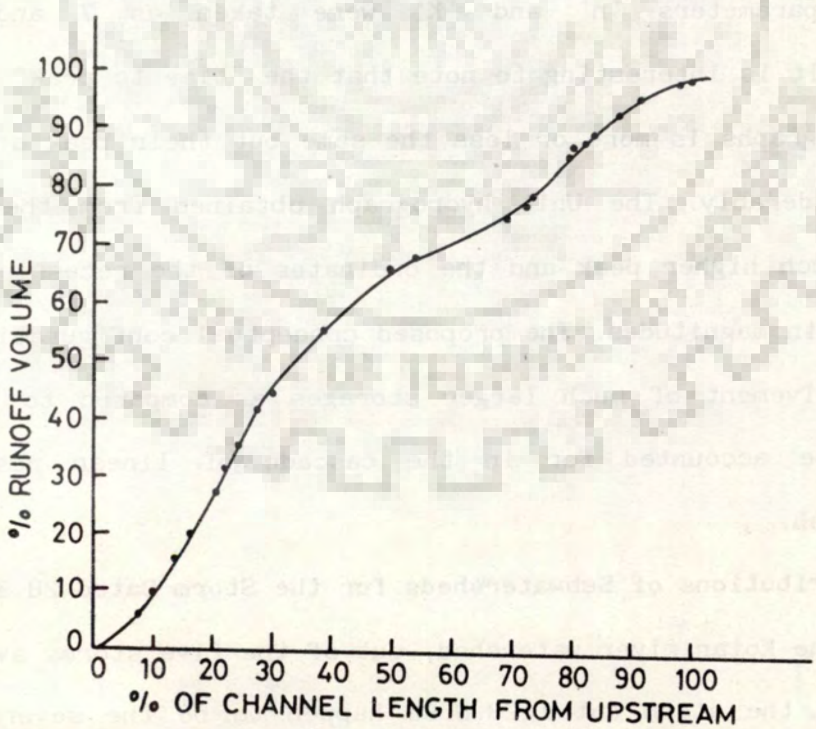


FIG. 5·27—PERCENTAGE VOLUME RUNOFF Vs. PERCENTAGE LENGTH OF MAIN CHANNEL FROM UPSTREAM

When the distributed lateral flows coming to the channel due to the unit pulse of the rainfall excess are routed through the main channel, using the concepts of DW theory, the outflow hydrograph obtained at the outlet of the Kolar river is shown in Figure 5.28. Conventionally, it is a unit hydrograph for the Kolar River using the proposed Physiographic Model-II and the concepts of the KW theory for the overland routing and the DW theory for the channel flow routing. The values of runoff at the watershed outlet worked out to be 7.823 MCM. Comparing the volume of the lateral flows to the main channel with the computed outflow volume at the outlet, the volume error works out to be 6.6%. This has been compared with the 1 hour unit hydrograph obtained from the Instantaneous Unit Hydrograph (IUH) obtained from the two parameter model consisting of a cascade of linear reservoirs as proposed by Nash (1957). In one such study (Panigrahi 1991), the average values of the two parameters 'n' and 'K' were taken as 7 and 1.022 respectively. It is interesting to note that the "time to peak" for the two unit hydrographs is more or less the same but their peak ordinates differed considerably. The Unit hydrograph obtained from the Nash's model has a much higher peak and the ordinates of the recession parts are much less in magnitudes. The proposed conceptual configuration thus indicates involvement of much larger storages as compared to the one which could be accounted for in the cascade of linear reservoirs proposed by Nash.

5.7.2 Contributions of Subwatersheds for the Storm Dated 28.8.1983

On the Kolar river watershed, out of the five storms available for this work, the storm dated 28.8.83 happen to be the severe most, involving a total rainfall depth of 297mm. The computed rainfall excess function for this storm is given in Appendix-II-C. The KW theory was used to compute the runoff volumes coming from the subwatershed planes

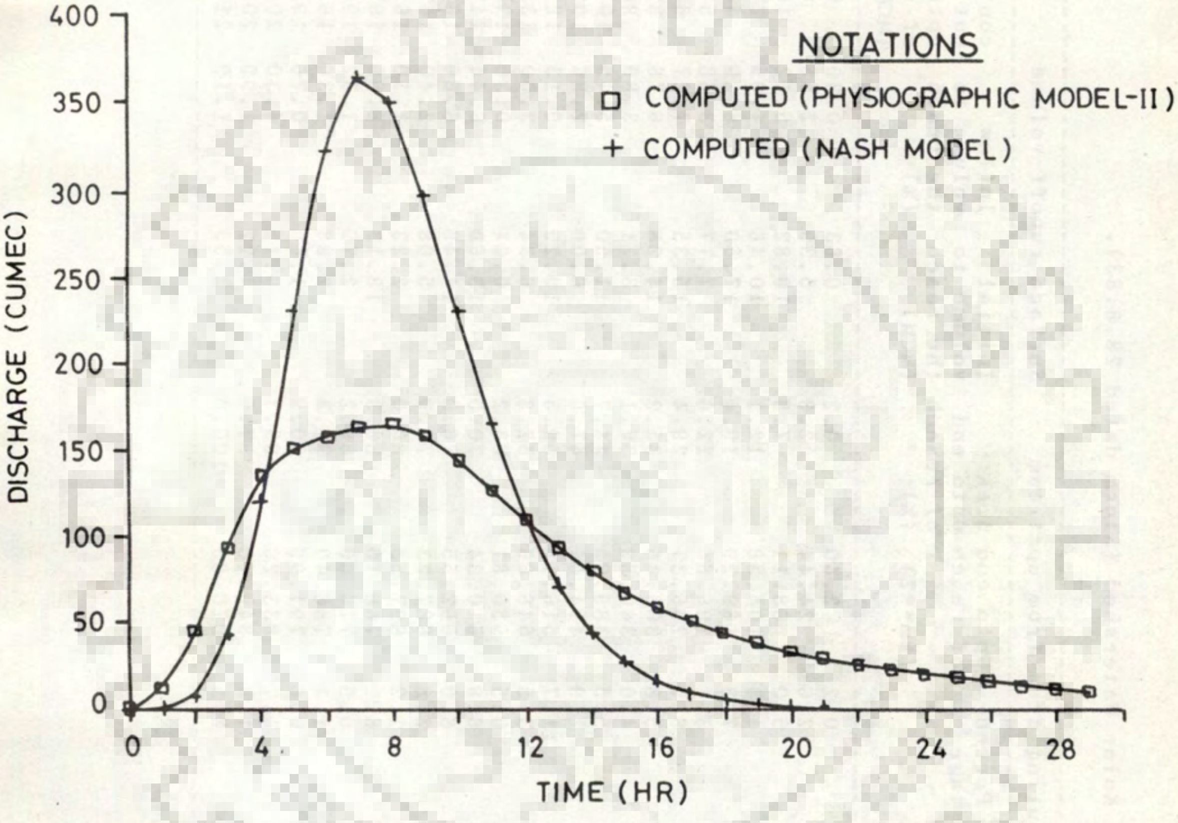


FIG. 5·28- UNIT HYDROGRAPHS FOR KOLAR WATERSHED

Table 5.20 : Water Balance Study of Kolar Watershed (Storm dated 28.8.83).

Sl. No.	Reach length	Distance from Upstream (km)	Contributing area from upstream				Surface runoff volume			
			Area (km**2)	Percentage	Upto end of reach (km**2)	Area upto end of reaach (%)	Initial volume to the reach (MCM)	Inflow volume to the reach (%)	contribution upto end of reach (MCM)	contribution upto end of reach (%)
1	0.5	0.5	2.04	0.2	2.40	0.2	0.63	0.30	0.634	0.30
2	3.0	3.5	22.40	2.6	24.44	2.8	5.39	2.55	6.026	2.80
3	3.5	7.0	43.67	5.0	68.11	7.8	10.82	5.12	16.846	7.98
4	2.5	9.5	37.27	4.3	105.38	12.1	10.46	4.95	27.310	12.90
5	3.5	13.0	63.91	7.3	169.29	19.4	17.50	8.29	44.814	21.23
6	1.5	14.5	27.37	3.1	196.66	22.6	7.77	3.68	52.587	24.91
7	3.0	17.5	59.69	6.9	256.37	29.4	15.35	7.27	67.933	32.18
8	3.0	20.5	50.11	5.8	306.46	35.2	14.28	6.76	82.217	38.95
9	2.0	22.5	34.43	4.0	340.89	39.1	8.21	3.89	90.428	42.84
10	3.0	25.5	48.16	5.5	389.05	44.7	11.50	5.45	101.930	48.29
11	2.0	27.5	25.59	2.9	414.64	47.6	6.00	2.86	107.987	51.16
12	8.0	35.5	101.54	11.7	516.18	59.3	19.29	9.13	127.274	60.30
13	5.0	40.5	30.68	3.5	546.86	62.8	6.34	3.00	133.611	63.30
14	0.5	41.0	3.95	0.5	550.81	63.2	0.64	0.30	134.456	63.60
15	9.0	50.0	59.53	6.8	610.34	70.0	13.60	6.44	147.859	70.05
16	13.5	63.5	52.21	6.0	662.55	76.0	14.48	6.86	162.343	76.92
17	3.5	67.0	24.68	2.8	687.23	78.9	5.08	2.40	167.425	79.32
18	1.0	68.0	10.30	1.2	697.53	80.0	2.33	1.10	169.735	80.42
19	5.5	73.5	71.23	8.2	768.70	88.3	18.13	8.59	187.886	89.01
20	1.0	74.5	7.86	0.9	776.60	89.2	2.79	1.32	190.675	90.33
21	1.5	76.0	9.77	1.1	786.40	90.3	2.88	1.36	193.557	91.70
22	1.0	77.0	5.60	0.6	791.99	90.9	1.60	0.76	195.165	92.46
23	5.0	82.0	41.22	4.7	833.20	95.7	9.15	4.30	204.319	96.80
24	3.0	85.0	16.19	1.8	849.40	97.5	4.22	1.99	208.538	98.80
25	7.5	92.5	21.45	2.5	870.85	100.0	2.53	1.19	211.070	100.00

to different reaches of the channel. Table 5.20 gives the details of contributing areas as well as the corresponding surface runoff volume contributed by them. The percentage of area contributions and the percentage of runoff volume for this storm event are also plotted on Figure 5.26. The two curves given in Figure 5.26 differ mainly due to differences in the rainfall excess generation over the overland planes. The error in the volume computations at the outlet compared to the observed hydrographs works out to be less than one percent.

The above analysis indicates the usefulness of the proposed Physiographic Model-II in obtaining the distributed responses coming to different reaches of the main channel.

Keeping in view the advantages of this model as mentioned above, the applicability of the same onto somewhat larger sized watersheds was considered worth trying. Thus, the proposed model has been applied onto the Barakar river stretch between Barkisuriya and Nandadhi (Section 5.8) which has a much larger drainage area of 2893 km^2.

5.8 APPLICATION OF THE PROPOSED PHYSIOGRAPHIC MODEL-II ONTO THE BARAKAR RIVER WATERSHED

In earlier sections, details of the application of the proposed physiographic model onto three watersheds (viz. Kolar, Railway Bridge No. 719 and Kassilian) have been discussed. In this section the model application has been extended for the study of the runoff mechanism for the Barakar river watershed which is much larger in size (2893 km^2) compared to the other three watersheds discussed earlier.

The drainage pattern of Barakar river watershed is given in Figure 4.6. The data availability on this watershed was discussed in Chapter IV. The distributed aspects of the physiography of this watershed are shown in Figure 5.29. Seven number of tributary

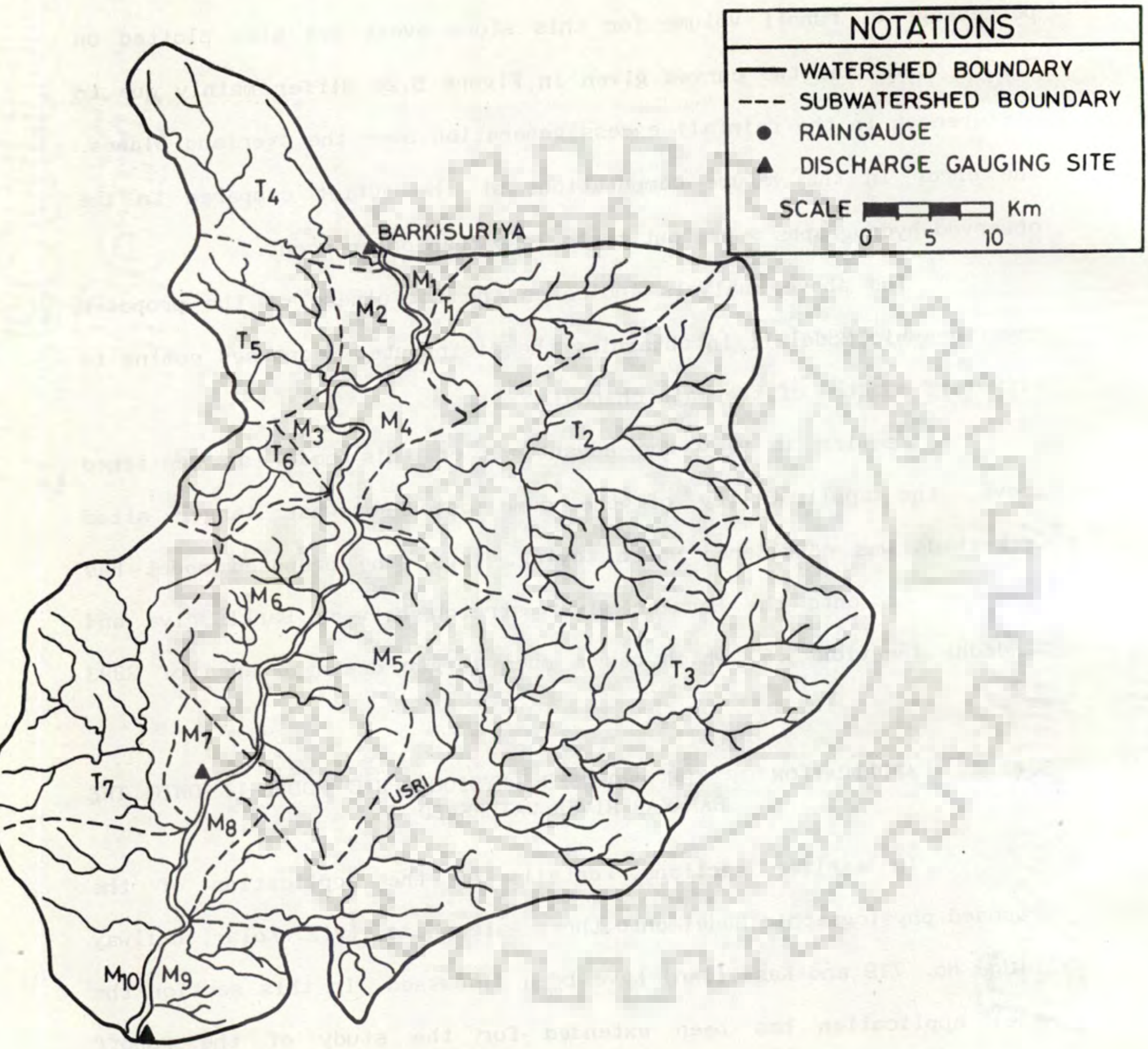


FIG. 5·29–THE BARAKAR RIVER WATERSHED AND ITS DISTRIBUTED SYSTEM subwatersheds and ten main channel subwatersheds have been identified for the modeling of this watershed. The values of the physiographic parameters for these subwatersheds are given in Tables 5.21 (a)and (b). The discretized patterns of the tributary subwatersheds and the main channel subwatersheds are given in Figures 5.30 and 5.31.

The Barakar river stretch has three gauging sites. The width of equivalent rectangular channel cross-section for the Barakar river worked out to be 176.366 m. A detailed analysis was carried out to compute the roughness. For this purpose the channel roughnesses corresponding to different stages of flood are worked out using the Manning's equation for steady state and uniform. Stage versus roughness so worked out are given in Figure 5.32. The average value of the channel roughness has been worked out from the three curves and the same is used in this model application.

For the average rectangular cross-section of the channel, corresponding to different stages (h_j), velocities (u_j) are worked out, considering the flow to be steady state and uniform. The computer programme given in Appendix-I-C is used for the purpose. The channel parameters as well as the coefficients of the stage-velocity regression relationship mentioned in the aforesaid paragraph are as under.

So =0.0026458 n =0.026

Ao=0.301686 A_1= 1.27977 A_2= -0.1634668 A_3=0.01583173

For the Barakar river, the rainfall data are not available for the three flood events under consideration. Therefore, the runoff depths obtained from the differences between the outflow and the inflow hydrographs of the river reach have been considered as input for the overland flow routing. These are computed by following the procedure given below. This way only the accountability of runoff mechanism through the proposed model fcr this watershed could be judged.

