EFFECT OF SURFACE WATER GROUND WATER INTERACTION ON ROUTING CHARACTERISTICS

NATIONAL INSTITUTE OF HYDROLOGY
JAL VIGYAN BHAWAN
ROORKEE - 247667 (UP) INDIA

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PREFACE

In alluvial rivers the phenomena of surface and sub-surface water interaction is very common. This complicates the routing phenomena to a great extent and affect the routing characteristics.

In the present report a program developed by USGS for the situation described above, has been applied to flood data of river Tapti. A sensitivity analysis has also been made. It is concluded that the model is more suitable for the situation when lean flow occurs in the stream to have significant storage in the aquifer in comparison to the flow in the stream and a distinct affect on routing.

This report entitled 'Effect of Surface Water Ground Water Interaction on Routing Characteristics' is a part of the work programme of 'Flood Studies Division' of this Institute. The study has been carried out by Sh. Surendra Kumar, Scientist 'B' under the Guidance of Dr. S.M. Seth Scientist 'F'.

SATISH CHANDRA
DIRECTOR

CONTENTS

LIST OF SYMBOLS

a	Retardation coefficient,
C	The Chezy's discharge coefficient,
Co	Wave Celerity,
F	Froude number,
F(t)	System input,
F'(t)	The time rate change of the system input,
g	Gravitational acceleration,
h(x,t)	Head relative to a datum,
$h(\tau)$	The system response function at location x,
K	Wave dispersion coefficient Hydraulic conductivity of the aquifer,
K'	Hydraulic conductivity of the confining bed,
l	Width of the aquifer
m'	Thickness of confining bed,
P(x,t)	The unit step response of the system,
q	Unit discharge,
q(t)	Stream depletion rate,
Q	The discharge per unit length of stream,
Qo	Selected base line discharge,
	Well pumpage,
S	Storage coefficient of the aquifer,
So	The channel slope,
t	Time,
T	Transmissivity of the aquifer,
U(x,t)	The instantaneous unit impulse response function.
Wo	Average channel width at selected base-line discharge,
x	-Distance along the channel

x	-Horizontal distance from the stream bank,
X	-Distance of well from stream,
y	Depth,
y(x,t)	The output function at some location x and time t,
	Aquifer diffusivity

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ABSTRACT

In alluvial rivers the phenomena of water exchange between aquifer and stream is very common. The flow from or into the stream decreases or increases the quantity of water flowing in the stream which affects the routing process. A program has been developed by USGS for stream flow routing taking into account the quantity of flow from or into the stream. In this study this program has been applied to flood data of River Tapti for the reach Hathnur to Gidhade. A sensitivity analysis has also been made. The storage coefficient, transmissivity, wave celerity, width of the aquifer for finite aquifer and the retardation coefficient for the case when the aquifer is semi-infinite and the stream is seperated from the aquifer by a confining bed, should be selected very carefully for the successful application of the model. Further it can be concluded that the model is more suitable for the situation when lean flow occurs in the stream to have significant storage in the aquifer in comparison to the flow in the stream and a distinct effect on routing.

1.0 INTRODUCTION

For river catchments in which there is significant interconnection between aquifers and the rivers, a through understanding of the way in which water is exchanged between the river and the underlying strata is important for resources planning. If the water is extracted from the aquifer, then knowledge of the subsurface water flows within the catchment is also important. The analysis of the interaction between surface and subsurface water has found wide application in hydrology, hydrogeology and land reclamation viz. in solving problems dealing with the use of water resources and with the prediction of changes in the water regime arising as a result of study of the ecological consequences of alterations in a natural water regime.

The problem is further complicated when streamflow routing is also involved with the interaction between surface and subsurface waters. In this the stream and the aquifer both are hydraulically connected where water is free to leave the stream and enter the aquifer and subsequently return to the stream. To meet the condition of mass conservation in stream flow routing, it is necessary to take the bank storage discharge in to account. The convolution technique has been applied for the situation assuming the system to be linear. The stream flow routing is based on diffusion analogy.

The streamflow routing model developed by U.S.G.S. has been designed to compute (i) the streamflow losses to and gains from bank storage for a reach between two gauging stations with streamflow record or (ii) the streamflow losses to and gains from bank storage and the resulting downstream hydrograph that may also have been influenced by wells and constant base flow.

In this report the modal has been applied to flood data of River

Tapti for the reach between Hathnur and Gidhade where most of the part of the reach is alluvial. A sensitivity analysis has been made. The most important variables of the model are wave celerity, wave dispersion coefficient, storage coefficient, transmissivity and the width of aquifer from the stream when the aquifer is finite aquifer and for the case when the stream bed is separated from the aquifer by a confining permeable bed, the retardation coefficient of that. These parameters should be accurately known for the successful use of the model. In this report some data were available and some have been suitably assumed.

The model is basically suitable to the situation when there is lean flow in the stream. For this case considerable effect of the interaction between surface and sub-surface water on routing occurs because of the comparable quantity of water going into or coming from the aquifer w.r.t. quantity of flow in the stream.

2.0 TERMS AND DEFINITIONS

The terms and definitions used in this report are described as below:

Bank Storage Discharge

During a flood period of a stream, groundwater levels are temporarily raised near the channel by inflow from the stream. The volume of water so stored and released after the flood,is referred to as bank storage.

Hydraulic Conductivity

The rate at which water of prevailing kinamatic viscosity is transmitted through a cross-section of unit area, measured at right angles to the direction of flow.

Confining Bed

Impermeable layer of soil which confines the flow is called the confining bed.

Storage Coefficient

It is defined as the volume of water that an aquifer realeases from or takes into storage per unit surface area of aquifer per unit change in the component of head normal to that surface.

Well Pumpage

This is the yield from well.

Transmissivity

The rate at which water of prevailing kinamatic viscosity is transmitted through a unit width of aquifer under a unit hydraulic gradient.

Aquifer

It may be defined as a formation that contains sufficient saturated permeable material to yield significant quantities of water to wells and springs.

Base Flow

Stream flow originating from groundwater discharge is referred to as ground water runoff or base flow.

Specific Retention

It is the ratio of the volume of water soil or rock will retain after saturation against the force of gravity to its own volume.

3.0 REVIEW

The work done in the past by various researchers related with the subject is described in brief as below:

Bouwer (1969) has brought together some of the advances in various scientific disciplines and their application to the analysis and prediction of sepage losses from open channels.

Freeze (1971) has developed a three-dimensional finite difference model for the treatment of saturated-unsaturated transient flow in small non-homogeneous anisotropic geologic basins. The uniqueness of the model lies in its inclusion of the unsaturated zone in a basin wide model that can also handle both confined and unconfined saturated aquifers under both natural and developed conditions. The model allows any generalized region shape and any configuration of time variant boundary conditions. When applied to natural flow systems, the model provides quantitative hydrographs of surface infiltration groundwater recharge, water table depth, and stream base flow.

Pinder and Sauer (1971) stated that the modification of a flood wave due to bank storage effects can be calculated by using numerical method. The dynamic equations describing one-dimensional open-channel flow and the equation for two-dimensional transient groundwater flow has been solved simulatenously, coupled by an expression for flow through the wetted perimeter of the channel. Numerical experiments indicate that flood waves may be modified considerably by bank storage, particularly in the lower segments of a long reach and that the degree of modification is influenced markedly by the hydraulic conductivity of the aquifer.

Keefer, T.N. anmd Mc Quivey, R.S. (1974) developed a multiple linearization technique which offers a useful and inexpensive improvement over a single linearization technique in one-dimensional convolution flow routing. From 10% to 50% reduction in error in magnitude of routed flows using hourly input can be achieved. The authors have examined two different linearization techniques, Harley's linear channel response model and the diffusion analogy model. They have shown that Harley's model proved difficult to apply to actual data because of the fixed relation between wave celerity and mean velocity. The further concluded that for most cases of practical interest the diffusion analogy will work well with multiple linearization.

Keefer (1976) in his paper compared a single-input linear system model and a multiple-input linear system model to a finite difference model. The comparisons are based on the ability of the models to predict discharge at the down-stream end of a 24.14 km. reach of prismatic channel. Four types of channels and two slopes covering a wide range of conditions have been evaluated. The single input model compares favourably in cases where flood wave celerity does not vary greatly with discharge. The multiple-input model can be made to compare favourable in all cases. He further concluded that the single-input model is approximately one-sixth as costly as the finite difference model, and the multiple-input model is approximately one-half as costly for the conditions investigated.

Rushton and Tomlinson (1979) have studied leakage between aquifers and rivers using an idealized one-dimensional problem. Various leakage mechanisms have been included and the effects on heads in the aquifer and flows to the river are noted. Leakage has been represented by a linear coefficient, a non-linear coefficient and the combination of the two. It has been shown that base-flow recessions are effectively independent of the magnitude of the linear leakage coefficient.

Vauclin, et. al. (1979) have studied transient two dimensional water flow in relative to the recharge of a water table aquifer. The approach is based on the physics of water transfer in the complete domain defined by both the saturated and unsaturated zones of soil. The validity of the model has been proved by the excellent agreement between simulated and experimental results.

Donald, et.al. (1980) stated that the aquifer contribution to a gaining stream can be conceptualised as having two parts; the first part is the intercepted lateral flow from the water table and the second is the flow across the stream-bed due to differences in head between the water level in the stream and the aquifer below. The amount intercepted is a function of the geometry of the cell, but the amount due to difference in head across the stream-bed is largely independent of cell geometry.

Reeder et.al. (1980) have investigated the effects of fluctuations in surface water depth on infiltration rates in to initially unsaturated soils by numerically solving the Richard's equation. They found that infiltration rates may increase with time in response to rapid rates of increase of water depth conditions under which this will occur, have also been identified.

Akan and Yen (1981)have developed a physically based mathematical model to simulate the surface-subsurface flow system. Surface flow is described by a set of one-dimensional dynamic wave equations and subsurface flow is assumed to be two-dimensional with potential gradients in the vertical as well as the surface flow direction. Verification of the model has been given by comparing the numerical results with some existing analytical solutions and limited available experimental results.

Miles and Rushton (1983) have described the formation of a model to represent surface and subsurface flows of water for a catchment in Central England. They have used a finite difference model to represent ground water flows in an aquifer with surface water flows being represented by flow balance techniques. Three hydrologically significant land types have been identified and the model contains three components which correspond to these. The purpose of the model is to assess the long term water resources of the catchment. The results show that it is possible to formulate a comprehensive model of a complete catchment, based upon measurable, physical parameters with the inflow of the water being calculated solely from rainfall and evapotranspiration estimates.

Crebas et. al. (1984) in their paper have described an approach to incorporating drainage and the interaction of groundwater and open-channel flow in a physically-based hydrologic response model. Two classes of exchange have been distinguished. The first is concerned with the influence of minor conduits which require too much refinement to be modelled on an individual basis, but amulgated make an important contribution to the water budget on an aerial basis. The second class of exchange is direct seepage from or into the large waterways in which the flow is significant and is modelled as a distinct hydrologic process. They have outlined a number of practical steps to limit the data and organisation required.

Miles (1985) has discussed the representation of flows between aquifers and rivers. The techniques described are suitable for regional groundwater flow models based on numerical techniques such as finite difference approximations. Two widely different methods of representing flows to partially penetrating rivers have been compared and shown to give same results. He has investigated the numerical statibility of one of the methods and demonstrated the effect of different depths of penetration.

Backer and Kundzewicz (1987) have analysed the category of multilinear models that embraces several approaches developed in apparently independent references. Multi-linear modelling is a means of describing a non-linear system with a set of linear models. This has been performed by distributing the inflow to the system (river reach) into inflows to submodels and treating these inflows with a family of different linear operators. The model considered by the authors are nearly as simple as linear models and almost as accurate as non-linear hydrodynamical models.

Vasiliev (1987) has given a survey of the mathematical (quantitative) modelling of the interaction between surface-water and ground-water. Besides describing the existing appraoch to such modelling and the main difficulties in its realization, he has presented a brief review of the principal works on the subject under consideration. Further he concluded stating that when developing mathematical models of real processes describing more or less completely the hydrologic cycle, many external effect should be taken into account and a great volume of information such as hydrological and soil data, geomorphological characteristics, charater of vegetation hydrological data on the regime of water bodies and streams, atmospheric conditions near the ground, precipitation, evaporation etc. should be analysed.

