FLOOD ROUTING IN A TYPICAL REACH OF NARMADA RIVER

A DISSERTATION

submitted in partial fulfillment of the
requirements for the award of the degree
of
MASTER OF ENGINEERING
in
WATER RESOURCES DEVELOPMENT

By
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CANDIDATE'S DECLARATION

I hereby declare that the work which is presented in this Dissertation entitled **"FLOOD ROUTING IN A TYPICAL REACH OF NARMADA RIVER"** in partial fulfilment of the requirement for the award of the degree of MASTER OF ENGINERING IN WATER RESOURCES DEVELOPMENT (Civil) submitted in Water Resources Development Training Centre, University of Roorkee, Roorkee, is a record of my own work carried out during the period from July 16^{th}, 2000 to December 16^{th}, 2000 under supervision of Dr. U.C. Chaube, Professor, WRDTC, University of Roorkee, Roorkee, and Dr. P.K. Mahapatra, Scientist, 'B', NIH, Roorkee.

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

(BISWAJIT SAHA)

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

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CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

X = Distance along the longitudinal direction.

t = time co-ordinate.

A = Cross-sectional area.

h = flow depth in meter.

Q = Discharge in Cumecs.

q = Lateral inflow per unit length of channel.

g = Acceleration due to gravity.

S_o = Bed Slope.

S_f = Friction slope.

n = Manning's roughness Co-efficient.

R = Hydraulic radius.

ΔX = Spacing parallel to the t-axis represent location of cross- sections.

Δt = Spacing parallel to the x-axis represent time line.

i,j = Subscript for numerical rectangular grid identification designates the x position and particular time level.

K = Represents any variable Q or A.

θ = Weighting factor.

N = Total numbers of nodes.

ABSTRACT

Flood Routing is a technique by which a flood hydrograph at any section in a channel is determined from a known hydrograph at some upstream section in the same channel. It is important in flood forecasting, design of hydraulic structures estimation of sediment /pollution transport. In this work, reaches in Narmada River are attempted to perform the flood routing studies. For the purpose, a computer program for channel routing is used. The equations governing the flow are One-dimensional Saint Venant equations and this solved numerically by four point preissmann's scheme. However, the main limitation in the model is that the channel bed is assumed rigid. Required data for river cross-sections and observed hydrographs at different locations are collected from various sources. In the present study, first, the model parameters are calibrated using an indirect method for conservation of water flow. Flood routing in Narmada from Mortakka to Mandleswar is performed for five different flood events. The contributions to the flow from intermediate regions are considered by assuming four tributaries at equal intervals between the measurement sites. Flood routing computations show that the computed and observed hydrographs at Mandleswar match satisfactorily. In addition, generalized equations are derived for the peak discharge and time to peak discharge. These equations can predict well, the flood peaks and time to attain the peak at any place between Mortakka and Mandleawar, using the peak and time to peak values of hydrograph at Mortakka. In the study for the reach from Jamtara to Bermanghat, a methodology is developed to estimate the discharge hydrograph in two tributaries, Hiran and Sher using the observed hydrographs at Jamtara and Bermanghat. Taking the tributary discharges into account flood routing computations show the computed hydrograph and observed hydrograph at Bermanghat match satisfactorily.

Chapter-1

INTRODUCTION

1.1 GENERAL

Rivers are associated with human civilization since time immemorial. However, all rivers exhibit floods causing great losses to lives and properties. A flood is an unusually high stage in a river - normally the level at which the river overtops its banks and inundates the adjoining area (Subramanya 1998). Such an event may be generated by intense rainfall, snowmelt and/or collapse of dam. The general characteristics of a flood flow are: (a) high discharge, (b) flows in flood plains, (c) change of river course, (d) sediment transport, (e) three dimensional flow, and, (f) turbulence. It is important to analyze the flow in a river during floods. Flow computation during flood is very much essential for design of hydraulic structures, flood forecasting, preparing flood inundation maps, estimation of sediment and pollution transport and river channel improvements. The main concern to a hydraulic engineer is the modeling of the movement of abnormal amount of water along a river.

Propagation of flood waves in rivers shows several distinctive phenomena; (a) translation, (b) attenuation, and (c) distortion. Translation of the flood is simply the recognition that the peak of the disturbance normally propagates in the downstream direction. This is coupled with the notion that the bulk of the water is also moving in the downstream direction. However, it is important to distinguish between the speed of propagation of the disturbance and the speed of the bulk of the water. The speed of the flood wave depends on parameters such as depth, width and velocity of the flow. In particular, a flood wave that is in-bank will travel faster than a flood that is over-bank and inundates a wide plain. Attenuation of a flood is a decrease in the peak between inflow and outflow. Moreover, it happens due to storage detention, withdrawal and roughness characteristics of channel. Generally, in-bank flood experiences less attenuation than over-bank flood. The peak discharge of outflow may be more than that of the inflow if there is considerable amount of lateral flows into the main channel. In this case, the slope and shape of the basin play an important role. Distortion refers to change in shape of the wave profile i.e. discharge hydrograph at upstream and downstream. It takes place due to storage between upstream and downstream, and, differences of wave speed in in-bank and out-bank.

The computation of the height and velocity of a flood as it propagates in a body of water is referred to as flood routing (Chaudhary, 1993). Flood routing is a technique by which a flood hydrograph at any section in a channel is determined from a known hydrograph at some upstream section in the same channel. Based on the flow condition, there are two broad categories of routing, viz., (a) reservoir routing, and, (b) channel routing. In reservoir routing, the effect of a flood wave entering a reservoir is studied to propagate the variations of reservoir elevation and outflow with time. In channel routing, the change on the shape of hydrograph as it goes down a channel is studied. Information on the peak discharge attenuation and computing peak time are obtained by channel routing. In channel routing, the water surface slope is assumed non-zero whereas it is assumed zero in case of reservoir routing.

Channel routing can be performed in three methods; (i) experimental, (ii) analytical, and, (iii) numerical. The advantages and disadvantages of the above methods are given in Table 1·1 (Anderson et al. 1984). Analytical methods are only for highly idealized cases and generalized closed form solutions are not possible due to the complexities involved with the governing equations. Experimental studies are very helpful in a better understanding of the flow phenomenon. However, after the advent of fast computers and availability of robust numerical techniques, it becomes popular among the engineers to use numerical modeling for studies in flood routing. It may be noted here that numerical models can never replace experimental models.

In most of the numerical models, channel routing is done solving the one-dimensional Saint Venant equations. However, due to complexities of the Saint Venant equations, various simplified approximations of flood wave propagation are also in use. Mamood and Yevjevich (1975) have presented an excellent review of such methods. Weinmann, (1979) also, presented a review on approximate flood routing methods. The simplified methods may be categorized as (a) empirical, (b) linearization of the Saint Venant equations, (c) hydrological, (d) simplified hydraulic and (e) complete hydraulic (Fread, 1985). Empirical methods are limited in application with sufficient observations of inflows and outflows to calibrate the essential coefficients. They require minimum computational resources. Some empirical models are; (a) lag models, and (b) gauge relations. In lag models, concept of lag, this is defined as the difference in time between inflow and outflow within the routing reach. In gauge relation, the flow at a downstream point to that at an upstream station is related. It is based on flow, water elevations and combination of each. Linearized models ignore the least important terms of momentum equation assuming that cross section is rectangular, bottom slope are constant, no lateral inflow and the friction slope term is linearized with respect to velocity and depth.

Table 1.1 : Advantages and disadvantages of various methods of channel routing

APPROACHES	ADVANTAGES	DISADVANTAGES
Experimental	1. Capable of being most realistic.	1. Equipment's required. 2. Scaling problems. 3. Measurements difficulties. 4. High operating cost 5. Time consuming
Analytical	1. It is handy and general information is in formula form. 2. Used for preliminary studies. 3. Computers are not essential.	1. Restricted to simple geometry and physics. 2. Usually restricted to linear problems.
Numerical	1. No restriction to linearity. 2. Complicated physics can be treated. 3. Time evolution of flow can be obtained. 4. Less time for analysis.	1. Truncation errors. 2. Boundary condition problems. 3. Computer cost.

In hydrologic routing methods, conservation of mass (continuity equation) and an approximate relation between flow and storage is used. Following are some of the hydrological models (a) Muskingum model, (b) Muskingum-Cunge model, and (c) Kalinin-Milyukov model.

Simplified hydraulic routing methods use the continuity equation and a simplified form of momentum equation. If the simplified momentum equation is the steady uniform flow equation, the routing procedure is *kinematic routing* and if an additional term for the slope of the water surface is included, the method is called *diffusion routing*. In the complete hydraulic routing, all the terms of the momentum equation are used. Sometimes it is also referred as *dynamic routing*. The above routing methods are in increasing order of accuracy.

For the numerical solution of the partial differential equations governing the flow, following methods may be used; (a) Method of Characteristics (MOC), (b) Finite Difference Method (FDM), (c) Finite Element Method (FEM), (d) Finite Volume Method (FVM), (e) Finite Analytic Method (FAM), and (f) Spectral Method (SM). In MOC, the governing equations are converted into a pair of ordinary difference equations and then are solved by finite difference scheme. In FDM, the governing equations are replaced by an equivalent set of algebraic equations and are then solved numerically. Of the various numerical methods, FDM have been used very extensively. It may be either explicit or implicit. In the explicit scheme, the differentials are expressed as a function of variables at known time level where as in the implicit scheme they are in terms of variables in an unknown time level. In FEM, the system is divided into a number of elements and partial differential equations are integrated within the elements. FVM and FAM are variations of the FDM. SM is appropriate for problems with periodic boundary and studies using SM for channel routing show no significant benefits over other methods (Sinha et al., 1995).

Out of the many computational models for channel routing, the channel routing model by Fread (1988) is widely used by field engineers and researchers. This uses one-dimensional Saint Venant equations as the governing equations. The numerical solution is based on the Four-point Preissmann Scheme, an implicit Finite Difference Method. However, the points that make it different from other models are; ease in data preparation, consideration of field situations and computational efficiency.

