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CATCHMENT ROUTING USING KINEMATIC WAVE APPROACH

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PREFACE

The second basic component in stormwater models is the surface runoff models, i.e. overland flow and open channel flow. The approach usually taken is a simplified one-dimensional flow approximation. The main problem associated with deterministic surface runoff modelling is the difficulty associated with the solving the equations of motion. However, in the last decade there have been significant advances in the science of surface water hydraulics which has resulted in the development of a substantial simplification of the flow equations. This simplification is called the kinematic wave approximation and it is now clearly established that the approximation can be made under almost all conditions of overland flow and for many conditions associated with stormwater flows in open channels.

Using the kinematic wave approach, a model has been developed for the upper Ramganga basin, falling in Himalayan region. Observed data has been made use of in calibration of the model and many derived/observed flood events occurred at the outlet of the basin that is Kalagarh, have been simulated using the model. a sensitivity analysis of the model parameters/input has also been made to study the behaviour of change in parameter/input values on the response of the system.

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(S M Seth)
DIRECTOR

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ABSTRACT

Estimation of flood peak magnitude and their time of occurrence still remains a concern to the flood hydrologists. Various theories dealing with the catchment hydrology have come up with the time. Many of the distributed models like SHE (System Hydrologic European) model, HEC-1 etc. which are highly sophisticated in nature are available in literature. But the quantum of data and expertise required to handle such models, to a great extent, restrict their use in field.

The present study is concentrated to the simulation of major flood events observed at Kalagarh on Ramganga river in Himalaya region. The well established kinematic wave theory which is applicable to steep/hilly regions has been applied. Major observed floods including 50, 100, 500 and 1000 yrs. and probable maximum flood have been simulated using this approach. The results fairly match with these observed /derived flood events. The sensitivity of the model parameters/inputs indicate that the celerity in the channel plays most significant role in the establishment of peak flood magnitudes at the outlet (at Kalagarh) of the upper Ramganga basin.

1.0 INTRODUCTION

Engineers and soil conservationists often need to estimate runoff rates and volumes from ungauged watersheds. Traditional formula methods are useful for some some purpose but for many objectives more precise knowledge is required about the hydrologic response of a watershed or model sensitivity to various physical factors or assumptions. For this purpose physically based distributed models are becoming more widely used.

Catchment routing refers to the calculation of flows in time and space within a catchment. The objective of catchment routing is to transform effective rainfall into stream flow. This is accomplished either in a lumped mode (e.g. time-area method) or in a distributed mode (e.g. kinematic wave method). Methods for catchment routing are of two types: (1) hydrologic and (2) hydraulic. Hydrologic methods are based on the storage concept and are spatially lumped to provide a runoff hydrograph at the catchment outlet. Hydraulic methods use kinematic or diffusion waves to simulate surface runoff within a catchment in a distributed context. Unlike hydrologic methods, hydraulic methods can provide runoff hydrographs inside the catchment.

The concepts of translation and storage are central to the study of flow routing, whether in catchments, reservoirs, or in stream channels. They are particularly important in catchment routing because they can be studied separately, unlike in reservoir and channel routing. Translation may be interpreted as the movement of water in a direction parallel to the channel bottom while the storage in a direction perpendicular to the channel bottom. Translation is synonymous with runoff concentration and storage with runoff diffusion. In reservoir routing, storage is the primary mechanism, with translation almost non-existent. In stream channel routing, the situation is reversed, with translation being predominant mechanism and storage playing only a minor role. This is the reason why kinematic and diffusion waves are useful models for stream channel routing. In catchment routing, translation and storage are about equally important and, therefore, they are often accounted for separately. The translation effect can be related to runoff concentration whereas the storage effect can be simulated with linear reservoirs.

Hydraulic catchment routing using kinematic waves was introduced by Wooding (1965). Since then, the kinematic wave approach has been widely used in deterministic modelling. This approach can be lumped or distributed, depending on whether the parameters are kept constant or allowed to vary in space. Analytical solutions are suited to lumped modelling, whereas numerical solutions are more appropriate for distributed modelling. In this report the kinematic wave approach is applied to the upper Ramganga catchment in Himalayan region.

2.0 REVIEW

Whenever and wherever the rate of rainfall exceeds the infiltration rate at the surface, the excess water begins to accumulate in static surface storage. the capacity of this storage is governed by the extent to which geometrical surface irregularities and surface tension can develop forces to balance the increasing gravitational forces. When the local static storage capacity is exceeded, surface runoff begins as a thin sheet flow. Surface irregularities cause a focusing of the local gravitational potential gradients and a component gathering the runoff into discrete stream channels. These channels form a tree like network which ensures that the flow immediately below each confluence exceeds that in either of the merging branches. It is obvious that there exists a whole spectrum of channel geometries and flow type. Eagleson(1970) lists the extremes to be examined for the analytical solution under each circumstances:

1. At one extreme lies the thin sheet flow called overland flow. it is likely to be the primary flow type in surface runoff from very small natural areas having little topographic relief.

2. The next distinctive flow type is found in the smallest stream channels, which gather the overland flow in a continuous fashion along their length to form the lowest order of stream flow.

3. As those smallest streams merge with one another, they form streams of higher order which will have concentrated tributary inflows as well as continuous lateral inflow. At the low order end of this range, the continuous lateral inflow is primarily overland flow and is thus only positive. At the higher end, this inflow is primarily from ground water and may be positive or negative. The computation is done through flood routing.

A contemplation of the works carried out on the solution of flow behaviour under the circumstances as explained above, is given below in brief.

Iwagaki(1955) was the first to use the kinematic wave method incorporating a continuous lateral inflow to analyze in detail the hydrographs of overland flow and stream flow. Henderson and Wooding(1964) and Wooding(1965, 1965, 1966) simplified the methods of Lighthil and Whitham(1955) and of Iwagaki(1955) in actual application of the method to natural catchments. Their methodology utilised basically the method of characteristics for the solution of kinematic wave routing equation.

Hager(1984) analysed the effects of local bottom slope and roughness coefficient variations in overland flow over a plane using the kinematic wave theory. He found the effect of three parameters being significant on the resulting surface profile but at the same time these could be ignored regarding the discharge characteristics provided the valleys are not flat in the upper and in the lower zones. In addition, the local effects of roughness coefficient must be accounted for when surfaces are significantly rougher at the upper than at the lower zones of the catchment area.

Abrahams and others (1986) in their works utilised the concept of Horton(1945) by assuming the overland flow as laminar flow. They examined the relation between the Darcy-Weisbach friction factor 'f' and the Reynold's number'Re' for overland flow on the experimental plots in Arizona, USA. They described the relation in terms of the simultaneous operation of two processes. the first being the progressive inundation of roughness elements and increase in their wetted upstream area as discharge decreases. This process causes flow resistance to increase. The second being the progressive increase in depth of flow over already inundated parts as discharge increases. This process causes flow resistance to decline. They indicated its importance in the applications on the mathematical modelling of overland flow on desert hill slopes.

Woolhiser and Liggett(1967) reported an analytical solution of the overland flow problem of the characteristic plane for an initially dry surface, neglecting the momentum of the rain. The energy gradient term was approximated by Chezy formula. They gave a parameter known as kinematic flow number. The work of Henderson and Wooding(1964) is a specific case of the solution given by Woolhiser and Liggett(1967). Almost similar conclusions were arrived at by Schreiber and Bender (1972) and Shen and Li (1973) who studied the effect of rainfall on the flow behaviour on very short planes.

Overton (1970) found an explicit mathematical relation from the kinematic equation between hydrologic lag time to equilibrium. Further, he found that the shape of the dimensionless rising hydrographs were very sensitive to small changes in the choice of time of equilibrium.

Hager and Hager (1985) compared the three techniques namely dynamical, zero inertia and the kinematic wave approximations for pseudo-steady flow conditions. They showed that the kinematic wave approach is well suited for overland runoff process. The zero inertia approach may be regarded as an intermediate formulation which retains the effect of surface slope but neglects dynamical flow properties.

The solution of kinematic wave routing equation, is dealt by analytical and numerical methods. A thorough examination of the method of characteristics falling under the fist category of methods, has been made by Woolhiser and Liggett(1967).

There are two types of numerical methods or finite difference techniques for solving the shallow water equations or the characteristic equations. They are the explicit and implicit schemes. Miller has reported a complete treatise on the subject and the work of Amein and Fang(1970) provides notable computational examples.

3.0 METHODOLOGY

The mathematical modelling of catchment is well established in hydrologic research and practice. Among the various aspects of catchment modelling, the conversion of rainfall into runoff, the one that perhaps received the most attention. Procedures to accomplish this invariably resort to kinematic wave theory. With the steep slopes usually converted in upland watersheds, flow conditions are such that kinematic waves are reasonably good approximations of the unsteady flow phenomena. Exceptions are cases involving mild slopes coupled with fast rising hydrographs, for which diffusion and dynamic waves are better representations than kinematic waves.

Procedures for kinematic wave computations are either analytical or numerical. Analytical solutions provide answers for a simplified class of problems while problems of a more general type are handled with numerical solutions discretizing the solution domain. Numerical solutions of kinematic waves are known to introduce variable amounts of numerical diffusion with the solution resembling a diffusion wave rather than a kinematic wave. In scheme first order, the numerical diffusion is uncontrolled resulting in catchment response being dependant on grid size. In schemes of second order, the numerical diffusion disappears, but a certain amount of numerical dispersion (of the third order) still remains(Ponce and others, 1979).