4.0 PROBLEM DEFINITION

Routing process is affected by the flow going into or coming from the stream from or to the aquifer. This becomes significant for the lean flows. The objective of this study is to apply the model developed by U.S.G.S. (Land, 1977), for stream-flow routing taking bank storage into account to the flood data of River Tapti for the reach Hathnur to Gidhade.

5.0 BASIS FOR MODEL DEVELOPMENT

The model presented, consists basically three hydrologic components, which are:

(a) Stream flow routing algorithm which applies one dimensional diffusion analogy and the convolution technique of Keefer (1974).

(b) The bank-storage discharge which is calculated analytically, uses equations for an abrupt change in stream stage and the convolution technique to compute the discharge from a series of abrupt changes at fixed time intervals.

(c) The stream depletion which is computed analytically.

Convolution Process:

Convolution is a concept basic to linear system theory. A system input is combined through the convolution process with a system response function to produce the predicted system output. In the case of routing, the system input is the upstream discharge, and the system output is the downstream discharge. This is expressed mathematically by

$$Y\ (x,\ t) = \int_{-\infty}^{t} Y\ (o,\ t-\tau)\ h(\tau)\ d\tau \qquad \text{...(1)}$$

where,

$Y\ (x,\ t)$ = the output function at some location x and time t

$Y\ (o,\ t-\tau)$ = the input function at x = o and at time $t-\tau$; and

$h\ (\tau)$ = the system response function at location x.

If the input and output are discrete numbers rather than continuous functions, Eq. 1 may be expressed in matrix form as:

$$[Y] = [H]\ [X] \qquad \text{...(2)}$$

where,

$[Y]$ = a column vector of system output values;

$[H]$ = a column vector response function and;

$[X]$ = an input matrix made up of Y (o,t) values.

The physical appearance of the matrix and vectors is shown in Fig.1 and has been examined by O'Donnell(1966).

(a) in matrix notation;

$$[X] \times [H] = [Y] \qquad \text{...(3)}$$

(b) expressed in complete form:

where,

n = no. of response ordinates

j = no. of inputs

m = no. of outputs = j+n - 1

$$\underbrace{\begin{bmatrix} Y(0,1) & & 0 & 0 & 0 \\ Y(0,2) & & Y(0,1) & 0 & 0 \\ Y(0,3) & & Y(0.2) & Y0,2) & 0 \\ Y(0,4) & & Y(0.3) & Y(0.2) & \\ & & & & \\ & & & & \\ & & & & \\ Y(0,j-1) & & & & \\ Y(0,j) & & & & \\ 0 & & & & \\ 0 & & & & \\ 0 & & & & \\ 0 & & & & \\ & 0 & Y(0,j) & & Y(0,j-1) \\ & 0 & 0 & & Y(0,j) \end{bmatrix}}_{\xleftrightarrow{\quad n \quad}} \begin{bmatrix} h_1 \\ h_2 \\ h_3 \\ \\ \\ h_{n+1} \\ h_n \\ \\ \\ \\ \\ \\ \\ \\ \\ \end{bmatrix} \begin{bmatrix} Y(x,1) \\ Y(x,2) \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ \\ Y(x,m-1) \\ Y(x,m) \end{bmatrix}$$

This assumes Y (0,t) = 0 for time $t < 0$ and $t > j$ time steps.

Figure 1 : MATRIX CONVOLUTION

The convolution technique discussed above can be applied in stream flow routing and bank storage discharge because the systems are assumed to be linear. This allows superimposing individual responses to obtain a composite response. The technique first requires determining the system's response to a single unit of input. For a single time step the observed inflow would be multiplied by each of the unit-response ordinates to obtain a series of calculated outflow hydrographs. A composite outflow hydrograph is obtained by summing the outflow values at common times.

(i) Streamflow Routing:

The streamflow routing theory used in this model is based on diffusion analogy which is described as below:

The differential equations governing flow in a wide rectangular channel with no lateral inflow may be written as:

$$\frac{\partial q}{\partial x} + \frac{\partial y}{\partial t} = 0 \qquad \text{...(4)}$$

and,

$$(gy^3 - q^2)\frac{\partial y}{\partial x} + 2y.\ q\frac{\partial q}{\partial x} + y^2\frac{\partial q}{\partial t}$$

$$= gy^3\left(S_o - \frac{g^2}{C^2\ y^3}\right) \qquad \text{...(5)}$$

where,

x = distance along the channel;

t = time;

g = gravitational acceleration;

y = depth;

q = unit discharge;

S_o = the channel slope, and

C = the Chezy's discharge coefficient.

When combined, Eqs. 4 and 5 yield a highly non-linear second order partial differential equation. By confining the attention to a small range of fluctuations about a base discharge q_o, this equation can be reduced to the following linear form:

$$(gy_o - u_o^2) \frac{\partial^2 q}{\partial x^2} - 2u_o \frac{\partial^2 q}{\partial x . dt} - \frac{\partial^2 q}{t^2} = 3gSo \frac{\partial q}{\partial x} + \frac{2gSo}{u_o} \frac{\partial q}{\partial t} \qquad \text{...(6)}$$

Three different expressions for the diffusion analogy model can be derived from Eqn. 6 by neglecting the last two terms on the left. All may be written as:

$$\frac{\partial q}{\partial t} = K. \frac{\partial^2 q}{\partial x^2} - Co \frac{\partial q}{\partial x} \qquad \text{...(6a)}$$

where,

K = wave dispersion coefficient ; and

Co = wave calerity.

when the Froude number is less than one-half, Co may be neglected and K expressed as:

$$K = qo/2So \qquad \text{...(7)}$$

When the Froude number must be accounted for

$$K = \frac{qo}{2So} [1 - F^2] \qquad \text{...(8)}$$

If instead of merely neglecting terms, the kinamatic wave equation is used as a first approximation to the solution of Eq. 6 and an order of magnitude analysis is performed, then

$$K = \frac{qo}{2So} (1 - \frac{F^2}{4}) \qquad \text{...(9)}$$

The 4 appears in the denominator to account for the neglected terms. Eq. 6a with K expressed as in Eq. 7 may be traced back to Hayami (1951). Eqn. 6a is the classic diffusion equation. The solution for a delta function input (the channel response function) is:

$$q\,(x,t) = \frac{1}{\sqrt{4\pi K}} \cdot \frac{x}{t^{3/2}} \ \exp.\left[\frac{-(C_o t - x)^2}{4Kt}\right] \qquad \ldots(10)$$

K from Eq. 7 can be calculated as:

$$K = Qo/(2So\ Wo) \qquad \ldots(11)$$

where,

Qo = Selected base-line discharge,

So = Channel slope, and

Wo = average channel width at selected base line discharge.

K is used to smooth the outflow hydrograph. Wave celerity is approximated by:

$$Co = \frac{1}{Wo} \ \frac{dQo}{dy} \qquad \ldots(12)$$

Equation (13) is applicable to one-dimensional confined flow in a homogeneous isotropic aquifer, and it is a good approximation for unconfined flow if changes in the water level are small in comparison with saturated thickness.

Equation 1 used for describing convolution technique, can also be written by using the notations used in this sub-section as below:

$$h\ (x,t) = \int_0^t F\ (\tau)\ U\ (x,t - \tau)\ d\tau \qquad \ldots(14)$$

or as

$$h\ (x,t) = \int_0^t F'\ (\tau)\ P\ (x,t - \tau)\ d\tau \qquad \ldots(14a)$$

where

$U(x,t)$ = the instantaneous unit impulse response function (T^{-1});

$P(x,t)$ = the unit step response of the system (dimensionless);

$F(t)$ = system input (the fluctuation of head at source) (L);

$F'(t)$ = the time rate change of the system input (LT^{-1})

The discharge into or out of the stream at x = 0 can be obtained by applying Darcy's law (Cooper and Rarabaugh, 1963) to equation (14a)

$$Q = T \int_0^t F'(t) \frac{\partial P(o, t-\tau)}{\partial x} d\tau \qquad ...(15)$$

where,

Q = the discharge per unit length of stream;

T = aqufer transmissivity

The model considered can be applied for three cases as below:

i) Semi-infinite aquifer

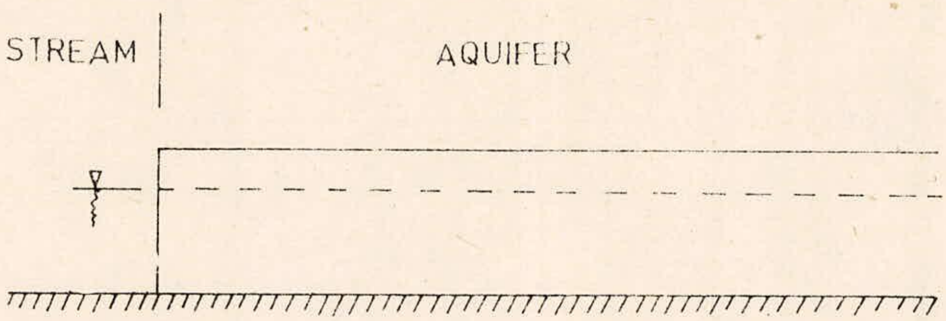
ii) Finite aquifer

iii) Semi-infinite aquifer with a permeable confining bed covering the stream

(i) Semi-infinite aquier:

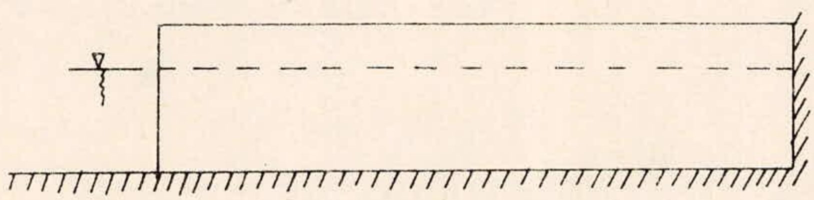
The simplest case being considered is that of a plane source in a semi-infinite aquifer as shown in Fig. 2 (a). The unit response function for this case is given by Carslow and Jaeger (1959) and can be expressed as:

$$P(x,t) = \text{erfc} \left(\frac{x}{(4\alpha t)^{1/2}} \right] \qquad ...(16)$$



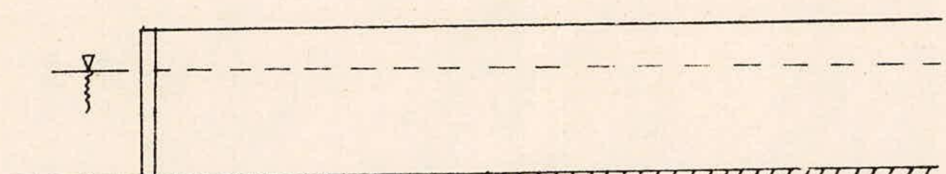
Case 1 — Semi-infinite aquifer

(a)



Case 2 — Finite aquifer

(b)



Case 3 — Semi-infinite aquifer with permeable confining bed separating stream and aquifer.

(c)

FIG. 2 - SELECTED STREAM AQUIFER BOUNDARY CONDITIONS.

where, erfc is the complementary error function. The instantaneous unit impulse response function can be shown by differentiation with respect to time to be:

$$U(x,t) = \frac{x}{(4\pi\alpha)^{1/2}\, t^{3/2}} \exp\left(-\frac{x^2}{4\alpha t}\right) \qquad \ldots(17)$$

The derivative of equation (16) with respect to distance for use in equation 14(a) is:

$$\frac{dP\,(0,\ t)}{d\ x} = -\frac{1}{(4\pi\alpha t)^{1/2}} \qquad \ldots(18)$$

(ii) Finite aquifer

The unit step response function for the finite aquifer, illustrated in Fig. 2(b), appears in different forms in the literature. The form found by Hall and Moench (1972) to converge most rapidly is that given by Cooper and Rorabaugh (1963).

$$P\,(x,t) = 1 - \frac{4}{\pi} \sum_{n=1}^{\infty} \frac{\mathrm{Sin}\ (cx)\ \mathrm{Exp}\ (-c\,\alpha t)}{(2\,n-1)} \qquad \ldots(19)$$

where,

$c = (2n-1)\,\pi/2l$;

l = width of the aquifer

Differentiation of eqn. 19 with respect to time yields the instantaneous unit impulse response function,

$$U(x,t) = \frac{\pi\alpha}{l^2} \sum_{n=1}^{\infty} (2n-1)\ \exp\ (-c^2\alpha t)\ \ \mathrm{Sin}\ (cx) \qquad \ldots(20)$$

and differentiation with respect to distance yields

$$\frac{dp\,(o,t)}{dx} = -\frac{2}{l}\sum_{n=1}^{\infty} \exp(-c^2 \alpha t) \qquad \ldots(21)$$

If the width of aquifer 'l' is allowed to become fairly larne, i.e. $l \rightarrow 0$, then equation 19 to 21 will give the same results for the semi-infinite aquifer as do equations 16 to 18.