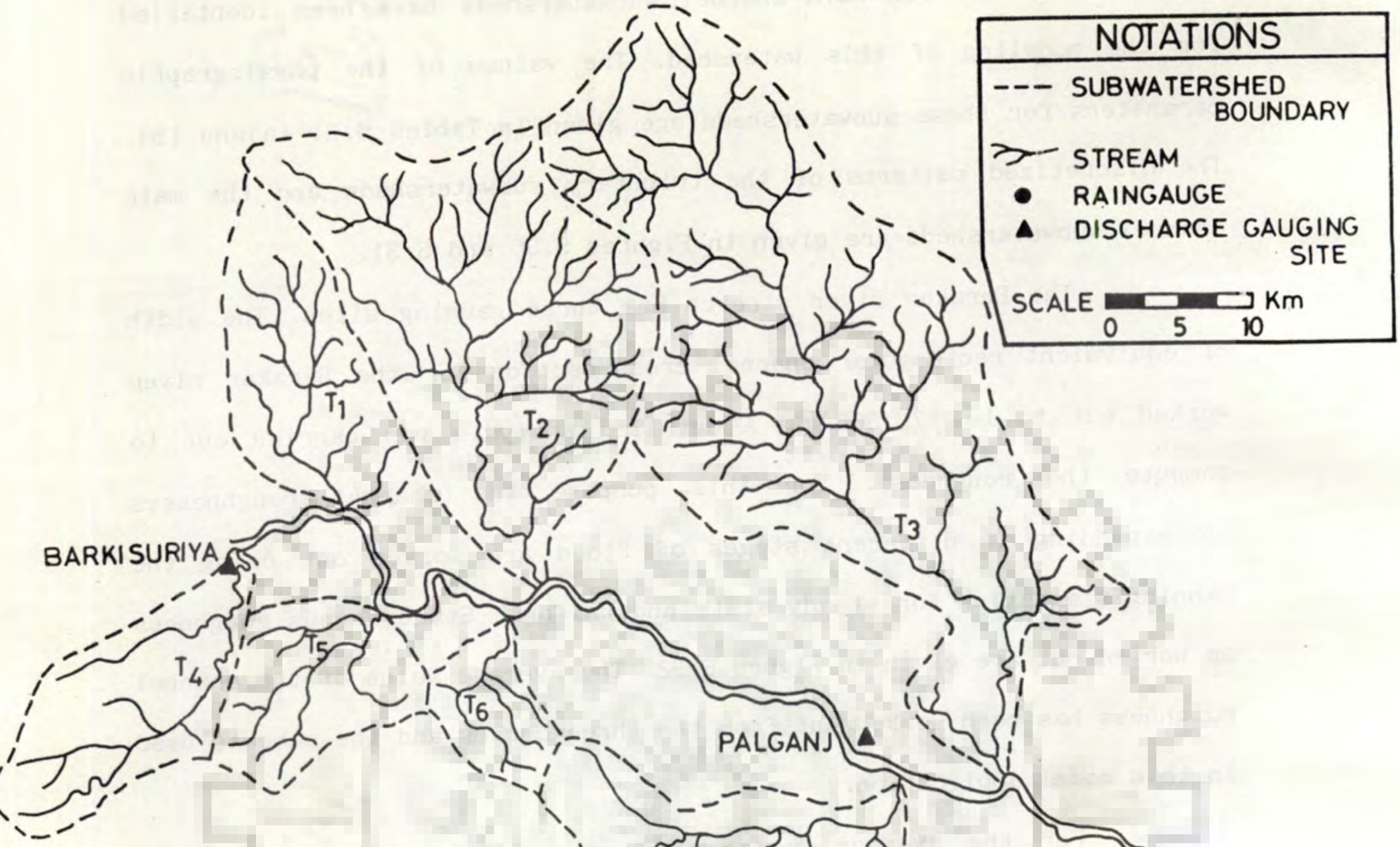


FIG. 5·30 – TRIBUTARY DRAINAGE PATTERN OF BARAKAR RIVER WATERSHED

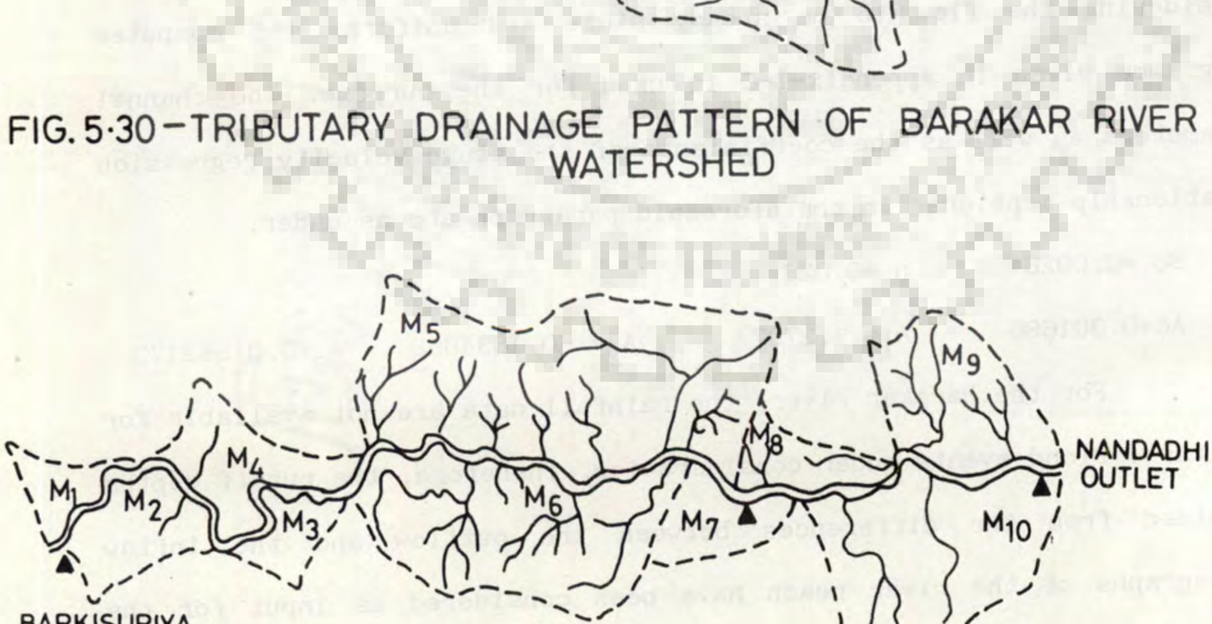


FIG. 5·31 – THE MAIN CHANNEL SUBWATERSHEDS OF BARAKAR RIVER WATERSHED

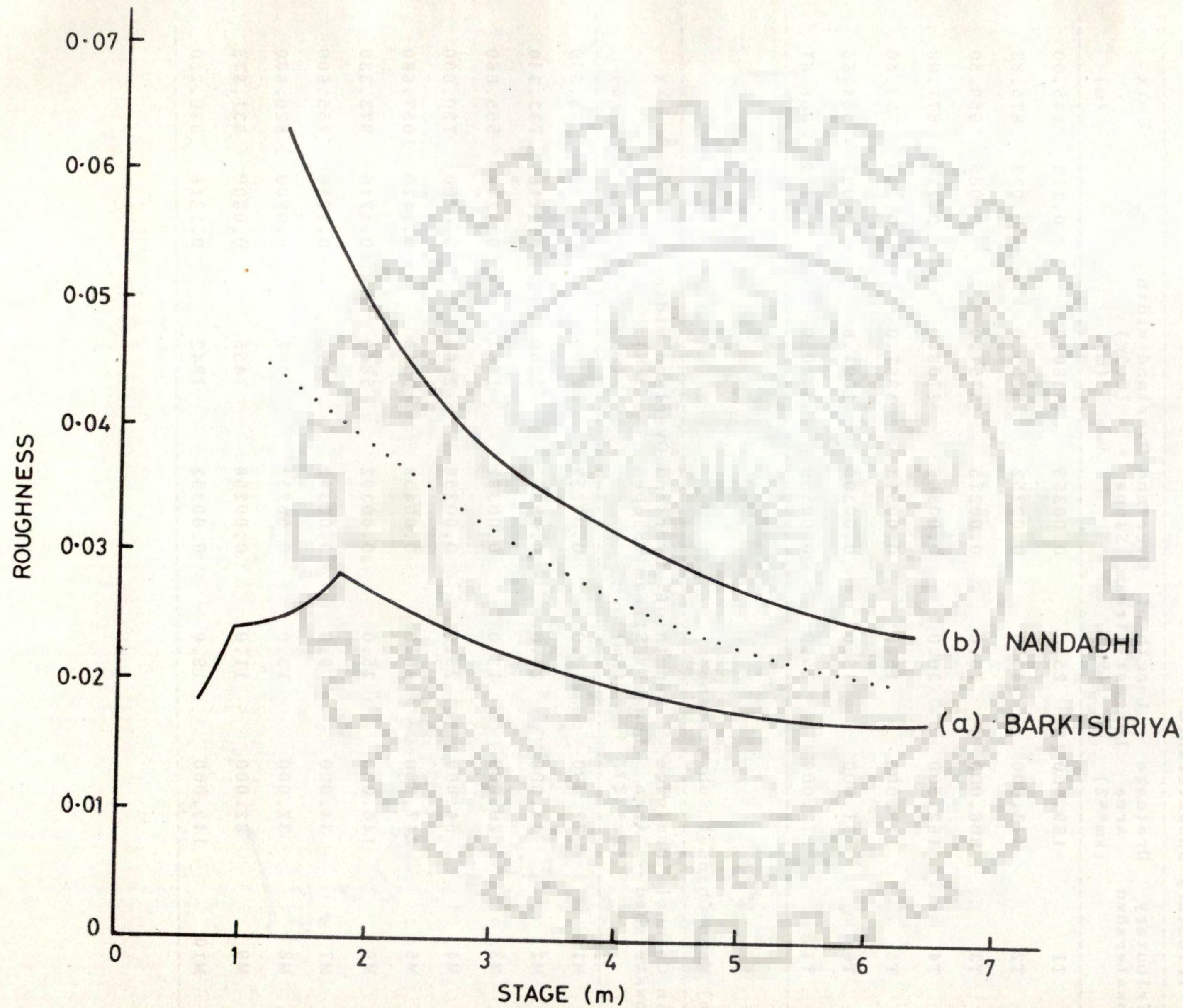


FIG. 5·32–THE STAGE–ROUGHNESS CURVES FOR THE TWO CROSS SECTIONS OF BARAKAR RIVER

Tabel 5.21 Physiographic Parameters of Barakar River Subwatersheds

(a) Tributary Subwatershed

Sl. No.	Tributary subwatershed	Drainage Area (km**2)	Length of Longest stream (km)	Overland Slope	Overland width (0.5W) (m)	α	ΔX (m)
1	T1	169.000	25.0	0.00365	3380.0	0.113	845.00
2	T2	473.000	45.0	0.00252	5255.0	0.078	875.83
3	T3	806.000	70.0	0.00275	5757.0	0.085	959.50
4	T4	162.500	30.0	0.00366	2708.0	0.114	677.00
5	T5	119.000	15.0	0.00513	3967.0	0.159	793.40
6	T6	61.000	18.0	0.00306	1694.0	0.095	564.66
7	T7	272.000	25.0	0.00550	5440.0	0.171	906.67

(b) Main Channel Subwatersheds

Sl. No.	Main Channel subwatershed	Drainage Area (km**2)	Length of Longest Stream (km)	Overland Slope	Overland width (W) (m)	α	ΔX (m)
1	M1	22.000	10.0	0.00258	2200	0.0180	733.330
2	M2	44.000	20.0	0.00392	2200	0.1216	733.330
3	M3	20.000	12.0	0.00392	1667	0.1216	555.660
4	M4	54.000	25.0	0.00258	2160	0.0800	720.000
5	M5	247.500	26.0	0.00458	9519	0.1420	1057.660
6	M6	146.000	25.0	0.00392	5840	0.1216	973.330
7	M7	34.000	9.0	0.00392	3778	0.1216	755.600
8	M8	32.000	13.0	0.00258	2462	0.0800	820.660
9	M9	82.000	11.0	0.00258	7455	0.0800	931.875
10	M10	149.000	19.0	0.00392	7842	0.1216	980.250

As shown in Figure 5.33, the inflow and outflow hydrographs have a time lag of δt in their CG's as well as in their peaks. This is assumed to be mainly introduced due to overland runoff contribution coming to the channel. Thus, the difference (D_j) between the outflows (Q_j) and the inflow hydrograph ordinates (I_j) are worked out as under.

$$D_j = Q_j - I_{j-\delta t} \tag{5.6}$$

The runoff volumes over the durations (δt_j) are computed corresponding to these differences (D_j) in ordinates, and the runoff depths are worked out for the entire contributing watershed. The rainfall excess is considered equal to the runoff depth. While comparing δt works out to be 3 hours.

The conceptualized rectangular subwatersheds are placed parallel to the main channel for the distributed physiographic representation as shown in Figure 5.34. The widths of these rectangular planes have been further divided into smaller space steps (Δx_j) for the overland flow routing (Table 5.21(a) and (b)). The superposition of lateral flows (q_{oj}) from different overlapping rectangular planes is accomplished through computer programme given in Appendix-I-A(10) and superimposed values are given in Table 5.22. Thus, the channel reaches having uniform lateral flows have been identified. Length of these reaches and the values of space step into which the reaches are divided are also given in Table 5.22.

The KW theory has been used to carry out the overland flow routing over the subwatersheds and the computer programme is given in Appendix-I-A(1). For this purpose a time step of 300 seconds has been adopted. The values of runoff depths discussed above have been fed to KW programme for overland flow computations. The lateral flows (q_{oj})

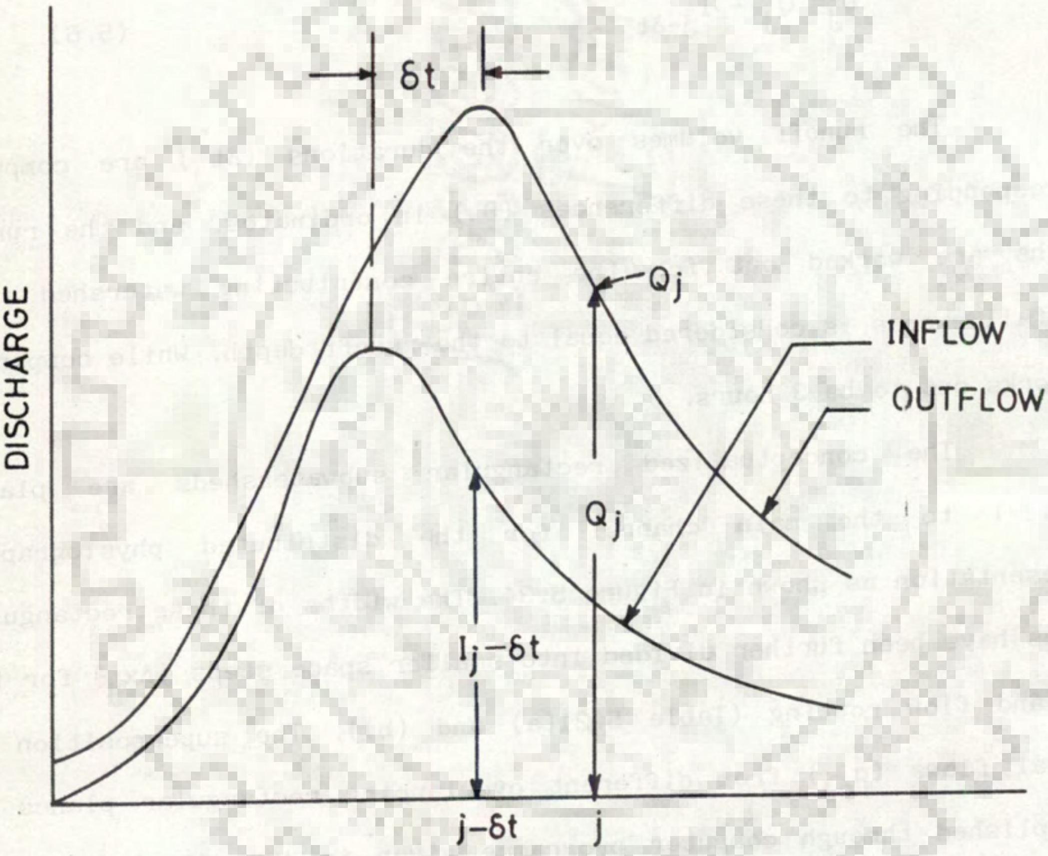


FIG. 5·33 — COMPUTATION OF LATERAL FLOWS FOR BARAKAR RIVER WATERSHED

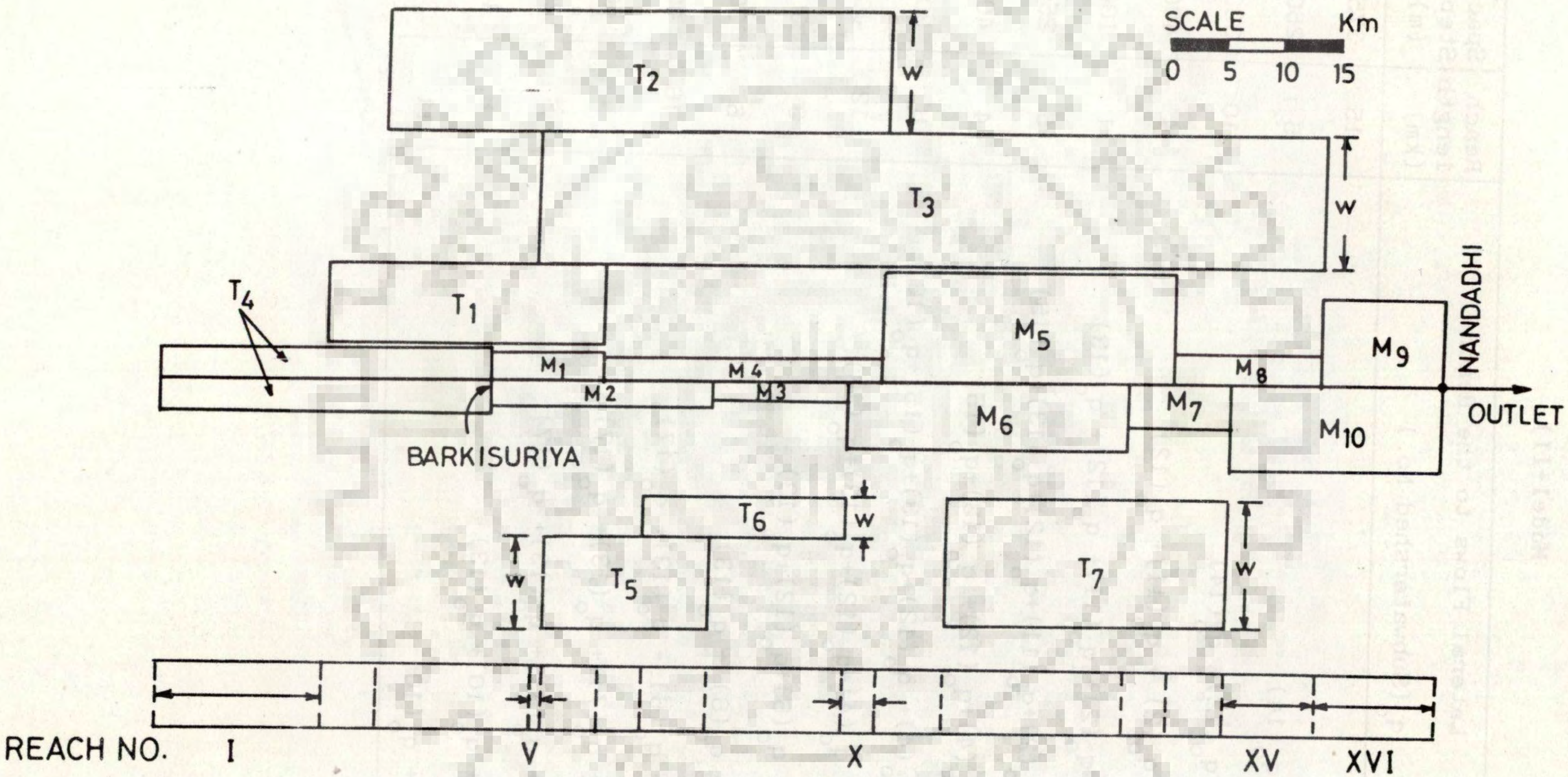


FIG. 5·34- CONCEPTUAL CONFIGURATION OF BARAKAR RIVER WATERSHED FOR PHYSIOGRAPHIC MODEL-II

Table 5.22 Superimposed Lateral Flows to Barakar river (Physiographic Model-II)

Reach No.	Lateral Flows to the Reach q_o(Subwatershed No.)	Reach length (km)	Space Step (m)
1	$q_o(14)$	15	3750
2	$q_o(11)+ q_o(14)$	5	2500
3	$q_o(11)+ q_o(12)+ q_o(14)$	10	3333.33
4	$q_o(1) + q_o(2) + q_o(11)+ q_o(12)$	4	4000
5	$q_o(1) + q_o(2)+ q_o(11)+ q_o(12)+ q_o(13)$	1	1000
6	$q_o(1)+q_o(2)+q_o(11)+q_o(12)+q_o(13)+q_o(15)$	5	2500
7	$q_o(2)+q_o(4)+q_o(12)+ q_o(13)+q_o(15)$	4	4000
8	$q_o(2)+q_o(4)+ q_o(12)+q_o(13)+q_o(15)+q_o(16)$	6	3000
9	$q_o(3) + q_o(4)+q_o(12)+q_o(13)+q_o(16)$	12	4000
10	$q_o(4) + q_o(6)+q_o(12)+q_o(13)$	3	3000
11	$q_o(5) + q_o(6) + q_o(13)$	6	3000
12	$q_o(5) + q_o(6) + q_o(13)+ q_o(17)$	16	4000
13	$q_o(5) + q_o(7) + q_o(13)+ q_o(17)$	4	4000
14	$q_o(7) + q_o(8) + q_o(13)+ q_o(17)$	5	2500
15	$q_o(8) + q_o(10)+ q_o(13)$	8	4000
16	$q_o(9) + q_o(10)$	11	3666.66

obtained through the application of KW model, have been routed through the main channel. The dynamic wave theory has been used for the channel flow computations. For this the time step (Δt) has been taken as 60 minutes and the other model parameter θ is taken as 0.67.

The inflow hydrographs at the upstream section (Barkisuriya) were available and the same were given for the upstream boundary conditions. The initial conditions have been adopted as the minimum flow ordinates of the inflow hydrographs. The routing is performed for the unit width of the rectangular channel section. The data of three available storms have been used for the proposed model and the computed and observed hydrographs at the outlet are compared as shown in Figures 5.35(a) to (c). The significant parameters of the observed and the computed hydrographs are given in Table 5.23. The model efficiencies worked out for the three storm events are given in Appendix-IV. These close comparisons and the model efficiencies do suggest that the proposed model has worked satisfactorily.