1.2 REVIEW OF LITURATURE

In this section, a brief review on the earlier studies concerning flood routing is presented. In addition, previous works on Narmada Basin are also given. As mentioned earlier, flood routing can be performed by any one of the three methods, viz., analytical, experimental and numerical. As the present work uses a numerical method, the following paragraphs are with respect to numerical studies only.

1.2.1 Equations Used in Flood Routing

The most important component of a numerical model is the mathematical equations used to represent the flow phenomena. As seen in the literature, there are a large number of equations available for channel routing. Three-dimensional Navier-Stokes equations provide a complete description of fluid flow at any Reynolds number. These are for unsteady flows and represent the most generalized equations. These equations use the assumption of a Newtonian fluid. Direct solution of these equations can describe a turbulent flow. Due to lack of computer space and speed, these equations cannot be solved directly for large Reynolds numbers. Navier-Stokes equations can be simplified to Reynolds equations where the velocity and pressure terms are divided into their average and fluctuating parts. In addition, some sort of turbulence modeling is performed to evaluate the Reynold stresses. Numerical solutions, for Navier-Stolkes equations, based on various techniques (Lagrangian approach, Eulerian approach, and, Mixed Lagrangian-Eulerian approach) are available in the literature (Hyman 1984). Various turbulence models are described by Rodi (1984).

The theory that incorporates vertical accelerations, to a limited extent, in approximations to the horizontal motion equation is called Boussinesq Theory (Boussinesq 1872). Many forms of the equations attributed to Boussinesq are found in literature. Variations are due to the order of accuracy of terms retained and methods of derivation. Boussinesq Equations are also known by various other names, such as *Serre Equation*, *Perigrine Equation* etc. Boussinesq theory can be applied to finite amplitude quasi-long waves propagating in shallow water. Some studies using Boussinesq equations are by Gharanzik and Chaudhry (1991), Carmo et al. (1993).

Assuming the velocity to be uniform along a vertical direction and integrating along the depth, two-dimensional Saint Venant Equations can be derived from three-dimensional Navier-Stokes equations (Lai 1997). These equations can be further simplified to one-dimensional equations, neglecting the variations in transverse direction. Assuming a radial flow, Townson and Salihi (1989) used one-dimensional Saint-Venant equations in radial coordinates. Most of the numerical studies in flood routing use Saint-Venant equations (Chaudhry 1993). Vertically Averaged Moment (VAM) Equations are depth averaged equations and can be derived by taking the moment of the momentum equations (Khan and Steffler 1996). As suggested by Paterson and Apelt (1988) depth averaged equations can be derived by assuming various forms of distribution for the velocity in a vertical plane. For example, they have derived equations for four different types of assumptions.

1.2.2 Flood Routing

Technical papers on flood routing are many in numbers. Only, some references, which are relevant to the present work, are cited in the following paragraphs.

Palaniappan et.al.,(1986),They have used a Muskingum-Cunge method for flood routing in Narmada river from Mortakka to Gurudeswar.

Ragan (1966) investigated the numerical flood routing technique for channels subject to lateral inflows, in laboratory. He faced one of the practical problems found with the numerical technique. The results were highly sensitive to the values of the Mannigs roughness coefficient used to describe the channel roughness. He described that as the percent error in the roughness co-efficient increased, the time required for the converge the actual hydrograph increased.

Amein et al. (1970) attempted to solve the unsteady flow equations by the method of characteristics and it was presented for stream flow routing. The procedure was applied to the routing of a flood hydrograph through a channel. He found from that the friction forces alone can reduce flood peak and flatten out the rising and recession limbs of the hydrograph. Moussa and Bocquillon (1996) developed a modified form of diffusive equation considering the two parameters of the equation. These parameters are celerity and diffusivity, which are the function of discharge. Keskin and Agiralioglu (1997) developed a simplified dynamic model with a new form of momentum equation consisting of cross sectional area and discharge to solve the Saint Venant equation for flood routing. The channel cross-section was assumed rectangular with a constant width and an explicit finite difference formulation was used. The results were compared with a dynamic model, which gave satisfactory results. Thus, they showed that the simplified dynamic model is easier to formulate and compute.

Fread et al. (1997) developed an explicit numerical scheme for the solution of one-dimensional unsteady flow in natural rivers. It was tested with the implicit scheme, which gave accurate results. In some situations the scheme i.e. implicit and explicit multiple routing was introduced to incorporate the advantages of using both the schemes. Ping et al. (1999) developed a hydraulic flood routing method for multibranch rivers based on the double sweeping method by means of imaginary channel length for the simultaneous solution of the implicit finite difference scheme of Saint Venant equations. The model provides an effective tool to handle the problem of flood routing for multibranch rivers and to treat the problem of weir control on flood routing.

1.2.3 Narmada Basin

In the comprehensive study of Narmada river basin, several studies of Narmada had been done at National Institute of Hydrology (NIH), for investigating the hydrological, hydrometrological and physical characteristics of the difference sub basins. The rating curve had also been developed for some of the gauging sites located in the Narmada River. Singh R.D., (1999). S.M.Seth et.al.(1985), They have evaluated rating curves for some of the gauging sites on Narmada River.

Seth et al. (1990) applied the SHE model to Narmada Basin from its origin upto Mannot. All land phase components of hydrologic cycle were considered either by finite difference representation of the partial differential equation of mass, momentum conservation form or in the form of empirical equations. Seth et al. (1990) has evaluated

SHE model, which is also a deterministic, distributed and physically based hydrological modeling upto Barna basin. SETH et al. (1990) have evaluated a SHE model, a deterministic and physically based hydrological modeling to Hiran sub basin of Narmada which can make use of all available information i.e. topography, soil and land use and a new ideas on hydrological process. Jain et al. (1990) application of SHE model to Kolar sub-basin of river Narmada has been described. The SHE model is a deterministic and physically based model where in land phase components of hydrologic cycle mainly considered. Tripathi et. al.(1992) In this paper agriculture water quality issues on farm irrigation management, aquifer recharge and discharge, have been discussed.for Narmada basin.

Choubey et al. (1995) carried out a land capability classification in a part of Narmada considering the texture, depth, permeability and salinity of the soil, which is very important for proper management of agricultural land. Jain et al. (1995) evaluated a fluvial geomorphologic characteristic of four sub-basins of upper Narmada for developing the hydrological models to simulate hydrological response of the basin, which was widely used for simulating hydrological response of ungauged basin. Goel et.al. (1998), have developed a systematic methodology for multivariate modeling of flood flows and application and validation of this methodology have been made by using daily flows of Narmada river in Garudeswar.

From review of papers related to Narmada river, it is seen that most of the work done so far is based on rainfall and runoff, stage-discharge rating relation, flood routing using Muskingum-Cunge method and etc. In this work, a numerical model using complete Saint Venant equations, used to route the flood and generalized equations for the peak discharge and time to peak discharge have been formulated. A methodology is also developed for finding out the tributary outflow using the observed flows at nearby places in the main river.

OBJECTIVES AND ORGANIZATION OF STUDY

The objectives of the present work are; (1) calibrating the parameters of the flood routing model. (2) flood routing in Narmada from Mortaka to Mandleswar, (3) developing a generalized equation for peak discharge and time to peak and apply it for the above study reach, and (4) developed and apply a methodology for estimating the discharge hydrograph of tributaries (Hiran and Sher) from observed inflow and outflow hydrographs of a river reach(Narmada at Jamtara and Berminghat).

Importance of flood routing and review of literature relevant to the present work was presented in this chapter. Mathematical model using the governing equations and their numerical solution is presented in chapter two. In chapter three, the study area is described. Results of the present study are presented in chapter four and important conclusions and recommendations for future work are given in the last chapter.

Chapter 2

MATHEMATICAL MODELING

In the previous chapter, importance of flood routing in various engineering applications, review of literature and objectives of the present study have been presented. In this chapter, mathematical modeling using partial differential equations governing the flow and their numerical solution is presented.

2.1 GOVERNING EQUATIONS

In the present work, one-dimensional Saint Venant equations in rectangular coordinate system are used as the governing equations. These equations are (Chaudhry 1993):

Continuity Equation:

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}-q^{*}=0 \tag{1}$$

Momentum Equation:

$$\frac{\partial Q}{\partial t}+\frac{\partial}{\partial x}\left(\frac{Q^2}{A}+\frac{gAh}{2}\right)=gA\left(S_o-S_f\right) \tag{2}$$

In the above equations, x and t are the distance coordinate along the longitudinal direction and time coordinate, respectively. A is the cross-sectional area of flow, h is the flow depth, Q the discharge of flow and q is the lateral inflow per unit length of the channel. g is the acceleration due to gravity and S_o and S_f are the bed slope and friction slope, respectively. A definition sketch is presented in Fig.2.1.

Equations 1 and 2 are obtained making the following assumptions (Chaudhry 1993).

(a) Channel bed slope is small.

(b) Pressure distribution at a section is hydrostatic along vertical direction. This is true if the vertical acceleration is small, i.e., the water surface variation is gradual.

(c) The friction losses in unsteady flows may be computed using formulas for the steady state friction losses.

(d) The velocity distribution at a channel cross section is uniform.

(e) The channel is straight and prismatic.