A widely accepted formulation for catchment modelling is the selection of the kinematic wave equation together with a simplified spatial representation of the catchment in the form of an open book. Such a configuration consists of two planes adjacent to one channel (Fig. 1)

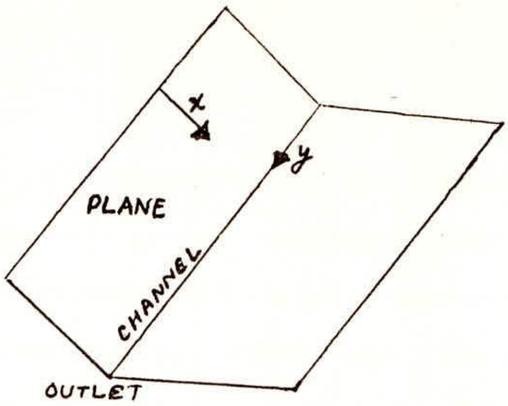

Fig.1- Open Book Catchment Schematization

Rainfall is the inflow to the plane and the runoff from the planes is by overland flow in the direction perpendicular to the channel alignment. Inflow to the channel is by the lateral contribution from the planes and the outflow from the channel is the catchment response.

A source of uncertainty in this type of modelling is the adequacy of the frictional representation. Flow over the planes is usually of such small depths that laminar flow may prevail. In contrast, flow in the channel is likely to be of a turbulent nature. yet in certain cases, especially near the downstream end of the planes the flow may be in the transitional regime. For routine applications, frictional coefficients are generally estimated in such a way that they account for laminar flow in the overland flow in the overland flow planes (HEC,1985).

To derive the kinematic wave equations, the usual statement of conservation of mass in a control volume is coupled with a simplified form of conservation of momentum which accounts for frictional and body forces. The statement of conservation of mass is:

$$\frac{\delta Q}{\delta x} - \frac{\delta A}{\delta t} = q_L \qquad (1)$$

where,

Q = flow rate (cumecs)

A = flow area (sq. m)

q_L = lateral inflow (cumec/m) of channel length.

The simplified momentum statement is $S_0=S_f$ where, S_0 is the bed slope and S_f is the friction slope. This is in fact a statement of uniform flow momentum statement:

$$Q = \alpha A^{\beta} \qquad (2)$$

where, α and β are coefficient and exponent, respectively. The values of α and β contain information on frictional and cross-sectional characteristics (for instance, $\beta=3$ for a wide channel with laminar friction; $\beta=5/3$ for a wide channel governed by Manning's friction; $\beta=3/2$ for a wide channel with Chezy friction; and $\beta=4/3$ for a triangular channel with Manning's friction.

The speed of propagation of kinematic waves is obtained from Eq. 2 :

$$c = \left.\frac{\delta Q}{\delta A}\right|_{x_0} = \beta \frac{Q}{A} = \beta . v \qquad (3)$$

where, c is the celerity of kinematic wave; v is the mean flow velocity.

The kinematic wave equation is obtained by multiplying Eqs. 1 and 3 and making use of chain rule to obtain:

$$\frac{\delta Q}{\delta t} + c \frac{\delta Q}{\delta x} = c\, q_L \qquad (4)$$

Eq. 4 contains convection terms on the left hand side and a source term on the right hand side. Significantly, Eq. 4 does not contain a diffusion term. This equation may be solved by analytical or numerical means. A typical class of numerical solutions is that in which the values of $\delta Q/\delta t$, $\delta Q/\delta x$, c and q_L are expressed in terms of some or all of four discrete adjacent values of Q, c and q_L in space and time (Fig. 2). Alteratively, the value of c could be kept constant, leading to the linear kinematic wave solution.

SOLUTION SCHEMES

A numerical solution of Eq. 4 needs that the spatial and temporal derivatives are perfectly centered in the rectangular grid (Fig. 2). Otherwise, a certain amount of diffusion will be generated in the numerical solution. However, a small amount of numerical dispersion will still remain (Ponce and others, 1979). The dispersion is most often simply a nuisance, but in certain cases it may be of such magnitude as to invalidate the whole solution.

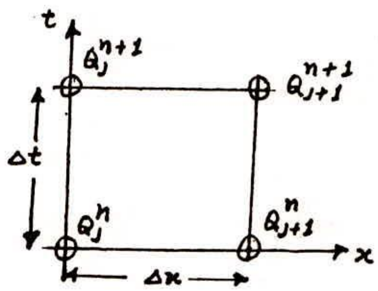

Fig. 2 Spatial and Temporal Discretization

Ponce and others(1979) have shown that of the three possible fully-centered schemes (Fig. 3), scheme I and II are conditionally stable and convergent to the analytical solution as the Courant number (defined as the ratio of physical celerity c to the grid celerity $\Delta x/\Delta t$) approaches 1.0. Scheme I is stable for Courant number less than or equal to 1.0, amounts to 'upward differencing'. Scheme II is stable for Courant Nos. greater than or equal to 1.0. This is exactly the opposite image of Scheme I. Scheme III is unconditionally stable but it is non-convergent for any value of Courant number.

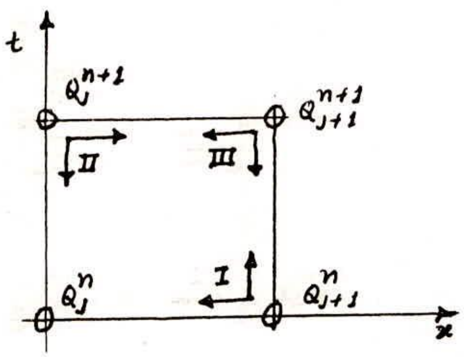

Fig. 3 Three Fully Off-Centered Numerical Schemes

The solution of Eq. 4 by different schemes result in the following:

Scheme I

$$\frac{Q_{j+1}^{n+1} - Q_{j+1}^{n}}{\Delta t} + \bar{c}\,\frac{Q_{j+1}^{n} - Q_{j}^{n}}{\Delta x} = \bar{c}\,\bar{q}_L \qquad (5)$$

Scheme II

$$\frac{Q_{j}^{n+1} - Q_{j}^{n}}{\Delta t} + \bar{c}\,\frac{Q_{j+1}^{n+1} - Q_{j}^{n+1}}{\Delta x} = \bar{c}\,\bar{q}_L \qquad (6)$$

Scheme III

$$\frac{Q_{j+1}^{n+1} - Q_{j+1}^{n}}{\Delta t} + \bar{c}\,\frac{Q_{j+1}^{n+1} - Q_{j}^{n+1}}{\Delta x} = \bar{c}\,\bar{q}_L \qquad (7)$$

Routing Procedure

Routing is basically needed for two segments (1) plane and (2) channel. Referring to open book concept (Fig. 1), the routing on planes is converted out using Scheme I (Eq. 5). Application of Eq. 5 in linear mode result in the following form:

$$Q_{j+1}^{n+1} = C_1 Q_j^n + C_2 Q_{J+1}^n + C_3 Q_L \qquad (8)$$

where, C1=C, C2=1-C and C3=C, with C being the Courant Number ($C=\beta v_p \frac{\Delta t}{\Delta x}$). Here, Δx is referred for plane and v_p is the average velocity of flow on the plane. The term Q_L is the lateral inflow (m^3/s). For routing in the plane, the lateral inflow is equal to the effective rainfall (cm/hr) times the applicable area. (sq. m).

For channel routing, Scheme II is adopted. Linear transformation of Eq. 6 yields:

$$Q_{j+1}^{n+1} = C_0 Q_j^{n+1} + C_1 Q_J^n + C_3 Q_L \qquad (9)$$

where, $C_0=(c-1)/C$; $C_1=1/C$; and $C_3=1$. Here, C is the Courant No. $C=\beta v_c \frac{\Delta t}{\Delta y}$; v_c is the average velocity of flow in the channel and Δy is the length of the channel. The term Q_L in channel routing is the average uniformly distributed lateral inflow ($m^3/s/m$) multiplied by the channel length (m).

Catchment Routing

The routing procedure for overland flow as depicted by Eqs. 8 and 9, has been adopted as the basis of catchment routing. A watershed can be presented in the simplified form as in Fig. 4. L_1, L_2, L_3 etc. are the lengths of the channels of sub-catchments.

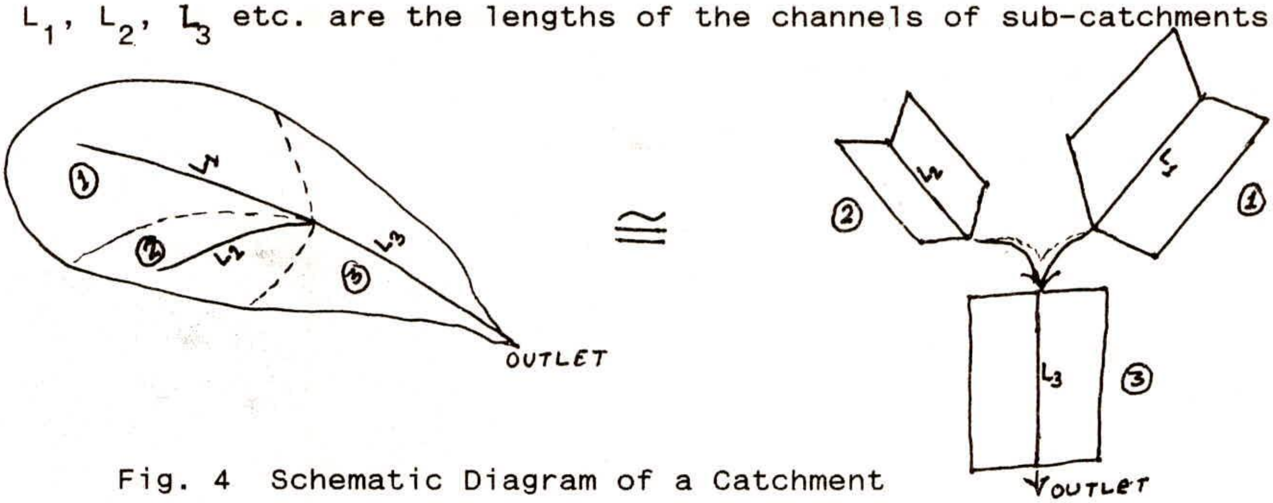

Fig. 4 Schematic Diagram of a Catchment

The resulting outflow from sub-catchments (1) and (2) are joined together. The combined flow at the moment acts as inflow to (3). The simplified structure can be extended to the user requirement for representing the whole catchment. It is worth mentioning that the computation time increases with the increase in segments.