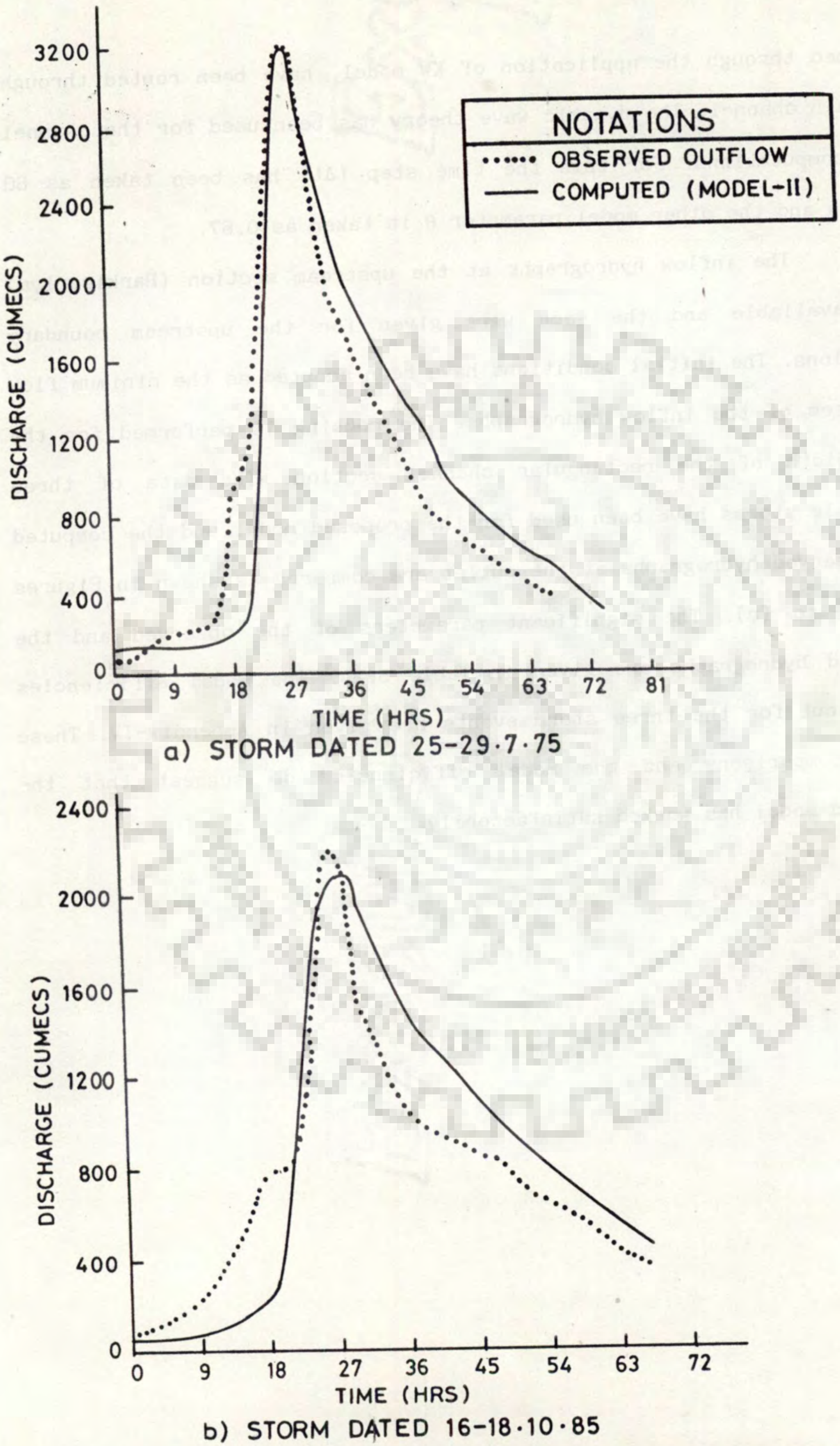


FIG. 5·35-COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF BARAKAR RIVER WATERSHED

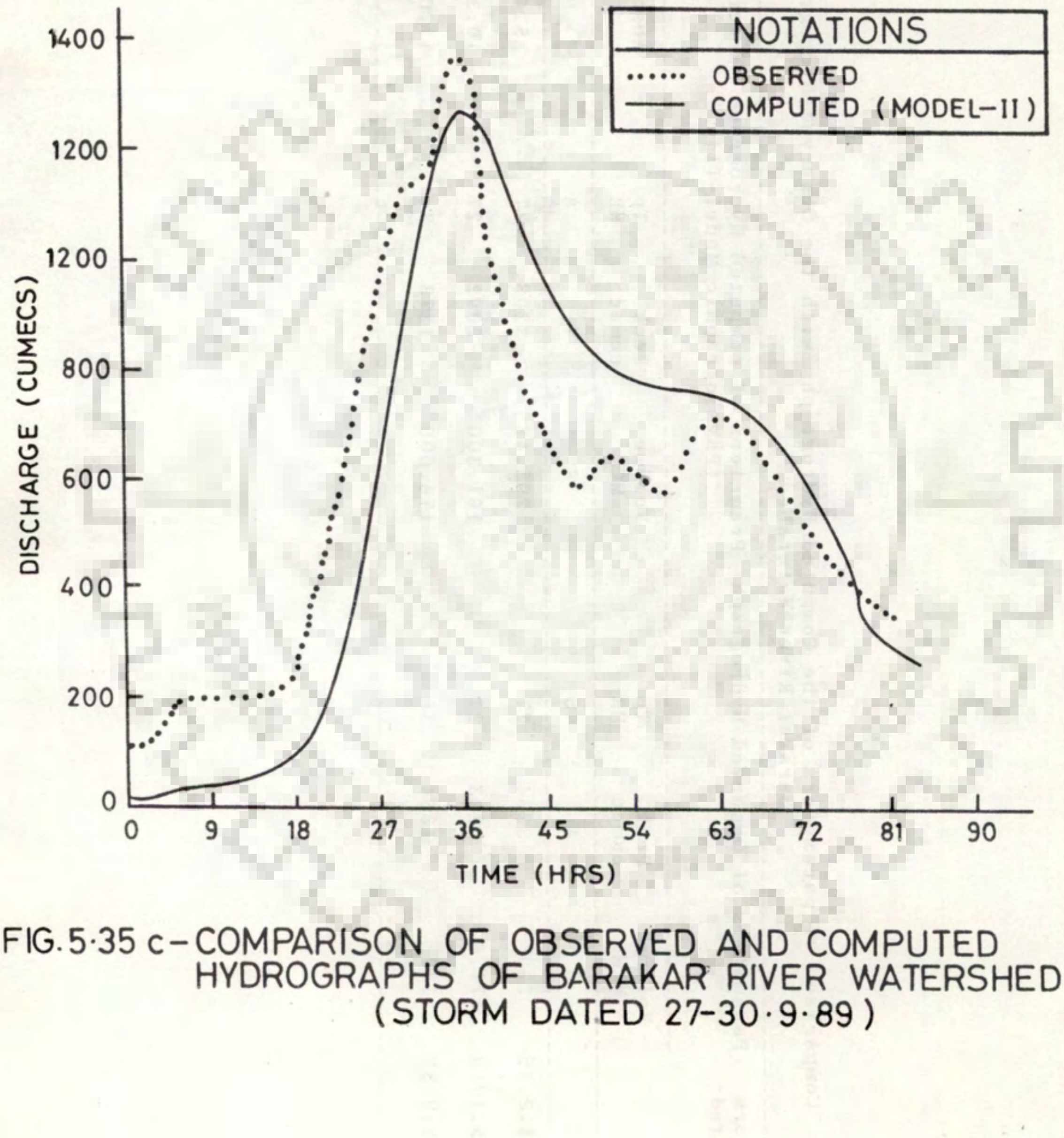


FIG. 5·35 c – COMPARISON OF OBSERVED AND COMPUTED HYDROGRAPHS OF BARAKAR RIVER WATERSHED (STORM DATED 27-30·9·89)

Table 5.23 Comparison of parameters of the Computed Hydrographs with the Observed Hydrographs of Barakar River Watershed

Sl. No.	Storm Dated	Parameters of Observed Hydrograph			Prameters of computed Hydrograph (Physiographic Model-II)			Error in Prediction	
		Volume MCM	Peak cumec	Tp HR	Volume MCM	Peak cumec	Tp HR	Absolut	Relative %
1	2	3	4	5	6	7	8	9	10
1	25-28.9.75	290	3234	27	295.485	3208	27	5.485	1.9
2	16-19.10.8	171.6	2203	27	161.300	2092	27	10.300	6.0
3	27-30.9.89	187	1364	36	176.000	1273	36	11.000	5.8

CHAPTER VI

DISCUSSION OF RESULTS AND CONCLUSIONS

The proposed study was taken up with a view to develop a suitable surface hydrologic model capable of taking into account the distributed nature of the physiography and landuse of the watersheds of the tropical countries. The watersheds may vary in size ranging from a few square kilometers to a few thousands square kilometers. The model should have the capabilities of accounting for landuse changes which might be introduced due to watershed management practices which in turn may influence the runoff process (Chapter-I). A literature survey conducted during the course of the study (Chapter-II) revealed that most of the popular models currently in use in the tropical countries are based on the unit hydrograph theory, and therefore are lumped in nature. The utility of such models lies in their limited requirements of data. However, owing to their inherent lumped nature, these models are not capable of taking into account the distributed nature of the physiographic parameters and are therefore incapable of assessing their influence on the runoff process.

As discussed in Chapter-III, the Kinematic Wave (KW) theory and the Dynamic Wave (DW) theory based hydrologic models currently being used for solving the St. Venant's equations, have the capability of taking into account the distributed nature of the watershed physiography provided suitable physiographic models are adopted for use. In the present study, both the theories (i.e. the KW and DW) have been applied. The KW theory (Section 3.4) is applied for routing the water flows onto the overland planes. The Lax-Wendroff explicit scheme has been used for the mathematical formulation. The theory has its limitations in application that the appropriate conditions must be satisfied for its application. The criteria adopted for its application is $Fr^2K > 5$, where Fr = Froude number and K = Kinematic Wave number (Morris and Woolhiser, 1980). There are many other criteria which have been applied in the past (Section 2.6), but the above mentioned criteria was considered to be most appropriate. This criteria has suitably been satisfied while applying the KW theory onto the watersheds. As a sample of the calculations, the computed values of the overland flow depths, velocities, Froude numbers and Kinematic Wave numbers for the two storm events i.e. dated 11.8.1965 for the watersheds of Railway Bridge No. 719 and dated 10.8.1984 for the Kolar river watershed are given in Appendix-V.

The DW theory has been preferred for routing the flows through the channel. This may have the advantage of routing the high flows as well as the low flows (i.e. for the flows where the KW theory may not be applicable). The mathematical formulation of the problem is discussed in Section 3.3. The 'four point implicit scheme' is used for solving the governing equations with appropriate initial and boundary conditions as discussed in Section 3.3.

The above mentioned mathematical theories have been incorporated in the two physiographic models proposed in this study.

i) Physiographic Model-I; Consisting of tributary subwatersheds and a single consolidated main channel subwatershed.

ii) Physiographic Model-II; Consisting of tributary subwatersheds and distributed main channel subwatersheds.

The details of these models have been discussed in Section 3.5. The later model is an extension to the first model. The watershed under consideration is split up into its subwatersheds of its tributaries which remain common to both the models. The remaining area is considered as a single main channel subwatershed in the first case whereas it is further split up into smaller units in the second physiographic model given above. Drainage characteristics of the areas happen to be the criteria adopted for the demarcation of the subwatersheds. In order to obtain the conceptual configuration, the surface runoffs coming from each of these subwatersheds (i.e. tributary subwatersheds and main channel subwatersheds) are folded onto the main channel (Section 3.5). The final physiographic pattern so arrived at will remain unique for the watershed under consideration. The surface runoffs from the overlapping overland planes are superimposed to compute the lateral flows coming to the main channel. Flows are routed through the main channel to compute the outflow hydrographs at the outlet. For the proposed configuration each of the subwatersheds becomes the elementary unit from which the runoff responses are to be computed. Any changes on its landuse can be appropriately taken care of by suitably modifying the values of the physiographic parameters and thus affecting the runoff process.

The requirements of data for application onto natural watersheds have been discussed in Chapter IV. The data could be procured for the following three watersheds.

(i) watershed of Railway Bridge No. 719 (14 km^2)

(ii) Kassilian river watershed (67 km^2)

(iii) Kolar river watershed (870 km^2)

Short durationed runoff data was available for Barakar river (2893 km^2) of the DVC system but, the short durationed rainfall data of the recording raingauges were not available. However, the model was tested on all these four watersheds (Chapter V).

In Sections 5.2 and 5.3, the application of the proposed two models have been discussed in depth and details for the watershed of

Kolar river (870 km^2). This watershed happens to be the largest among all the three watersheds considered for testing and where the type of data needed for the application were available. The other watersheds happen to be smaller in sizes (Railway Bridge No.719, 14 km^2 and Kassilian watershed 67 km^2). In order to draw logical conclusions about the applicability of the model in general, in Section 5.5.1 and 5.5.2 the proposed models have also been applied onto the watershed of the Railway Bridge No. 719 and Kassilian watershed. The open book type physiographic model which had been commonly used by the researchers in the past, has also been applied to all the three watersheds for the sake of comparison of the proposed models with it. The comparison of the computed hydrographs with the observed one are shown in Figure 5.9 for Kolar, in Figure 5.18 for Railway Bridge No. 719 and in Figure 5.19 for the Kassilian watershed. The comparison of the hydrograph parameters (Tables 5.5, 5.14a and 5.14b) and the worked out model efficiencies given in Appendix IV (Nash et al. 1970) do suggest that the physiographic model-II consisting of tributary subwatersheds and the distributed main channel subwatersheds gives better results. At the same time the proposed physiographic model-I is comparatively simpler and easy to apply. The open book type physiographic model has in general not produced satisfactory results.

The proposed physiographic model have the added advantage that the distributed responses of the surface runoff coming to the channel can be obtained, to arrive at the water balance of the channel flows. Sample calculations carried out for the Kolar watershed for the storm event dated 28.8.1983 have thus been reported in Section 5.7.

It may also be interesting to note that the proposed physiographic model-II when run for a unit pulse of rainfall excess having duration of one hour, did produce a response which could be termed as unit hydrograph at the outlet. This when compared with the unit hydrograph obtained by using a two parameter cascade model of linear reservoirs (Nash, 1957) differed significantly (Figure 5.28). The analysis indicated that the storages associated with the process were significantly more than the same which could be taken into account by the Nash's conceptual model. The runoff mechanism of the proposed model has further been tested for its applicability to a much larger watershed (viz. Barakar river with 2893 km^2 area). In the absence of short durationed rainfall data, the uniformly distributed runoff depth resulting the downstream hydrograph was considered as an input function (Section 5.8). A comparison of the computed and observed hydrographs for the three storm events suggests that the proposed model yields quite satisfactory results.

In the proposed approach, computation of the rainfall excess distribution and estimation of the surface roughness are the two important aspects which influence the results significantly. In all future works, this need be dealt with care and be investigated thoroughly to improve the general performance of the model. However, this is to mention that the computation of the rainfall excess and its distribution in space and time continues to be the weakest link in surface hydrology. This aspect could be strengthened further if the proposed model is linked with infiltration based ground water models (Ahmed, et al., 1991 and 1993). Efforts are on in this direction in the Department of Hydrology, University of Roorkee, Roorkee (India) and it is hoped that in future suitable models will be developed to cater to the needs of tropical countries.