(iii) Semi-infinite aquifer with a permeable confining bed covering the stream bed:

The unit step response for this case illustrated in Fig. 2(c) can be obtained from the solution of a radiation boundary problem given by Carslaw and Jaeger (1959):

$$P(x,t) = \operatorname{erfc}\left(\frac{x}{(4\alpha t)^{1/2}}\right) - \exp\left(\frac{x}{a} + \frac{\alpha t}{a^2}\right) \operatorname{erfc}\left(\frac{x}{(4\alpha t)^{1/2}} + \frac{(\alpha t)^{1/2}}{a}\right) \qquad \ldots(22)$$

where,

a = retardation coefficient;

$= \frac{m' K'}{K}$

m' = thickness of confining bed;

k' = hydraulic conductivity of confining bed;

K = hydraulic conductivity of aquifer.

The retardation coefficient, as defined by Hantush (1965), is the effective thickness of aquifer required to cause the same head loss as the confining bed of the stream bed. The implicit assumption is made that the confining bed has no storage.

Differentiation of equation 22 with respect to time yields the instantaneous unit impulse response function.

$$U(x,t) = \frac{(\alpha)^{1/2}}{a\,(\pi t)^{1/2}} \exp\left(\frac{-x^2}{4\alpha t}\right) - \frac{\alpha}{a^2} \exp\left(\frac{x}{a} + \frac{\alpha t}{a^2}\right) \operatorname{erfc}\left(\frac{x}{(4\pi t)^{1/2}} + \frac{(\alpha t)^{1/2}}{a}\right) \quad \ldots(23)$$

and, differentiation of equation 22 with respect to distance yields

$$\frac{d\,P(o,t)}{dx} = -\frac{1}{a} \exp\left(\frac{\alpha t}{a^2}\right) \operatorname{erfc}\left(\frac{(\alpha t)^{1/2}}{a}\right) \quad \ldots(24)$$

Depletion of Stream by Wells:

Glover and Balmer (1959) developed an analytical expression for computing the discharge, as a function of time, from a stream to an aquifer resulting from pumpage of wells. The resulting expression for semi-infinite aquifer is:

$$q(t) = Q_o \left(1 - \operatorname{erfc}\left(\frac{x}{\sqrt{4\ t\ T/S}}\right)\right) \quad \ldots(25)$$

where,

x = distance of well from stream ;

Q_o = Well pumpage;

$q(t)$ = Stream depletion rate; and

erfc = an error function, described as below:

$$\operatorname{erfc}(Z) = \frac{2}{\sqrt{\pi}} \int_{z}^{\infty} e^{-t^2}\, dt \qquad \ldots(26)$$

The approximate solution of the expression at right hand side of the equation 26 is:

$$\frac{2}{\sqrt{\pi}} \int_{z}^{\infty} e^{-t^2}\, dt = \frac{1}{[\,1 + a_1 Z + a_2 z^2 + \cdots + a_6 z^6]^{16}} + \epsilon(Z) \qquad \ldots(27)$$

Here,

$$|\,\epsilon(Z)\,| \leq 3 \times 10^{-7}$$

Neglecting the term $\epsilon(z)$,

$$\operatorname{erfc}(Z) = \frac{1}{[\,1 + a_1 z + a_2 z^2 + \cdots + a_6 z^6]^{16}} \qquad \ldots(28)$$

where,

a_1 = 0.07052 30784 $\quad$ a_2 = 0.04228 20123

a_3 = 0.00927 05272 $\quad$ a_4 = 0.00015 20143

a_5 = 0.00027 65672 $\quad$ a_6 = 0.00004 30638

The expression for error function (erfc) is from Hastings, Jr (1955).

The assumption in applying equation (25) is that all of the pumped water would eventually reduce the stream-flow by an equal amount. This could result in a stream flow depletion or a reduction in base-flow. If some of the well's water is not at the stream's expense, this fraction must be subtracted from Q_0 The scope of this model is not to analyze the source of water to wells but to adjust the timing of streamflow depletion caused by wells.

Base Flow:

This model is not designed to compute a base-flow component of stream flow. However, the base flow for the [illegible] period is known and reasonably constant, a constant value can be added to the streamflow.

6.0 DATA REQUIREMENT:

For the model discussed earlier, following data are required for the use of the model. The model calculates the bank storage discharge hydrograph, routes the upstream hydrograph taking into account the flow from the stream to aquifer or aquifer to stream and depletion due to wells if they exist. Data required, are grouped seperately for the purposes discussed above in seperate heads as below:

(A) Bank Storage Discharge:

(i) Discharge:

The discharge hydrographs (preferably hourly data) at every section considered are required. The total number of data points of the discharge hydrograph will vary with the duration of study.

The base flow if available should be provided for every reach.

(ii) Rating Table:

Rating at every section of the reach considered as above should be available which covers the entire range of discharge. If there is a shift in the rating from time to time the shift data should also be given.

(iii) Reach Properties:

The channel length of the reach and alluvial length of the reach are required. The estimated travel time of the flood wave for reach is also required.

(iv) Aquifer Properties:

Aquifer characteristics for the reach are required. It includes transmissivity and storage coefficient. Soil retention factor should be provided to calculate fraction of bank storage retained in aquifer. This water may go to satisfy a soil moisture deficiency above the original water table or to plants.

Retardation coefficient which describes the impedence to flow between a stream and an aquifer due to a permeable confining bed covering the stream bank, will be required for the aquifer which is semi-infinite and the stream is lined with permeable confining bed. If the aquifer is finite, the width of aquifer from stream to boundary is required.

(B) Routing with Bank Storage Discharge:

Besides the data required for the case discussed above the following data are also required.

(1) Channel Properties:

(a) Wave dispersion Coefficient:

This describes the spreading of a hydrograph pulse from the upstream to downstream points of a reach.

(b) Wave Celerity:

It controls the travel time between ends of a reach for a hydrograph pulse.

Realistic values of the two parameters defined above should be provided, but the value of the wave celerity should be quite accurate.

(2) Diversion and Well Data:

The no. of diversions or wells, if exist, should be provided. If the distance from the stream is less than 3. m (approx), a direct diversion is assumed. The rate of diversion or well pumpage (a negative value assumes withdrawal and positive value assumes recharge) data with their duration of working should be provided.

7.0 APPLICATION

The model developed by USGS has been applied to River Tapti. A general picture of the basin is given below:

7.1 Description of Catchment

7.1.1 River system and basin characteristics

Tapti is second largest river of Central India which flows westward and discharges into Gulf of Cambay (Arabean Sea). Tapti takes its origin in Multai hills in the Gavilgarh hills ranges of Satpura mountain in Madhya Pradesh.

The Tapti basin extends over an area of 65, 145 sq.km. and lies between east longitude of 72°38' to 78°17' and north latitude 20°5' to 20°3' situated in Deccan Plateau. The river is 724 Km long out of which the reach length of 130 km is selected for the study which lies between Hathnur and Gidhade.

A basin map of Tapti river is enclosed vide Fig. No. 3.

7.1.2 Physiography of the reach

The reach considered has rich fertile plains generally black soil. The average bed slope of the reach is around 0.52×10^{-3}.

7.1.3 Tributaries

The Tapti receives several tributaries and in the reach considered there are four tributaries which joins Tapti between Bhusaval and Gidhade. Between Hathnur and Bhusaval there is no tributary. A line diagram showing these details, is given in Fig. 4.

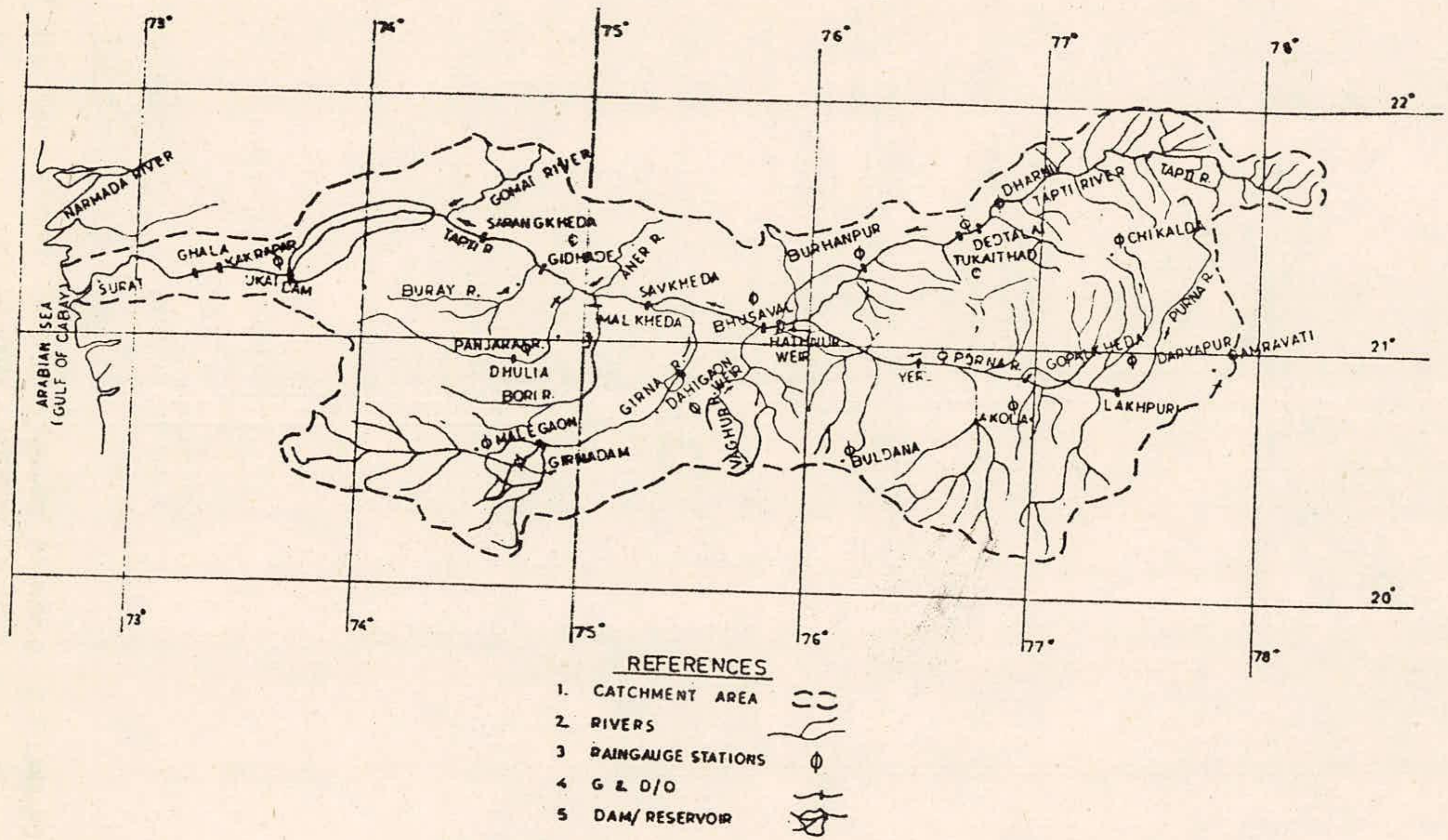
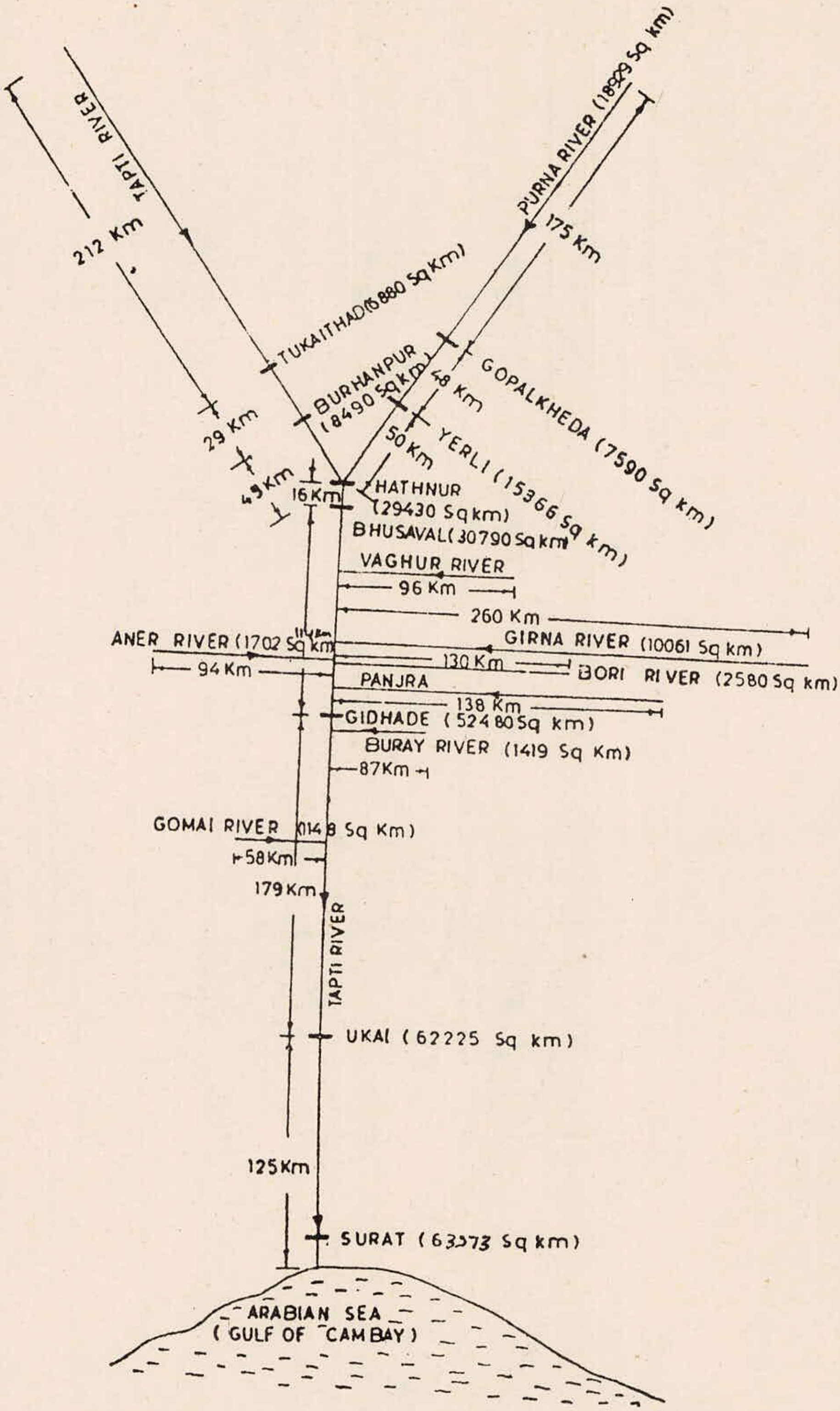