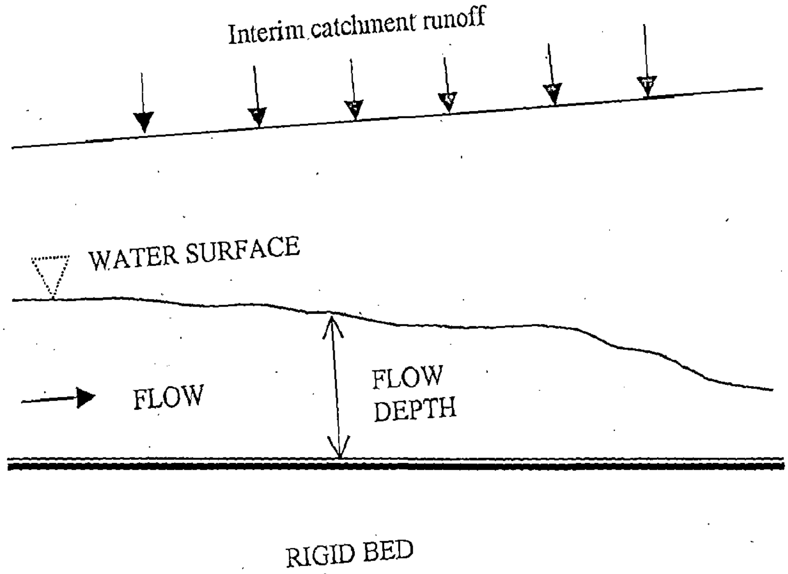


Figure 2.1 Definition sketch for flow in a channel

The above assumptions are valid for most of the gradually varied flow situations. However, the governing equations do not account for the effective stresses, which arise due to (i) laminar viscous stresses, (ii) turbulent stress and (iii) stress due to depth averaging.

Extra turbulent stress-like terms appear while depth averaging the momentum equations because of the non-uniformity of the velocity in the vertical direction. Based on experiments in the laboratory channels, Odggrd and Bergs (1988) have shown that the error introduced by uniform velocity assumptions is negligible. Flokstra (1977) also showed that away from walls the effective stresses are dominated by the bottom stress. However, it should be noted that these effective stresses should be considered while simulating circulating flows (Flokstra 1977). Few models are available simulating effective stresses using turbulence closure models. However, these methods as applied in open channels (Rastogi and Rodi 1978) are at best successful for fixed bed channels. In addition, their application may be required only if one is interested in three-dimensional flow structure and actual sediment movement.

The friction slope S_f used in Eq.2 is calculated using the Manning's equation.

$$S_f = \frac{n^2 Q^2}{A^2 R^{\frac{4}{3}}} \qquad (3)$$

In Eq. 3, n is Manning's roughness coefficient and R is the hydraulic radius represented by the ratio of area and wetted perimeter.

2.2 NUMERICAL SOLUTION

The governing equations for unsteady channel flows as discussed in the previous section constitute a set of non-linear hyperbolic partial differential equations (Lyn 1987, Chaudhary 1993). Analytical solution for these equations is available only for highly idealized cases. Therefore, they are solved numerically. In this section, a numerical scheme is presented for the solution of the above one-dimensional Saint Venant equations. In the present work, a method developed by Preissman (1961) and earlier adopted by Fread (1988) has been used. This method uses the weighted four point Preissman implicit scheme, a finite difference method, for the numerical solution of the governing equations. The solution strategy is presented in Fig. 2.2.

The Saint Venant equations are hyperbolic partial differential equations having two independent variables, x and t; and, two dependent variables, A and Q. The remaining terms are either functions of x, t, h, and/or Q, or they are constants. Eqs. 1 and 2 are solved numerically by performing two basic steps. First, the partial differential equations are represented by a corresponding set of finite difference algebraic equations and second, the system of algebraic equations is solved in conformance with prescribed initial and boundary conditions.

Equations 1 and 2 can be solved by either explicit or implicit finite difference techniques (Liggett and Cunge, 1975). Explicit methods, although simpler in application, are restricted by mathematical stability considerations to very small computational time steps. Such small time steps cause the explicit methods to be very inefficient. Implicit methods, on the other hand, have no restriction on the size of the time step due to mathematical stability (Preissmann 1961, Amein and Fang 1970,

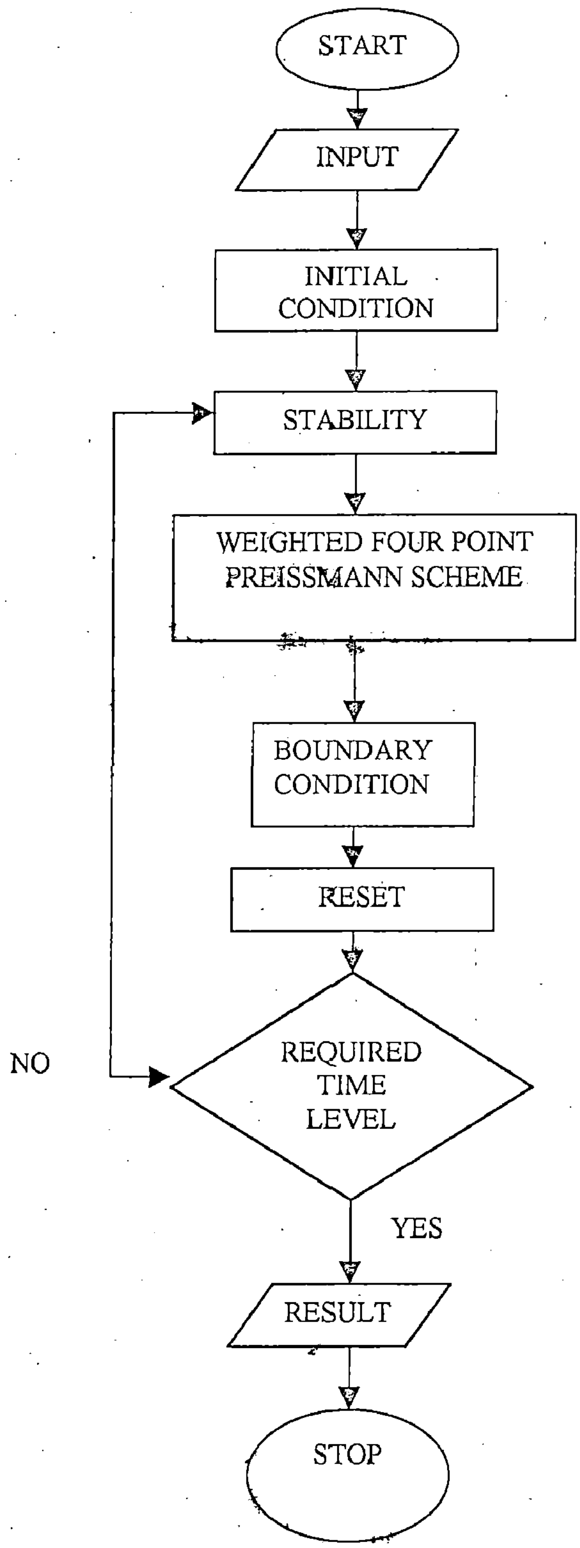


Figure 2.2 Flow chart for the numerical solution

Strelkoff 1970) However, convergence considerations may require its size to be limited (Fread, 1974).

Of the various implicit schemes, weighted four-point scheme used by earlier researchers (Preissmann 1961, Chaudhary and Contractor 1973 and Fread and Lewis 1988) appears most advantageous. Because, it can readily be used with unequal distance steps and its stability – convergence properties can be conveniently controlled. In this scheme, the continuous *x-t* region is represented by a rectangular grid of discrete points. The grid points are determined by the intersection of line drawn parallel to the *x* and *t* axes. Those parallels to the *t*-axis represent locations of cross sections and they have a spacing of ΔX, which need not be constant. Those parallels to the *x*- axis represent time lines and they have spacing of Δt, which also need not be constant (Fig. 2.3).

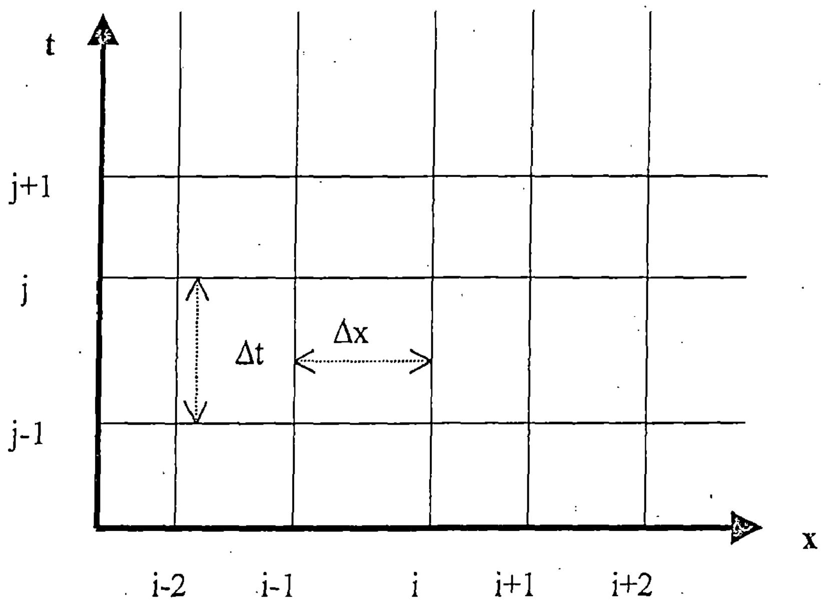


Figure 2.3 Numerical grid in x and t directions

Each point in the numerical rectangular grid can be identified by a subscript *j, and I*, which designates the *x*-position and particular time level. The time derivatives are approximated by a forward difference approximation centered between the ith and i+1th points along the x-axis, i.e.

$$\frac{\partial K}{\partial t}=\frac{K_i^{j+1}+K_{i+1}^{j+1}-K_i^j-K_{i+1}^j}{2\Delta t_j} \tag{4}$$

Where K represents any variable Q, or A. The spatial derivatives are approximated by a forwarded finite difference approximation positioned between two adjacent time lines according to weighting factors of θ and $1-\theta$.

$$\frac{\partial K}{\partial x}=\theta\left[\frac{K_{i+1}^{j+1}-K_i^{j+1}}{\Delta x_i}\right]+(1-\theta)\left[\frac{K_{i+1}^j-K_i^j}{\Delta x_i}\right] \tag{5}$$

Variables other than derivatives are approximated at the time level when the spatial derivatives are evaluated by using the same weighting factors i.e.