APPENDIX-I-A(1)

```
C  ************************************************************
C  KINEMATIC WAVE PROGRAME FOR OVERLAND FLOW ROUITING
C  USING LAX-WENDROFF EXPLICIT SCHEME.
C  ************************************************************
C  THE PROGRAMME IS PART OF THE
C   "MODELING OF FLOOD FLOWS IN NATURAL WATERSHEDS"
C   DEVELOPED BY MOHAMMAD REZA NAJAFI SHAHRI(PhD. STUDENT)
C   GUIDED BY DR. B.S. MATHUR (PROF. AND HEAD)
C         AND DR. D. KASHYAP PROFESSOR,
C   DEPARTMENT OF HYDROLOGY UNIVERSITY OF ROORKEE, ROORKEE,
C   INDIA.
C*************************************************************
C   NPL       = NO. OF OVERLAND PLANES.
C   NC        = NO. OF COMPUTATIONAL NODES OF MAIN CHANNEL
C   DTK       = COMPUTATIONAL TIME STEP FOR KW FLOW ROUTING.
C   NTP       = NO. OF TIME STEPS.
C   NCP(I)    = NO. OF COMPUTATIONAL NODES FOR PLANE I.
C   DXP(I)    = SPACE INTEVAL FOR THE PLANE I
C   SO(I)     = OVERLAND SLOPE OF PLANE I.,
C   REF(I)    = OVERLAND MANNING'S ROUGHNESS FOR PLANE I.
C   ALPHA(I) = SO(I)**0.5/REF(I)
C   H(I,LK,J)= OVERLAND FLOW DEPTH (M) FOR THE NODE LK OF
C              PLANE I, AT TIME STEP J.
C   Q(I,LK,J)= DITRIBUTED INTERMEDIATE FLOW TO THE PLANE I,
C              FOR THE NODE LK, AND AT TIME LEVEL J.
C   DTRE      = DURATION OF EACH RE BLOCK (SEC.).
C   NREB      = NO OF RE BLOCKS.
C   NRS       = NO OF RAINGAUG STATIONS.
C   AREA      = PLANE AREA IN METRE.
C   DX        = SPACE-STEP-LENGTH OF MAIN CHANNEL
C   RE(I,J)   = Jth RAINFALL EXCESS BLOCK (MM) FOR THE Ith PLANE .
C ***************************************************************
      DIMENSION H(26,12,50),QLL(26,12,50),QL(12,50),
      DIMENSION NCP(26),VS(12,20),DX(39)
      DIMENSION DXP(26),SO(26),REF(26),ALPHA(26),AREA(12)
      COMMON/CA/QF(26),QC(39),NC
      COMMON/RAN1/RE(4,25),QPL(26)
```

```
      COMMON/RAN2/JRE
      OPEN(UNIT=1,FILE='WAVE-K',STATUS='OLD')
      OPEN(UNIT=3,FILE='KIN3.OUT',STATUS='NEW')
      OPEN(UNIT=4,FILE='WAVE-K.OUT',STATUS='NEW')
      OPEN(UNIT=5,FILE='INCK.DAT',STATUS='OLD')
      OPEN(UNIT=6,FILE='INCK.OUT',STATUS='NEW')
      OPEN(UNIT=7,FILE='PARA.OUT',STATUS='NEW')
CC***********************************************
CC***  DTK SHOULD BE SELECTED SUCH THAT DTRE IS DIVISIBLE BY DTK.
CC***  NTP SHOULD BE MULTIPLE OF DTRE/DTK.
CC***********************************************
      READ(1,*)NPL,EPSQ,DTK,NTP,IT,NC
      READ(1,*)(AREA(I),I=1,NPL)
      READ(1,*)(DX(I),I=1,NC-1)
      J=1
      READ(5,*)SUMR,SUMQF,SUMCH
      DO 420 I=1,NPL
      READ(1,*)SO(I),REF(I),DXP(I),NCP(I)
      NCPK=NCP(I)
      READ(5,*)(H(I,LK,1),LK=1,NCPK)
      READ(5,*)(QLL(I,LK,1),LK=1,NCPK)
      ALPHA(I)=SQRT(SO(I))/REF(I)
  420 CONTINUE
CC    WRITE(*,*)(ALPHA(I),I=1,NPL)
      READ(1,*)NREB,DTRE,NRS
      DO 75 LI=1,NRS
  75  READ(1,*)(RE(LI,J),J=1,NREB)
      WRITE(*,20)
  20  FORMAT(2X,'ENTER THE MAIN CHANNEL WIDTH IN METER')
      READ(*,*)W
CC*********************************
CC    CONVERT RAINFALL DEPTH INTO RAINFALL INTENSITY.
      DO 105  LI=1,NRS
      DO 105 JT=1,NREB
      RE(LI,JT)=RE(LI,JT)/DTRE
  105 CONTINUE
CC *** EEX IS THE EXPONENT m.
```

```
      EEX=5./3.
      EX2=EEX-1.0
      G=9.81
      JRE=1
      ICH=JRE
      NDT=DTRE/DTK
CC    DO J=1,NTP-1
C****************** TIME LOOP***************
      DO 50 J=1,NTP
CC     WRITE(7,120)J+1
C120   FORMAT(7X,'TIME LINE= ',I3)
      IT=IT+DTK
      IF(J.LT.NTP)GO TO 51
      WRITE(6,*)IT
51    WRITE(*,*)J
CC***************** PLANE LOOP **************
      DO 100 K=1,NPL
      NCPK=NCP(K)
      IF(J*DTK.GT.NREB*DTRE)GO TO 350
73    IF(K.GT.1)GO TO 400
CC***********************************
      CALL RAIN
CC***********************************
400   SUMR=SUMR+QPL(K)*DTK*AREA(K)
      DO 300 LK=2,NCPK
      QLL(K,LK,J+1)=QPL(K)
300   CONTINUE
      H(K,1,J+1)=0.0
      SUMH=H(K,1,J+1)
      QLL(K,1,J+1)=QPL(K)
      GO TO 199
350   DO 450 LK=1,NCPK
450   QLL(K,LK,J+1)=0.0
199   CONTINUE
CC********* NODE LOOP ***********
      DO 200 LK=2,NCPK
      QL(LK,J)=QLL(K,LK,J)
```

```
      IF(LK.EQ.NCPK)GO TO 30
      QL(LK,J+1)=QLL(K,LK,J+1)
      QL(LK-1,J)=QLL(K,LK-1,J)
      QL(LK+1,J)=QLL(K,LK+1,J)
      FP=H(K,LK,J)-DTK*(0.5*ALPHA(K)*(H(K,LK+1,J)**EEX-
     1H(K,LK-1,J)**EEX)/DXP(K)-QL(LK,J))
      BM=0.25*ALPHA(K)*DTK**2*EEX/DXP(K)
      GGG=(H(K,LK+1,J)**EEX-H(K,LK,J)**EEX)
      QQQ=0.5*(QL(LK+1,J)+QL(LK,J))
      AAA=(ALPHA(K)*GGG/DXP(K)-QQQ)
      SP=(H(K,LK+1,J)**EX2+H(K,LK,J)**EX2)*AAA
      QA=0.5*(QL(LK,J)+QL(LK-1,J))
      ALP=ALPHA(K)*(H(K,LK,J)**EEX-H(K,LK-1,J)**EEX/DXP(K)-QA)
      TP=(H(K,LK,J)**EX2+H(K,LK-1,J)**EX2)*ALP
      FOP=2.*DXP(K)/(EEX*ALPHA(K)*DTK)*(QL(LK,J+1)-QL(LK,J))
      H(K,LK,J+1)=FP+BM*(SP-TP+FOP)
      GO TO 40
  30  CONTINUE
      ALF=ALPHA(K)*(H(K,LK-1,J)**EEX-H(K,LK,J)**EEX)/DXP(K)
      H(K,LK,J+1)=H(K,LK,J)+DTK*(ALF+QL(LK,J))
 40   CONTINUE
      HH=H(K,LK,J+1)
      IF(HH-0)41,42,42
 41   H(K,LK,J+1)=0.0
 42   CONTINUE
      SUMH=SUMH+H(K,LK,J+1)
      IF(LK.LT.NCPK)GO TO 198
CC; ***QF(K) IS THE OUTFLOW FROM SUBCATCHMENT K AT TIME J+1*****
      QF(K)=ALPHA(K)*H(K,LK,J+1)**EEX
CC **** PLANE SURFACE VELOCITY *********
 198  VS(K,LK)=ALPHA(K)*(H(K,LK,J+1))**EEX
  200 CONTINUE
      VL=ALPHA(K)*(H(K,NCPK,J+1))**EEX
CC    COMPUTE AVE. OVERLAND FLOW DEPTH.
      HAVE=SUMH/NCP(K)
CC    AVERAGE FLOW VELOCITY
      V=ALPHA(K)*HAVE**EEX
```

```
CC      FROUDE NUMBER
        DENO=(G*HAVE)**0.5
        FR=V/DENO
CC      KINEMATIC FLOW NUMBER "K"
        AKINK=SO(K) * (DX(K)*(NCP(K)-1)) / (FR**2*HAVE)
CC      KINEMATIC WAVE CELERITY
        CELE=ALPHA(K)*EEX*HAVE**EX2
CC      COMPUTE TOTAL DISCHARGE VOLUME TO MAIN CHANEL
        SUMQF=SUMQF+QF(K)*DTK*(AREA(K)/(DXP(K)*(NCP(K)-1)))
        IF(J*DTK-JRE*DTRE)77,84,77
 84     WRITE(7,355)K,HAVE,V,FR,AKINK,CELE,VL
 355    FORMAT(2X,I2,2X,3F8.4,2X,F16.1,2X,F6.4,1X,F8.6)
 77     CONTINUE
        SUMQF=SUMQF+QF(K)*DTK*(AREA(K)/(DXP(K)*(NCP(K)-1)))
        IF(J.LT.NTP)GO TO 100
        WRITE(6,*)(H(K,LK,J+1),LK=1,NCPK)
        WRITE(6,*)(QLL(K,LK,J+1),LK=1,NCPK)
 100    CONTINUE
        IF(ICH-JRE)65,60,65
 60     WRITE(3,363)
 363    FORMAT(10X,'RAINFALL INTENSITY')
        WRITE(3,*)(QPL(I),I=1,NPL)
        ICH=ICH+1
 65     CONTINUE
 C65     WRITE(3,15)NPL,IT
  15    FORMAT(3X,'OUTFLOW OF SUBCATCHMENTS 1 TO ',I2,' AT TIME '
       1,I8,'SEC ',/)
CC       WRITE(3,26)(QF(K),K=1,NPL)
 26     FORMAT(2X,7F10.8)
CC*****************************
        CALL CANALQ
CC*****************************
        DO 1000 LU=1,NC
        QC(LU)=QC(LU)/W
 1000   CONTINUE
        DO 28 I=1,NC-1
        SUMCH=SUMCH+QC(I+1)*DX(I)*DTK
```

```
 28     CONTINUE
CC      WRITE(3,25)IT
CC       WRITE(8,*)(QC(I),I=1,NC)
CC       WRITE(8,88)
CC      WRITE(3,*)(QC(ID),ID=1,NC)
CC 25   FORMAT(3X,/,'LATERAL INFLOW TO MAIN RIVER CHANNEL (q) AT TIME',
CC      1I8,' SEC',/,3X,'(M**3/SEC/UNIT L/UNIT WIDTH OF MAIN)',/)
        IF(J*DTK-JRE*DTRE)50,64,50
C64     ITIME=JRE*DTRE
        WRITE(3,362)J+1
 362    FORMAT(1X,'SURFACE VLOCITY AT TIME LINE ',I3)
        DO 367 K=1,NPL
        WRITE(3,*)K,(VS(K,LK),LK=1,NCP(K))
 367    CONTINUE
        WRITE(3,365)
 365    FORMAT(1X,'OUTFLOW FROM PALNES')
        WRITE(3,76)(QF(K),K=1,NPL)
 76     FORMAT(2X,7F10.8)
        WRITE(7,120)J+1
 120    FORMAT(7X,'TIME LINE= ',I3)
CC      WRITE(4,*)J+1,ITIME,IT
        WRITE(4,88)
 88     FORMAT(/)
        WRITE(4,*)(QC(I),I=1,NC)
        JRE=JRE+1
        ICH=JRE
 50     CONTINUE
        WRITE(6,*)SUMR,SUMQF,SUMCH
        STOP
        END
```

APPENDIX-I-A(2)

```
C*********************************************************
C SUBROUTINE FOR COMPUTING THE WEIGHTED RAINFALL
C  INTENSITY  FOR THE SUBWATERSHEDS OF KOLAR RIVER
C (PHYSIOGRAPHIC MODEL-I)
C*********************************************************

      SUBROUTINE RAIN COMMON/RAN1/RE(4,25),QPL(26)
      COMMON/RAN2/J
      QPL(1)=RE(1,J)
      QPL(2)=RE(2,J)
      QPL(3)=RE(3,J)
      QPL(4)=RE(4,J)
      QPL(5)=RE(1,J)
      QPL(6)=0.59*RE(1,J)+0.41*RE(2,J)
      QPL(7)=RE(2,J)
      QPL(8)=0.038*RE(1,J)+0.962*RE(2,J)
      QPL(9)=RE(2,J)
      QPL(10)=0.5*RE(3,J)+0.5*RE(4,J)
      QPL(11)=RE(1,J)
      QPL(12)=0.83*RE(1,J)+0.17*RE(1,J)
      QPL(13)=0.5*RE(2,J)+0.5*RE(3,J)
      QPL(14)=RE(3,J)
      QPL(15)=RE(3,J)
      QPL(16)=0.99*RE(3,J)+0.01*RE(4,J)
      RETURN
      END
```

APPENDIX-I-A(3)

```
C*************************************************
C      SUBROUTINE FOR SUPERIMPOSING LATERAL FLOWS
C     FROM OVELAPPING SUBWATERSHEDS FOR COMPUTING
C     NET LATERAL FLOWS TO  CHANNEL  REACHES  (KOLAR
C     WATERSHED, PHYSIOGRAPHIC MODEL-I)
C*************************************************
      SUBROUTINE CANALQ
      COMMON/CA/QF(16),QC(43),NC
      QC(1)=2*QF(1)
      QC(2)=QC(1)
      QC(3)=QC(1)+2*QF(11)
      QC(4)=QC(3)
      QC(5)=QC(3)+2*QF(5)
      QC(6)=QC(5)
      QC(7)=QC(5)+2*QF(6)
      QC(8)=QC(7)+2*QF(12)
      QC(9)=QC(8)
      QC(10)=2*(QF(1)+QF(5)+QF(6)+QF(8)+QF(12))
      QC(11)=2*(QF(1)+QF(6)+QF(8)+QF(12))
      QC(12)=QC(11)+2*QF(7)
      QC(13)=2*(QF(1)+QF(6)+QF(7)+QF(8)+QF(9)+QF(12))
      QC(14)=2*(QF(1)+QF(7)+QF(8)+QF(9)+QF(12))
      QC(15)=2*(QF(2)+QF(7)+QF(8)+QF(9))
      QC(16)=2*(QF(2)+QF(8)+QF(9))
      QC(17)=QC(16)
      QC(18)=QC(16)
        QC(19)=2*(QF(2)+QF(9))
      QC(20)=QC(19)
      QC(21)=2*(QF(2)+QF(13))
      DO 10 I=22,24
10    QC(I)=QC(21)
      QC(25)=2*QF(2)
      QC(26)=QC(25)
      QC(27)=2*QF(3)
      DO 20 I=28,30
20    QC(I)=QC(27)
      QC(31)=2*(QF(3)+QF(14))
      QC(32)=QC(31)
      QC(33)=QC(31)+2*QF(16)
      QC(34)=QC(33)+2*QF(15)
      QC(35)=QC(34)
      QC(36)=2*(QF(3)+QF(15)+QF(16))
      QC(37)=2*(QF(3)+QF(16))
      QC(38)=QC(37)+2*QF(10)
      QC(39)=QC(38)
      QC(40)=2*(QF(3)+QF(10))
      QC(41)=2*QF(4)
      QC(42)=QC(41)
      QC(43)=QC(41)
      RETURN
      END
```

APPENDIX-I-A(4)

```
C***********************************************
C  SUBROUTINE FOR COMPUTING THE WEIGHTED RAINFALL
C INTENSITY FOR THE SUBWATERSHEDS OF KOLAR RIVER
C (PHYSIOGRAPHIC MODEL-II)
C***********************************************
      SUBROUTINE RAIN
      COMMON/RAN1/RE(4,25),QPL(26)
      COMMON/RAN2/J
      QPL(1)=RE(1,J)
      QPL(2)=RE(1,J)
      QPL(3)=0.91*RE(1,J)+0.09*RE(2,J)
      QPL(4)=0.98*RE(1,J)+0.02*RE(2,J)
      QPL(5)=RE(2,J)
      QPL(6)=RE(2,J)
      QPL(7)=RE(2,J)
      QPL(8)=RE(2,J)
      QPL(9)=0.24*RE(2,J)+0.76*RE(3,J)
      QPL(10)=RE(3,J)
      QPL(11)=RE(3,J)
      QPL(12)=0.77*RE(3,J)+0.23*RE(4,J)
      QPL(13)=RE(4,J)
      QPL(14)=RE(4,J)
      QPL(15)=RE(1,J)
      QPL(16)=0.59*RE(1,J)+0.41*RE(2,J)
      QPL(17)=RE(2,J)
      QPL(18)=0.038*RE(1,J)+0.962*RE(2,J)
      QPL(19)=RE(2,J)
      QPL(20)=0.5*RE(3,J)+0.5*RE(4,J)
      QPL(21)=RE(1,J)
      QPL(22)=0.83*RE(1,J)+0.17*RE(2,J)
      QPL(23)=0.5*RE(2,J)+0.5*RE(3,J)
      QPL(24)=RE(3,J)
      QPL(25)=RE(3,J)
      QPL(26)=0.99*RE(3,J)+0.01*RE(4,J)
      RETURN
      END
```

APPENDIX-I-A(5)

```
C*************************************************
C     SUBROUTINE  FOR SUPERIMPOSING LATERAL FLOWS
C     FROM OVELAPPING SUBWATERSHEDS FOR COMPUTING
C     NET LATERAL FLOWS TO CHANNEL REACHES (KOLAR
C     WATERSHED, PHYSIOGRAPHIC MODEL-II)
C*************************************************
      SUBROUTINE CANALQ
      SUBROUTINE CANALQ
      COMMON/CA/QF(26),QC(39),NC
      QC(1)=2*QF(1)
      QC(2)=QC(1)
      QC(3)=2*QF(1)+2*QF(21)
      QC(4)=QC(3)+2*QF(15)
      QC(5)=QC(4)+2*QF(16)
      QC(6)=QC(5)+2*QF(22)
      QC(7)=2*QF(22)+QF(3)+QF(2)+2*QF(15)+2*QF(16)+2*QF(18)
      QC(8)=2*QF(22)+QF(3)+QF(4)+2*QF(15)+2*QF(16)+2*QF(18)
      QC(9)=2*QF(22)+QF(3)+QF(4)+2*QF(16)+2*QF(17)+2*QF(18)
      QC(10)=2*QF(22)+QF(3)+QF(4)+2*QF(17)+2*QF(18)+2*QF(19)
      QC(11)=2*QF(22)+QF(3)+QF(5)+2*QF(17)+2*QF(18)+2*QF(19)
      QC(12)=QF(6)+QF(5)+2*QF(17)+2*QF(18)+2*QF(19)
      QC(13)=QF(6)+QF(7)+2*QF(18)+2*QF(19)
      QC(14)=QC(13)
      QC(15)=QC(13)
      QC(16)=QF(6)+QF(8)+QF(19)
      QC(17)=QC(16)
      QC(18)=2*QF(23)+QF(6)+QF(8)+2*QF(19)
      QC(19)=2*QF(23)+QF(6)+QF(9)
      QC(20)=QC(19)
      QC(21)=QC(19)
        QC(22)=QF(9)+QF(10)
      QC(23)=QC(22)
      QC(24)=QC(22)
      QC(25)=QC(22)
      QC(26)=2*QF(24)+QF(10)+QF(9)
      QC(27)=2*QF(24)+2*QF(26)+QF(9)+QF(10)
      QC(28)=2*QF(24)+2*QF(25)+2*QF(26)+QF(9)+QF(10)
      QC(29)=QC(28)
      QC(30)=2*QF(25)+2*QF(26)+QF(9)
      QC(31)=2*QF(26)+QF(11)+QF(9)
      QC(32)=2*QF(26)+QF(11)+QF(12)
      QC(33)=QC(32)+2*QF(20)
      QC(34)=QC(33)
      QC(35)=QF(12)+QF(14)+2*QF(20)
      QC(36)=QF(13)+QF(14)
      QC(37)=QC(36)
      QC(38)=QC(36)
      RETURN
      END
```

APPENDIX-I-A(6)

```
C**************************************************
C     SUBROUTINE  FOR SUPERIMPOSING LATERAL FLOWS
C     FROM OVELAPPING SUBWATERSHEDS FOR COMPUTING
C     NET LATERAL FLOWS TO CHANNEL REACHES (RAILWAY
C    BRIDGE NO.719 WATERSHED, PHYSIOGRAPHIC MODEL-I)
C**************************************************
      SUBROUTINE CANALQ
      COMMON/CA/QF(4),QC(10),NC
      QC(1)=2*QF(1)
      QC(2)=QC(1)
      QC(3)=QC(1)
      QC(4)=2*(QF(1)+QF(2)+QF(3)+QF(4))
      QC(5)=QC(4)
      QC(6)=QC(4)
      QC(7)=2*(QF(1)+QF(2)+QF(3))
      QC(8)=QC(7)
      QC(9)=QC(1)
      QC(10)=QC(1)
      RETURN
      END
```

APPENDIX-I-A(7)

```
C**************************************************
C     SUBROUTINE  FOR SUPERIMPOSING LATERAL FLOWS
C     FROM OVELAPPING SUBWATERSHEDS FOR COMPUTING
C     NET LATERAL FLOWS TO CHANNEL REACHES (RAILWAY
C    BRIDGE NO.719 WATERSHED, PHYSIOGRAPHIC
C             MODEL-II)
C**************************************************
      SUBROUTINE CANALQ
      COMMON/CA/QF(8),QC(12),NC
      QC(1)=2*QF(1)
      QC(2)=QC(1)
      QC(3)=QC(1)
      QC(4)=QC(1)+2*QF(8)
      QC(5)=2*(QF(1)+QF(6)+QF(7)+QF(8))
      QC(6)=QC(5)
      QC(7)=QC(5)+QF(3)
      QC(8)=QF(2)+QF(3)+2*(QF(6)+QF(7))
      QC(9)=QC(8)
      QC(10)=QF(2)+QF(3)+QF(5)
      QC(11)=QF(2)+QF(3)+QF(4)+QF(5)
      QC(12)=QF(2)+QF(5)
      RETURN
      END
```

APPENDIX-A(8)

```
C*************************************************
C     SUBROUTINE  FOR SUPERIMPOSING LATERAL FLOWS
C     FROM OVELAPPING SUBWATERSHEDS FOR COMPUTING
C     NET LATERAL FLOWS TO CHANNEL REACHES (KASSILIAN
C     WATERSHED, PHYSIOGRAPHIC MODEL-I)
C*************************************************
      SUBROUTINE CANALQ
      COMMON/CA/QF(6),QC(25),NC
      QC(1)=2*(QF(1)+QF(2))
      QC(2)=QC(1)
      QC(3)=2*(QF(1)+QF(2)+QF(4))
      DO 10 I=4,9
  10  QC(I)=QC(3)
      QC(10)=2*(QF(1)+QF(4))
      QC(11)=2*(QF(1)+QF(4)+QF(3))
       QC(12)=2*(QF(1)+QF(3))
      DO 20 I=13,15
  20  QC(I)=QC(12)
      QC(16)=2*(QF(1)+QF(3)+QF(5))
      QC(17)=QC(16)
      QC(18)=2*(QF(1)+QF(5))
      QC(19)=QC(18)+2*QF(6)
      DO 30 I=20,2230
      QC(I)=QC(19)
      QC(23)=2*(QF(1)+QF(6))
      RETURN
      END
```

APPENDI-I-A(9)

```
C*************************************************
C     SUBROUTINE  FOR SUPERIMPOSING LATERAL FLOWS
C     FROM OVELAPPING SUBWATERSHEDS FOR COMPUTING
C     NET LATERAL FLOWS TO CHANNEL REACHES (KASSILIAN
C     WATERSHED, PHYSIOGRAPHIC MODEL-II)
C*************************************************
    SUBROUTINE CANALQ
    COMMON/CA/QF(9),QC(24),NC
    QC(1)=2*(QF(1)+QF(5))
    QC(2)=QC(1)
    QC(3)=QC(1)+2*QF(7)
    DO 10 I=4,9
10  QC(I)=QC(3)
    QC(10)=QF(2)+2*QF(7)
    QC(11)=QF(2)+2*(QF(6)+QF(7))
    QC(12)=QF(2)+QF(4)+2*QF(6)
    DO  20 I=13,16
20  QC(I)=QC(12)
    QC(17)=QF(2)+QF(4)+2*(QF(6)+QF(8))
    QC(18)=QC(17)
    QC(19)=QF(3)+QF(4)+2*QF(8)
    QC(20)=QC(19)+2*QF(9)
    DO 30 I=21,2330
    QC(I)=QC(20)
    QC(24 )=QF(3)+2*QF(9)
    RETURN
    END
```