FIG.3 — INDEX MAP OF TAPTI BASIN



REMARKS: FIG. GIVEN IN BRACKET IS C.A. UP TO THAT SITE/RIVER
(TOTAL C.A. OF BASIN = 65145 Sq Km)

FIG. 4—LINE DIAGRAM OF THE TAPTI RIVER

7.1.4 Meteorological Conditions:

(a) Monsoon and rainfall:

The average annual rainfall of the catchment of the river Tapti is 78.8 cm. More than 90% of rainfall occurs during South-West monsoon from mid of June to mid of October and about 50% of rainfall is received in the month July and August.

(b) Temperature:

The mean minimum temperature varies from 11.1°c to 14.4°c but the temperature below freezing point has also been recorded. The mean maximum temperature ranges from 38°C to 48°C. The mean temp. in the basin varies from 25°c to 30°c.

(c) Evaporation:

The evaporation losses assumed for Ukai Project is 138mm/ year and for Upper Tapti Basin is 244 mm/year.

As seen from the Fig. 4, most of the tributaries are joining at the middle or d/s of the middle of the reach Bhusaval to Gidhade. It is presumed that the lateral inflow contributed by the tributaries is joining at the middle of the above said reach. Now the reach Hathnur to Gidhade can be broken into three subreaches namely, Hathnur to Bhusaval, Bhusaval to middle of the reach Bhusaval to Gidhade and middle of the latter reach to Gidhade. The analysis has been done for reach no. 2 and 3.

The flow data at three sections, rating curves, length of reach data are taken from the M.E. Theses presented by Bhandari (1988).

Storage coefficient and transmissivity are on the basis of the information provided by CGWB in the surrounding area and rest of the data are assumed.

8.0 RESULTS AND DISCUSSION

The model developed by USGS for streamflow routing with losses to bank storage and wells, in this study as discussed earlier has been applied to River Tapti for the flood occured in August 1983. The reach selected lies between Hathnur and Gidhade. The observed data were available at Hathnur, Bhusaval and Gidhade. As seen from the Fig. 5 and 6 the observed and calculated hydrographs at Bhusaval and Gidhade are fairly matching. Though the peaks of calculated hydrographs are slighthtly higher then the observed hydrographs. This can be due to the assumption involved in diffusion analogy that the fluctuations about the selected base line discharge are negligible. This can be true for the case when there is lean flow in the stream.

Further, a sensitivity analysis has been made. The results are tabulated in Table-1 and are plotted in graphs (Fig. 7, 8 and 9). The analysis has been done for volume to bank storage, peak flow and travel time

Volume of bank storage and peak of flow to bank storage are more sensitive to storage coefficient and transmissivity. Volume to bank storage is less sensitive to the retardation coefficient in case 3 but insensitive to other parameters. Peak of flow to bank storage is more sensitive to wave celerity when the value of wave celerity is decreased. But the peak is insensitive to wave dispersion coefficient. It slightly deviates only for large variation in wave dispersion coefficient. The peak is significantly dependent upon the retardation coefficient for case-3. For finite aquifer when the width decreases the peak reduces rapidly but it does not change with the