Assessment of Kinematic Routing Technique:

The kinematic wave model provides translation and diffusion, the latter, however, due only to the finite grid size. The method can be linear or non-linear and lumped or distributed, depending on numerical scheme and input data. The method is applicable to small catchments with steep slopes where diffusion is small and can be controlled by grid refinement. Theoretically, the method could also be applicable to midsize catchments, as long as physical diffusion remains small. In practice the larger are the catchments, the more likely it is that the physical diffusion be negligible. The distributed nature of kinematic wave models results in substantial data needs; the use of average parameters would render the model lumped, with the consequent loss of detail. Another important consideration in kinematic wave model is the validity of the geometric configuration

4.0 PROBLEM DEFINITION

The catchment routing can be preformed using time-area method and its improved version the Clarke method, cascade of linear reservoirs (Nash model) and kinematic or diffusion wave methods. The latter methods are distributed in nature whereas the former methods are not. This added advantage has led to the popularity of kinematic and diffusion wave methods. The kinematic wave methods, if solved analytically, does not result in diffusion, thereby resulting higher peaks. However, the diffusion wave which is an improvement over the kinematic wave does offer diffusion. The numerical solution of the kinematic wave offers diffusion but due to the involvement of the errors in the solution and is termed as numerical diffusion. Therefore, the kinematic wave approach has extensively been used in catchment routing, especiallly, to the watersheds which are steep (slope in the range of 0.01-0.001).

The kinematic wave routing techniques has been in use for channel routing since long. The extension of the concept to catchment routing is relatively recent. This technique has been recommended for for the steep, hilly terrain catchments. Therefore, upper Ramganga catchment, in himalayan region, up to Kalagarh where a multi-purpose water resources project is in operation, was selected for the study through kinematic wave approach.

The study is to concentrate on the development of a model, for the Ramganga catchment, to transform rainfall excess into runoff using kinematic wave approach and to study the effect of change in its parameters on the outflow hydrograph, obtained at the outlet of the basin, through sensitivity analysis.

5.0 STUDY AREA

Looking at the applicability of the kinematic routing approach, a basin in Himalayan region is selected for study. The computer model is developed for the upper catchment of Ramganga and has been applied. A brief description of the catchment follows:

Ramganga river is a major tributary of River Ganga. It originates at a place known as Diwali khel in the Himalayas. It emerges out of hills at kalagarh where a dam named Ramganga dam is situated. The total length of the river up to the dam site is approximately 158 Kms. It further continues its journey in the planes for another 370 Kms before joining Ganga at Farrukhabad. During its travel up to Kalagarh, the river is joined by the following main tributaries.

1. Ganges
2. Binoo
3. Khatraun
4. Nair
5. Badangad
6. Mandal
7. Helgad
8. Sona Nadi

The catchment area of the Ramganga up to Ramganga dam is shown in Fig. 5. and is 3134 sq. km in size. The catchment area lies between elevation 262m and 2926 m and is considerably below the perpetual snow line of the Himalayas. Normally, 50% of the drainage basin is covered with forest and 30% is under cultivation on terraced fields.

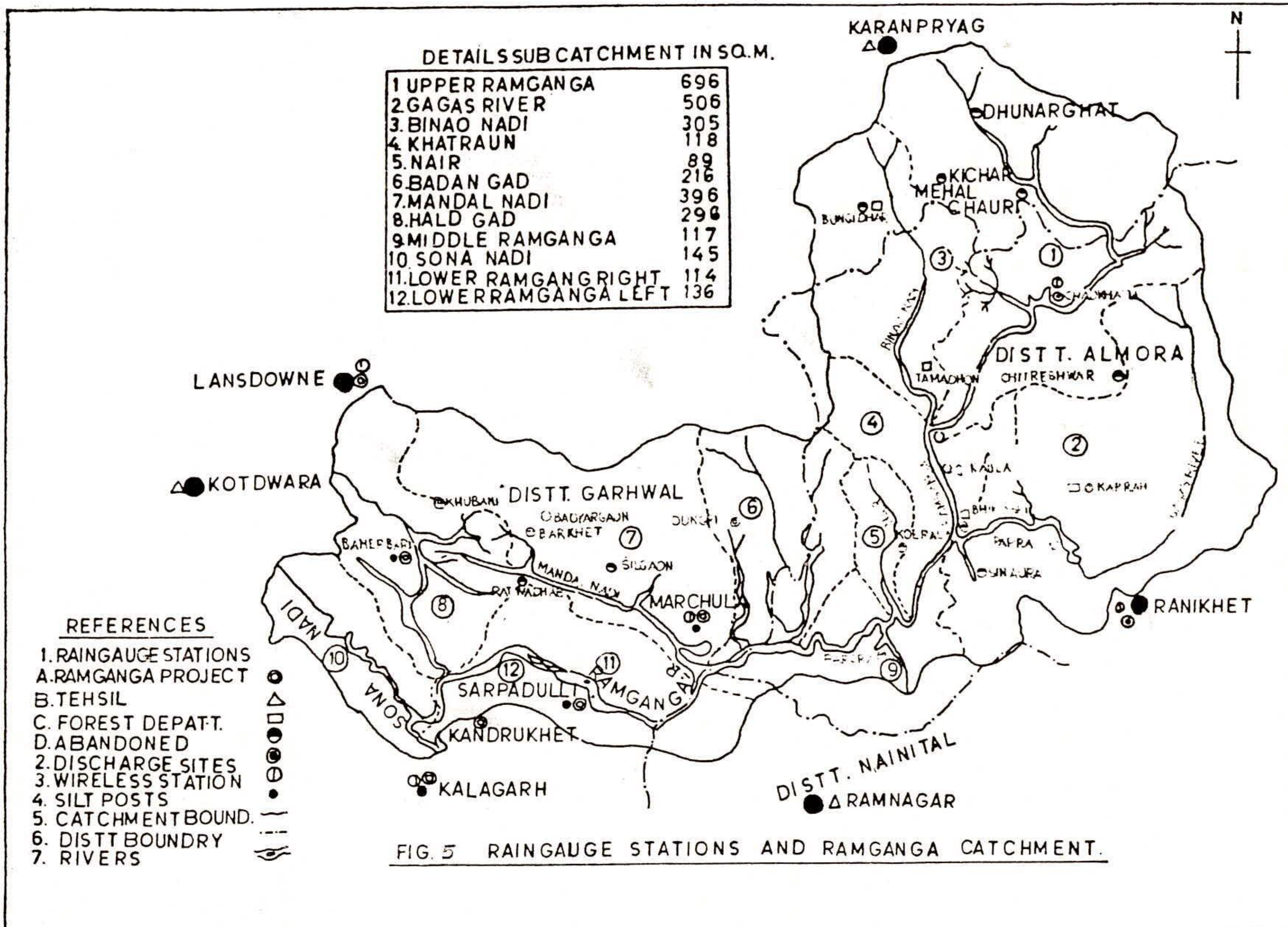

FIG. 5 RAINGAUGE STATIONS AND RAMGANGA CATCHMENT.

Precipitation

Ramganga valley experiences an annual precipitation of 1552mm estimated using the recorded rainfall data from 1955 to 1974. The annual precipitation varies from 654mm to 2436mm.

Runoff

Stream flow records of the Ramganga river including river stages, instantaneous as well as monthly, are available at Kalagarh from the year 1958. Mandi discharge site was established in the year 1958 and functioned up to June 1963. Later on, due to construction of permanent bridge near the site, it was shifted about 910m u/s of the bridge and named as new discharge site which is under operation. From 1968 three more discharge sites were established at Sarpdulli and Marchulla on the main Ramganga river and Baherbari on one of its tributaries. Records from 1960-61 onwards are available. Various high floods were recorded at site e.g. on 16.09.1993, 2607.1966, 08.08.1969, and 02.09.1978. The data has been obtained from various design memos. on Flood Hydrology for the Ramganga River Project, prepared by Central Design Directorate, Irrigation Dept., U.P., Lucknow.

Flood Forecasting Measures

A network of the following wireless stations was established in Ramganga catchment for transmitting the river stage and runoff data at different sites so as to estimate the likely volume and stage of water at Kalagarh dam site.

1. Kalagarh
2. Marchulla
3. Naula
4. Lansdown
5. Chaukhatia
6. Ranikhet

A central control room was established at Kalagarh for receiving message from these stations and transmitting them to the concerned authorities for taking necessary protective measures.

Arrangements has been made with the India Meteorological department for receiving rainfall warnings from their Lucknow and Delhi stations and these were also included in the messages delivered from the control room.

Evaporation

Data on evaporation loss from free water surface is not available. The evaporation loss from Ramganga reservoir during a year, is computed as 0.07097 m-acre-ft (0.00833 m-ha-m) with the help of Rowhers formula and records available.

Schematic Representation of the Catchment

For describing the topology of the basin up to dam site for computer use, the whole catchment is sub-divided in the planes and channels as described through Fig. 4.