In the above equations θ is a weighting factor and selection of θ is important.

$$K=\theta\left[\frac{K_i^{j+1}+K_{i+1}^{j+1}}{2}\right]+(1-\theta)\left[\frac{K_i^j+K_{i+1}^j}{2}\right] \tag{6}$$

$\theta = 0$ is fully explicit and $\theta = 1$ is fully implicit. For $0.55 < \theta < 1$ the scheme is stable. When the finite-difference operators, defined by Eqs. 4-6, are used to replace the derivatives and other variables in Eqs. 1-2, the following weighted, four–point implicit, finite difference equations are obtained:

$$\theta\left[\frac{A_i^{j+1}+A_{i+1}^{j+1}}{2}\right]+(1-\theta)\left[\frac{A_i^{j}+A_{i+1}^{j}}{2}\right]+\theta\left[\frac{Q_i^{j+1}+Q_{i+1}^{j+1}}{2}\right]$$

$$+(1-\theta)\left[\frac{Q_i^{j}+Q_{i+1}^{j}}{2}\right]-q=0 \qquad (7)$$

$$\theta\left[\frac{Q_i^{j+1}+Q_{i+1}^{j+1}}{2}\right]+(1-\theta)\left[\frac{Q_i^{j}+Q_{i+1}^{j}}{2}\right]+\theta\left[\frac{(Q^2/A)_i^{j+1}+(Q^2/A)_{i+1}^{j+1}}{2}\right]+(1-\theta)$$

$$\left[\frac{(Q^2/A)_i^{j}+(Q^2/A)_{i+1}^{j}}{2}\right]+\frac{gh}{2}\left[\theta\left(\frac{A_i^{j+1}+A_{i+1}^{j+1}}{2}\right)+(1-\theta)\right.$$

$$\left.\left(\frac{A_i^{j}+A_{i+1}^{j}}{2}\right)\right]-gA(S_o-S_f)=0 \qquad (8)$$

In the above equations (Eqs. 7-8), the terms associated with the j^{th} time are for a known time level and $j+1^{th}$ is the unknown time level. In the beginning of the program, j refers to the initial conditions. The equations stated above cannot be solved in an explicit or direct manner for the unknowns since there are four unknowns and only two equations. However, if Eqs. 7-8 is applied to each of the $N-1$ nodes, a total of $2N-2$ equations with $2N$ unknowns will be formulated. Here, N denotes the total number of nodes locations where computations are performed to find the flow variables. Then, the other two equations are obtained from boundary conditions in order to get a solution. These $2N$

non-linear equations are solved iteratively by a fast converging Newton- Raphson method (Chaudhry 1993).

BOUNDARY CONDITIONS:

Upstream Boundary Condition: In the present mathematical model, the upstream boundary condition plays an important role in the solution of the governing equations. In most of the cases, this information is obtained from the specified hydrograph i.e. $Q_I = QI$ (t). In which the flow at section 1 (the most up stream cross section), and QI (t) represents the specified flow at time (t). The upstream flow should not be zero and the time specification should not be less than the required time of flood routing.

Down stream boundary condition: In case of down stream boundary condition, the model uses one of the following four conditions. (i) Specified stage hydrograph, (ii) single valued rating curve, (iii) critical flow condition. (iv) generated dynamic loop rating curve.

Initial Condition: The flow conditions are also specified at the beginning of the unsteady flow conditions, which is known as initial condition. The initial condition (flow at t=0) are specified by assuming a steady non-uniform flow. The flow depth (h) is specified by numerically integrating the steady state gradually varied flow equation. Presence of any internal boundary requires special treatment, as governing equations are not valid at these locations. In such cases, established empirical formula or relations replace momentum equation.

STABILITY: Although an implicit formulation is used which requires no stability criteria, a small time step is used in the model to obtain results with small truncation error. The time step, Δt is calculated by,

$$\Delta t < 0.075 C t_r (Z/D)^{0.5}$$

where C =Wave celerity = $\sqrt{gh}$

g = gravitational force.

h= water depth.

t_r = time of rise of hydrograph

$$Z = (1-\varepsilon^2)/\left[4\theta^2\varepsilon^2 - (2\theta - 2)^2\right]$$

In which ε is the permissible error ratio ($0.90<\varepsilon<0.99$) and θ is the weighting factor.

DATA REQUIREMENT:

In this work, following data are required.

(1) Inflow hydrograph ordinates and corresponding time interval.

(2) Total computational time.

(3) Cross-section at observed inflow and outflow stations and their location.

(4) Elevation corresponding to each top width of cross-section

(5) Top width corresponding to each elevation of cross-section

(6) Manning's roughness coefficient.

(7) Expansion and Contraction coefficient between the cross-section.

(8) Computational distance(Δx)

(9) Initial size of time step.(Δt)

(10) Down stream boundary parameter.

LIMITATIONS:

The limitations of the model constitute the assumptions used in the derivation of governing equations. Some of the limitations are:

(1) The model assumes one-dimensional flow, but there are some instances where the flow is more nearly two-dimensional than one-dimensional. In many cases where the wide flood plain is bounded by rising topography, the significance of neglecting the transverse velocities and water surface variations is confined to a transition reach in which the flow changes from one dimensional to two dimensional and back to one dimensional along the x- direction.

(2) The model assumes a rigid bed channel, but during floods it can cause significant scour i.e. degradation of alluvial channels. This enlargement in channel cross sectional area is neglected in model since the equation for the sediment transport and channel bed armoring are not included in the governing equations.

(3) Bed roughness is a complex function of various flow and bed characteristics. The uncertainty associated with the selection of the Mannings n can be quite significant for the floods due to (i) the great magnitude of the flood produces flow in portions of floodplains which are very infrequently or never before inundated; this necessitates the selection of the n value without the benefit of previous evaluations of n from measured elevation/ discharges or the use of calibration techniques for determining the n values; (ii) The effect of transported debris can also alter the Mannings n. However, the model assumes a constant value of Mannings roughness coefficient n all time level.

Chapter 3

THE STUDY AREA

The governing equations and their numerical solution to perform the flood routing for specific reaches in the Narmada River were presented in the previous chapter. The study area and available data for the present study have been described in the following sections.

3.1 NARMADA BASIN

The Narmada River situated in Central India is a west flowing river. It originates from Maikala hills at Amarkantak in Madhya Pradesh at an elevation of 1058 m and flows through Madhya Pradesh, Maharastra and Gujarat. Total length of the river is 1312 Km and the basin area is about 98796 Km^2. The basin is bounded on the north by the Vindhyas, on the east by the Maikala range, on the south by the Satpuras, and on the west by the Arabian Sea (Fig. 3.1). The river has 41 tributaries penetrating the catchment in notrh-south directions. Some of the major tributaries are Hiran, Sher, Shakkar, Tawa, Burner, Chotta tawa and Kundi. The Narmada basin is of elongated shape. Topography of the Narmada basin is hilly with forest cover in the upper reaches, lower reaches are flat with abundance of farmland. Major part of the Narmada basin consists of variety of black soils with a large content of clay. Mixed red and black soil, red and yellow soil and skeletal soil are observed at isolated areas. The vegetation in the Narmada catchment includes a variety of agricultural crops on the plains and forest of varying density in the upland areas. The climate is humid tropical ranging from sub-humid in the east to semi-

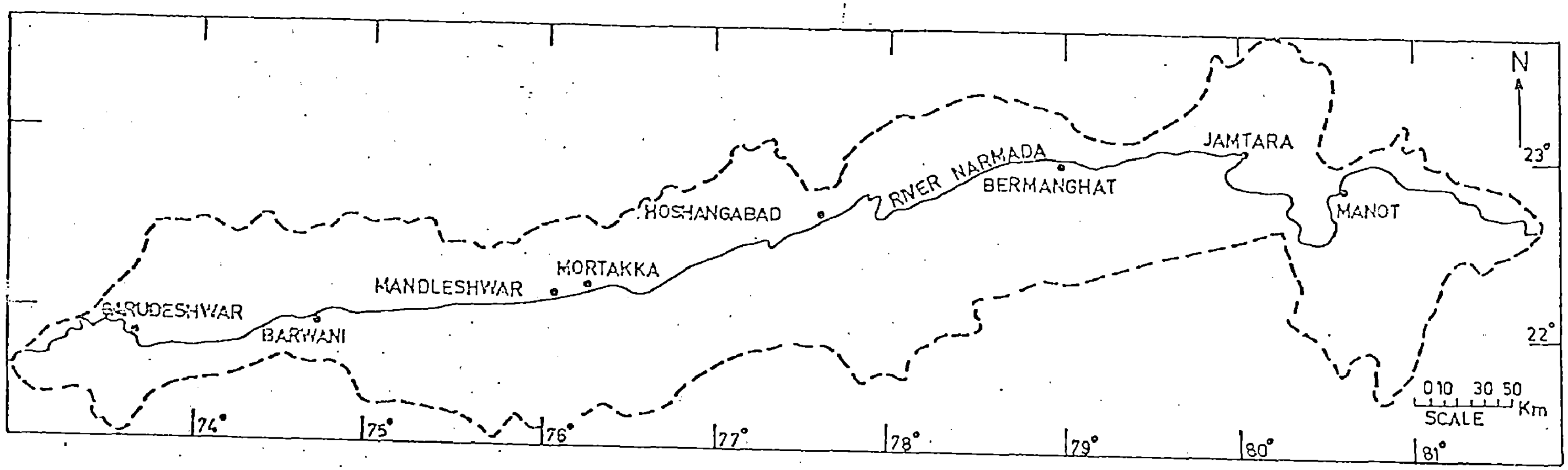


Fig. 3.1 Index map showing gauging sites on the river Narmada

arid in the west. The average annual rainfall is 1230 mm. The Southwest monsoon is the main source accounting for 90% of the annual rainfall, of which 60% falls during the months of July and August. There are seven important gauging sites (Mannot, Jamtara, Bermanghat, Hosangabad, Mortaka, Mandleswar and Gurudeswar) in the main river. For this dissertation work, the specific reach considered is from Mortaka to Mandleswar for five different events. In addition, the reach from Jamtara to Bermanghat, which contains two major tributaries, is also attempted for one flooding event.