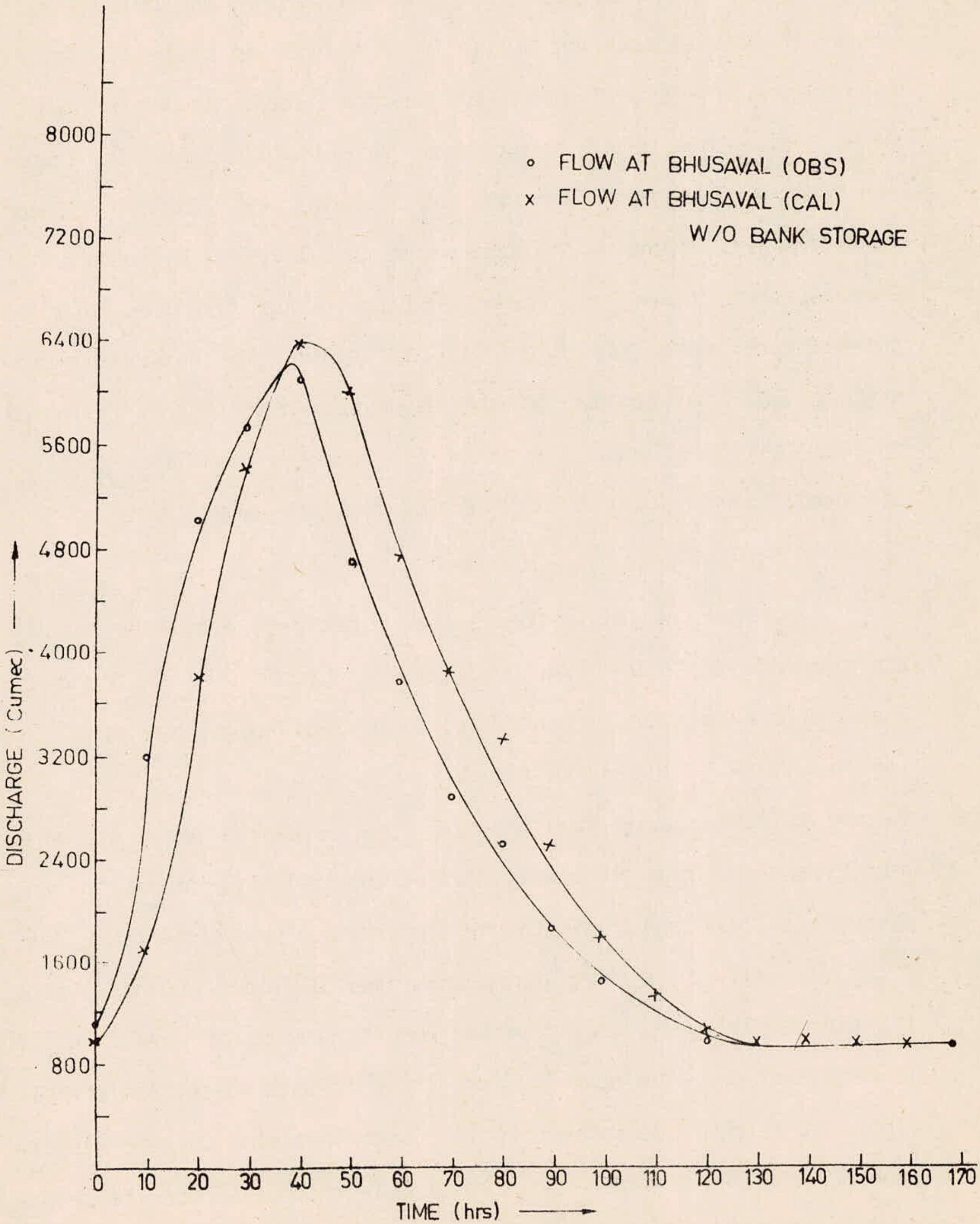


FIG. 5 — OBSERVED AND CALCULATED FLOW AT BHUSAWAL

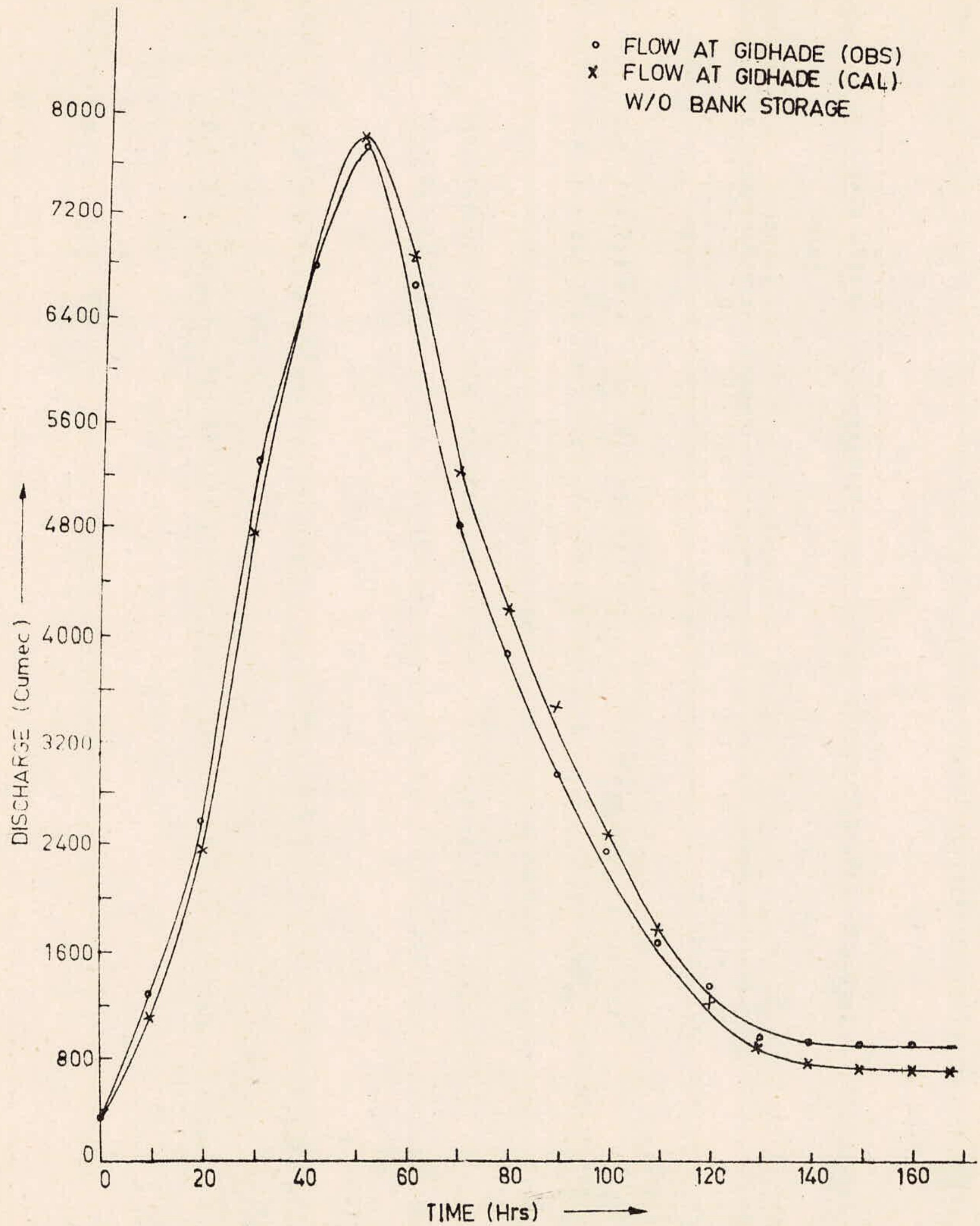


FIG. 6 — OBSERVED AND CALCULATED FLOW AT GIDHADE

TABLE-1 : RESULTS OF SENSITIVITY ANALYSIS

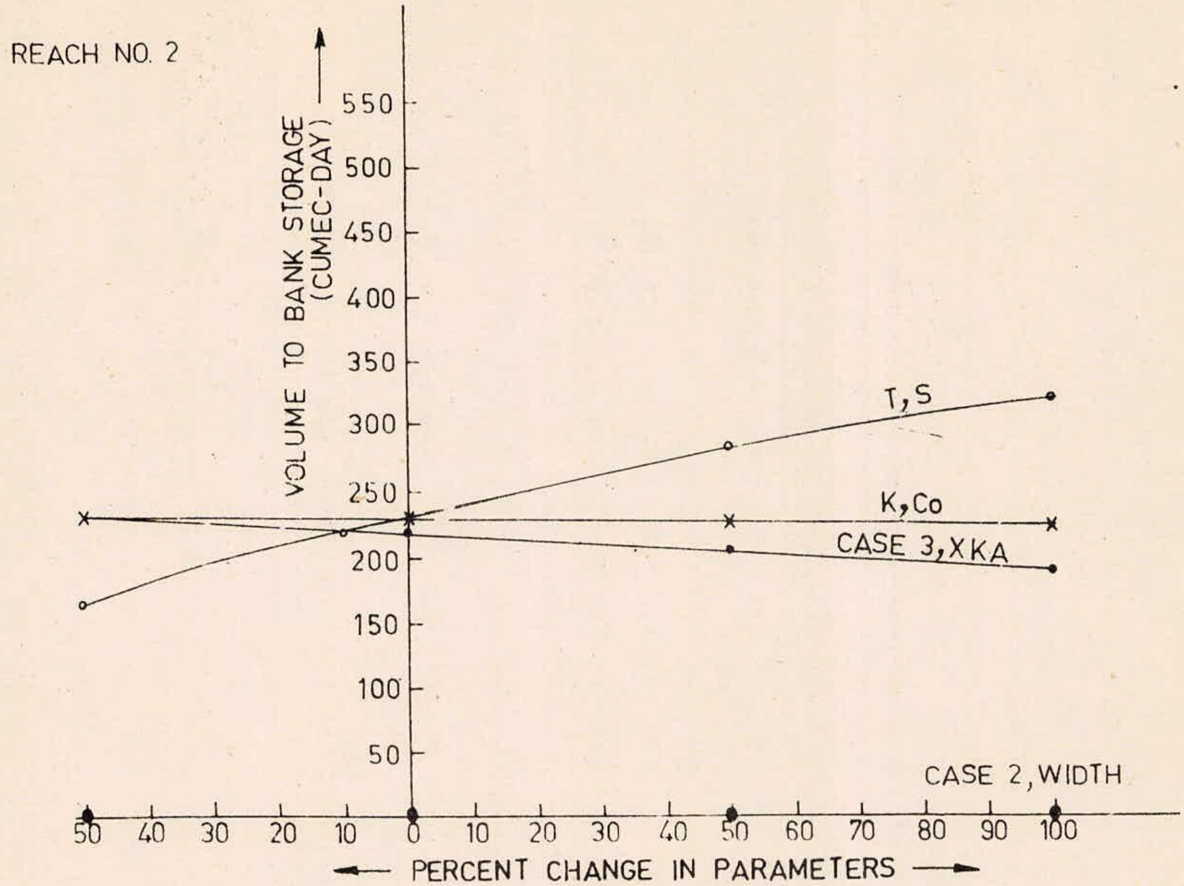
REACH NO. 2

PARAMETERS	VOLUME TO BANK STORAGE (CUMEC-DAY)						PEAK OF FLOW TO BANK STORAGE (CUMEC)						TRAVEL TIME (HRS)			
	CHANGE						CHANGE						CHANGE			
	-50	-10	0	+10	+50	+100	-50	-10	0	+10	+50	+100	-50	0	+50	+100
AQUIFER PROPERTIES :																
T : 210 sq. m/day	163	219	231	243	283	326	251	337	355	373	434	500	4.5	4.5	4.5	4.5
S : 0.03	164	219	231	243	283	327	251	337	355	372	434	501	4.5	4.5	4.5	4.5
CHANNEL PROPERTIES :																
K :3625 sq. m/day	232	232	231	231	231	231	346	346	355	355	355	337	4.9	4.5	4.1	3.8
CO:2.58 m/sec	233	232	231	231	231	231	320	346	355	355	363	366	7.7	4.5	3.2	2.5
BOUNDARY CONDITIONS : CASE : 2																
WIDTH :1524 m	0	0	0	0	0	2	225	355	355	355	355	355	4.5	4.5	4.5	4.5
CASE : 3 RETARD. COEFF. :61 m	231	226	221	217	207	194	240	183	173	163	134	109	4.5	4.5	4.5	4.5
SOIL RETENTION FACTOR 0			145						235					4.6		
0.5			224						285					4.6		
1.0			303						285					4.6		

TABLE-1 (CONTD.)

REACH NO. 3

AQUIFER PROPERTIES :																
T : 210 sq. m/day	103	138	145	153	178	205	201	271	285	300	348	401	4.6	4.6	4.6	4.6
S : 0.03	104	139	145	152	175	206	202	271	285	299	289	402	4.6	4.6	4.6	4.6
CHANNEL PROPERTIES :																
K :3210 sq. m/day	145	147	145	145	145	146	285	285	285	285	314	314	5.0	4.6	4.5	4.0
CD:2.58	149	147	145	146	146	144	236	285	285	315	337	338	7.9	4.6	3.2	2.5
BOUNDARY CONDITIONS :																
CASE : 2 WIDTH :1524 m	0	1	1	1	1	2	277	282	283	283	285	285	4.6	4.6	4.6	4.6
CASE : 3 RETARD. COEFF. :61 m	146	143	139	137	129	121	181	133	125	118	97	77	4.6	4.6	4.6	4.6
SOIL RETENTION FACTOR																
0			145						285					4.6		
0.5			224						285					4.6		
1.0			303						285					4.6		



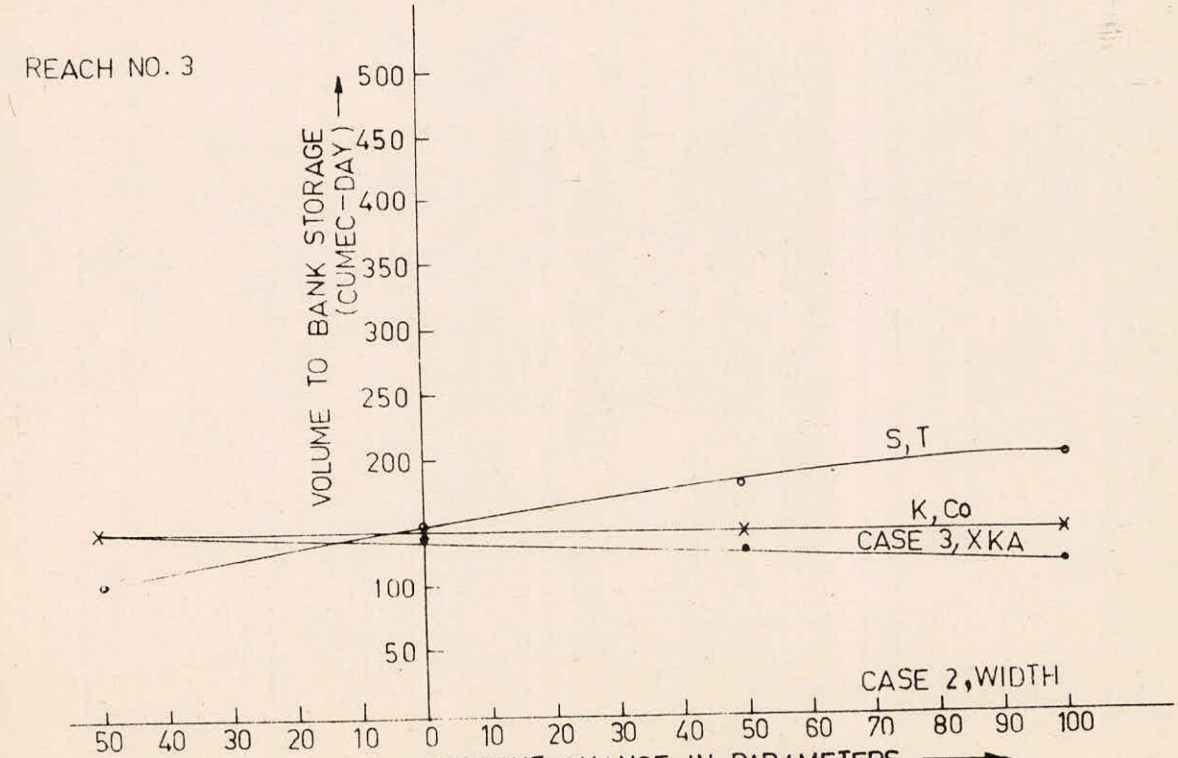
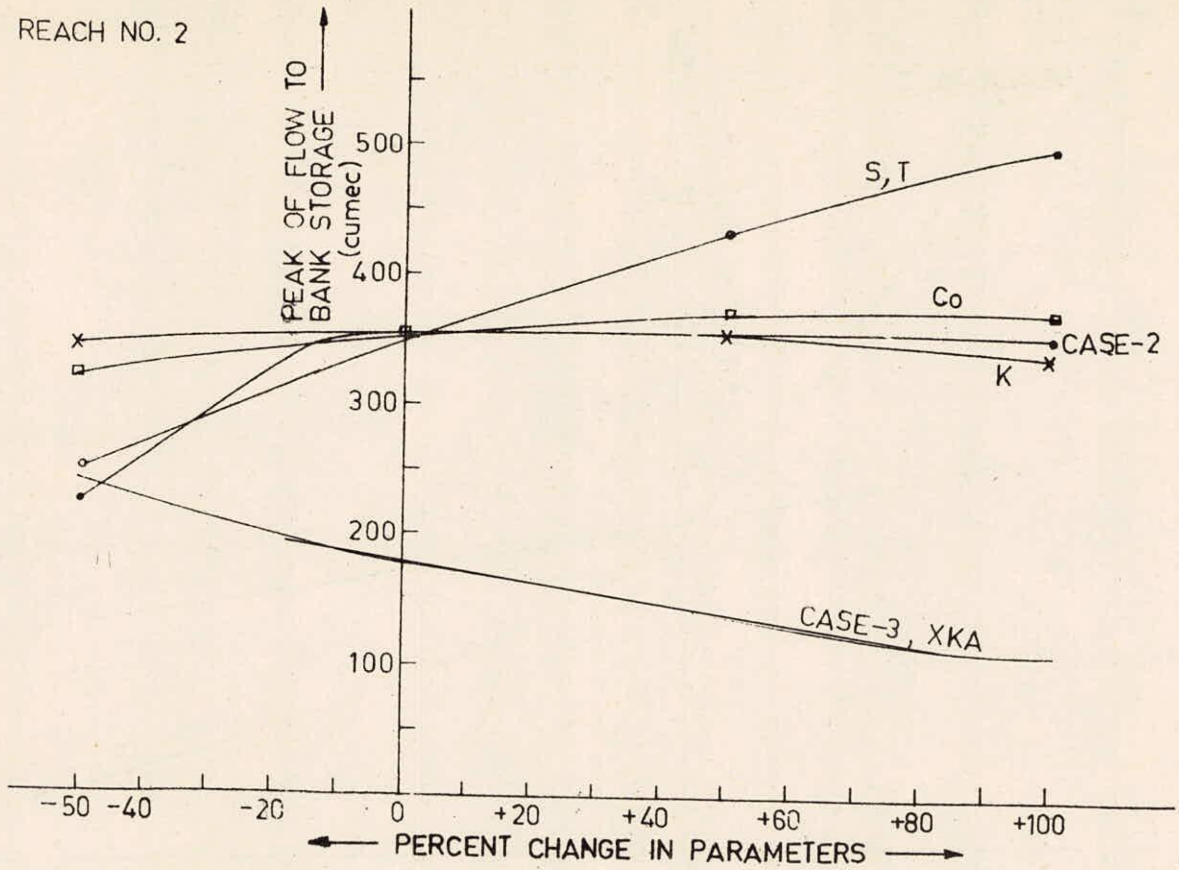