6.0 RESULTS AND DISCUSSION

As indicated earlier, the kinematic wave approach can be comfortably used for the hilly, steep terrains and so is with the Ramganga basin which has been sub-divided into 13 units (or open books) for computer application. The open books are assumed to be of the length of the segments (or flow length of the channels/tributaries) and their left and right plane lengths are derived using the physical topography of each segments.

The study has been made three fold: (A) sensitivity analysis of the developed model, (B) the calibration of the model, and (C) simulation of various flood events keeping the sensitivity of the parameters in view.

A. Sensitivity Analysis

Kinematic wave model applied to open book deals with the flow in the planes and channels. The wave celerity of the flow on the planes is very low in comparison to that in the channels due to dominance of roughness on the planes. The flow from the planes is assumed to be uniformly distributed lateral inflow to the channel. The lateral inflow, so joining the reach, contributes to the flow rise along the channel. The spatial distribution of the flow along the channel can be handled by the kinematic wave approach and this is one of the advantages over the storage routing methods. However, it is worth mentioning the diffusion occurring at the d/s end is due to the numerical error of the solution. The number of increments (NDX and NDY) play a great role in deciding the extent of the errors. The following given is the description of input parameters/data and their significance with respect to the response.

The plane celerity and channel celerity of the flow are responsible for the representation of dynamic behaviour of the flow on the planes and in the channels. A parameter named Courant parameter ($c.\Delta t/\Delta s$) where, Δs is Δx or Δy for planes or channels, respectively. This parameter, in turn take care of the dynamics of the flow. The combined response of Δx, Δt and wave celerity is represented by the parameter. The best results are obtained when the number approaches 1.00. However, in the field situations, it is difficult to maintain the relation and therefore, is a possible source of error.

Rainfall intensity and its duration are very much responsible for the outflow peak magnitude and its dispersion. There are basically following parameters responsible for describing flow response:

1. NDX :no. of increments in the planes used for deciding Δx.
2. NDY :no. of increments in the channels used for deciding Δy.
3. Δt :time interval at which computations are desired to be :made.
4. c_p :wave celerity of flow on the planes (m/s)
5. c_c :wave celerity of flow in the channel (m/s)
6. Rainfall Intensity: intensity of the excess rainfall (cm/hr)
7. Time Duration : time duration of the rainfall. If excess rainfall is unity, the duration is for so many hrs. unit hydrograph.

The above seven inputs out of which first five are the parameters. The sensitivity of the model inputs is presented in Table-1. The sensitivity of each parameter/input is described below:

TABLE-1 SENSITIVITY ANALYSIS

PARAMETER	NDX	NDY	NDT	t (min)	c_p (m/s)	c_c (m/s)	VARIATION IN C: Plane	VARIATION IN C: Channel	RAINFALL: Intensity (cm.hr)	RAINFALL: Time Duration (min.)	RAINFALL: Volume (MCM)	OUTFLOW: Peak Discharge (cumecs)	OUTFLOW: t_p (min)	OUTFLOW: t_b (min)	OUTFLOW: Runoff Vol. (MCM)
NDX	1	1	120	60.0	0.25	2.5	0.0254	0.1978	0.5	120.0	31.3	285.67	660	7200	30.5
							1.3657	1.1866							
	2	1	120	60.0	0.25	2.5	0.0507	0.1978	0.5	120.0	31.3	285.14	720	7200	31.0
							2.7314	1.1866							
	5	1	120	60.0	0.25	2.5	0.1268	0.1978	0.5	120.0	31.3	289.67	720	7200	31.3
							6.8285	1.1866							
	10	1	120	60.0	0.25	2.5	0.2537	0.1978	0.5	120.0	31.3	301.16	480	6480	31.3
							13.6571	1.1866							
NDY	1	2	120	60.0	0.25	2.5	0.0254	0.3955	0.5	120.0	31.3	312.25	780	7200	30.5
							1.3657	1.3791							
	1	5	120	60.0	0.25	2.5	0.0254	0.9889	0.5	120.0	31.3	316.72	780	7200	30.5
							1.3657	5.9328							
	1	10	120	60.0	0.25	2.5	0.0254	1.9777	0.5	120.0	31.3	304.61	780	7200	30.5
							1.3657	11.8655							
NDX & NDY	2	2	120	60.0	0.25	2.5	0.0507	0.3955	0.5	120.0	31.3	318.35	780	7200	31.0
							2.7314	2.3731							
NDT & DELTA t	2	2	60	120.0	0.25	2.5	0.1015	0.7911	0.5	120.0	31.3	319.88	840	7200	31.2
							5.4628	4.7462							
	2	2	90	80.0	0.25	2.5	0.0677	0.5274	0.5	120.0	31.3	213.92	800	7200	20.7
							3.6419	3.1641							
	2	2	180	40.0	0.25	2.5	0.0338	0.2637	0.5	120.0	31.3	312.56	800	7200	31.0
							1.8209	1.5821							
	2	2	240	30.0	0.25	2.5	0.0254	0.1978	0.5	120.0	31.3	310.54	780	7200	31.1
							1.3657	1.1866							
RAINFALL INTENSITY & TIME DURATION	2	2	120	60.0	0.25	2.5	0.0507	0.3955	0.25	240.0	31.3	314.09	840	7200	31.0
							2.7314	2.3731							
	2	2	120	60.0	0.25	2.5	0.0507	0.3955	0.125	480.0	31.3	300.17	960	7200	31.0
							2.7314	2.3731							
	2	2	120	60.0	0.25	2.5	0.0507	0.3955	0.0625	960.0	31.3	284.53	1140	7200	30.9
							2.7314	2.3731							
	2	2	120	60.0	0.25	2.5	0.0507	0.3955	1.0000	60.0	31.3	300.69	360	7200	31.0
							2.7314	2.3731							

TABLE-1 CONTD.

PARAMETER	NDX	NDY	NDT	t (min)	c_p (m/s)	c_c (m/s)	VARIATION IN C: Plane	VARIATION IN C: Channel	RAINFALL: Intensity (cm.hr)	RAINFALL: Time Duration (min.)	RAINFALL: Volume (MCM)	OUTFLOW: Peak Discharge (cumecs)	OUTFLOW: t_p (min)	OUTFLOW: t_b (min)	OUTFLOW: Runoff Vol. (MCM)
c_p	2	2	120	60.0	0.125	2.5	0.0254	0.3955	0.5	120.0	31.3	265.21	720	>7200	29.4
							1.3657	2.3731							
	2	2	120	60.0	0.0625	2,5	0.0127	0.3955	0.5	120.0	31.3	214.11	840	>7200	25.3
							0.6829	2.3731							
	2	2	120	60.0	0.03125	2.5	0.0063	0.3955	0.5	120.0	31.3	148.90	960	>7200	20.5
							0.3414	2.3731							
	2	2	120	60.0	0.50	2.5	0.1015	0.3955	0.5	120.0	31.3	329.88	300	7200	31.3
							5.4628	2.3731							
	2	2	120	60.0	1.00	2.5	0.2023	0.3955	0.5	120.0	31.3	335.24	240	7200	31.3
							10.9256	2.3731							
	2	2	120	60.0	2.00	2.5	0.4059	0.3955	0.5	120.0	31.3	332.56	240	7200	26.1
							25.8513	2.3731							
	2	2	120	60.0	3.00	2.5	0.6089	0.3955	0.5	120.0	31.3	331.97	240	7200	26.1
							32.7769	2.3731							
	2	2	120	60.0	4.00	2.5	0.8118	0.3955	0.5	120.0	31.3	331.83	240	7200	26.1
							43.7026	2.3731							
c_c	2	2	120	60.0	0.50	0.25	0.1015	0.0396	0.5	120.0	31.3	74.30	5700	>>7200	21.0
							5.4628	0.2373							
	2	2	120	60.0	0.50	0.50	0.1015	0.7910	0.5	120.0	31.3	76.39	720	>7200	30.9
							5.4628	0.4746							
	2	2	120	60.0	0.50	1.00	0.1015	0.1582	0.5	120.0	31.3	240.25	1740	7200	31.3
							5.4628	0.9492							
	2	2	120	60.0	0.50	2.00	0.1015	0.3164	0.5	120.0	31.3	386.04	960	6480	31.3
							5.4628	1.8985							
	2	2	120	60.0	0.50	3.00	0.1015	0.4747	0.5	120.0	31.3	484.56	720	6180	31.3
							5.4628	2.8477							
	2	2	120	60.0	0.50	5.00	0.1015	0.7911	0.5	120.0	31.3	632.01	420	6000	31.3
							5.4628	4.7462							
	2	2	120	60.0	0.50	10.00	0.1015	1.5822	0.5	120.0	31.3	965.00	240	5820	31.3
							5.4628	9.4924							
c &	2	2	120	60.0	0.50	2.50	0.1015	0.3955	1.0	120.0	62.7	876.17	840	6720	62.7
c							5.4628	2.3731							
RAIN-	2	2	120	60.0	0.50	1.00	0.1015	0.1582	1.0	120.0	62.7	480.50	1740	>7200	62.7
FALL							5.4628	0.9492							
INTEN-	2	2	120	60.0	0.50	1.00	0.1015	0.1582	2.0	120.0	125.4	981.00	1740	>7200	125.3
SITY							5.4628	0.9492							
	2	2	120	60.0	0.50	1.00	0.1015	0.1582	4.0	120.0	250.7	1921.99*	1740	7200	250.7
							5.4628	0.9492							
	2	2	120	60.0	0.50	1.00	0.1015	0.1582	8.0	60.0	250.7	1922.07*	1740	7200	250.7
							5.4628	0.9492							

1. NDX :

The effect of variation in NDX is shown in Fig. 6. Increase in the no. of increments in the planes increases the peak discharge mildly and runoff volume approaches to the rainfall volume. It is due to the distributed nature of the model. However, increase in the no. affects the computation time significantly. The time to peak is also affected.