APPENDIX-I-A(10)

```
C*****************************************************
C SUBROUTINE FOR COMPUTING THE WEIGHTED RAINFALL
C INTENSITY FOR THE SUBWATERSHEDS OF BARAKAR RIVER
C            (PHYSIOGRAPHIC MODEL-I)
C*****************************************************
      SUBROUTINE CANALQ
      COMMON/CA/QF(17),QC(35),NC
      QC(1)=2*QF(14)
      QC(2)=QC(1)
      QC(3)=QC(1)
      QC(4)=QC(1)
      QC(5)=QC(1)
      QC(6)=2*QF(14)+2*QF(11)
      QC(7)=QC(6)
      QC(8)=2*(QF(14)+QF(11)+QF(12))
      QC(9)=QC(8)
      QC(10)=QC(8)
      QC(11)=QF(1)+2*(QF(11)+QF(12))+QF(2)
      QC(12)=QF(1)+2*(QF(11)+QF(12))+QF(13)+QF(2)
      QC(13)=2*QF(15)+QF(1)+QF(2)+2*(QF(11)+QF(12)+QF(13))
      QC(14)=QC(13)
      QC(15)=2*QF(15)+QF(2)+QF(4)+2*(QF(12)+QF(13))
      QC(16)=2*(QF(15)+QF(16))+QF(4)+2*(QF(12)+QF(13))
      QC(17)=QC(16)
      QC(18)=2*QF(16)+QF(3)+QF(4)+2*(QF(12)+QF(13))
      QC(19)=QC(18)
      QC(20)=QC(18)
      QC(21)=QF(6)+QF(4)+2*(QF(12)+QF(13))
      QC(22)=QF(6)+QF(5)+2*QF(13)
      QC(23)=QC(22)
      QC(24)=2*QF(17)+QF(6)+QF(5)+2*QF(13)
      QC(25)=QC(24)
      QC(26)=QC(24)
      QC(27)=QC(24)
      QC(28)=2*QF(17)+QF(7)+QF(5)+2*QF(13)
      QC(29)=2*QF(17)+QF(7)+QF(8)+2*QF(13)
      QC(30)=QC(29)
      QC(31)=QF(10)+QF(8)+2*QF(13)
      QC(32)=QC(31)
      QC(33)=QF(10)+QF(9)
      QC(34)=QC(33)
      QC(35)=QC(33)
      RETURN
      END
```

APPENDIX-I-B

```
C*************************************************************
COMPUTER PROGRAMME FOR DYNAMIC WAVE CHANNEL FLOW COMPUTATIONS
C*************************************************************
C   THE PROGRAMME IS PART OF THE
C    "MODELING OF FLOOD FLOWS   IN   NATURAL WATERSHEDS"
C    DEVELOPED BY MOHAMMAD REZA NAJAFI SHAHRI(PhD. STUDENT)
C    GUIDED BY DR. B.S. MATHUR (PROF. AND HEAD)
C          AND DR. D. KASHYAP PROFESSOR,
C    DEPARTMENT OF HYDROLOGY UNIVERSITY OF ROORKEE, ROORKEE,
C    INDIA.
C*************************************************************
C VLDZ,VLD1,VLD2 AND VLD3 ARE THE COEFFICIENTS OF
C VELOCITY DEPH RELATION OF D/S BOUNDARY OF THE MAIN CHANNEL.
C NC=TOTAL NUMBER OF COMPUTATIONAL NODES ON THE MAIN CHANNEL.
C NT=TOTAL NO. OF TIME STEPS.
C DT=TIME STEP (SEC).
C DX=SPACE INCREMENT(M).
C KOUNT=MAX. NO. OF ITRATION.
C BS(I)=CHANNEL BED SLOPE.
C AMAN(I)=MANNINGS ROUGHNESS COEFFICIENT.
C UI(I) AND HI(I) ARE INITIAL VELOCITY AND DEPTH OF FLOW.
C UT(I) AND HT(I) ARE THE VELOCITY AND DEPTH  ORDINATES OF
C INFLOW HYDROGRAPH.
C Q(J,I)=LATERAL INFLOW AT TIME J AND NODE I.
C&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
      DIMENSION UI(52),HI(52),UT(61),HT(61),BAS(61)
      DIMENSION UC(52),HC(52),QINF(61),FRD(60)
      COMMON/AA/BS(52),U(52),Q(61,52),
     1DX(52),XU(52),XH(52)
      COMMON/AA2/X(104),NM2,LG
      COMMON/AB/H(52),AMAN(52),QINT(52)
      COMMON/C/IN1(104),IN2(104),IN3(104),NM
      COMMON/C2/Z(104,104),B(104,1)
      OPEN(UNIT=1,FILE='WAVE-D.DAT',STATUS='OLD')
      OPEN(UNIT=2,FILE='WAVE-D.OUT',STATUS='NEW')
      OPEN(UNIT=7,FILE='OUTFLOW.OUT',STATUS='NEW')
      OPEN(UNIT=8,FILE='INTERM.OUT',STATUS='NEW')
```

```
C&&&&&&& FEED THE DATA &&&&&&&&&&&&&&&&&&&&&&
      READ(1,*)VLDZ,VLD1,VLD2,VLD3
      READ(1,*) NC,NT,DT,KOUNT,EPS1,EPS2,TETA,CHW
      READ(1,*)(DX(I),I=1,NC-1)
      READ(1,*)(BS(I),I=1,NC)
      READ(1,*)(AMAN(I),I=1,NC)
      READ(1,*)(UI(I),I=1,NC)
      READ(1,*)(HI(I),I=1,NC)
      READ(1,*)(UT(J),J=1,NT+1)
      READ(1,*)(HT(J),J=1,NT+1)
      READ(1,*)(BAS(J),J=1,NT+1)
      READ(1,*)((Q(J,I),I=1,NC),J=1,NT+1)
C      WRITE(2,10) NC,NT,DT,KOUNT,EPS1,EPS2,TETA
C      WRITE(2,11)VLDZ,VLD1,VLD2,VLD3
C      WRITE(2,12)
C      WRITE(2,13)(DX(I),I=1,NC-1)
C      WRITE(2,14)
C      WRITE(2,44)(BS(I),I=1,NC)
C      WRITE(2,15)
C      WRITE(2,44)(AMAN(I),I=1,NC)
C      WRITE(2,16)
C      WRITE(2,*)(UI(I),I=1,NC)
C      WRITE(2,17)
C      WRITE(2,*)(HI(I),I=1,NC)
C      WRITE(2,18)
C      WRITE(2,*)(UT(J),J=1,NT+1)
C      WRITE(2,19)
C      WRITE(2,*)(HT(J),J=1,NT+1)
C      WRITE(2,21)
C      WRITE(2,*)((Q(J,I),I=1,NC),J=1,NT+1)
 10   FORMAT(2X,'NO. OF CROSS  SECTIONS ON MAIN CHANNEL =',I3
     1/,2X,'NO. OF TIME STEPS =',I5
     2/,2X,'TIME STEP =',F10.1,' (SEC)',
     1/,2X,'MAX. NO. OF ITERATION FOR EACH TIME LEVEL =',I4
     1/,2X,'CONVERGNCE CRITERIA IS  ABS{H(I,K)-H(I,K-1)} < ',F5.4
     1/,2X,'AND',21X,'ABS{U(I,K)-U(I,K-1)} < ',F5.4
     1/,2X,'WHERE K IS THE ITERATION NUMBER.',
```

```
     1/,2X,'WEITAGE COEFFICIENT TETA =',F5.3)
11    FORMAT(2X,'THE MORPHOMETRIC AND FLOW PARAMETERS OF MAIN CHANNEL',
     1//4X,'THE COEFFICIENTS OF D/S RATING CURVE ARE :',/4F12.8)
12    FORMAT(/4X,'DISTANCE INCREMENT DX ',/)
13    FORMAT(8F9.2))
14    FORMAT(/4X,'BED SLOPE ALONG THE CHANNEL ',F7.4 )
44    FORMAT(5(1X,8F7.4/))
15    FORMAT(/4X,'MANNINGS ROUGHNESS COEFF. ALONG THE CHANNEL ARE :',)
16    FORMAT(/4X,'INITIAL FLOW VELOCITY :',)
17    FORMAT(/4X,'INITIAL FLOW DEPTH :',)
18    FORMAT(/4X,'INFLOW HYDROGRAPH VELOCITY :',)
19    FORMAT(/4X,'INFLOW HYDROGRAPH OF DEPTH :',)
21    FORMAT(/4X,'LATERAL INFLOW TO THE CHANNEL q :',)
C&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
      WRITE(*,121)
C C IDS IS A NODE ON THE MOST D/S STRETCH OF THE MAIN CHANNEL.
      READ(*,*)IDS
 121  FORMAT(1X,'TYPE IN THE NODE NO. WHERE YOU REQUIRE THE COMPUTED OUT
     1FLOW ',)
1022  FORMAT(1X,'TYPE IN THE CHANNEL WIDTH IN METER')
C C***  AVERAGE SLOPE FOR A COMPUTATIONAL LINK.
      DO 309 KW=2,NC
      BS(KW)=(BS(KW-1)+BS(KW))/2.0
309   CONTINUE
      WRITE(7,115)
C C   WRITE(8,112)IDS
 112  FORMAT(2X,I4,2X,'IS THE NODE NO. WHERE THE COMPUTED
     1 HYDROGRAPH (PER UNIT WIDTH)',
     1/8X,'IS REQUIRED.',//)
C C   WRITE(8,137)
      SUMIN=0.0
      G=9.81
      BD=4./3.
      NM=2*(NC-1)+2
      NM2=NM
      DO 90 IY=1,NC
      U(IY)=UI(IY)
```

```
      H(IY)=HI(IY)
 90   CONTINUE
C     CALCULATING THE VOLUME OF OBS. INFLOW HYDROGRAPH
      DO 75 JK=1,NT+1
      QINF(JK)=UT(JK)*HT(JK)
      SUMIN=SUMIN+QINF(JK)
 75   CONTINUE
      VOLUMI=SUMIN*DT
      QINT(1)=U(1)*H(1)
      SUMOUT=QINT(1)
      DRH=(QINT(1)-QINF(1))*CHW
      I6=1
      WRITE(7,117)I6,UT(1),HT(1),QINF(1),U(1),
     1H(1),QINT(1),DRH,BAS(1),DRH*CHW
C     FLOW SIMULATION  FOR DIFFERENT TIME STEPS
      DO 150 J=1,NT
      LH=J+1
      HTT=HT(LH)
      UTT=UT(LH)
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
      CALL NEWTON (G,NC,LH,DT,BD,VLDZ,VLD1,VLD2,
     1VLD3,KOUNT,HTT,UTT,J,EPS1,EPS2,TETA)
C*******************************************
C       WRITE(2,88)
C  88   FORMAT(20X,5HU(I),/)
C       WRITE(2,*)(U(I),I=1,NC)
C       WRITE(2,86)
C  86   FORMAT(20X,5HH(I),/)
C       WRITE(2,*)(H(I),I=1,NC)
C     DO 96 I=1,NC
C     FRD(I)=U(I)/(SQRT(G*H(I)))
C96   CONTINUE
C       WRITE(2,*)(FRD(I),I=1,NC)
C     CALCULATING THE VOLUME OF COMPUTED OUT FLOW HYDROGRAPH
      SUMOUT=SUMOUT+QINT(NC)
      DRH=(QINT(NC)-QINF(LH))*CHW
      COMPQ=DRH+BAS(LH)
```

```
      WRITE(7,117)LH,UT(LH),HT(LH),QINF(LH),U(NC),
     1H(NC),QINT(NC),DRH,BAS(LH),COMPQ
117   FORMAT(I4,2F7.3,F9.3,1X,F6.3,1X,F6.3,1X,F9.3,2X,F10.3,1X,
     1F6.2,1X,F10.2)
C     WRITE(8,119)LH,U(1),H(1),QINT(1),U(IDS),H(IDS),
     1QINT(IDS)
119   FORMAT(I4,2F7.4,F8.3,1X,F7.3,1X,F7.3,1X,F7.3)
150   CONTINUE
C     VOLUME COMPUTATION.
      VOLUMO=SUMOUT*DT
      VOLUMI=VOLUMI*CHW
      VOLUMO=VOLUMO*CHW
      DVOL1=VOLUMO-VOLUMI
      EROR=(DVOL1)/VOLUMI*100.0
115   FORMAT(3X,'T'4X,'UT',5X,'HT',4X,'QINF',4X,'UOUT',4X,'HOUT',
     15X,'QOUT',6X,'DRH',3X,'BASE FLOW',3X,'COMP. Q'/)
137   FORMAT(3X,'T'4X,'UT',5X,'HT',4X,'QINF',4X,'UOUT',4X,'HOUT',
     15X,'QOUT',)
C     WRITE(7,212)VOLUMI,VOLUMO,DVOL1,EROR
212   FORMAT(2X,/,'INFLOW VOLUME =',F17.3
     1     /,2X,'OUTFLOW VOLUME =',F17.3
     2    /2X,'OUTFLOW VOLUME -INFLOW VOLUME=',F17.3
     3   /,2X,'PERCENTAGE OF VOLUME ERROR=',F10.5)

517   FORMAT(10X,24HCONVERGENCE NOT ACHIVED,/)
      STOP
      END
C   ************************************************
      SUBROUTINE  NEWTON (G,NC,LH,DT,BD,VLDZ,VLD1,
     1VLD2,VLD3,KOUNT,HTT,UTT,J,EPS1,EPS2,TETA)
C   ************************************************
      COMMON/AA/BS(52),U(52),Q(61,52),
     1DX(52),XU(52),XH(52)
      COMMON/AA2/X(104),NK2,LG
      COMMON/AB/H(52),AMAN(52),QINT(52)
      COMMON/C/IN1(104),IN2(104),IN3(104),NK
      COMMON/C2/F(104,104),BF(104,1)
```

```
      DO 660 IO=1,NC
      IV=2*IO
      IK=IV-1
      X(IK)=H(IO)
      X(IV)=U(IO)
  660 CONTINUE
      AD=7.0/3.0
C       ITERATING FOR UNKNOWNS
      X(1)=HTT
      DO 430 IK2=1,KOUNT
      WRITE(*,*)LH,IK2
      DO 45 IK=1,NK
      DO 441 JK=1,NK
      F(IK,JK)=0.0
 441  CONTINUE
  45  CONTINUE
C     DEFINE  UPSTREAM BOUNDARY CONDITION OF MAIN CHANNEL
C     X(1)=HTT
      F(1,1)=1.0
      BF(1,1)=0.0
C     DOWNSTREAM BOUNDARY EQUATION OF MAIN RIVER CHANNEL.
      F(NK,NK-1)=-(VLD1+2.*VLD2*X(NK-1)+3.*VLD3*X(NK-1)**2)
      F(NK,NK)=1.0
      BF(NK,1)=-(X(NK)-(VLDZ+VLD1*X(NK-1)+VLD2*X(NK-1)**2+VLD3
     1*X(NK-1)**3))
      TETA2=TETA*TETA
      TETAL=TETA*(1-TETA)
      TETAH=(1-TETA)**2
      DO 330 IS=2,NK-1
      KC=IS/2
      IF(IS/2*2.EQ.IS)GO TO 35
      ILL=(IS-1)/2
      IL=2*ILL
      AMA=AMAN(KC)
      AMA2=AMA**2
C     MOMENTUM EQUATION
      RAT=DT/DX(KC)
```

```
 F(IS,IL-1)=TETA*(-2.0*G*RAT-BD*G*AMA2*DT
1*X(IL)**2/X(IL-1)**AD-
2DT*Q(LH,KC)*X(IL)/X(IL-1)**2.0)
 F(IS,IL)=1.0+RAT*(-2*TETA2*X(IL)-2*
1TETAL*U(KC))+
2TETA*(G*AMA2*DT*2*X(IL)/X(IL-1)**BD+
3DT*Q(LH,KC)/X(IL-1))
 F(IS,IL+1)=TETA*(2*G*RAT-BD*G*AMA2*DT*
1X(IL+2)**2/X(IL+1)**AD
2-DT*Q(LH,KC+1)*(X(IL+2)/X(IL+1)**2))
 F(IS,IL+2)=1.0+RAT*(TETA2*2*X(IL+2)+2*
1TETAL*U(KC+1))
2+TETA*(G*AMA2*DT*(2*X(IL+2)/X(IL+1)**BD)+
3DT*Q(LH,KC+1)/X(IL+1))
 BF(IS,1)=-(X(IL+2)+X(IL)+RAT*(TETA2*
1(X(IL+2)**2-X(IL)**2)
1+2*TETAL*(X(IL+2)*U(KC+1)-U(KC)*X(IL))+
2TETAH*(U(KC+1)**2-U(KC)**2))
3+TETA*(2*G*RAT*(X(IL+1)-X(IL-1))
4+G*AMA2*DT*(X(IL+2)**2/X(IL+1)**BD+
4(ABS(X(IL))*X(IL))/X(IL-1)**BD)
5+DT*(Q(LH,KC+1)*X(IL+2)/X(IL+1)+Q(LH,KC)*
5X(IL)/X(IL-1)))
6+(1-TETA)*(2*G*RAT*(H(KC+1)-H(KC))+
7G*AMA2*DT*(ABS(U(KC+1))*U(KC+1)/H(KC+1)**BD+
7ABS(U(KC))*U(KC)/
8H(KC)**BD)
9+DT*(Q(J,KC+1)*U(KC+1)/H(KC+1)+Q(J,KC)*U(KC)/H(KC)))
9-U(KC+1)-U(KC)-2*G*DT*BS(KC))
  GO TO 330
35  ILL=IS/2
    IL=2*ILL
C   WRITE(2,*)KC,DX(KC)
    RAT=DT/DX(KC)
C**** CONTINUITY EQUATION
    F(IS,IL-1)=1.0+2*RAT*(TETA2*(-X(IL))-TETAL*U(KC))
    F(IS,IL)=2*RAT*(-TETA2*X(IL-1)-TETAL*H(KC))
```

```
      F(IS,IL+1)=1.+2*RAT*(TETA2*X(IL+2)+TETAL*U(KC+1))
      F(IS,IL+2)=2*RAT*(TETA2*X(IL+1)+TETAL*H(KC+1))
      BF(IS,1)=-(X(IL+1)+X(IL-1)+2*RAT*(TETA2*(X(IL+2)*
     1X(IL+1)-X(IL)*X(IL-1))
     2+TETAL*(X(IL+2)*H(KC+1)-X(IL)*H(KC)+U(KC+1)*X(IL+1)
     3-U(KC)*X(IL-1))
     4+TETAH*(U(KC+1)*H(KC+1)-U(KC)*H(KC)))
     5-DT*(TETA*(Q(LH,KC+1)+Q(LH,KC))+
     6(1-TETA)*(Q(J,KC+1)+Q(J,KC)))
     7-H(KC+1)-H(KC))
  330 CONTINUE
C  ************************************
      CALL MATINV
C  ************************************
  459 DO 735 IV=1,NC
      JV=2*IV
      KD=JV-1
      XH(IV)=BF(KD,1)+X(KD)
      XU(IV)=BF(JV,1)+X(JV)
  735 CONTINUE
C      WRITE(*,*)(XU(IV),IV=1,NC)
C      WRITE(*,*)(XH(IV),IV=1,NC)
      DO 185 IT=1,NC
      IKR=(2*IT)
      IKC=IKR-1
      IGH=IT
      DBF=ABS(BF(IKC,1)-EPS1)
      TH=ABS(BF(IKC,1))
      IF(ABS(BF(IKR,1))-EPS2)185,185,186
      IF(ABS(BF(IKC,1))-EPS1)185,185,186
  185 CONTINUE
      GO TO 189
  186 DO 67 IR=1,NK
      X(IR)=BF(IR,1)+X(IR)
   67 CONTINUE
  430 CONTINUE
      WRITE(2,518)
```

```
518   FORMAT(10X,24HCONVERGENCE NOT ACHIVED,/)
      STOP
189   DO 360 IV=1,NC
      U(IV)=XU(IV)
      H(IV)=XH(IV)
      QINT(IV)=U(IV)*H(IV)
360   CONTINUE
      WRITE(2,216)
 216  FORMAT(20X,23HINTERMEDIATE DISCHARGE/)
      WRITE(*,216)
      WRITE(*,*)(QINT(ID),ID=1,NC)
      RETURN
      END


         SUBROUTINE MATINV
CCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCCC
C        MATRIX INVERSION
         COMMON/C/IN1(104),IN2(104),IN3(104),N1
 COMMON/C2/Z(104,104),B(104,1)
         M=1
         N=N1
  10     DETERM=1.00
  15     DO 20 J=1,N
  20     IN3(J)=0
  30     DO 550 I=1,N
  40     AMAX=0.00
  45     DO 105 J=1,N
         IF(IN3(J)-1)60,105,60
  60     DO 100 K=1,N
         IF(IN3(K)-1)80,100,999
 999     ID=2
         GOTO 740
  80     IF(AMAX-ABS(Z(J,K)))85,100,100
  85     IROW=J
  90     ICOLUM=K
         AMAX=ABS(Z(J,K))
```

```
 100    CONTINUE
 105    CONTINUE
        IN3(ICOLUM)=IN3(ICOLUM)+1
 260    IN1(I)=IROW
 270    IN2(I)=ICOLUM
 130    IF(IROW-ICOLUM)140,310,140
 140    DETERM=-(DETERM)
 150    DO 200 L=1,N
 160    SWAP=Z(IROW,L)
 170    Z(IROW,L)=Z(ICOLUM,L)
 200    Z(ICOLUM,L)=SWAP
        IF(M)310,310,210
 210    DO 250 L=1,M
 220    SWAP=B(IROW,L)
 230    B(IROW,L)=B(ICOLUM,L)
 250    B(ICOLUM,L)=SWAP
 310    PIVOT=Z(ICOLUM,ICOLUM)
 330    Z(ICOLUM,ICOLUM)=1.00
 340    DO 350 L=1,N
 350    Z(ICOLUM,L)=Z(ICOLUM,L)/PIVOT
 355    IF(M)380,380,360
 360    DO 370 L=1,M
 370    B(ICOLUM,L)=B(ICOLUM,L)/PIVOT
 380    DO 550 L1=1,N
 390    IF(L1-ICOLUM)400,550,400
 400    T=Z(L1,ICOLUM)
 420    Z(L1,ICOLUM)=0.0
 430    DO 450 L=1,N
 450    Z(L1,L)=Z(L1,L)-Z(ICOLUM,L)*T
 455    IF(M)550,550,460
 460    DO 500 L=1,M
 500    B(L1,L)=B(L1,L)-B(ICOLUM,L)*T
 550    CONTINUE
 600    DO 710 I=1,N
 610    L=N+1-I
C 620          IF(IN1(L)-IN2(L))630,710,630
 620    IF(IN1(L)-IN2(L))630,710,630
```

```
630     JROW=IN1(L)
640     JCOLUM=IN2(L)
650     DO 705 K=1,N
660     SWAP=Z(K,JROW)
670     Z(K,JROW)=Z(K,JCOLUM)
700     Z(K,JCOLUM)=SWAP
705     CONTINUE
710     CONTINUE
        DO 730 K=1,N
        IF(IN3(K)-1)715,720,715
715     ID=2
        GO TO 740
720     CONTINUE
730     CONTINUE
        ID=1
740     RETURN
        END
```