3.2 RIVER REACH FROM JAMTARA TO BERMANGHAT

The sites Jamtara and Bermanghat on the main river are located at 399 Km and 510 Km away, respectively, from the source at Amarkantak. Both the sites are in Madhya Pradesh. The major tributaries - Hiran and Sher, between Jamtara and Berminghat, join the main river at a distance of 461 and 494 Km, respectively, from the source. The Hiran joins the main river from right side, whereas the Sher is from the left. The lengths of these tributaries are 187 Km and 128 Km, respectively and their catchment areas are 4480 and 2867 sq. Km. In between Jamtara and Berminghat, there are five ordinary rain gauge stations (Narsingpur, Lakhandon, Ghansore, Chhindwara and Patan) and two self recording rain gauge stations (Jabalpur and Adhartal). The reach between Jamtara and Berminghat is in the upper plain region. Major part of this area consists of black soils with clay and about 40% of which are under forest and remaining 60% is under agricultural, grass and wasteland.

3.3 RIVER REACH FROM MORTAKKA TO MANDLESWAR

The sites Mortaka and Mandleswar are situated in the main river Narmada at 886 Km and 926 Km away from the source. Between these two sites, there are 10 ordinary rain gauge stations (Khargone, Bhikangaon, Lachore, Kasrawad, Maheswar, Mandu, Manpur, Bagaud, Choral and Patalpani). This reach is situated in the middle plain region of the basin. This reach mainly contains black soils and mixed red yellow soils. The vegetation in the region is an agricultural crop in the plains and forest of varying density.

3.4 AVAILABLE DATA

The Narmada basin has been studied extensively, covering various aspects of hydrology, at National Institute of Hydrology (NIH), Roorkee. The data, which have been used in this study, are, (1) observed flood hydrographs at Mortakka and Mandleswar for five cases, and data adopted from Seth et. al. (1985),Vol.2 of Final report, and (2) river cross-sections at Mortakka and Mandleswar have been extracted from Seth and Palanippan (1985),(CS-6). In addition, for the study on river reach from Jamtara to Bermanghat, one observed hydrograph at Jamtara and Baramanghat for the one event and cross-sections were also taken from the same references stated above.

Chapter 4

RESULTS AND DISCUSSION

The study area and the available data have been described in chapter three and results for flood routing at two reaches of Narmada are presented in this chapter. The results of the present work are presented in four parts. In the first part, calibration showing determination of various model parameters such as, roughness coefficient (n) and distance step size (Δx) in the numerical grid is presented. Results of flood routing from Mortakka to Mandleswar for five different events are presented in the second part. The third part constitutes derivation of generalized equations from cases of studies in part 2, for peak outflow at any intermediate location and time to achieve this value, as functions of inflow parameters and length. In the last part, flow hydrographs in the tributaries (Hiran and Sher) using observed hydrographs at Jamtara and Bermanghat in the Narmada River, are determined. Before discussing the results, the used input data for the present study is presented in the following section.

4.1 INPUT DATA

In the present analysis of flood routing from Mortakka to Mandleswar in the Narmada River, the inflow hydrographs at Mortakka are prescribed as the measured discharges at that place. These values are adopted from Seth et al. (1985). Different events have been considered for the purpose. The cross-section details at Mortakka and Mandleswar have been extracted from Seth and Palaniappan (1985). These two cross-sections are presented in Fig. 4.1. As seen from this figure, the cross-sections are not

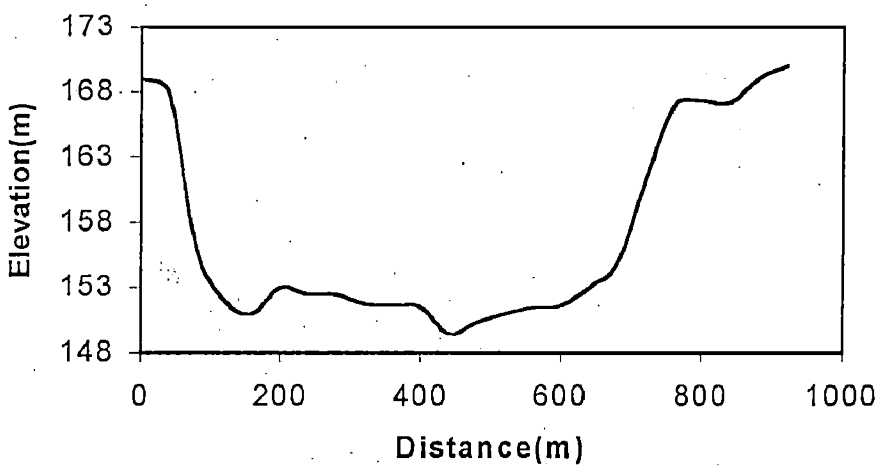


Figure 4.1(a) Cross-section at Mortaka

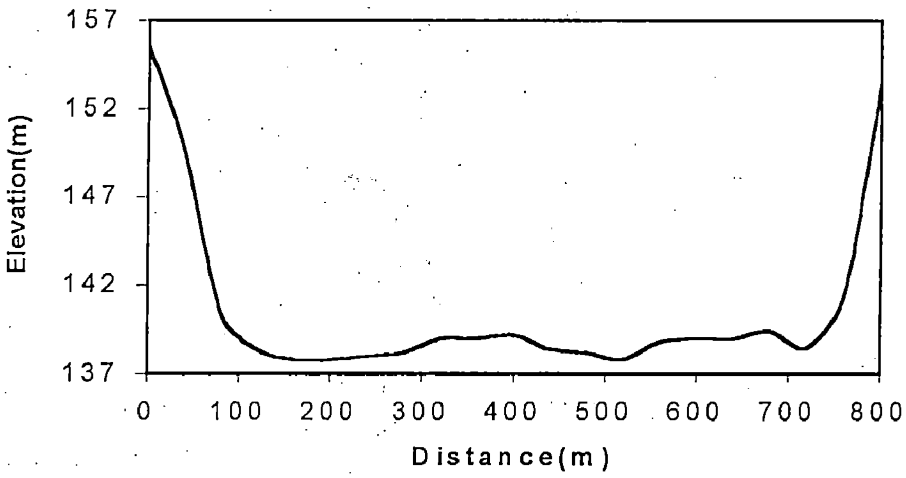


Figure 4.1(b) Cross-section at Mandleswar

defined at higher elevations. Thus, the present study is limited to the flow in the main river only and higher flows involving the flood plains cannot be analyzed using these cross-sections.

Further, three more cross-sections at equal intervals between Mortakka to Mandleswar are also used. These cross-sections are obtained using linear interpolations, as no measured values are available. The river bed friction characteristics is defined by the Manning's roughness coefficient, *n* and its value for the specific reach from Mortakka to Mandleswar is *0.03*. In the section for calibration, it will be shown that this value gives a better match with the observed outflow hydrograph. The incremental step size, $\Delta x = 1$ *Km* is used in this study. As the width of the cross-sections are approximately of the same order, used values of expansion/contraction coefficients are zero. The lateral inflows due to the contribution from the intermediate catchment area are determined by deducting the observed hydrographs at Mandleswar and Mortakka. An input data file used for the computer program is presented in Appendix-1.

4.2 CALIBRATION

The model parameters, such as distance step size, Δx, and Manning's roughness coefficient, *n*, are calibrated for the reach from Mortakka to Mandleswar. on Narmada river. As the flood waves moves downstream, the peak discharge is reduced and the shape of the hydrograph is flattened. However, if there is considerable amount of lateral flow from the intermediate catchment area, the flood peak at a downstream location may be more as compared to that at a station located upstream. In the present case as seen from the measured discharges, the lateral inflow from the area between Mortakka and Mandleswar is significant and therfore, the peak discharge at Mandleswar is higher than that at Mortakka. In the absence of any information about lateral inflow, it is difficult to calibrate the model parameters. Therefore, an indirect method is adopted to calibrate the Manning's roughness coefficient.

4.2.1 Determination of Manning's *n*

Let *A* and *C*, be two stations, on the main river. Observed hydrographs at *A* and *C* (Q_o^A and Q_o^C) are available. Cross-sections at *A* and *C* are known. An intermediate station *B* which is located at mid point of *A* and *C* is assumed. The cross-section at *B* is determined using linear interpolation of values for cross-section at *A* and *C*. The observed hydrograph at *B* is assumed as the average of the observed hydrographs at *A* and *C*. Using the above information and assuming a arbitrary *n* values, two flood routings, first from *A* to *C* and then from *B* to *C* are performed. The first set of results gives the computed hydrographs at station *B* (Q_c^{A-B}) and at *C* (Q_c^{A-C}) due to observed hydrograph at *A*. The second set of results gives the computed hydrograph at *C* (Q_c^{B-C}) due to the observed hydrograph at *B*. The contributions from *A* to *B*, *B* to *C*, and *A* to *C* are calculated separately. A Check is made to verify the contribution from *A* to *C* equals the combination of contribution from *A* to *B* and *B* to *C*. The procedure is repeated with different *n* values till the above condition is satisfied. The calibration strategy to determine optimum *n* values is presented in figure 4.2.