2. NDY :

The variation in peak discharge at the outlet and others is shown in Fig. 7. Increase in the no. tends to increase the peak discharge but is more sensitive than the NDX. It does not have bearing on the time to peak. It probably smooths the outflow hydrograph.

3. Δt :

Fig. 8 depicts the effect of variation in NDT on peak discharge, time to peak etc. Greater the NDT lesser would be delta t. Reduction in the time interval causes the flood peaks to vary in magnitude. For Δt=80 min. and rainfall duration ie. 120 min there occurs a loss of mass, violating the mass conservation law. It is because the ratio of the rainfall duration and Δt not being an integer value is the cause of loss of mass. The ordinates at this interval are required to be readjusted according to the rainfall duration for avoiding the unusualness.

4. c_p :

The behaviour of c_p is shown through Fig. 9. The plane celerity has a consistent bearing on the results of outflow peaks. Increase in the wave celerity increases the peak outflow

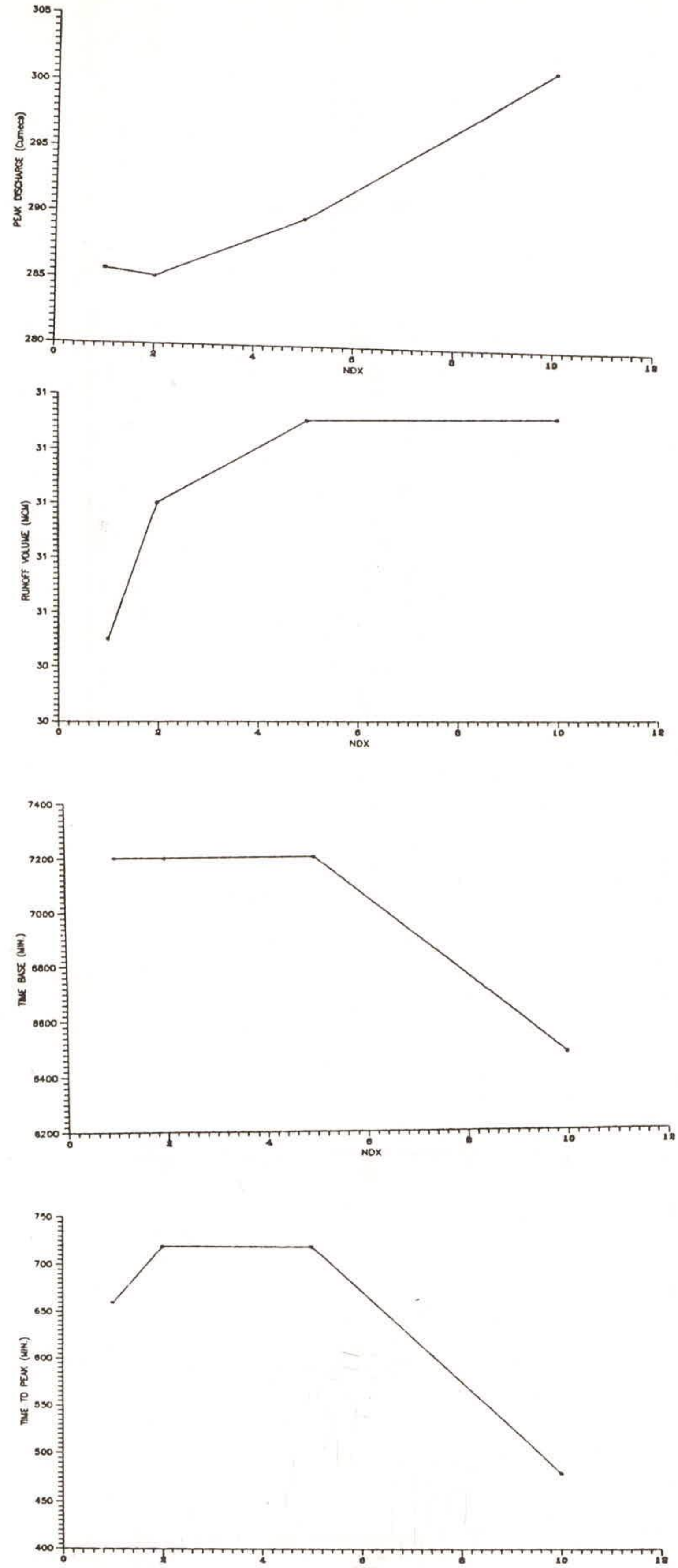

Fig. 6 : Sensitivity of NDX

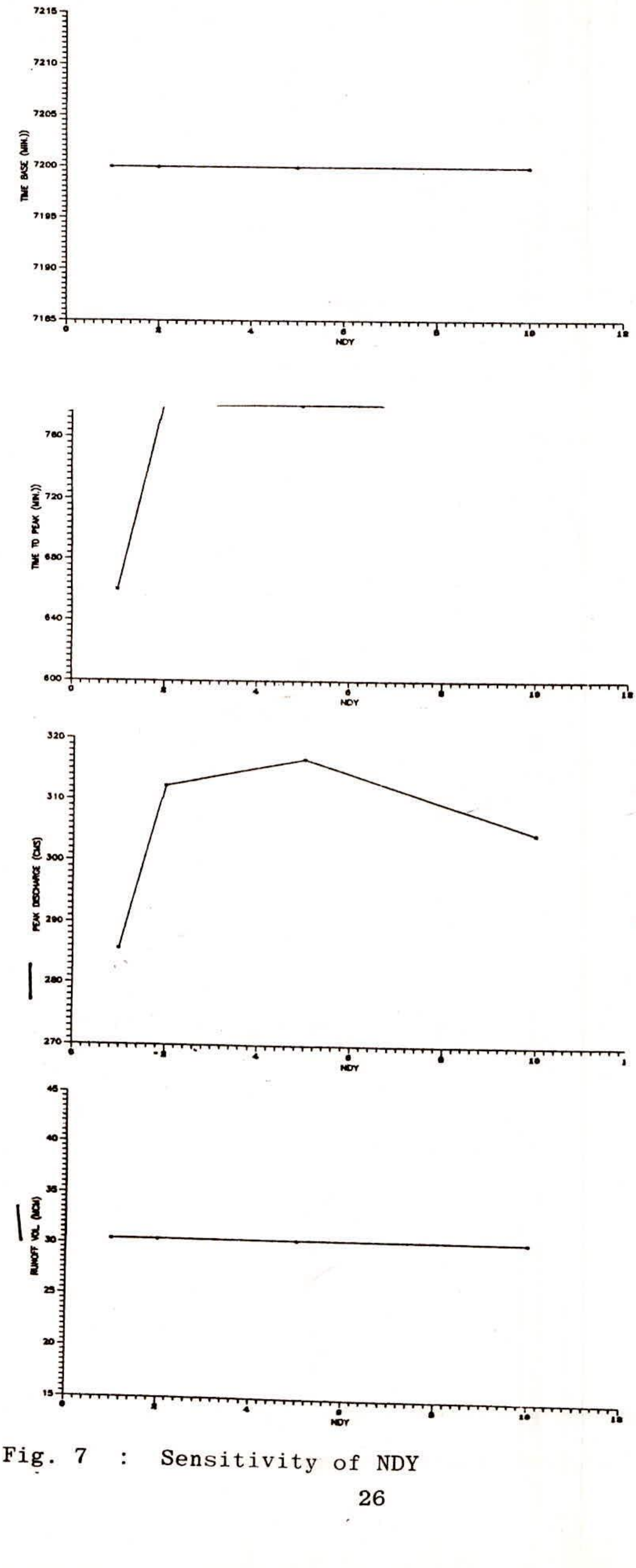

Fig. 7 : Sensitivity of NDY

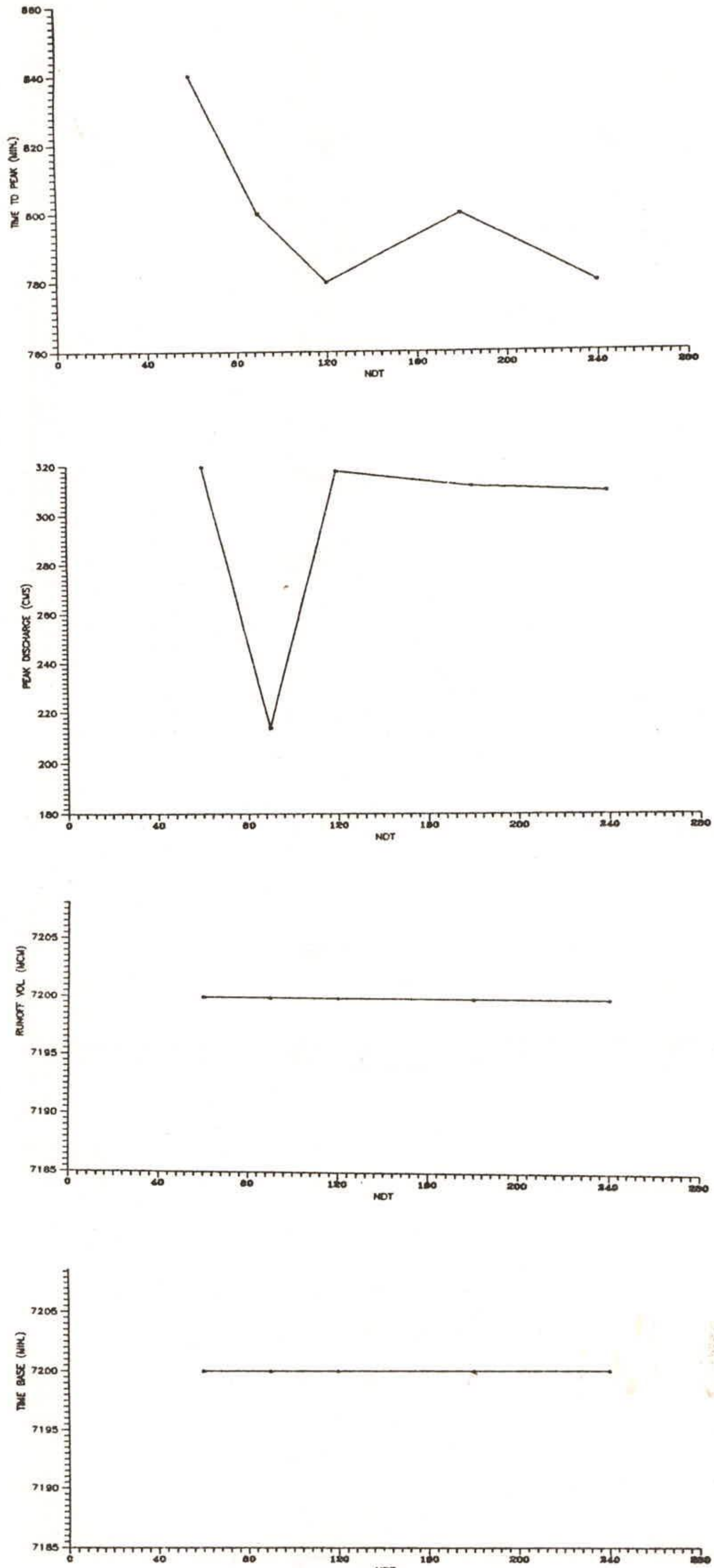

Fig. 8 : Sensitivity of NDT

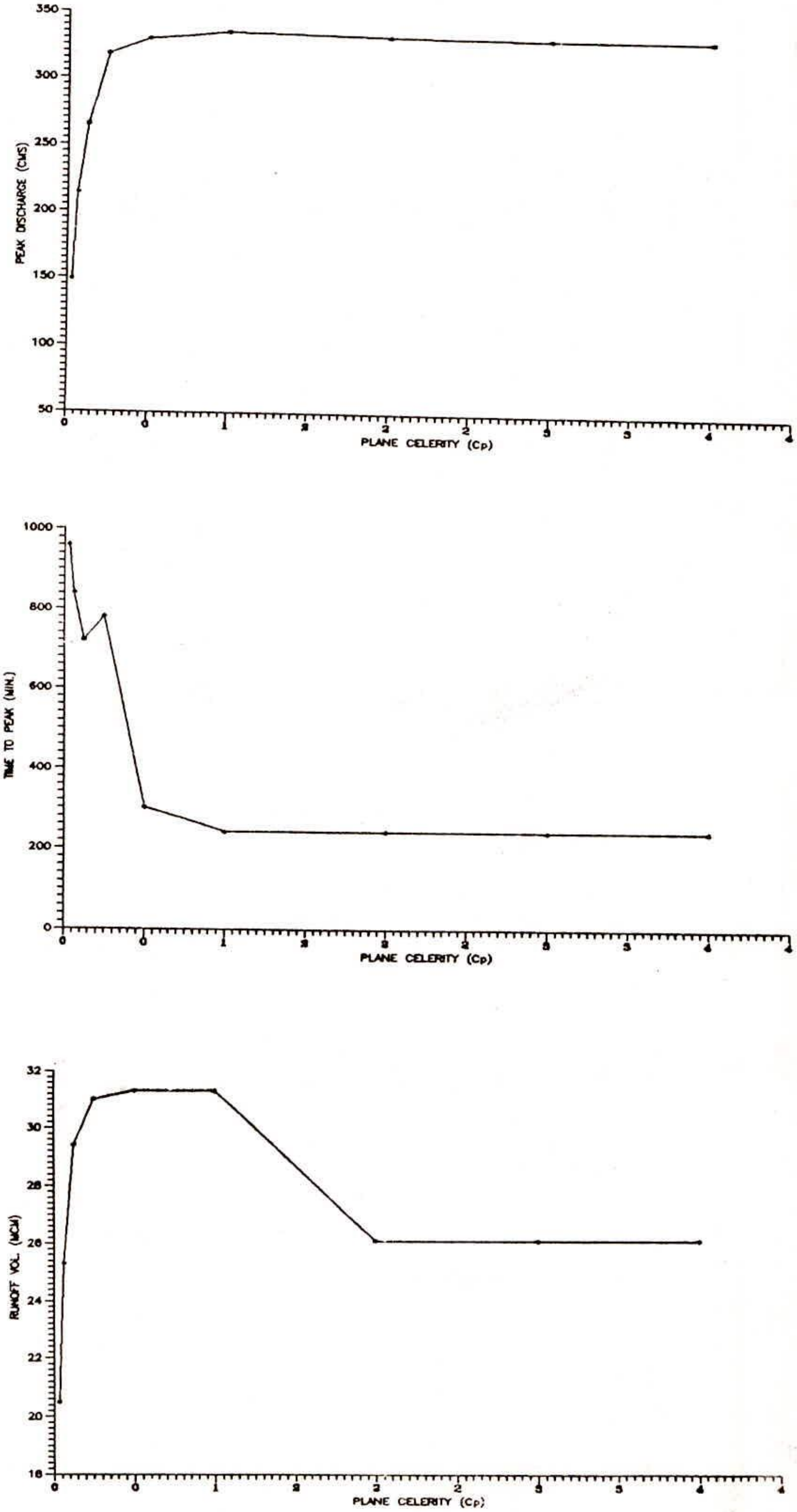

Fig. 9 : Sensitivity of C_p

consistently. The same behaviour is reflected by the time to peaks but much increase in the (>1.0m/s) does not affect t_p.

5. c_c :

Fig. 10 shows the effect of variation in channel celerity. Increase in the wave celerity in the channel sharply affects the peak discharge. Low values result in the low peak discharge and vice versa. Reverse affect is seen on time to peak values. The peaks occur earlier for the higher wave celerities.

6. Rainfall Intensity and its Time Duration :

For the unit input of rainfall on the catchment with different durations result in different rainfall intensities. From Fig. 11, it is seen that increase in the intensity values increases the peak discharges but at a lower rate. On the other hand, the time to peak values decreases with the increase in rainfall intensities.

B. Calibration

The calibration of the model parameters is based on the Aug. 1948 flood event. The parameters e.g. NDX, NDY, NDT, celerity over the planes and in the channels has been fixed using trial and error technique. The excess rainfall intensities and their durations were available and have been used in the model calibration for the above parameters. The calibrated results are shown in Fig. 12. the peak discharge and the time to peak exactly matches with the observed. however, a little variation is seen at the start of the rising and receding limbs of the hydrograph. But as the emphasis of this study is towards the simulation of