APPENDIX-I-C

```
C*****************************************************************
C    COMPUTATION OF STEADY STATE FLOW VELOCITIES CORRESPONDING
C    TO DIFFERENT STAGES OF FLOOD.

C*****************************************************************
C Z=SIDE SLOPE, B= BED WIDTH, SLOP= BED SLOPE, ROUF=
C MANNING'S ROUGHNESS, H=FLOW DEPTH, DIS=DISTANCE BETWEEN
C THE  SITES. DX=DISTANCE FROM SITE,FOR WHICH PARAMETRS
C TO BE CALCULATED.
C  NSITE= NO. OF SITES, HMAX= MAXIMUM DEPTH, DH=INCREAMENTAL
C DEPTH FOR EACH TRIAL, NH= NO. OF DEPTH FOR WHICH OTHER
C PARAMETRS SHOULD BE EVALUATED.
      DIMENSION Z(10),B(10),SLOP(10),ROUF(10),DIS(10),DX(10)
     1,Q(50),H(28000),ZCH(50),BCH(50),SLOPCH(50),ROUFCH(50)
      OPEN(UNIT=1,FILE='BES.DAT',STATUS='OLD')
      OPEN(UNIT=2,FILE='TRICA.OUT',STATUS='NEW')
      READ(1,*)NSIT,HMAX,DH,NH,EPS,DT
      DO 50 IT=1,NSIT
      READ(1,*)Z(IT),B(IT),SLOP(IT),ROUF(IT)
      WRITE(2,*)Z(IT),B(IT),SLOP(IT),ROUF(IT)
   50 CONTINUE
      DO 60 IZ=1,NSIT-1
      READ(1,*)DIS(IZ),DX(IZ)
      WRITE(2,*)DIS(IZ),DX(IZ)
   60 CONTINUE
      READ(1,*)(Q(I),I=1,NH)
      KOUNTH=HMAX/DH
      ZCH(1)=Z(1)
      BCH(1)=B(1)
      SLOPCH(1)=SLOP(1)
      ROUFCH(1)=ROUF(1)
      BF=2.0/3.0
      DO 100 J=1,NSIT-1
      NDX=DIS(J)/DX(J)
      NC=NDX+1
      DZ=(Z(J+1)-Z(J))/NDX
     DB=(B(J+1)-B(J))/NDX
```

```
      DSLOP=(SLOP(J+1)-SLOP(J))/NDX
      DROUF=(ROUF(J+1)-ROUF(J))/NDX
      WRITE(2,*)NC
      DO 200 I=2,NC
      ZCH(I)=ZCH(I-1)+DZ
      BCH(I)=BCH(I-1)+DB
      SLOPCH(I)=SLOPCH(I-1)+DSLOP
      ROUFCH(I)=ROUFCH(I-1)+DROUF
  200 CONTINUE
      DO 10 N=1,NC
      WRITE(2,15)N,ZCH(N),BCH(N),SLOPCH(N),ROUFCH(N)
   10 CONTINUE
  100 CONTINUE
   15 FORMAT(2X,I3,2X,F7.4,2X,F9.3,2X,F7.5,2X,F7.5)
      TOTAL=0.0
      DO 110 J=1,NSIT-1
C*****************************************
      IF (J.GT.1)GO TO 500
C*****************************************
      WRITE(2,*)J
      H(1)=0.0
      DO 240 L=1,NH
      DO 300 K=2,KOUNTH
      H(K)=H(K-1)+DH
      A=(B(J)+Z(J)*H(K))*H(K)
      P=B(J)+2.0*H(K)*(SQRT(Z(J)**2+1.0))
      QC=(A/ROUF(J))*(A/P)**BF*SQRT(SLOP(J))
      QERROR=((ABS(Q(L)-QC))/Q(L))*100
      IF(QERROR.LT.EPS)GO TO 310
      HC=H(K)
  300 CONTINUE
  310 HR=A/P
      TOPW=B(J)+2.0*Z(J)*HC
      V=QC/A
C     WRITE(2,25)L,HC,V,QC,Q(L),A,P,HR,TOPW
C****** QUNIT=DISCHARGE PER UNIT WIDTH***************
      QUNIT=HC*V
```

```
      TOTAL=TOTAL+QUNIT
      WRITE(2,*)V,HC,QC,QUNIT
      WRITE(*,*)V,HC,QC,QUNIT
      H(1)=HC-DH
C     H(1)=0.0
240   CONTINUE
      VOLUM=TOTAL*DT
      WRITE(2,550)VOLUM
110   CONTINUE
25    FORMAT(I3,2X,F9.4,2X,F8.3,2X,F10.3,2X,F10.3,2X,F8.3,2X,3F8.3)
550   FORMAT(2X,'DISCHARGE VOLUME PER UNIT WIDTH =',F15.4)
500   STOP
      END
```

APPENDIX-I-D

INPUT DATA FILE (WAVE-D·DAT)

```
0.40270350  1.82908200  -0.23624940  0.01625036
38 38 3600.0  6 0.006 0.01 0.67 80.0

500.00  3000.00  3500.00  2500.00  3500.00  1500.00  3000.00  3000.00
2000.00  3000.00  2000.00  2666.66  2666.66  2666.66  2500.00  2500.00
500.00  3000.00  3000.00  3000.00  3375.00  3375.00  3375.00  3375.00
3500.00  1000.00  2750.00  2750.00  1000.00  1500.00  1000.00  2500.00
2500.00  3000.00  2500.00  2500.00  2500.00

.0030  .0030  .0030  .0030  .0030  .0030  .0030  .0030
.0030  .0030  .0030  .0030  .0030  .0030  .0030  .0030
.0030  .0030  .0030  .0030  .0030  .0030  .0030  .0030
.0030  .0030  .0030  .0030  .0030  .0030  .0030  .0030
.0030  .0030  .0030  .0030  .0030  .0030

.0260  .0260  .0260  .0260  .0260  .0260  .0260  .0260
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2.339988E-01  2.339988E-01  2.339988E-01
```

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2.339988E-01	2.339988E-01	2.339988E-01	2.339988E-01	2.339988E-01
2.339988E-01	2.339988E-01	2.339988E-01	2.339988E-01	

54 54 54.91 55.83 56.74 57.65 58.56 59.48 60.39 61.3 62.2 63.11 64.03
64.94 65.85 66.77 67.68 68.59 69.5 70.42 71.33 72.24 73.16 74.09 75 14*75

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.0000000	.0000000	.0000000	.0000000	
.0000000	.0000000	.0000000	.0000000	
.0000000	.0000000	.0000000	.0000000	
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2.964769E-06	3.778008E-06	3.778008E-06	2.085820E-06	1.743033E-06
1.456781E-06	1.585964E-06	1.585964E-06	3.482719E-07	.0000000
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1.772701E-07 2.081061E-07 2.349433E-07 1.864454E-07 1.245393E-07
1.014007E-07 6.361159E-08 7.023426E-010 7.023426E-010 7.023426E-010
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6.519905E-04 5.352409E-04 5.852049E-04 3.918469E-04 3.186514E-04
2.008075E-04 3.767252E-06 3.658322E-06 3.658322E-06 3.658322E-06
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1.733471E-04 1.493697E-04 1.739849E-04 1.739849E-04 6.270477E-05
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1.809210E-03 1.200648E-03 9.782219E-04 6.721852E-04 6.165066E-05
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1.000502E-03 8.899341E-04 8.899341E-04 8.899341E-04 3.403911E-04
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4.271221E-04 5.251802E-04 5.251802E-04 3.345063E-04 2.819976E-04
2.387298E-04 4.134547E-04 4.134547E-04 4.027273E-04 3.104203E-04

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6.479163E-04	8.004472E-04	9.928712E-04	1.161012E-03	1.226179E-03
1.143435E-03	1.257433E-03	1.257854E-03	1.164031E-03	1.155654E-03
1.155654E-03	1.155654E-03	4.230042E-04	4.230042E-04	7.141741E-04
4.732352E-04	4.732352E-04	4.732352E-04	1.957536E-04	1.957536E-04
1.957536E-04	1.957536E-04	3.311952E-04	4.676273E-04	5.766064E-04
5.766064E-04	3.616860E-04	3.049375E-04	2.409554E-04	3.755718E-04
3.755718E-04	2.687379E-04	1.342961E-04	1.342961E-04	1.342961E-04
1.900931E-04	1.900931E-04	3.409694E-04	5.623613E-04	6.882694E-04
8.513047E-04	8.584281E-04	9.211499E-04	7.938310E-04	8.260095E-04
7.832597E-04	6.606242E-04	6.855796E-04	6.855796E-04	6.855796E-04
2.548766E-04	2.548766E-04	5.001483E-04	4.811028E-04	4.811028E-04
4.811028E-04	3.736741E-04	3.736741E-04	3.736741E-04	3.736741E-04
6.792907E-04	9.709386E-04	1.221543E-03	1.221543E-03	7.235515E-04
5.974658E-04	5.008664E-04	6.381605E-04	6.381605E-04	2.514814E-04
4.583057E-05	4.583057E-05	4.583057E-05	1.716248E-04	1.716248E-04
3.138891E-04	5.192266E-04	6.466476E-04	8.013222E-04	8.917679E-04
9.611451E-04	8.648654E-04	9.052614E-04	8.847244E-04	7.680227E-04
7.544381E-04	7.544381E-04	7.544381E-04	2.920183E-04	2.920183E-04
5.574386E-04	4.917966E-04	4.917966E-04	4.917966E-04	3.404093E-04
3.404093E-04	3.404093E-04	3.404093E-04	6.415781E-04	9.504028E-04
1.189258E-03	1.189258E-03	7.254854E-04	5.969920E-04	5.041673E-04
6.298294E-04	6.298294E-04	2.234833E-04	1.960381E-05	1.960381E-05
1.960381E-05	2.065628E-04	2.065628E-04	3.830954E-04	6.369854E-04
7.590959E-04	9.289677E-04	8.738280E-04	9.531662E-04	7.699978E-04
7.619981E-04	6.871918E-04	4.976080E-04	5.068792E-04	5.068792E-04
5.068792E-04	1.801254E-04	1.801254E-04	3.975552E-04	4.279613E-04
4.279613E-04	4.279613E-04	3.640345E-04	3.640345E-04	3.640345E-04
3.640345E-04	6.890196E-04	1.014390E-03	1.275472E-03	1.275472E-03
7.624092E-04	6.258214E-04	5.404497E-04	6.724353E-04	6.724353E-04
2.292504E-04	1.011413E-05	1.011413E-05	1.011413E-05	3.345878E-04
3.345878E-04	6.279272E-04	1.043613E-03	1.218005E-03	1.471669E-03
1.244075E-03	1.379256E-03	1.025810E-03	9.522705E-04	7.995974E-04
4.454555E-04	4.518380E-04	4.518380E-04	4.518380E-04	1.670670E-04
1.670670E-04	3.982308E-04	4.830958E-04	4.830958E-04	4.830958E-04
4.564578E-04	4.564578E-04	4.564578E-04	4.564578E-04	8.658360E-04
1.273577E-03	1.602586E-03	1.602586E-03	9.569077E-04	7.846430E-04
6.774761E-04	8.316233E-04	8.316233E-04	2.710677E-04	5.920143E-06
5.920143E-06	5.920143E-06	4.701508E-04	4.701508E-04	8.791360E-04

1.463081E-03	1.688669E-03	2.033049E-03	1.655590E-03	1.848994E-03
1.331392E-03	1.207792E-03	1.000531E-03	4.878108E-04	4.801400E-04
4.801400E-04	4.801400E-04	1.751645E-04	1.751645E-04	4.173334E-04
4.855619E-04	4.855619E-04	4.855619E-04	4.349171E-04	4.349171E-04
4.349171E-04	4.349171E-04	8.315897E-04	1.234920E-03	1.551219E-03
1.551219E-03	9.336263E-04	7.655203E-04	6.632782E-04	8.262239E-04
8.262239E-04	2.772194E-04	3.781273E-06	3.781273E-06	3.781273E-06
4.266849E-04	4.266849E-04	7.754833E-04	1.285119E-03	1.499784E-03
1.823832E-03	1.624419E-03	1.782657E-03	1.390251E-03	1.345906E-03
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2.954887E-04	2.954887E-04	5.691557E-04	4.856566E-04	4.856566E-04
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6.086287E-04	9.046810E-04	1.129318E-03	1.129318E-03	6.930250E-04
5.713875E-04	4.809721E-04	6.184954E-04	6.184954E-04	2.211620E-04
2.573044E-06	2.573044E-06	2.573044E-06	2.548406E-04	2.548406E-04
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1.212467E-03	1.056515E-03	1.120182E-03	1.098363E-03	9.362944E-04
9.201933E-04	9.201933E-04	9.201933E-04	3.406239E-04	3.406239E-04
5.828849E-04	3.901357E-04	3.901357E-04	3.901357E-04	1.608345E-04
1.608345E-04	1.608345E-04	1.608345E-04	2.969368E-04	4.459815E-04
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3.231939E-04	3.231939E-04	1.296208E-04	1.836924E-06	1.836924E-06
1.836924E-06	1.672201E-04	1.672201E-04	2.931359E-04	4.825732E-04
5.713812E-04	7.078366E-04	6.786452E-04	7.293737E-04	6.149421E-04
6.522049E-04	6.220136E-04	5.201496E-04	5.414475E-04	5.414475E-04
5.414475E-04	1.991596E-04	1.991596E-04	3.332393E-04	2.164906E-04
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6.964595E-05	1.361649E-06	1.361649E-06	1.361649E-06	1.169810E-04
1.169810E-04	2.055787E-04	3.383385E-04	3.961438E-04	4.902483E-04
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2.766924E-04	2.955382E-04	2.955382E-04	2.955382E-04	1.104293E-04
1.104293E-04	1.830994E-04	1.196692E-04	1.196692E-04	1.196692E-04
3.264712E-05	3.264712E-05	3.264712E-05	3.264712E-05	6.214886E-05
9.722267E-05	1.193910E-04	1.193910E-04	7.753968E-05	6.437368E-05
5.379440E-05	8.394885E-05	8.394885E-05	4.057304E-05	1.040351E-06
1.040351E-06	1.040351E-06	7.411202E-05	7.411202E-05	1.293657E-04
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2.517997E-04	2.631657E-04	2.492537E-04	2.002248E-04	2.095208E-04
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7.984356E-05	7.984356E-05	7.984356E-05	1.879024E-05	1.879024E-05
1.879024E-05	1.879024E-05	3.466863E-05	5.395364E-05	6.577375E-05
6.577375E-05	4.349485E-05	3.637455E-05	2.933164E-05	4.803426E-05
4.803426E-05	2.459921E-05	8.148778E-07	8.148778E-07	8.148778E-07
6.703455E-05	6.703455E-05	1.192683E-04	1.966218E-04	2.276339E-04
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1.760082E-04	1.194994E-04	1.264217E-04	1.264217E-04	1.264217E-04
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4.800027E-05	1.092556E-05	1.092556E-05	1.092556E-05	1.092556E-05
2.039005E-05	3.204531E-05	3.904606E-05	3.904606E-05	2.587469E-05
2.161876E-05	1.762915E-05	2.987015E-05	2.987015E-05	1.591057E-05
6.516702E-07	6.516702E-07	6.516702E-07	4.858073E-05	4.858073E-05
8.600489E-05	1.418835E-04	1.662565E-04	2.052054E-04	1.882466E-04
2.055861E-04	1.671518E-04	1.711167E-04	1.610662E-04	1.243092E-04
1.260452E-04	1.260452E-04	1.260452E-04	4.787112E-05	4.787112E-05
7.534157E-05	4.311318E-05	4.311318E-05	4.311318E-05	8.458269E-06
8.458269E-06	8.458269E-06	8.458269E-06	1.450957E-05	2.204661E-05
2.649940E-05	2.649940E-05	1.813530E-05	1.540750E-05	1.134811E-05
1.973754E-05	1.973754E-05	1.083463E-05	5.304088E-07	5.304088E-07
5.304088E-07	3.529572E-05	3.529572E-05	6.195765E-05	1.021735E-04
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1.188872E-04	1.115831E-04	8.580104E-05	8.831992E-05	8.831992E-05
8.831992E-05	3.294956E-05	3.294956E-05	5.232126E-05	3.052595E-05
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7.707336E-06	4.382837E-07	4.382837E-07	4.382837E-07	2.615996E-05
2.615996E-05	4.551835E-05	7.496354E-05	8.637509E-05	1.067689E-04
9.201723E-05	1.006598E-04	7.944026E-05	8.214298E-05	7.646829E-05
5.789298E-05	6.096675E-05	6.096675E-05	6.096675E-05	2.247883E-05
2.247883E-05	3.613064E-05	2.175348E-05	2.175348E-05	2.175348E-05
4.068026E-06	4.068026E-06	4.068026E-06	4.068026E-06	6.975283E-06
1.065566E-05	1.277572E-05	1.277572E-05	8.795598E-06	7.481633E-06
5.502852E-06	9.920854E-06	9.920854E-06	5.683322E-06	3.669365E-07
3.669365E-07	3.669365E-07	1.970631E-05	1.970631E-05	3.399399E-05
5.589627E-05	6.389162E-05	7.907106E-05	6.596291E-05	7.208674E-05
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4.298579E-05	4.298579E-05	1.577558E-05	1.577558E-05	2.560886E-05
1.587697E-05	1.587697E-05	1.587697E-05	2.933840E-06	2.933840E-06
2.933840E-06	2.933840E-06	5.072539E-06	7.796469E-06	9.349795E-06
9.349795E-06	6.432753E-06	5.465507E-06	4.061214E-06	7.416056E-06
7.416056E-06	4.316960E-06	3.107500E-07	3.107500E-07	3.107500E-07
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5.954206E-05	4.824147E-05	5.265325E-05	4.007987E-05	4.169011E-05
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1.136991E-05	1.136991E-05	1.857788E-05	1.180248E-05	1.180248E-05
1.180248E-05	2.168621E-06	2.168621E-06	2.168621E-06	2.168621E-06
3.789393E-06	5.864559E-06	7.037549E-06	7.037549E-06	4.834227E-06
4.100736E-06	3.086065E-06	5.695431E-06	5.695431E-06	3.361204E-06
2.658491E-07	2.658491E-07	2.658491E-07	1.168773E-05	1.168773E-05
1.985292E-05	3.255529E-05	3.671995E-05	4.556560E-05	3.596366E-05
3.919782E-05	2.934301E-05	3.056805E-05	2.802743E-05	2.021045E-05
2.252363E-05	2.252363E-05	2.252363E-05	8.383602E-06	8.383602E-06
1.375630E-05	8.914746E-06	8.914746E-06	8.914746E-06	1.638196E-06
1.638196E-06	1.638196E-06	1.638196E-06	2.897009E-06	4.516295E-06
5.424637E-06	5.424637E-06	3.718395E-06	3.148227E-06	2.402587E-06
4.473981E-06	4.473981E-06	2.672220E-06	2.294995E-07	2.294995E-07
2.294995E-07	9.189689E-06	9.189689E-06	1.551120E-05	2.540472E-05
2.850533E-05	3.541485E-05	2.732847E-05	2.974391E-05	2.193438E-05
2.286618E-05	2.091797E-05	1.483965E-05	1.674997E-05	1.674997E-05
1.674997E-05	6.306022E-06	6.306022E-06	1.037884E-05	6.834839E-06
6.834839E-06	6.834839E-06	1.261475E-06	1.261475E-06	1.261475E-06
1.261475E-06	2.259543E-06	3.548852E-06	4.267369E-06	4.267369E-06
2.918054E-06	2.465479E-06	1.909057E-06	3.582466E-06	3.582466E-06
2.162603E-06	1.997319E-07	1.997319E-07	1.997319E-07	7.320307E-06
7.320307E-06	1.229018E-05	2.010533E-05	2.246147E-05	2.793506E-05
2.115657E-05	2.299585E-05	1.674031E-05	1.745347E-05	1.594190E-05
1.113734E-05	1.269745E-05	1.269745E-05	1.269745E-05	4.830471E-06
4.830471E-06	7.970300E-06	5.316704E-06	5.316704E-06	5.316704E-06
9.881830E-07	9.881830E-07	9.881830E-07	9.881830E-07	1.793554E-06
2.837939E-06	3.416734E-06	3.416734E-06	2.330404E-06	1.964695E-06
1.543567E-06	2.916144E-06	2.916144E-06	1.777216E-06	1.751024E-07
1.751024E-07	1.751024E-07	5.904113E-06	5.904113E-06	9.868860E-06
1.612566E-05	1.795115E-05	2.234493E-05	1.666702E-05	1.809474E-05
1.303037E-05	1.358185E-05	1.239218E-05	8.536811E-06	9.808020E-06
9.808020E-06	9.808020E-06	3.763826E-06	3.763826E-06	6.224344E-06