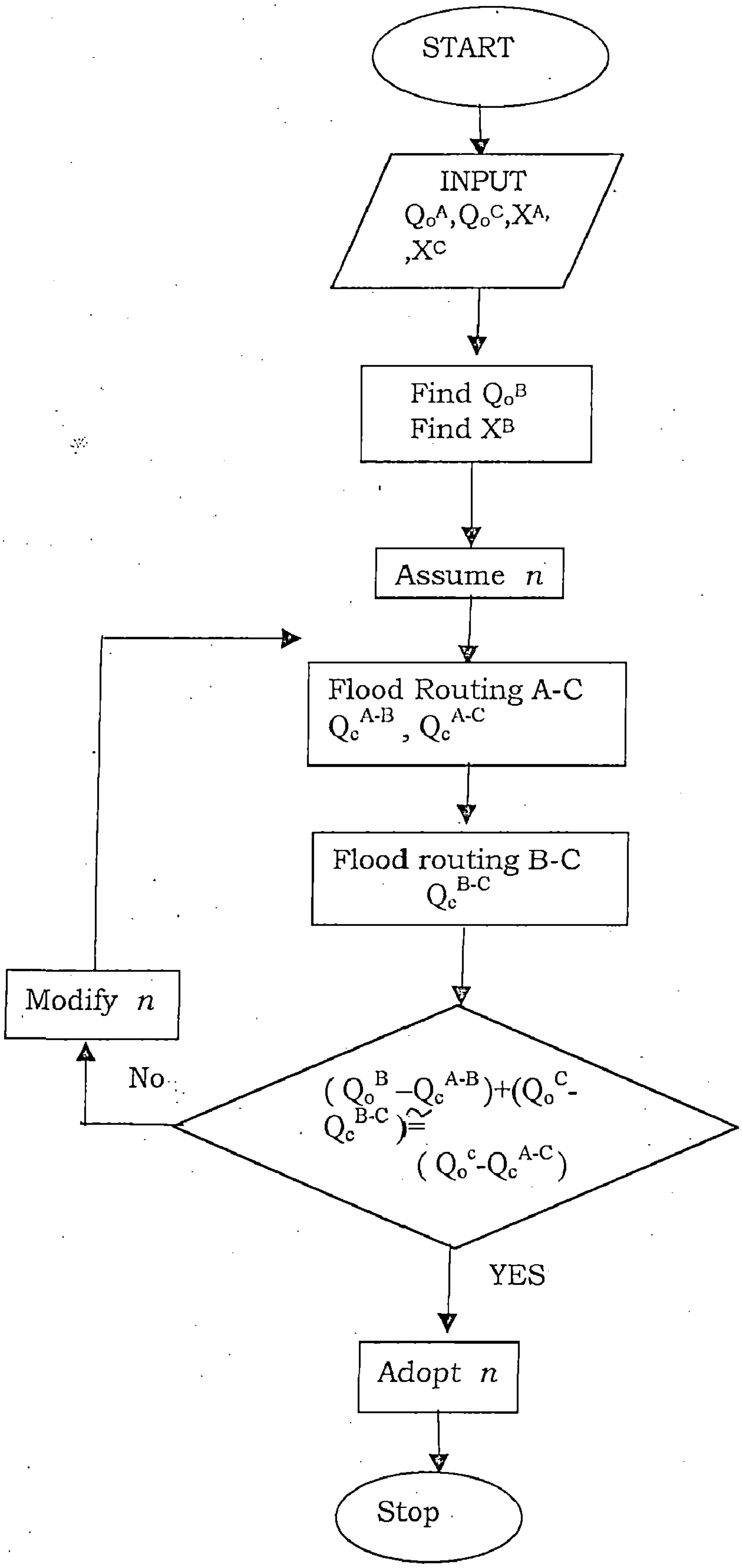


Figure 4. 2 Determination of Manning's *n* by calibration

Two cases for the reach Mortakka and Mandleswar have been considered to do the calibration, following the procedure mentioned above. It may be noted here that, A refers to Mortakka and C to Mandleswar. An imaginary station, I, is considered as B. The calibrated value of Manning's n is 0.030. Results of the above numerical study are presented in Figures 4.3, 4.4, 4.5 and 4.6. In these Figures the contributions from Mortakka (A) to Mandleswar(C) and combinations of contributions from Mortakka to I and I to Mandleswar are presented. The comparison shows a good match between the two hydrograph.

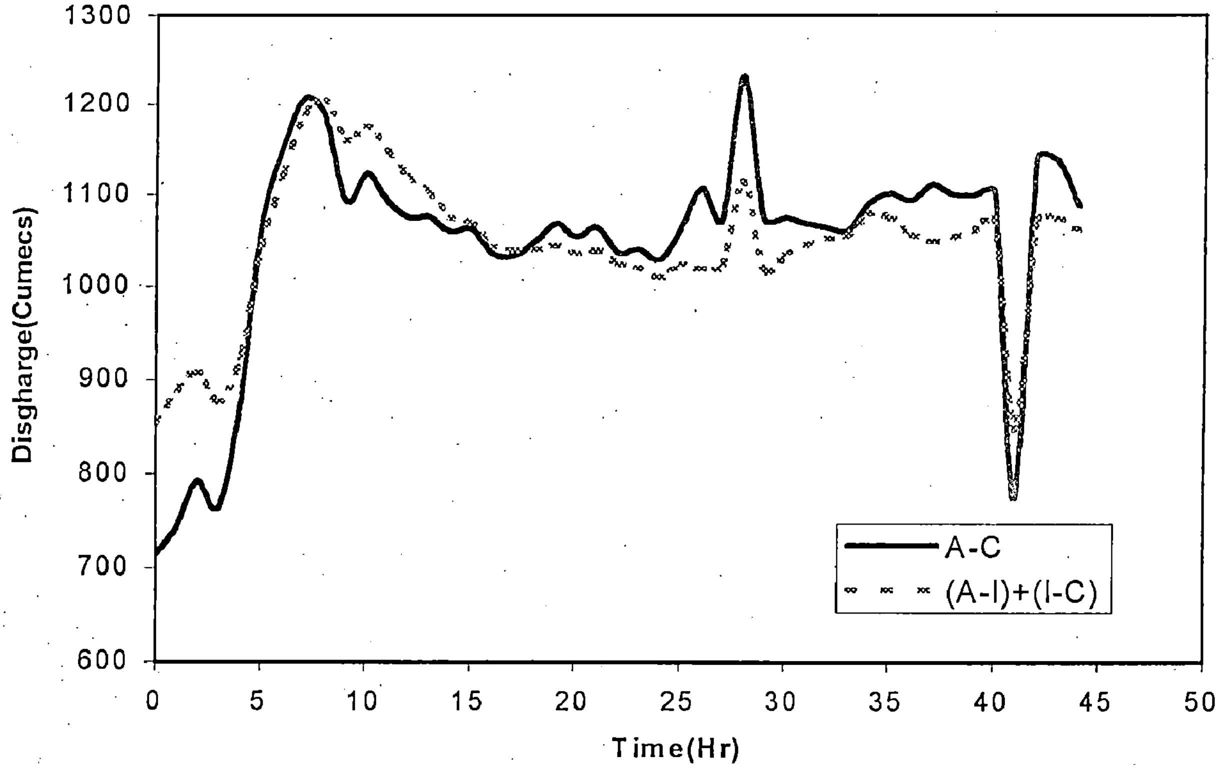


Figure 4. 3 Lateral inflows from Mortakka to Mandleswar (case 1)

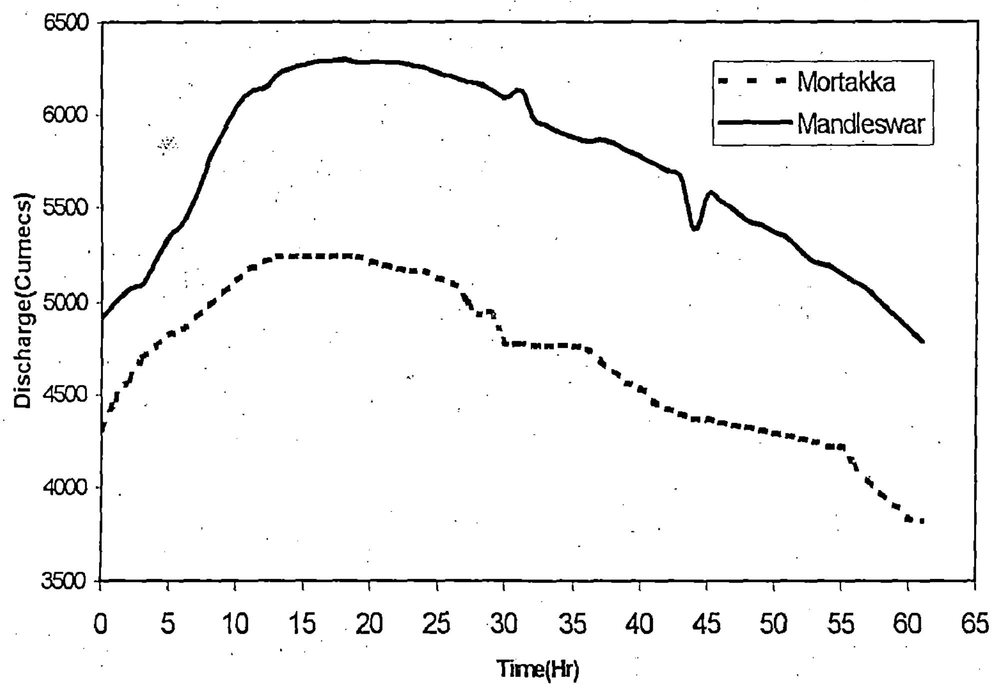
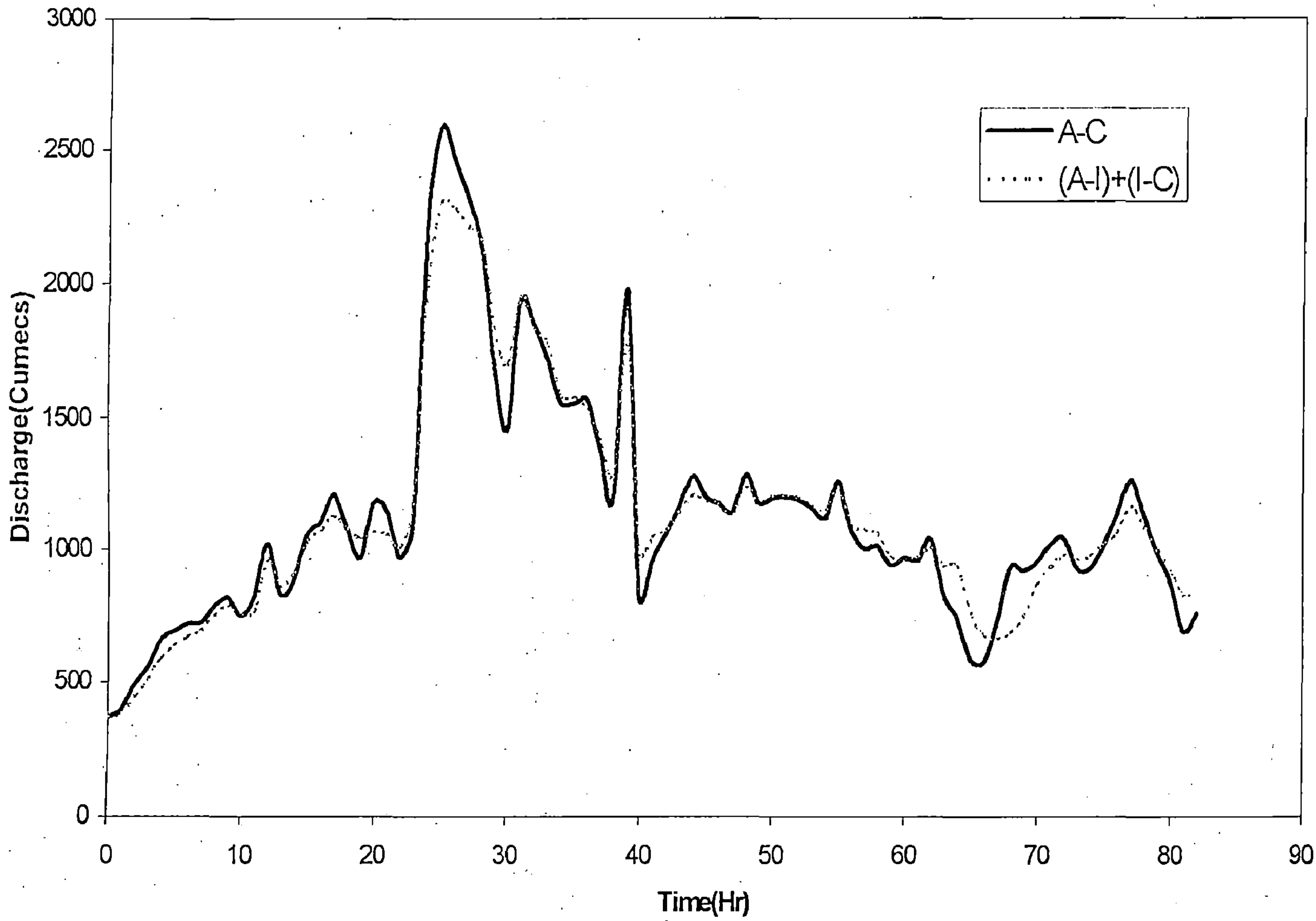