FIG 7— EFFECT OF PERCENT CHANGE IN PARAMETERS ON VOLUME TO BANK STORAGE



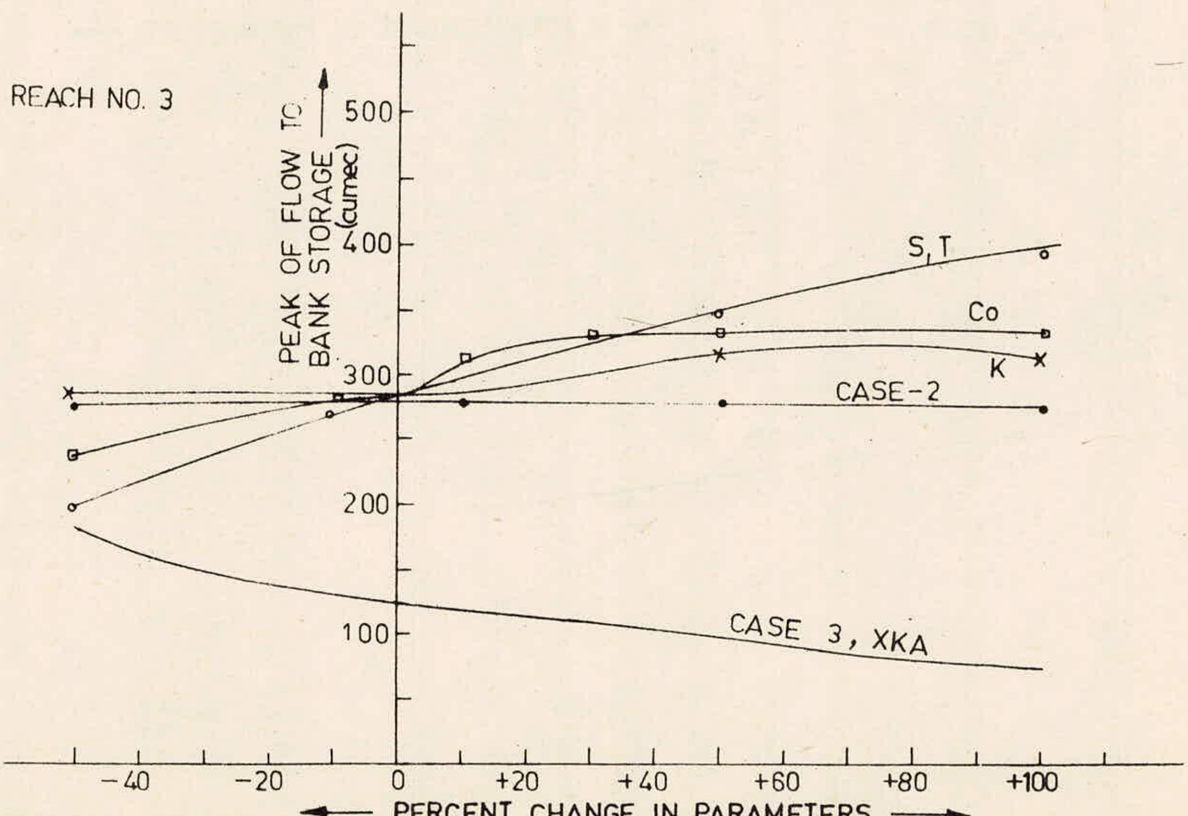
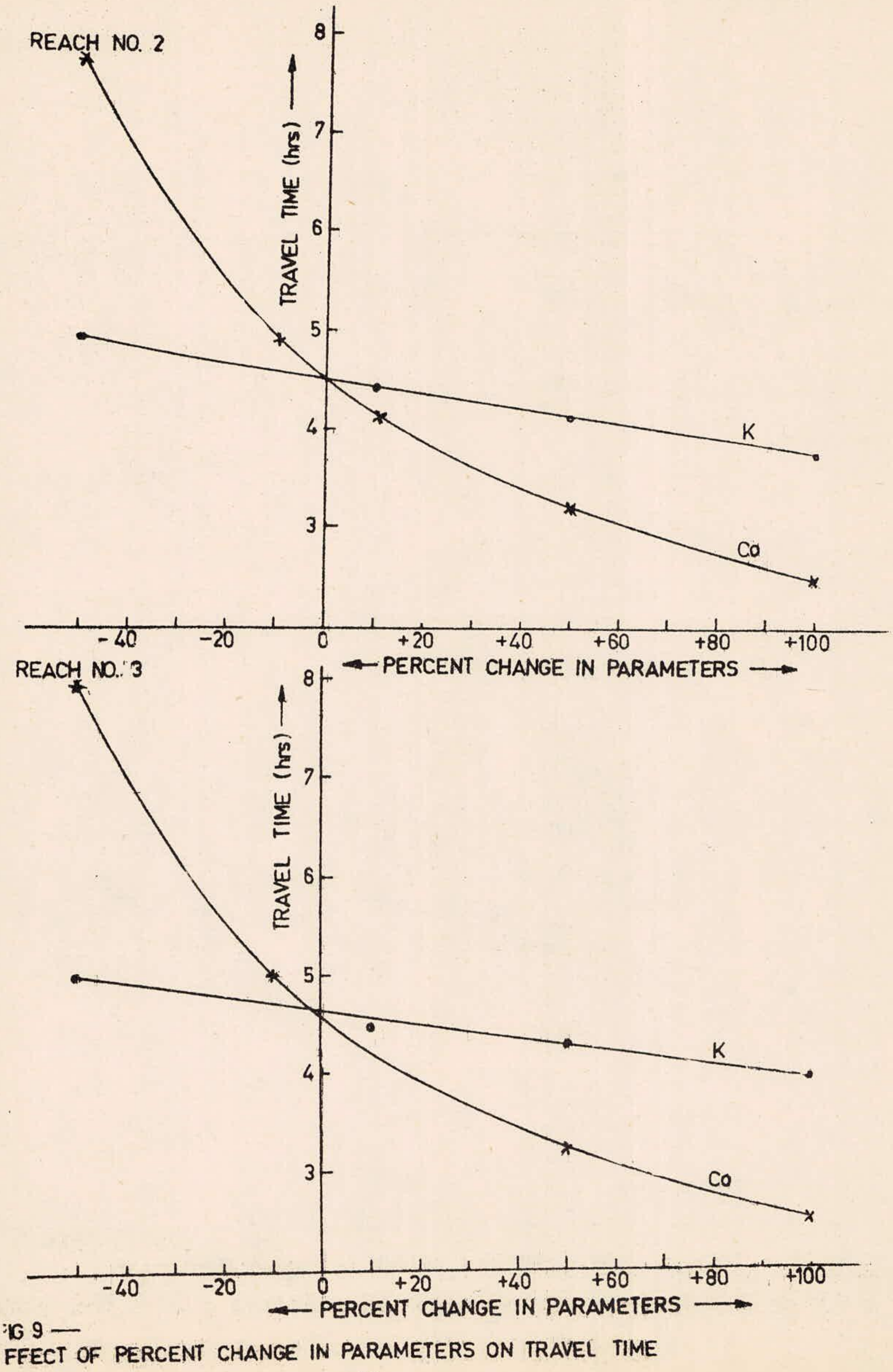


FIG 8 —
EFFECT OF PERCENT CHANGE IN PARAMETERS ON PEAK OF FLOW TO BANK STORAGE



'IG 9 —
FFECT OF PERCENT CHANGE IN PARAMETERS ON TRAVEL TIME

increase in width. Travel time as seen from the Table 1 and graph (Fig. 9) is largely sensitive to wave celerity and wave dispersion coefficient only and insensitive to other parameters. With wave celerity and wave dispersion coefficient, the travel time changes inversely. The travel time is much sensitive to wave celerity than wave dispersion coefficient.

The change with increase or decrease in soil retention factor is seen from Table 1 and graph (Fig. 10). The peak and travel time is not dependent upon soil retention factor. But the volume to Bank storage increases proportionally as the storage in aquifer is increased with increase in soil retention factor.

Some other runs are taken to provide wide concept of the phenomena. For non-route option, keeping the other parameters constant, the change in storage coefficient, changes the pattern of bank storage discharge hydrograph (Fig. 11). Now, the effect of change of transmissivity on the pattern is seen from Fig. 12. It is seen that the bank storage is more dependent upon storage coefficient than the transmissivity of the aquifer. For route option the change in bank storage discharge hydrograph with S and T are shown in Fig. 13 and 14. The negative sign indicates the flow from stream and vice versa. In this case also the bank storage discharge is more dependent upon storage coefficient than the transmissivity of the aquifer.