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7.862465E-07	7.862465E-07	1.446063E-06	2.304695E-06	2.778330E-06
2.778330E-06	1.890100E-06	1.589990E-06	1.266970E-06	2.407837E-06
2.407837E-06	1.480124E-06	1.545343E-07	1.545343E-07	1.545343E-07
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1.810926E-05	1.334236E-05	1.447098E-05	1.032768E-05	1.075950E-05
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3.355549E-06	6.345761E-07	6.345761E-07	6.345761E-07	6.345761E-07
1.182353E-06	1.897452E-06	2.290391E-06	2.290391E-06	1.554270E-06
1.304664E-06	1.053678E-06	2.013103E-06	2.013103E-06	1.247219E-06
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1.174478E-05	8.320201E-06	8.662845E-06	7.892524E-06	5.309034E-06
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1.911355E-06	1.911355E-06	1.294007E-06	1.083946E-06	8.864824E-07
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1.225155E-07	3.315672E-06	3.315672E-06	5.492316E-06	8.947253E-06
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7.077085E-06	6.444278E-06	4.292065E-06	4.998981E-06	4.998981E-06
4.998981E-06	1.951513E-06	1.951513E-06	3.246019E-06	2.231752E-06
2.231752E-06	2.231752E-06	4.295226E-07	4.295226E-07	4.295226E-07
4.295226E-07	8.200007E-07	1.332543E-06	1.612587E-06	1.612587E-06
1.089366E-06	9.107379E-07	7.535090E-07	1.452716E-06	1.452716E-06
9.125364E-07	1.099564E-07	1.099564E-07	1.099564E-07	2.791759E-06
2.791759E-06	4.615095E-06	7.512118E-06	8.287260E-06	1.033630E-05
7.428857E-06	8.041307E-06	5.633161E-06	5.857567E-06	5.331336E-06
3.519890E-06	4.109874E-06	4.109874E-06	4.109874E-06	1.610383E-06
1.610383E-06	2.683046E-06	1.852596E-06	1.852596E-06	1.852596E-06
3.594630E-07	3.594630E-07	3.594630E-07	3.594630E-07	6.939369E-07
1.133988E-06	1.373953E-06	1.373953E-06	9.263207E-07	7.730097E-07
6.463847E-07	1.251085E-06	1.251085E-06	7.907042E-07	9.915598E-08
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APPENDIX-II-A

BRIDGE NO. 719 (INDIA)
RAINFALL RUNOFF DATA

(i) STORM EVENT DATED: 24/25.7.1964

RUNOFF OBSERVATION		RAINFALL OBSERVATION			
TIME (HR)	DISCHARGE (cumec)	TIME (HR)	RAINFALL (mm)	Ø-INDEX (mm/HR)	Rainfall Excess (mm/0.5HR)
1	2	3	4	5	6
0.0	1.16	0.0-0.5	7.55	13.10	1.00
0.5	1.53	0.5-1.0	7.00		0.45
1.0	2.04	1.0-1.5	9.00		2.45
1.5	3.65	1.5-2.0	2.45		0.00
2.0	4.56	2.0-2.5	6.00		0.00
2.5	8.10	2.5-3.0	12.00		5.45
3.0	9.77	3.0-3.5	15.00		8.45
3.5	12.89	3.5-4.0	4.00		0.00
4.0	21.24				
4.5	25.25				
5.0	21.24				
5.5	16.99				
6.0	12.60				
6.5	8.69				
7.0	5.58				
7.5	4.42				
8.0	2.41				
8.5	1.16				

(ii) STORM EVENT DATED: 26/27.7.1964

1	2	3	4	5	6
0.0	1.20	0.0-0.5	6.00	5.80	3.10
0.5	2.50	0.5-1.0	8.00		5.10
1.0	4.60	1.0-1.5	3.84		0.94
1.5	6.60	1.5-2.0	2.36		0.00
2.0	8.50	2.0-2.5	6.29		3.39
2.5	11.80	2.5-3.0	8.90		6.00
3.0	13.93	3.0-3.5	13.00		10.10
3.5	18.21				
4.0	25.50				
4.5	33.84				
5.0	28.20				
5.5	22.14				
6.0	19.09				
6.5	16.09				
7.0	8.61				
7.5	6.88				
8.0	5.52				
8.5	3.65				
9.0	2.57				
9.5	1.20				

(iii) STORM EVENT DATED: 3.9.1964

1	2	3	4	5	6
0.0	1.13	0.0-0.5	2.36	13.78	0.00
0.5	2.13	0.5-1.0	1.96		0.00
1.0	3.96	1.0-1.5	8.08		1.19
1.5	5.09	1.5-2.0	9.63		2.74
2.0	6.79	2.0-2.5	11.41		4.52
2.5	8.49				
3.0	11.61				
3.5	10.76				
4.0	8.49				
4.5	6.85				
5.0	5.81				
5.5	4.43				
6.0	3.26				
6.5	2.65				
7.0	2.41				
7.5	1.70				
8.0	1.13				

(iv) STORM EVENT DATED: 11.8.1965

1	2	3	4	5	6
0.0	1.10	0.0-0.5	39.37	38.92	19.91
0.5	7.50	0.5-1.0	28.35		8.89
1.0	19.31	1.0-1.5	0.00		
1.5	27.10	1.5-2.0	2.36		
2.0	41.06				
2.5	31.86				
3.0	28.32				
3.5	21.83				
4.0	13.48				
4.5	10.22				
5.0	6.93				
5.5	2.09				
6.0	1.47				
6.5	1.10				

(v) STORM EVENT DATED: 16.9.1966

1	2	3	4	5	6
0.0	15.43	0.0-0.5	24.00	16.00	16.00
0.5	16.62	0.5-1.0	35.00		27.00
1.0	37.52	1.0-1.5	11.00		3.00
1.5	97.37				
2.0	86.47				
2.5	71.91				
3.0	57.35				
3.5	50.07				
4.0	40.95				
4.5	37.52				
5.0	31.86				
5.5	29.45				
6.0	25.97				
6.5	23.64				
7.0	22.37				
7.5	22.37				

(vi) STORM EVENT DATED: 19.9.1966

1	2	3	4	5	6
0.0	7.60	0.0-0.5	16.40	18.88	6.70
0.5	19.40	0.5-1.0	34.64		25.20
1.0	32.86				
1.5	46.42				
2.0	41.95				
2.5	37.52				
3.0	31.86				
3.5	28.32				
4.0	25.97				
4.5	22.30				
5.0	19.80				
5.5	18.50				
6.0	15.03				
6.5	14.50				
7.0	13.00				
7.5	12.03				
8.0	12.00				
8.5	10.50				
9.0	10.00				

APPENDIX-II-B

KASSILIAN WATERSHED (IRAN)
RAINFALL RUNOFF DATA

(i) STORM EVENT DATED: 21.1.1979

RUNOFF OBSERVATION		RAINFALL OBSERVATION			
TIME (HR)	DISCHARGE (cumec)	TIME (HR)	RAINFALL (mm)	Ø-INDEX (mm/HR)	Rainfall Excess (mm/HR)
1	2	3	4	5	6
0.00	0.96	0.0-0.1	0.00	2.68	0.00
1.00	0.98	1.0-2.0	0.30		0.00
2.00	1.00	2.0-3.0	3.60		2.26
3.00	1.05	3.0-4.0	1.70		0.36
4.00	1.65				
5.00	4.65				
6.00	7.50				
7.00	7.00				
8.00	6.25				
9.00	5.50				
10.00	4.80				
11.00	4.20				
12.00	3.65				
13.00	3.10				
14.00	2.80				
15.00	2.50				
16.00	2.25				
17.00	2.00				
18.00	1.80				
19.00	1.70				
20.00	1.60				
21.00	1.55				
22.00	1.35				
23.00	1.20				
24.00	1.10				
25.00	1.05				
26.00	1.00				
27.00	0.97				
28.00	0.96				

(ii) STORM EVENT DATED: 24.2.1979

1	2	3	4	5	6
0	0.40	-	-	-	-
1	0.42	1.0-2.0	0.20	2.254	0.000
2	0.45	2.0-3.0	0.90		0.000
3	0.43	3.0-4.0	0.90		0.000
4	0.44	4.0-5.0	2.70		1.073
5	0.44	5.0-6.0	1.40		0.000
6	0.46	6.0-7.0	0.40		0.000
7	0.54				
8	0.65				
9	0.95				
10	1.15				
11	1.40				
12	1.50				
13	1.45				
14	1.40				
15	1.35				
16	1.30				
17	1.25				
18	1.18				
19	1.15				
20	1.05				
21	1.02				
22	1.00				
23	0.95				
24	0.92				
25	0.90				
26	0.85				
27	0.82				
28	0.80				
29	0.78				
30	0.77				
31	0.76				
32	0.74				
33	0.43				
34	0.72				

(iii) STORM EVENT DATED: 23.5.1979

1	2	3	4	5	6
0	0.75	0.0-1.0	0.20	0.037	0.1815
1	0.75	1.0-2.0	0.80		0.7815
2	0.75	2.0-3.0	0.80		0.7815
3	0.80	3.0-4.0	0.40		0.3815
4	0.95	4.0-5.0	0.50		0.4815
5	5.80	5.0-6.0	0.70		0.6815
6	10.55	6.0-7.0	0.50		0.4815
7	13.00	7.0-8.0	0.20		0.1815
8	13.00	8.0-9.0	0.60		0.5815
9	10.75	9.0-10.0	1.30		1.2815
10	8.80				
11	7.75				
12	6.90				
13	6.10				
14	5.75				
15	5.00				
16	4.45				
17	3.90				
18	3.30				
19	3.00				
20	2.80				
21	2.45				
22	2.25				
23	2.00				
24	1.85				
25	1.65				
26	1.50				
27	1.35				
28	1.27				
29	1.10				
30	1.00				
31	0.90				
32	0.80				
33	0.75				

(iv) STORM EVENT DATED: 26.6.1979

1	2	3	4	5	6
0	0.24	0.0-1.0	0.90	0.67	0.565
1	0.26	1.0-2.0	2.10		1.765
2	0.28	2.0-3.0	2.20		1.865
3	0.30	3.0-4.0	0.80		0.465
4	0.35				
5	0.50				
6	0.65				
7	1.22				
8	1.65				
9	2.80				
10	5.25				
11	9.10				
12	8.15				
13	7.75				
14	7.00				
15	6.50				
16	5.85				
17	4.95				
18	4.40				
19	4.00				
20	3.55				
21	2.90				
22	2.75				
23	2.50				
24	2.30				
25	2.00				
26	1.80				
27	1.75				
28	1.70				
29	1.55				
30	1.45				

APPENDIX-II-C

KOLAR RIVER WATERSHED (INDIA)
RAINFALL RUNOFF DATA

(1) STORM EVENT DATED: 28.8.1983

RUNOFF OBSERVATION		RAINFALL OBSERVATION								
TIME (HR)	DISCHARGE (cumec)	TIME (HR)	BRIJESHNAGAR		BIRPUR		JHALIAPUR		REHTI	
			RAINFALL @ (mm)	RE (mm/HR)	RAINFALL (mm)	RE (mm/HR)	RAINFALL (mm)	RE (mm/HR)	RAINFALL (mm)	RE (mm/HR
1	2	3	4	5	6	7	8	9	10	11
0.0	8.34	0.0-1.0	0.00	0.000	0.00	0.000	5.00	1.532	0.25	0.000
1.0	8.24	1.0-2.0	2.00	0.000	0.00	0.000	6.50	3.032	0.75	0.000
2.0	8.00	2.0-3.0	4.50	0.398	1.75	0.000	5.50	2.032	10.50	9.074
3.0	7.90	3.0-4.0	53.50	49.398	3.00	0.586	21.00	17.532	2.60	1.174
4.0	7.81	4.0-5.0	50.00	45.898	8.75	6.336	27.25	23.782	18.60	17.174
5.0	10.34	5.0-6.0	43.50	39.398	6.50	4.086	10.50	7.032	14.75	13.324
6.0	27.51	6.0-7.0	5.50	1.398	40.00	37.586	15.75	12.282	39.00	37.574
7.0	125.57	7.0-8.0	17.00	12.898	25.00	22.586	16.35	12.882	3.60	2.174
8.0	328.17	8.0-9.0	18.50	14.398	11.00	8.586	37.50	34.032	0.00	0.000
9.0	663.74	9.0-10.0	15.50	11.398	24.00	21.586	28.00	24.532	0.00	0.000
10.0	2814.26	10.0-11.0	23.00	18.898	7.00	4.586	34.25	30.782	0.00	0.000
11.0	4069.34	11.0-12.0	39.00	34.898	13.00	10.586	41.75	38.282	0.00	0.000
12.0	4723.82	12.0-13.0	42.00	37.898	14.00	11.586	38.30	34.832	0.00	0.000
13.0	4879.21	13.0-14.0	26.00	21.898	26.00	23.586	25.75	22.282	0.00	0.000
14.0	4800.00	14.0-15.0	75.00	3.398	23.00	20.586	9.25	5.782	0.00	0.000
15.0	4665.00	15.0-16.0	11.00	6.898	7.00	4.586	4.25	0.782	0.40	0.000
16.0	4515.00	16.0-17.0	9.00	4.898	5.00	2.586	2.50	0.000	6.80	0.000
17.0	4329.15	17.0-18.0	0.00	0.000	6.50	4.086	3.50	0.032	0.00	0.000
18.0	4263.23	18.0-19.0	10.00	5.898	1.25	0.000	0.00	0.000	0.75	0.000
19.0	4056.60	19.0-20.0	1.00	0.000	6.50	4.086	0.00	0.000	0.35	0.000
20.0	3930.72	20.0-21.0	0.50	0.000	0.25	0.000	0.00	0.000	0.10	0.000
21.0	3463.08									
22.0	2875.39									
23.0	2001.16									
24.0	1067.13									
25.0	709.89									
				Ø =4.102 (mm/HR)		Ø =2.414 (mm/HR)		Ø =3.468 (mm/HR)		Ø =1.426 (mm/HR)

(ii) STORM EVENT DATED: 10.8.1984

1	2	3	4	5	6	7	8	9	10	11
0.0	59.76	0.0-1.0	0.50	0.00	4.75	2.28	0.75	0.000	2.50	0.000
1.0	55.30	1.0-2.0	1.25	0.00	3.00	0.53	0.50	0.000	1.50	0.000
2.0	57.95	2.0-3.0	7.00	2.61	6.00	3.53	3.50	0.000	10.00	3.885
3.0	59.76	3.0-4.0	10.00	5.61	6.25	3.78	6.50	2.525	20.00	13.885
4.0	70.35	4.0-5.0	4.00	0.00	8.55	6.08	5.00	1.025	30.00	23.885
5.0	118.44	5.0-6.0	13.50	9.11	2.70	0.23	10.50	6.525	30.00	23.885
6.0	160.13	6.0-7.0	13.50	9.11	10.50	8.03	15.50	11.525	20.00	13.885
7.0	232.69	7.0-8.0	10.25	5.86	8.50	6.03	11.50	7.525	0.00	0.000
8.0	705.61	8.0-9.0	20.25	15.86	9.00	6.53	18.50	14.525	0.00	0.000
9.0	991.55	9.0-10.0	8.25	3.86	7.00	4.53	19.00	14.525	10.00	3.885
10.0	1610.65	10.0-11.0	13.50	9.11	7.50	5.03	2.50	15.025	0.00	0.000
11.0	2075.62	11.0-12.0	22.00	17.61	12.50	10.03	0.00	0.000	0.00	0.000
12.0	2092.39	12.0-13.0	12.00	7.61	1.50	0.00	3.00	0.000	0.00	0.000
13.0	1960.50	13.0-14.0	2.50	0.00	3.00	0.53	13.75	0.000	0.00	0.000
14.0	1742.44	14.0-15.0	0.25	0.00	1.75	0.00	0.00	1.775	0.00	0.000
15.0	1683.08	15.0-16.0	0.00	0.00	0.40	0.00	0.00	0.060	0.25	0.000
16.0	1757.51	16.0-17.0	1.75	0.00	0.00	0.00	4.00	0.000	0.25	0.000
17.0	1471.70	17.0-18.0	0.75	0.00	4.35	1.88	2.75	0.025	6.75	0.635
18.0	1111.89	18.0-19.0	0.00	0.00	6.75	4.28	0.75	0.000	13.00	6.885
19.0	939.62	19.0-20.0	0.75	0.00	0.00	0.00	0.00	0.000	6.50	0.385
20.0	623.46									
21.0	383.80			φ =4.39 (mm/HR)		φ =2.47 (mm/HR)		φ =3.975 (mm/HR)		φ =6.115 (mm/HR)
22.0	75.53									
23.0	80.26									