Figure 4.4 Observed hydrographs at Mortakka and Mandleswar



Figures 4.5 Lateral inflows from Mortakka to Mandleswar (case2)

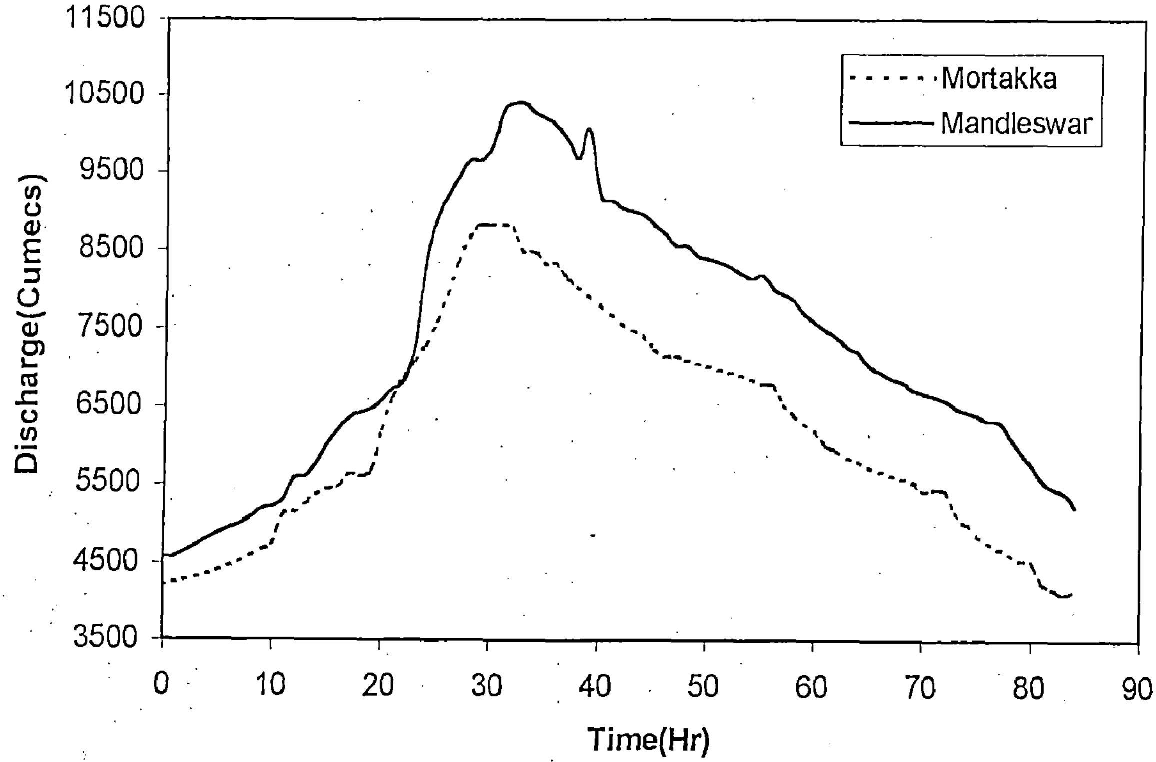


Figure 4. 6 Observed hydrographs at Mortakka and Mandleswar

4.2.2 Effect of Distance Time Step

The present model has been studied to see the effect of distance step size (Δx) on the results. For this purpose, the model has been executed using different Δx values (Δx=1.0Km, 2.0 Km and 5.0 Km). It is observed that as Δx reduces the results become closer (Figure 4.7). In rest of the studies, all the results have been obtained using Δx=1 km.

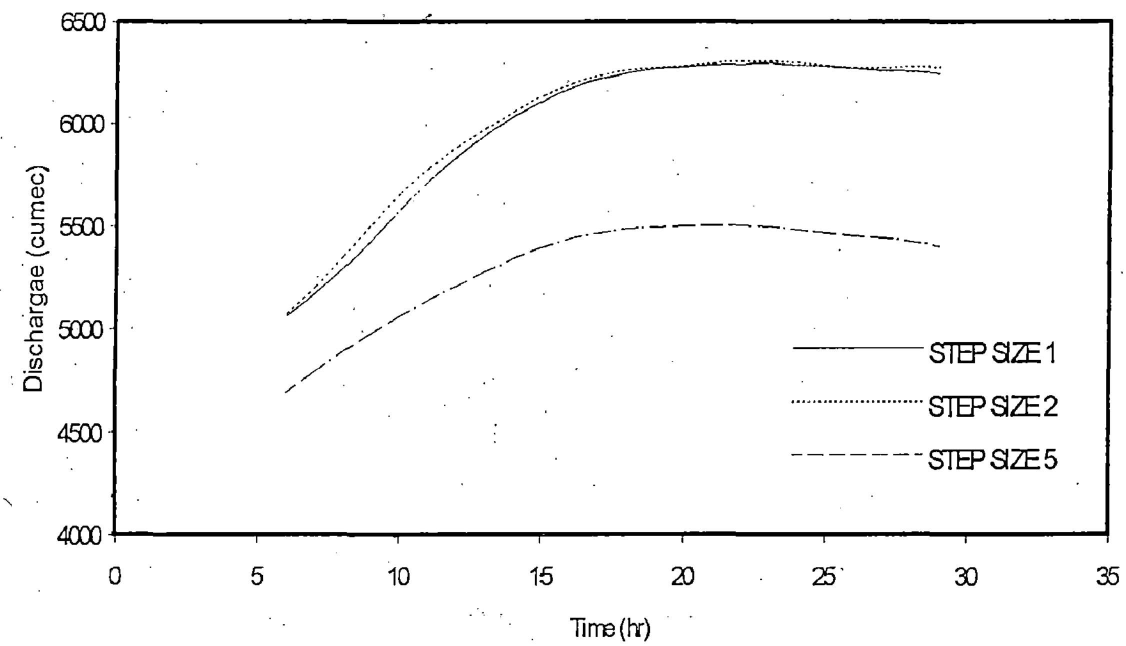


Figure 4. 7 Computed hydrograph at Mandleswar for different ΔX values

4.2 FLOOD ROUTING FROM MORTAKKA TO MANDLESWAR

In the present study, flood routing have been performed from Mortakka to Mandleswar for six events. The Cress-sections at Mortakka and Mandleswar are presented in Figure 4.1(a),(b). Observed inflow hydrographs at Mortakka for different cases are presented in Figures 4.8, 4.10, 4.12, 4.14, and 4.16. Summary of this hydrograph are presented in Table 4.1.

TABLE-4.1 SUMMARY OF INFLOW HYDROGRAPHS AT MORTAKKA

Cases	Peak discharge(Cumecs)	Time to peak (Hr)
Case 1	5679	4
Case 2	5243	13
Case 3	8824	29
Case 4	20813	4
Case 5	18477	21

Results obtained from the mathematical model for the above cases are presented in Figures 4.9, 4.11, 4.13, 4.15, and 4.17. In these Figures, the outflow hydrograph at Mandleswar as computed by the model are shown. In addition, the observed hydrograph at Mandleswar are also presented. For all the cases the match is satisfactory. The summary of outflow hydrographs at Mandleswar is given in Table 48. The results show that it is important to consider the contribution from the intermediate catchments. One case (Case 1) is also studied without considering the lateral inflow from the intermediate catchment. The computed hydrograph at Mandleswar (Figure 4.18) is underestimated when compared with the observed hydrograph at that place.