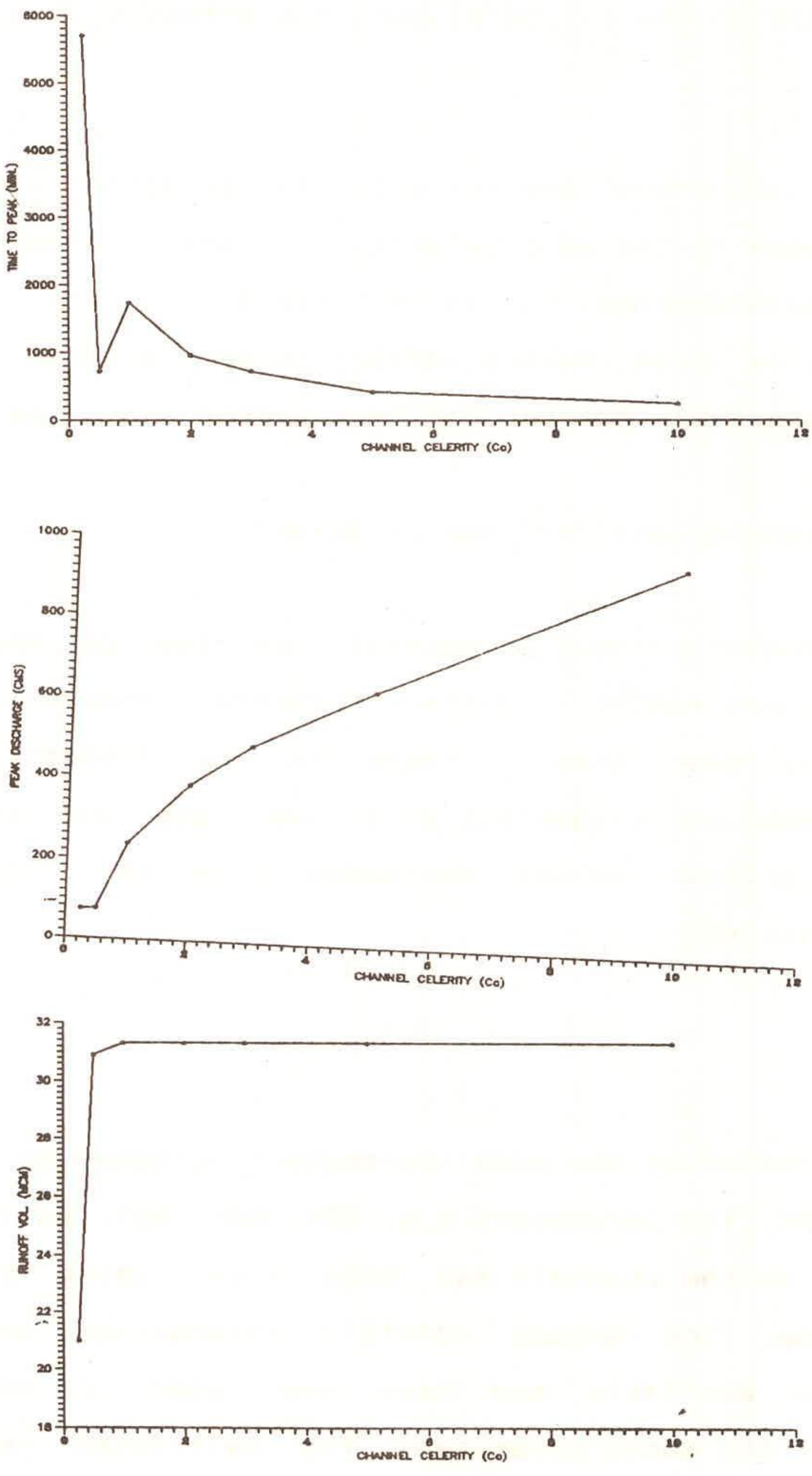

Fig. 10 : Sensitivity of C_c

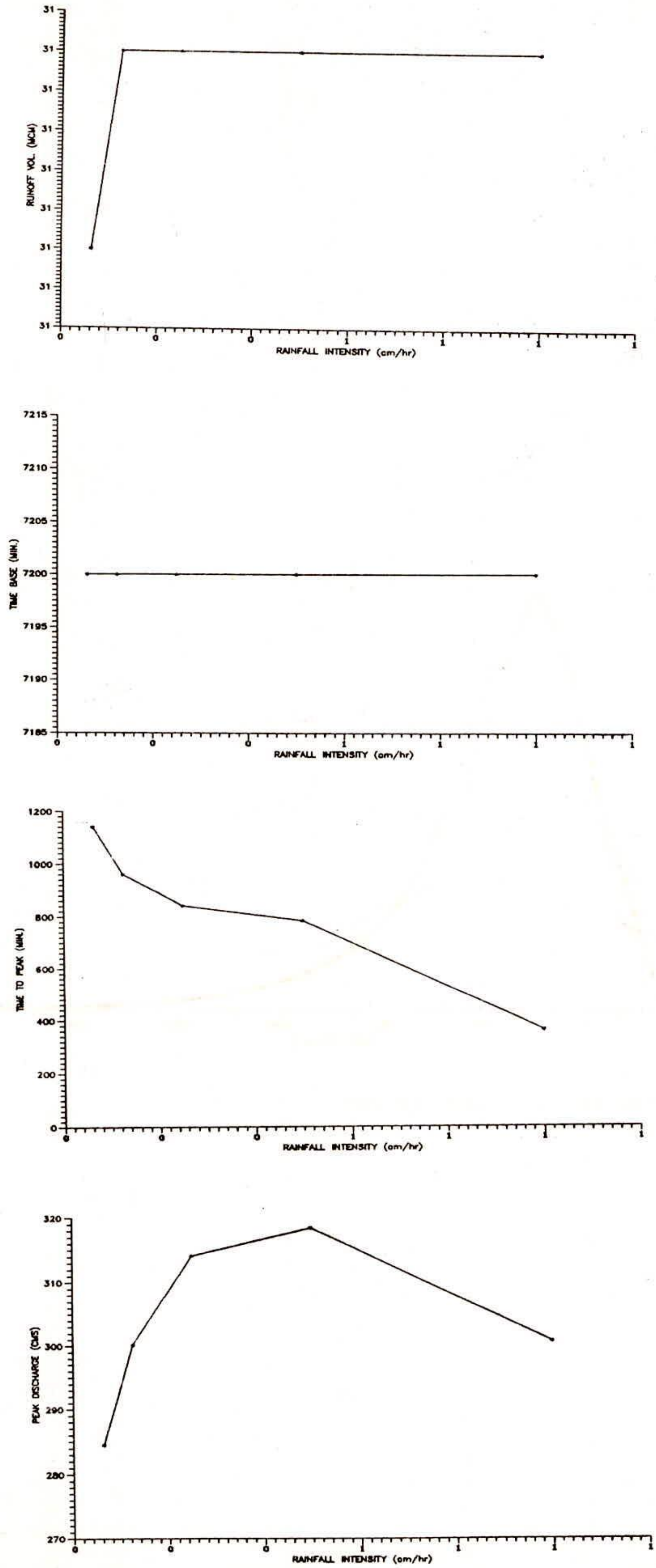

Fig. 11 : Sensitivity of Rainfall Intensity

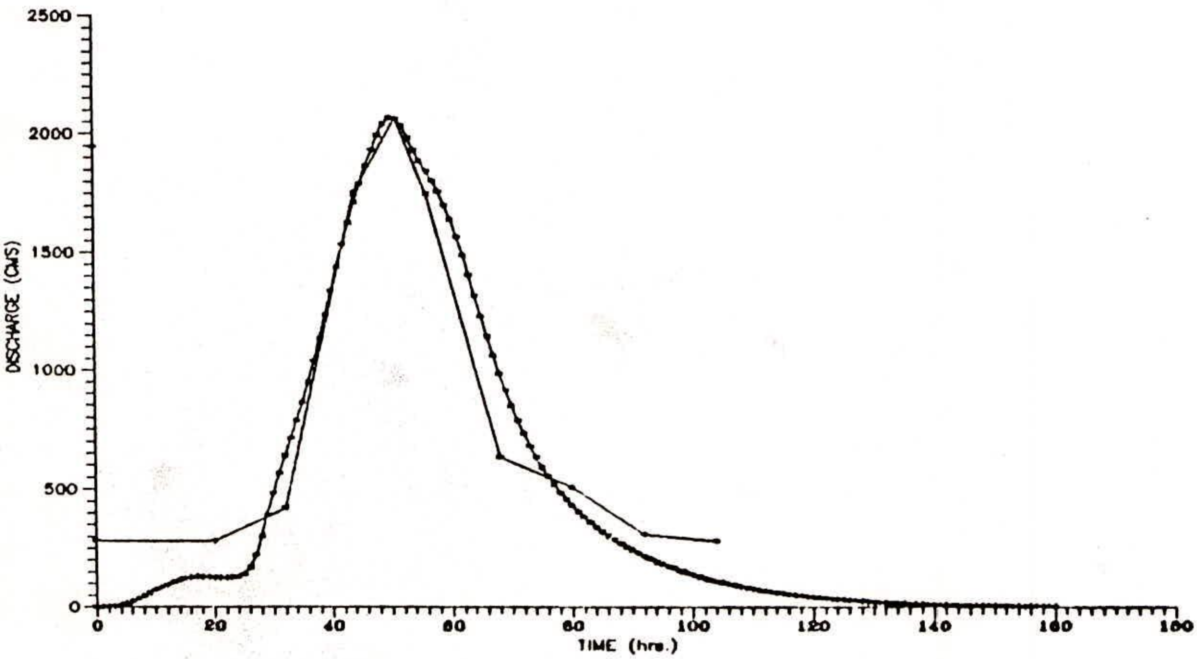

Fig. 12 : August 1948 Flood Event

the peak magnitudes and their time of occurrence, the variation, described above, may not be of much importance. The parameters so calibrated have been used in simulating the other derived/observed floods discussed below.

C. Simulation of Various Return Period Floods and Observed Floods:

Various return periods i.e. 50, 100, 500 and 1000 yrs. floods along with their corresponding rainfall inputs were available. Also, the maximum probable flood (PMF) derived from the transposed storm of 1880 was available for the project site. These flood hydrographs were derived by the Ramganga project authorities using the concept of unit hydrograph. Simulation results are shown in Table-2.

The simulation study of these floods was carried out using the calibrated parameters, described above. From the results of the sensitivity analysis, it can be inferred that the most sensitive input which affects the peak of the outflow hydrograph most is the wave celerity in the channel. The 1948 flood event was simulated using trial and error technique. Various accepted trial values of c_c were used and the flood event was simulated. The other floods were simulated keeping the same value of parameters.

The percentage variation in the peak discharges ranges from 4.5 to 8.44%. The percentage error is in acceptable range. The deviation can be further brought down if the wave celerity values are adjusted. The time to peak of the derived floods is at 84 hrs. while the simulation yields 73 hrs. This might be of concern to the forecasters due to decrease in the lead time available. The time base of the derived as well as simulated floods is same

TABLE-2 SIMULATION OF VARIOUS DERIVED FLOODS

FLOODS	DERIVED			SIMULATED				
	PEAK DISCHARGE (Cumecs)	TIME TO PEAK (Hrs)	TIME BASE (Hrs)	PEAK DISCHARGE (Cumecs)	TIME TO PEAK (Hrs)	TIME BASE (Hrs)	RUNOFF VOLUME (MCM)	RAINFALL VOLUME (MCM)
50-YRS.	6513	84	160	6513.18	73	160	989.7	991.9
100-YRS.	7362	84	160	7691.89	73	160	1144.7	1147.1
500-YRS.	8920	84	160	9571.33	73	160	1413.2	1416.2
750-YRS.	9344	84	160	10049.00	73	160	1480.8	1483.8
1000-YRS.	9628	84	160	10374.00	73	160	1526.8	1530.0
PMF	12035	84	160	13051.00	73	160	1882.0	1885.5

indicating that the catchment response is sufficiently represented by the model.