Some runs were taken to see the effect of well pumpage on the bank storage discharge. The increase or decrease in draft does not make any difference on bank storage discharge. But the

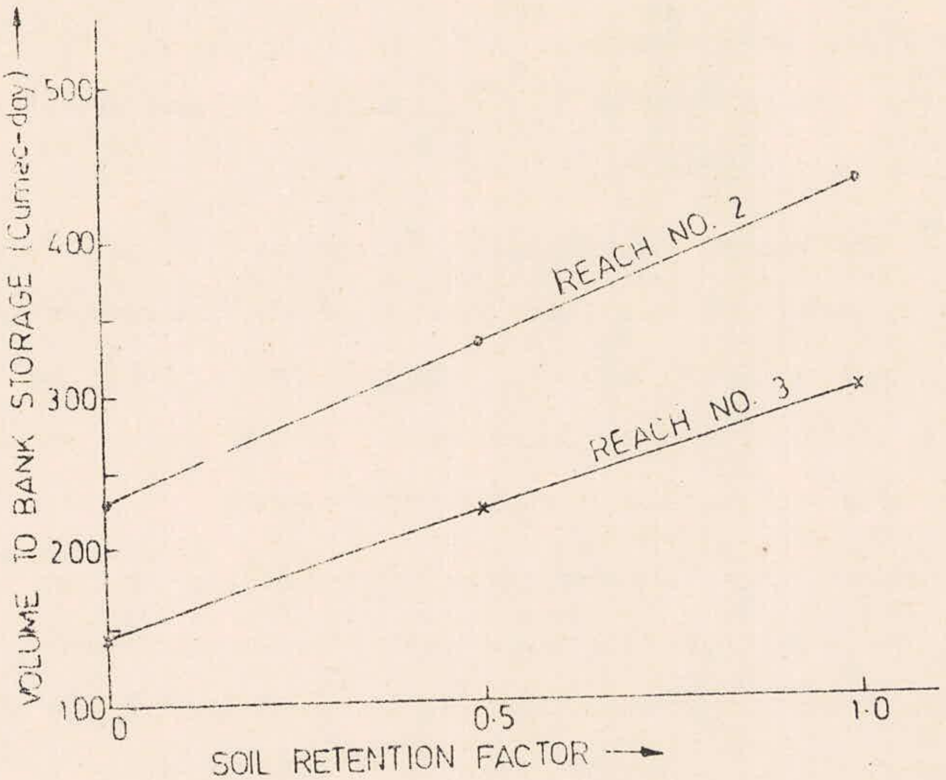


FIG. 10 – CHANGE IN VOL. TO BANK STORAGE WITH S.R.F

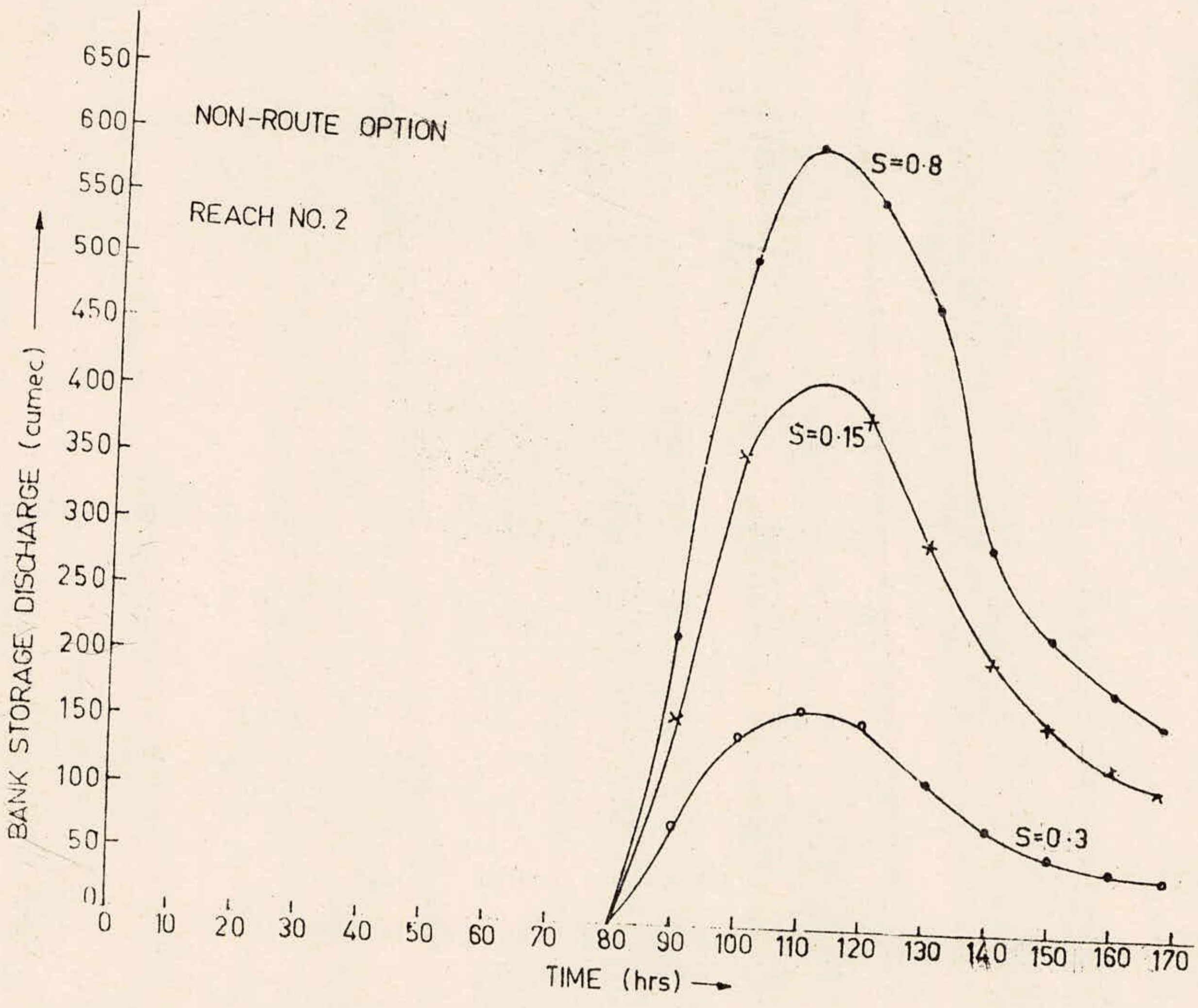


FIG. 11 – CHANGE IN BANK STORAGE DISCHARGE HYDROGRAPH WITH STORAGE COEFFICIENT (S)

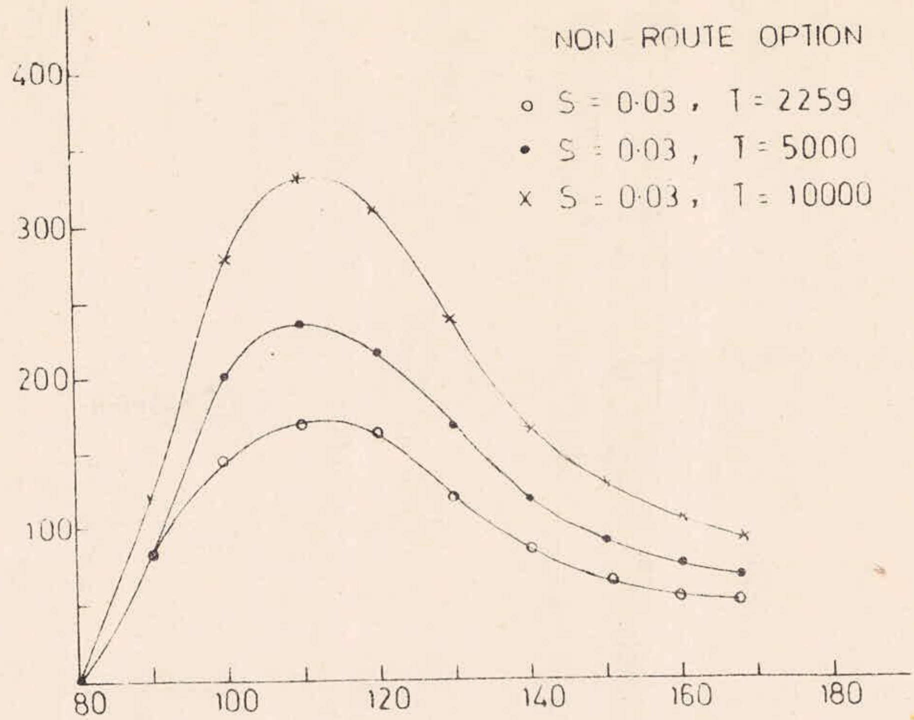


FIG.12 – CHANGE IN BANK STORAGE WITH TRANSMISSIVITY

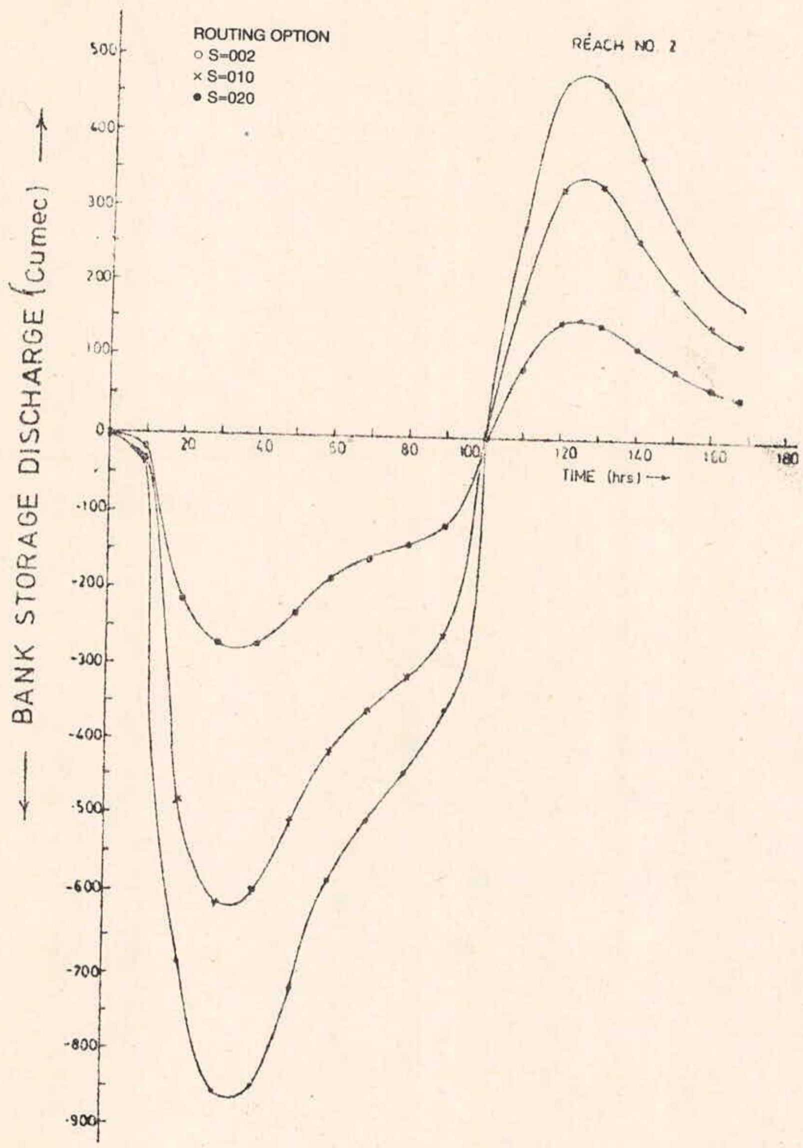


FIG.13- CHANGE IN BANK STORAGE DISCHARGE
HYDROGRAPH WITH STORAGE COEFFICIENT (S)