(iii) STORM EVENT DATED: 31.7.1985

1	2	3	4	5	6	7	8	9	10	11
0.0	7.56	0.0-1.0	1.00	0.00	9.50	0.000	14.00	0.00	5.00	0.00
1.0	7.81	1.0-2.0	19.50	7.13	10.00	0.000	38.50	18.37	20.30	8.35
2.0	8.07	2.0-3.0	18.00	5.63	3.00	0.000	2.50	0.00	9.25	0.00
3.0	8.07	3.0-4.0	2.25	0.00	6.00	0.000	9.00	0.00	6.50	0.00
4.0	8.50	4.0-5.0	7.25	0.00	10.00	0.000	20.00	0.00	12.50	0.55
5.0	8.88	5.0-6.0	18.00	5.63	3.00	0.000	10.00	0.00	18.00	8.05
6.0	10.34	6.0-7.0	9.50	0.00	3.00	0.000	5.00	0.00	9.50	0.00
7.0	23.75	7.0-8.0	4.50	0.00	5.00	0.000	9.00	0.00	9.50	0.00
8.0	31.02	8.0-9.0	20.00	7.63	6.50	0.000	13.00	0.00	33.50	21.55
9.0	59.76	9.0-10.0	21.00	8.63	12.00	0.793	53.00	32.87	16.00	4.05
10.0	60.68	10.0-11.0	13.00	0.63	30.00	18.793	15.00	0.00	8.00	0.00
11.0	62.54	11.0-12.0	1.50	0.00	25.50	14.293	7.00	0.00	6.00	0.00
12.0	1157.80	12.0-13.0	2.50	0.00	20.50	9.293	1.25	0.00	6.50	0.00
13.0	1169.48	13.0-14.0	1.00	0.00	8.00	0.000	1.00	0.00	0.00	0.00
14.0	1204.90	14.0-15.0	0.75	0.00	3.00	0.000	1.50	0.00	0.00	0.00
15.0	1234.93	15.0-16.0	0.00	0.00	11.00	0.000	0.25	0.00	0.00	0.00
16.0	1392.06	16.0-17.0	0.00	0.00	5.00	0.000	0.00	0.00	0.00	0.00
17.0	1253.16									
18.0	1045.17			ϕ =12.37		ϕ =11.207		ϕ =20.13		ϕ =11.954
19.0	817.39			(mm/HR)		(mm/HR)		(mm/HR)		(mm/HR)
20.0	603.90									
21.0	413.68									
22.0	226.34									
23.0	1487.57									
24.0	111.57									
25.0	99.83									

(iv) STORM EVENT DATED: 15.8.1986

1	2	3	4	5	6	7	8	9	10	11
0.0	42.33	0.0-1.0	1.25	0.000	0.00	0.000	0.50	0.000	1.00	0.000
1.0	41.80	1.0-2.0	6.75	0.000	1.75	0.000	0.75	0.000	3.00	0.000
2.0	41.55	2.0-3.0	17.50	7.902	5.00	0.000	20.50	8.567	19.25	2.545
3.0	41.20	3.0-4.0	19.25	9.652	12.50	0.000	12.50	0.567	25.25	8.545
4.0	40.78	4.0-5.0	3.75	0.000	20.00	4.755	4.00	0.000	15.50	0.000
5.0	40.00	5.0-6.0	1.50	0.000	3.75	0.000	7.00	0.000	2.50	0.000
6.0	40.00	6.0-7.0	7.25	0.000	2.00	0.000	5.00	0.000	14.50	0.000
7.0	40.02	7.0-8.0	12.75	3.152	10.25	0.000	5.50	0.000	7.00	0.000
8.0	97.52	8.0-9.0	20.00	10.402	14.00	0.000	22.50	10.567	11.50	0.000
9.0	210.00	9.0-10.0	10.00	0.402	13.50	0.000	15.00	3.067	34.00	17.295
10.0	340.10	10.0-11.0	15.00	5.402	36.50	21.255	25.00	13.067	47.00	30.295
11.0	715.64	11.0-12.0	15.00	5.402	40.00	24.755	16.50	4.567	10.75	0.000
12.0	869.20	12.0-13.0	6.95	0.000	16.50	1.255	0.25	0.000	7.75	0.000
13.0	1130.00	13.0-14.0	0.00	0.000	0.00	0.000	0.00	0.000	0.25	0.000
14.0	1333.00	14.0-15.0	0.75	0.000	0.00	0.000	0.00	0.000	0.00	0.000
15.0	1313.00	15.0-16.0	0.50	0.000	0.00	0.000	0.00	0.000	0.00	0.000
16.0	1294.06	16.0-17.0	0.00	0.000	0.00	0.000	0.25	0.000	0.00	0.000
17.0	1255.65	17.0-18.0	1.00	0.000	0.00	0.000	0.50	0.000	0.25	0.000
18.0	1217.78	18.0-19.0	3.75	0.000	0.00	0.000	0.25	0.000	0.25	0.000
19.0	1184.17	19.0-20.0	0.00	0.000	0.00	0.000	0.50	0.000	3.50	0.000
20.0	1036.63									
21.0	499.27									
22.0	226.64			Ø =9.598		Ø =15.245		Ø =11.933		Ø =16.705
23.0	208.00			(mm/HR)		(mm/HR)		(mm/HR)		(mm/HR)
24.0	193.63									
25.0	182.00									
26.0	178.04									

(v) STORM EVENT DATED: 27.8.1987

1	2	3	4	5	6	7	8	9	10	11
0.0	60.60	0.0-1.0	0.50	0.00	3.50	0.523	0.50	0.00	0.25	0.000
1.0	60.90	1.0-2.0	0.00	0.00	4.00	1.023	8.50	2.73	0.25	0.000
2.0	61.00	2.0-3.0	0.00	0.00	4.00	1.023	5.00	0.00	1.25	0.000
3.0	61.11	3.0-4.0	1.00	0.00	8.00	5.023	7.00	1.23	0.50	0.000
4.0	160.00	4.0-5.0	1.50	0.00	4.00	1.023	10.00	4.23	5.00	0.000
5.0	300.00	5.0-6.0	5.00	0.00	1.00	0.000	15.00	9.23	4.00	0.000
6.0	440.00	6.0-7.0	2.00	0.00	5.00	2.023	30.00	24.23	5.50	0.000
7.0	580.00	7.0-8.0	7.00	1.96	24.00	21.023	30.00	24.23	4.40	0.000
8.0	740.00	8.0-9.0	3.00	0.00	0.50	0.000	15.00	9.23	24.50	16.992
9.0	900.00	9.0-10.0	10.00	4.96	0.00	0.000	10.00	4.23	19.50	11.992
10.0	1125.50	10.0-11.0	20.00	14.96	0.50	0.000	7.00	1.23	21.00	13.492
11.0	1663.95	11.0-12.0	30.00	24.96	0.50	0.000	12.00	6.23	35.00	27.492
12.0	1934.50	12.0-13.0	13.50	8.46	1.00	0.000	10.50	4.73	23.00	15.492
13.0	2028.98	13.0-14.0	6.00	0.96	0.00	0.000	11.50	5.73	7.00	0.000
14.0	1981.00									
15.0	1663.95			Ø =5.04		Ø =2.977		Ø =5.77		Ø =7.508
16.0	1036.65									
17.0	745.24			(mm/HR)		(mm/HR)		(mm/HR)		(mm/HR)
18.0	361.09									
19.0	229.90									
20.0	244.06									
21.0	220.00									
22.0	210.00									
23.0	200.00									
24.0	200.00									
25.0	198.00									

(vi)

Average Weighted Rainfall (RF) and Rainfall Excess (RE) for Kolar Watershed

(All units in mm)

Storm dated	28.8.83		10.8.84		31.7.85		15.8.86		27.8.87	
Time (HR)	RF	RE	RF	RE	RF	RE	RF	RE	RF	RE
0-1	2.27	.00	1.36	.00	.56	.00	7.93	.00	1.63	.00
1-2	1.72	.00	2.40	.00	4.88	.00	21.07	10.76	3.83	.34
2-3	5.81	2.84	4.00	1.20	13.65	2.70	7.71	.00	2.93	.00
3-4	8.09	5.12	23.21	20.41	15.14	4.19	5.69	.00	5.26	1.77
4-5	7.19	4.22	26.76	23.95	10.52	.00	11.98	1.67	4.91	1.42
5-6	9.35	6.38	19.22	16.41	3.89	.00	10.15	.00	6.14	2.66
6-7	13.20	10.23	22.90	20.09	4.98	.00	5.82	.00	10.88	7.39
7-8	9.45	6.48	19.24	16.44	9.58	.00	6.14	.00	19.53	16.05
8-9	14.58	11.61	19.95	17.14	18.01	7.06	13.60	3.29	6.29	2.80
9-10	10.63	7.66	21.40	18.59	13.78	2.84	26.02	15.70	6.64	3.16
10-11	12.09	9.12	18.92	16.11	27.33	16.38	19.76	9.44	9.13	5.65
11-12	12.11	9.14	28.09	25.29	24.69	13.75	12.30	1.98	14.17	10.68
12-13	4.22	1.25	28.45	25.64	8.80	.00	9.17	.00	8.38	4.90
13-14	2.71	.00	24.74	21.93	.01	.00	3.61	.00	5.26	1.77
14-15	4.46	1.49	13.51	10.70	.23	.00	1.77	.00		
15-16	.16	.00	7.17	4.36	.15	.00	4.24	.00		
16-17	.54	.00	5.62	2.82	.07	.00	1.90	.00		
17-18	3.27	.30	3.41	.61	.45	.00				
18-19	3.90	.93	3.55	.74	1.22	.00				
19-20	.73	.00	2.79	.00	.30	.00				
20-21			.25	.00						

(RE = RAINFALL EXCESS, mm/hour)

APPENDIX-II-D

BARAKAR RIVER WATERSHED (INDIA)
RIVER DISCHARGE DATA

(i) STORM EVENT DATED: 25.9.1975

TIME (HR)	DISCHARGE (cumec)	
	BARKISURIYA	NANDADHI
1	2	3
0.0	97.50	91.60
3.0	137.10	82.80
6.0	131.70	77.80
9.0	118.80	81.10
12.0	137.10	88.00
15.0	148.20	180.30
18.0	145.40	209.30
21.0	170.00	230.00
24.0	181.50	268.40
27.0	157.90	905.60
30.0	1335.20	1176.50
33.0	980.00	2550.00
36.0	728.80	3234.10
39.0	619.30	2697.50
42.0	700.60	1953.70
45.0	690.00	1680.00
48.0	580.80	1462.00
51.0	525.90	1212.40
54.0	471.00	1047.90
57.0	425.20	817.80
60.0	365.00	773.50
63.0	305.50	658.60
66.0	268.00	592.60
69.0	240.00	518.00
72.0	201.40	460.00
75.0	175.20	410.80
78.0	162.90	361.90

(ii) STORM EVENT DATED: 16.10.1985

1	2	3
0.0	70.90	90.00
3.0	70.90	112.80
6.0	80.50	161.50
9.0	102.01	221.50
12.0	137.04	383.30
15.0	248.10	541.30
18.0	671.60	761.50
21.0	1047.10	831.10
24.0	1404.20	1303.40
27.0	1156.30	2203.70
30.0	977.80	1553.50
33.0	699.30	1312.80
36.0	699.30	1063.20
39.0	579.90	966.10
42.0	484.50	927.30
45.0	419.40	860.00
48.0	350.20	838.20
51.0	279.90	695.00
54.0	240.50	645.60
57.0	184.60	597.80
60.0	178.20	509.10
63.0	165.80	434.60
66.0	148.10	383.30

(iii) STORM EVENT DATED: 27.9.1989

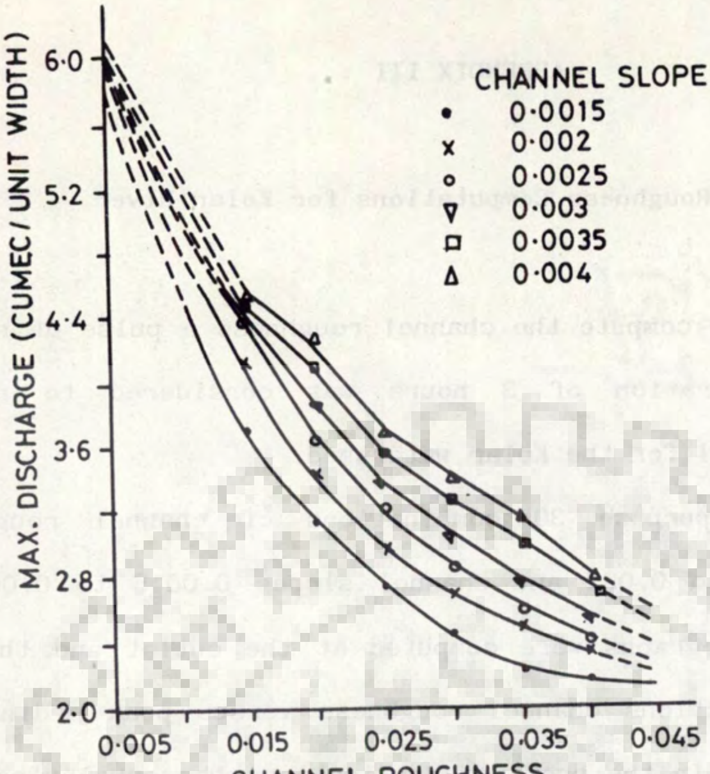
1	2	3
0.0	17.06	113.99
3.0	18.73	117.90
6.0	35.46	201.63
9.0	43.37	201.63
12.0	76.64	201.63
15.0	84.63	207.35
18.0	10.65	247.18
21.0	159.80	457.45
24.0	255.89	670.11
27.0	331.83	924.64
30.0	440.54	1126.00
33.0	519.15	1171.35
36.0	429.91	1364.93
39.0	388.91	1038.87
42.0	340.98	841.02
45.0	305.25	676.46
48.0	279.94	597.06
51.0	255.89	651.32
54.0	225.75	626.81
57.0	197.79	585.43
60.0	178.21	689.26
63.0	159.79	721.99
66.0	148.17	702.24
69.0	129.03	608.85
72.0	116.29	508.22
75.0	97.49	447.72
78.0	84.63	392.25

APPENDIX III

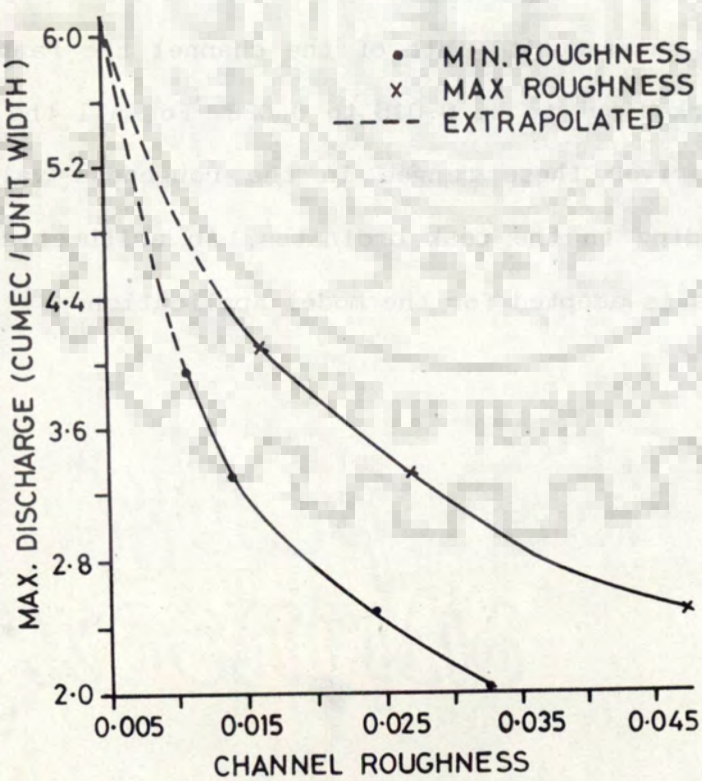
Channel Roughness Computations for Kolar River

In order to compute the channel roughness a pulse of rainfall excess having a duration of 3 hours was considered to run the Physiographic Model-II for the Kolar watershed.

For this purpose 30 combinations of channel roughnesses ranging from 0.015 to 0.035 and channel slopes 0.0015 to 0.004 were considered. The hydrographs were computed at the outlet and the plots were drawn with functions channel roughness versus peak ordinates as shown in figure A-i. In the proposed applications the enveloping curves given in Figure A-ii are used and corresponding to observed peak values the range of roughness is arrived at. For example for a Qmax corresponding to 3.2 cumecs/unit width of the channel the range of the channel roughness worked out to be 0.015 to 0.028. For all the observed flood of the Kolar river these ranges in the roughness values were determined corresponding to the peak ordinates. An arithmetic mean of all these roughnesses is adopted for the model application.



(i) RELATIONSHIP FOR DIFFERENT CHANNEL SLOPES



(ii) ENVELOPING CURVES

FIG. A-PEAK DISCHARGE Vs CHANNEL ROUGHNESS FOR KOLAR RIVER

APPENDIX IV

Model efficiencies for the Four Watersheds Under Study

Watershed	Storm Event	Efficiency of Physiographic Model		
		I	II	Open Book Type
Kolar River	(1) 28.8.83	94.60	95.40	93.80
	(2) 10.8.84	89.70	89.90	90.50
	(3) 31.7.85	77.14	73.80	61.00
	(4) 15.8.86	71.90	76.40	75.4
	(5) 27.8.87	60.52	61.10	73.6
Railway Bridge No. 719	(1) 24-25.7.64	29.80	87.60	76.45
	(2) 26-27.7.64	70.00	99.76	68.09
	(3) 3.9.64	75.50	85.35	_
	(4) 11.8.65	75.16	70.94	80.61
	(5) 16.9.66	70.50	73.89	80.43
	(6) 19.9.66	92.11	76.15	28.57
Kassilian River	(1) 21.1.79	86.70	90.69	81.91
	(2) 24.2.79	95.00	82.42	81.72
	(3) 23.5.79	30.36	37.39	89.28
	(4) 2.6.79	-	1.00	33.98
Barakar river	(1) 25-29.7.75	-	82.30	-
	(2) 16-18.10.85	-	74.89	-
	(3) 27-30.9.89	-	75.64	-

APPENDIX V

The computed Values of Overland Flow Depth, Velocity, F_r and KW Number

(a) Railway Bridge No. 719 (storm dated 11.8.1965, at the end of 2nd time interval)

Sl. No.	subwatershed	Overland flow depth (m)	Velocity (m/sec)	Froude number	Kinematic Wave Number $*10^4$
1	T1	0.0199	0.0080	0.0019	176543
2	T2	0.0179	0.0009	0.0022	375123
3	T3	0.0180	0.0007	0.0016	66558
4	M1	0.0185	0.0007	0.0017	169555
5	M2	0.0138	0.0004	0.0012	337956
6	M3	0.0197	0.0009	0.0021	117328
7	M4	0.0164	0.0006	0.0006	240884
8	M5	0.0156	0.0007	0.0019	28378

(b) Tributary Subwatersheds of Kolar River (storm dated 10.8.1984, at the end 5th time interval)

Sl. No.	subwatershed	Overland flow depth (m)	Velocity (m/sec)	Froude number	Kinematic Wave Number
1	T1	0.0073	0.0004	0.0014	830635
2	T2	0.0072	0.0003	0.0012	736443
3	T3	0.0073	0.0003	0.0011	418683
4	T4	0.0073	0.0010	0.0036	834730
5	T5	0.0079	0.0004	0.0014	375613
6	T6	0.0088	0.0003	0.0012	452852
7	T7	0.0070	0.0003	0.0010	1134726
8	T8	0.0074	0.0003	0.0009	718339
9	T9	0.0049	0.0002	0.0009	2601028
10	T10	0.0022	0.0001	0.0005	3842905
11	T11	0.0022	0.0001	0.0005	43616820
12	T12	0.0022	0.0001	0.0005	41469150

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