The other observed flood events (e.g. Sept. 1957; July, 1962; and 1980), shown in Figs. 13, 14, and 15, have been simulated using the above developed model. During the course of simulation the parameters fixed by the calibration procedure have been kept same. The simulated peak discharges of the 1957 and 1962 flood events are greater than the observed whereas the simulated peak discharge of the 1980 flood event is less than the observed. The cause of this anomaly lies in the variation of channel celerities with the flow magnitudes. The 1980 flood event has the peak discharge of the order of 15000 cumecs whereas the other flood events have the peak discharges of the order of 3000 cumecs. The results of sensitivity analysis indicate that the increase in the channel celerity increases the peak discharge at an almost proportionate rate. In the case of 1980 flood event, the channel celerity would have been more than the calibrated while in the other two cases, it would have been less than the calibrated to obtain more satisfactory results. However, the time to peak, in all the cases, faithfully simulates the observed with little variation and a better simulation ia achieved for the rising limbs of the hydrographs.

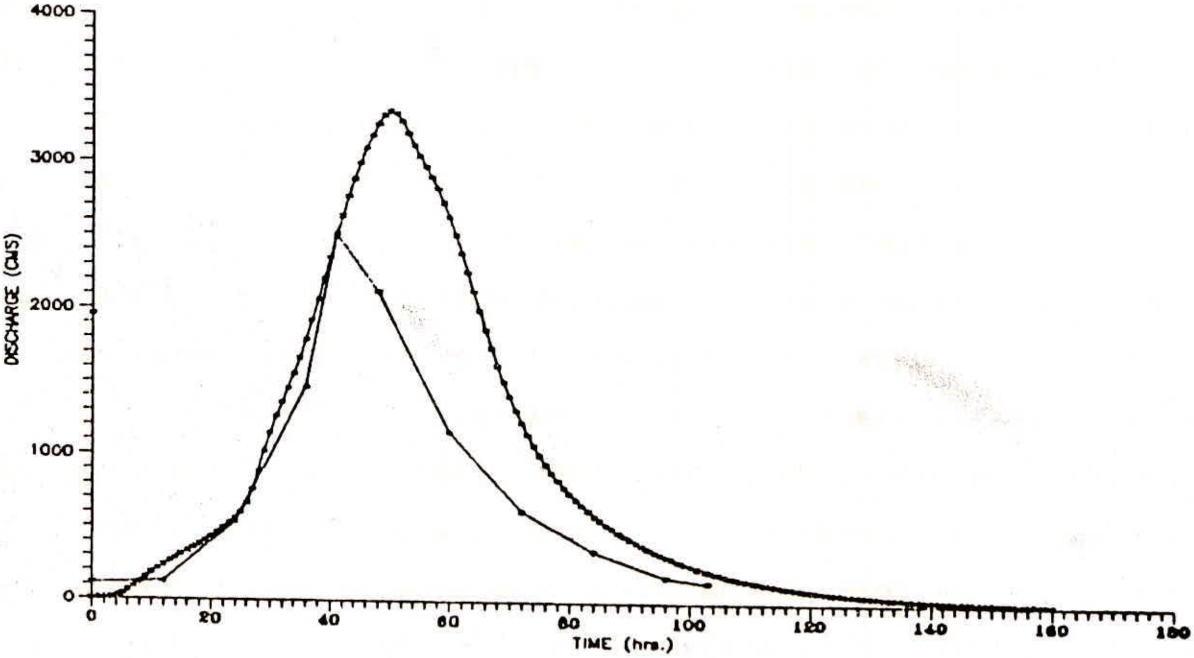

Fig. 13 : Sept. 1957 Flood Event

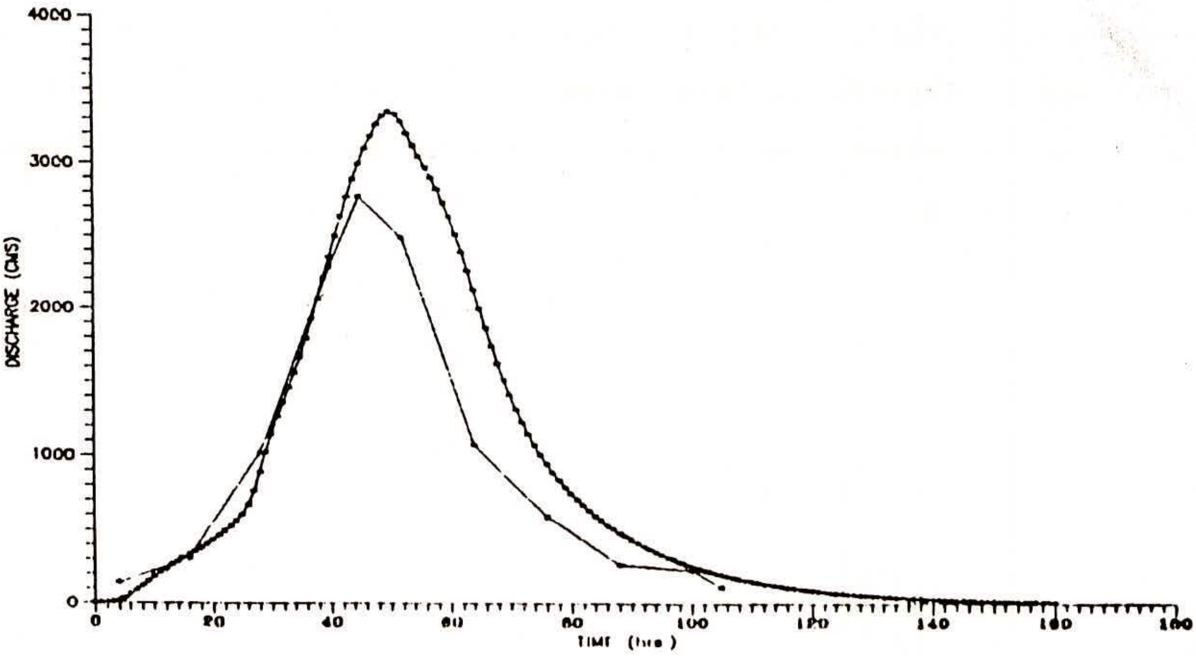

Fig. 14: July 1962 Flood Event

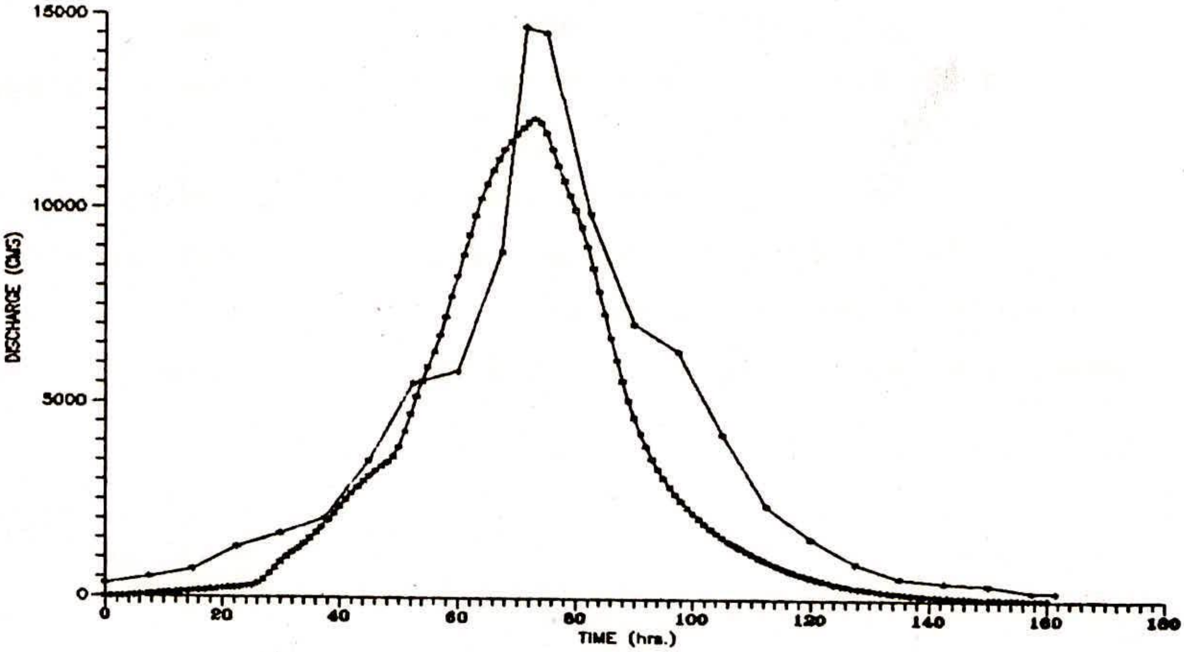

Fig. 15 : 1980 Flood Event

7.0 CONCLUSION

Kinematic wave approach has been extensively used as a tool in the area of flood routing. The concept has been extended to catchment routing using established open book theory. The whole catchment has been considered as a network of open books. Kinematic wave approach is used to deal with the flow behaviour on the plane and in the channels. Present study has concentrated on the applicability of kinematic wave approach to a large size catchment.

The model developed for the Ramganga catchment has been used to simulate various return period floods i.e 50, 100, 500, and 1000 yrs. and probable maximum flood which are derived using unit hydrograph theory and the extreme flood events observed at Kalagarh.. The kinematic wave model faithfully simulates the flood peaks and the overall time distribution of the flow at Kalagarh.

The developed model can be used for estimation of runoff for given rainfall excess. It can be used as rainfall-runoff model provided the model is coupled with a routine which takes into account the rainfall losses for the estimation of rainfall excess.

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