REACH NO.2

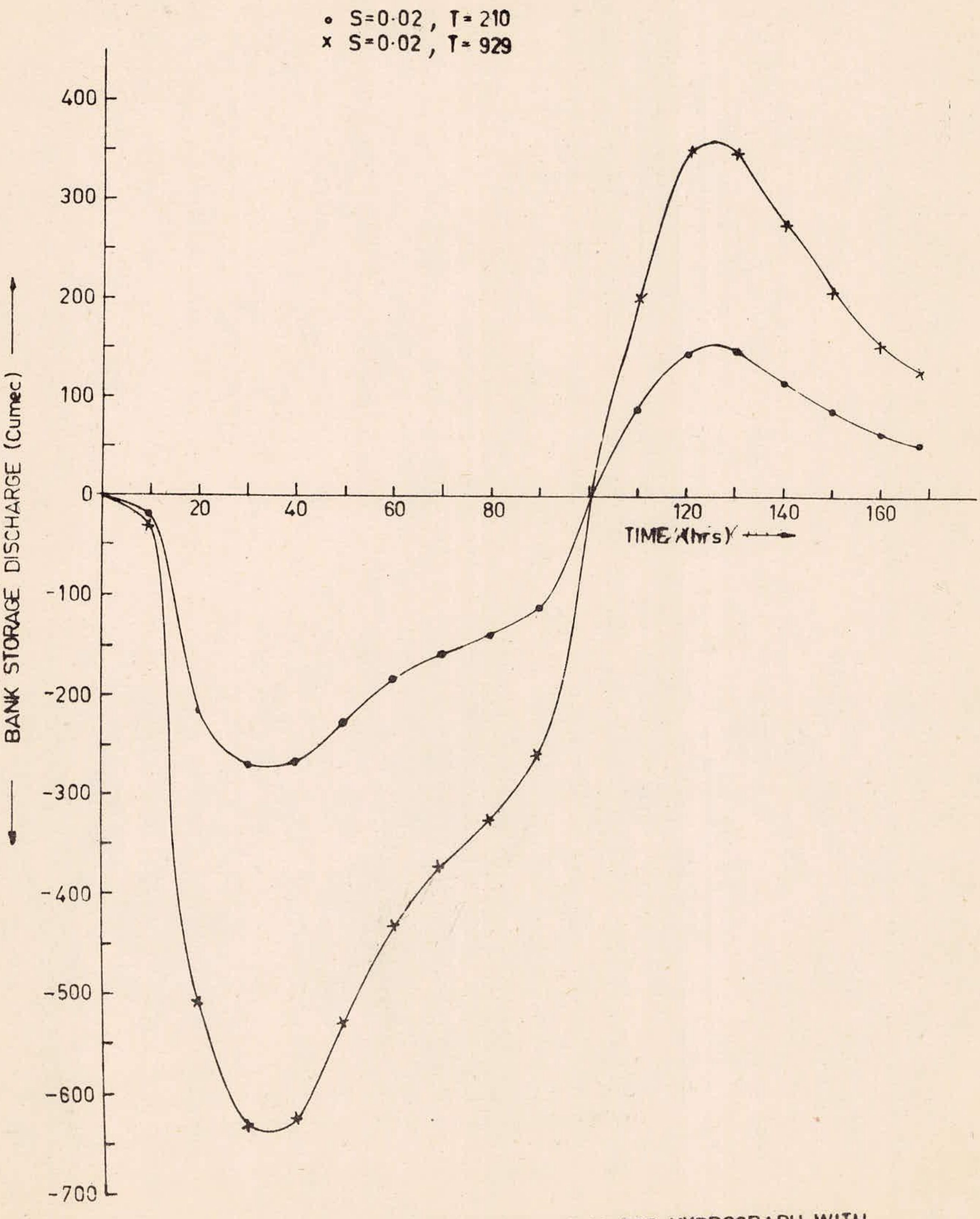
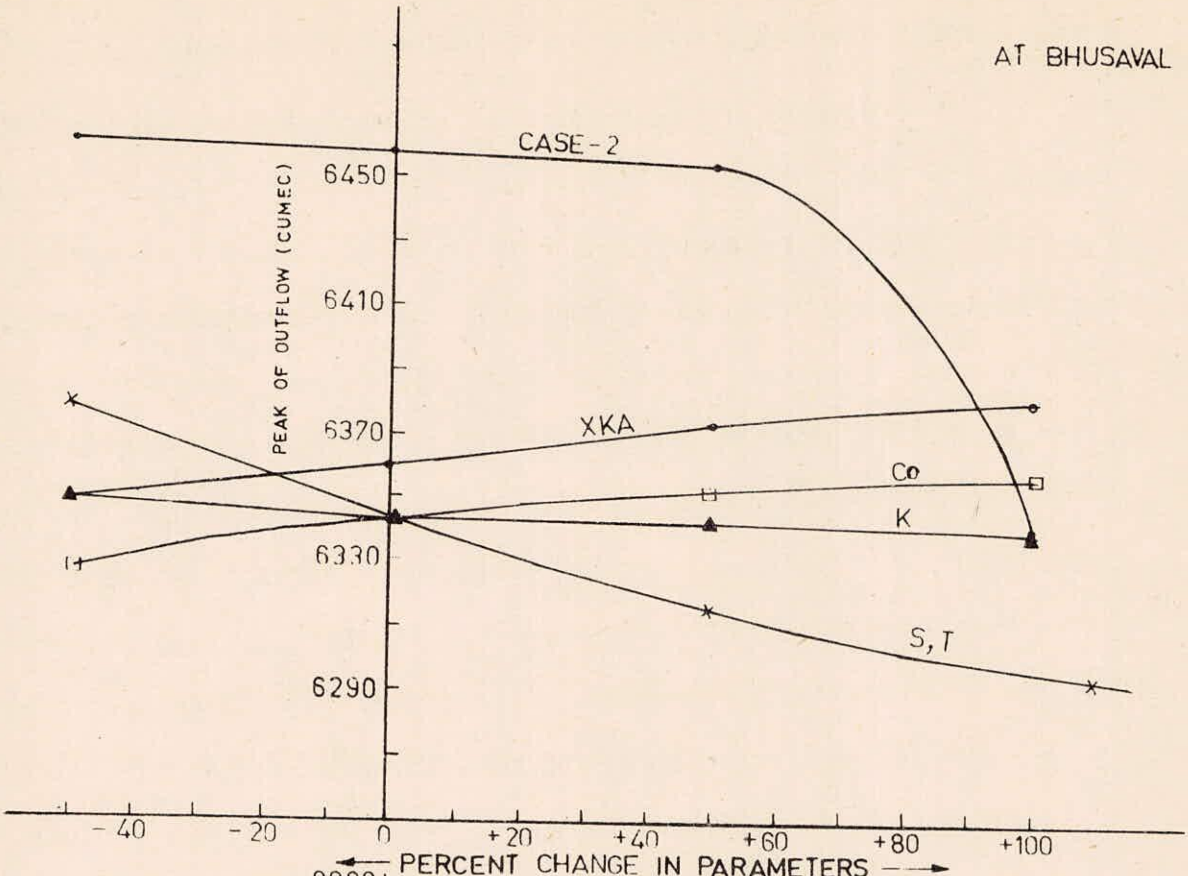


FIG. 14 - CHANGE IN BANK STORAGE DISCHARGE HYDROGRAPH WITH TRANSMISSIVITY (T).

peak/volume of outflow discharge is changed correspondingly.

The effect of the variation in parameter values is also seen on the peak of outflow hydrograph (See Fig.-15). The wave celerity has the most predominant effect on attenuation. The peak of outflow increases/decreases as the wave celerity increases/decreases. On the other hand the wave dispersion coefficient has negligible effect on attenuation. The peak of outflow discharge is inversely proportional to S and T values because of the bank storage discharge. As the value of S or T increases the bank storage discharge increases and hence the peak of outflow decreases. For the case of finite aquifer the peak of outflow hydrograph is increased significantly. This is because of the lesser bank storage discharge. But upto ± 50% of variation of the width of aquifer, there is negligible change in the peak but as the width increases more than + 50% the peak discreases rapidly. As the width is further increased the finite aquifer is treated to be semi-infinite aquifer. For case, when the aquifer is semi-finite aquifer with permeable confining bed separating the stream and the aquifer, the peak of outflow hydrograph is increased in comparison to the peak in semi-infinite aquifer because of the impedence caused by the confining bed to the flow going into the aquifer. Change in travel time does not affect the peak of the outflow hydrograph as it is calibrated in the model on the basis of the values of wave celerity and wave dispersion coefficient.



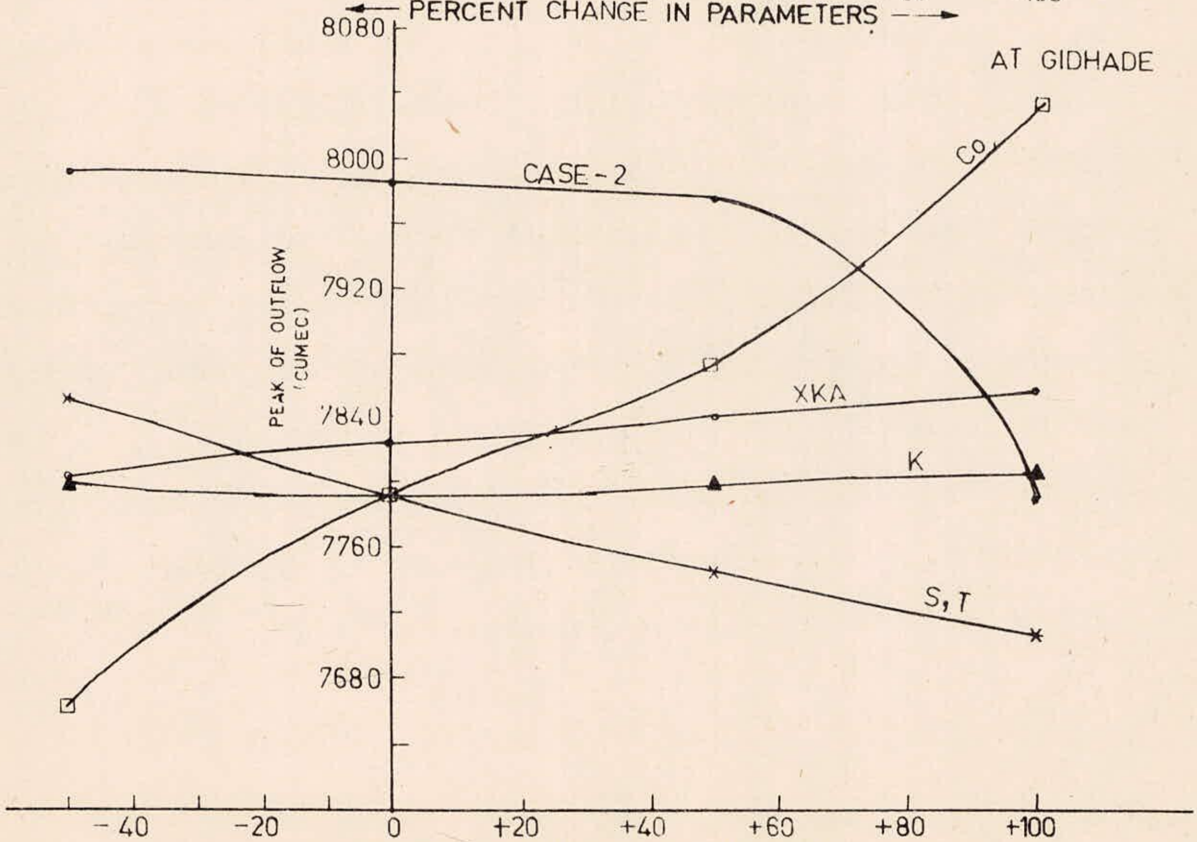


FIG. 15
EFFECT OF PERCENT CHANGE IN PARAMETERS ON PEAK OF OUT FLOW.

9.0 CONCLUSION

Routing process is significantly affected by the flow interacted between stream and aquifer. Seeing the results discussed in the previous section, it can be concluded that the bank storage is much more dependent upon the values of storage coefficient and transmissivity. To apply the model in alluvial rivers particularly when lean flow occurs in the stream, it is necessary to have reliable values of these parameters because these governing parameters may change the sciene of the flow. As seen from Fig. 8 wave celerity also has significant effect on the volume of bank storage hence a reliable value of it should be chosen. The wave dispersion coefficient has negligible effect on the bank storage hence this can be within 90% accuracy.

The peak of the outflow hydrograph is highly dependent upon the wave celerity. Hence the value of the wave celerity should be chosen very carefully. While wave dispersion coefficient has negligible effect on the peak of outflow and hence similar weightage can be given to the parameter.

For finite aquifer the width of the aquifer should be reliable as for a larger width the finite aquifer is treated to be semi-infinite aquifer.

For the case when the aquifer is semi-infinite and the stream is separated from the aquifer by a confining bed, the value of retardation coefficient should be accurate enough as the outflow peak/volume is affected by it proportionally.

The analysis presented earlier has been made using flood data of River Tapti. A more significant effect of the interaction of stream water with ground water on stream routing would have been seen if the model has been applied to the pre-monsoon (lean flow) data. In the latter case there would have been significant storage in the aquifer in comparison to the flow in the stream.

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DIRECTOR	DR. SATISH CHANDRA
SCIENTIFIC STAFF	SURENDRA KUMAR S M SETH
DOCUMENTATION STAFF	Y.P.S. VERMA