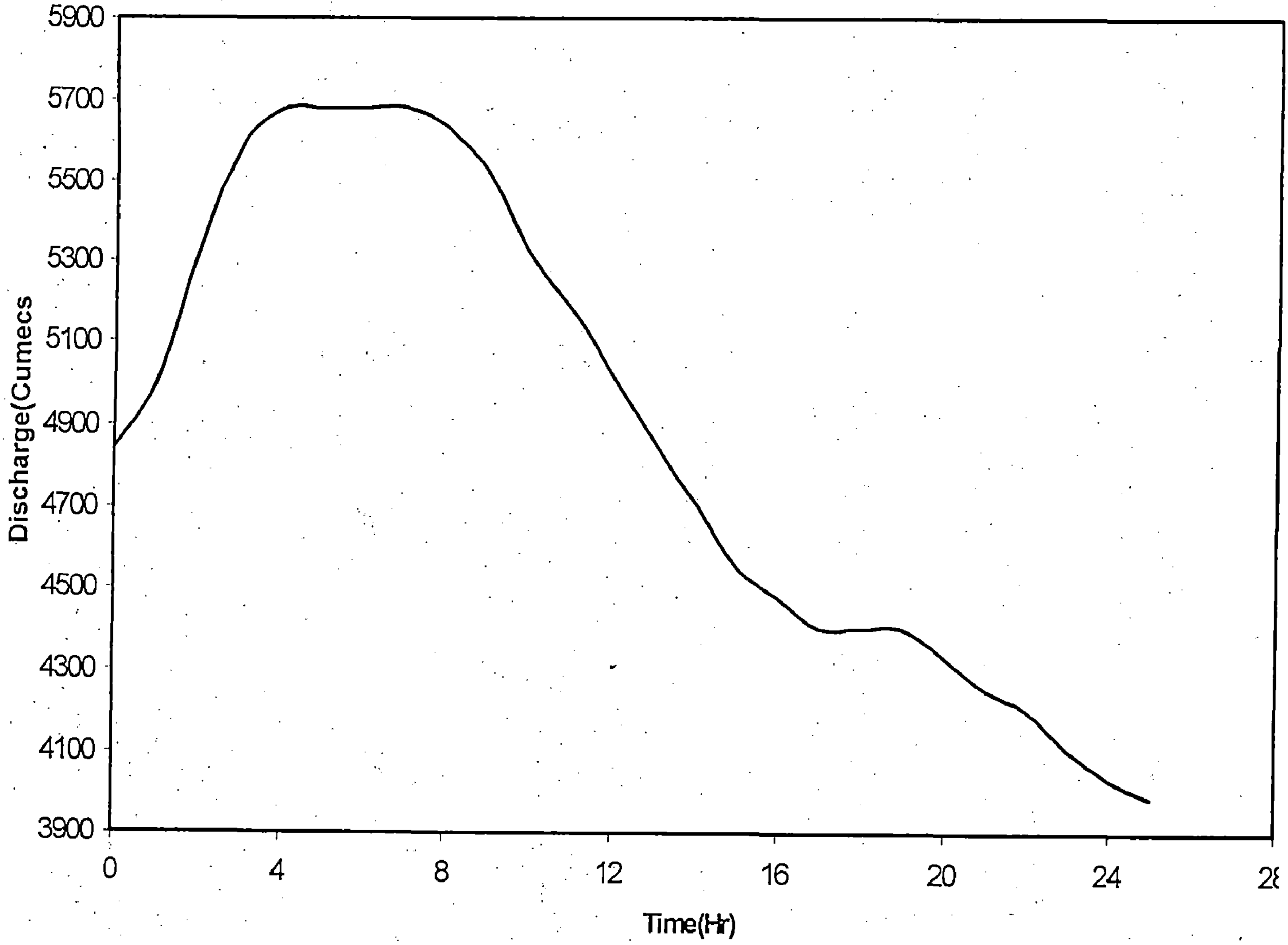


Figure 4. 8 Inflow Hydrograph at Mortakka (Case-1)

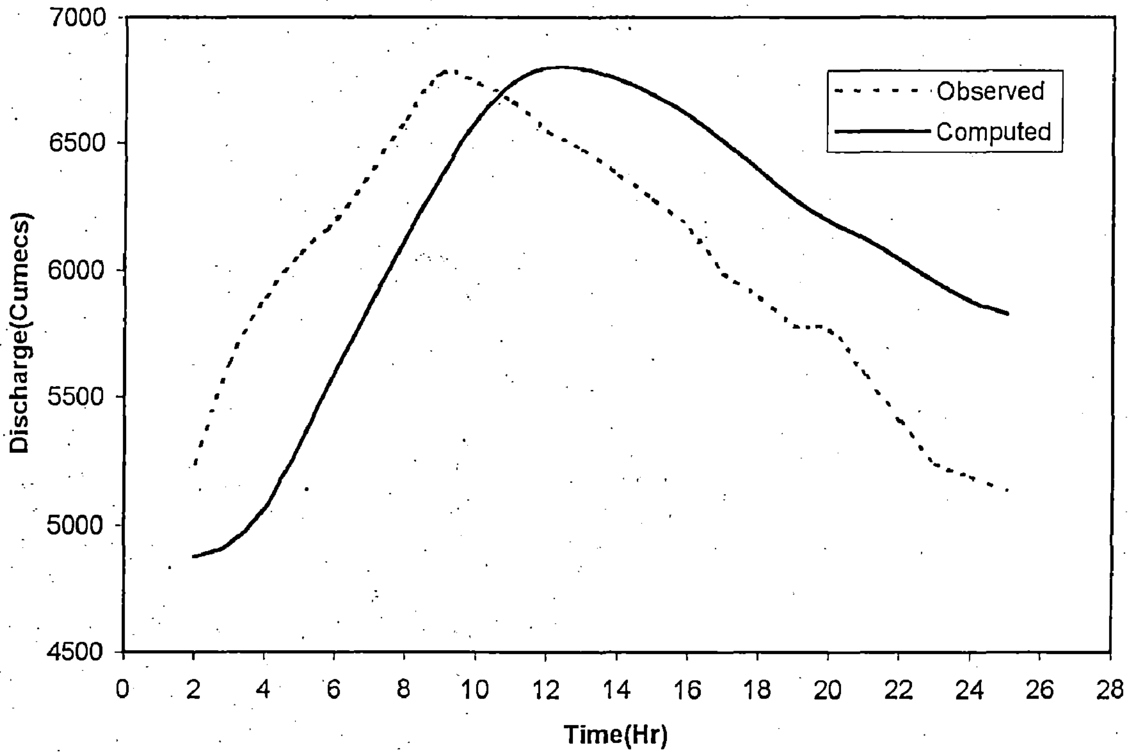


Figure 4.9 Outflow Hydrograph at Mandleswar (Case-1)

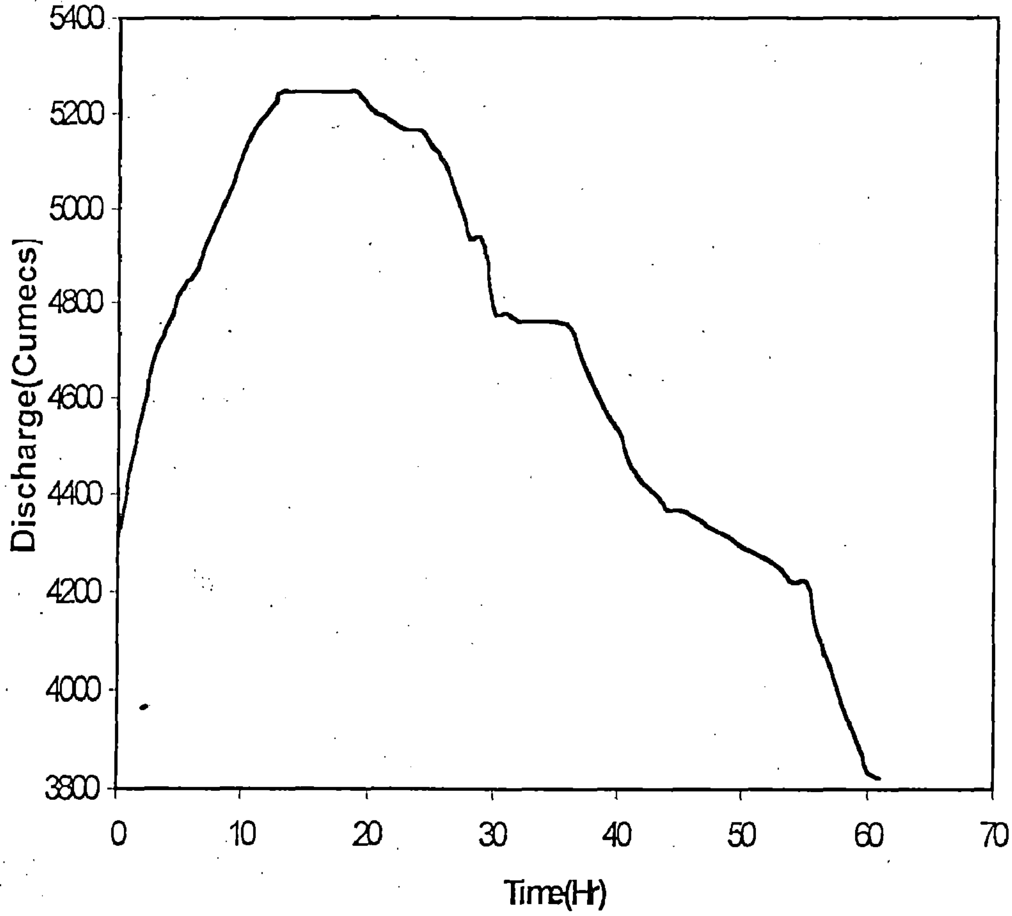


Figure 4.10 Inflow Hydrograph at Mortakka (Case-2)

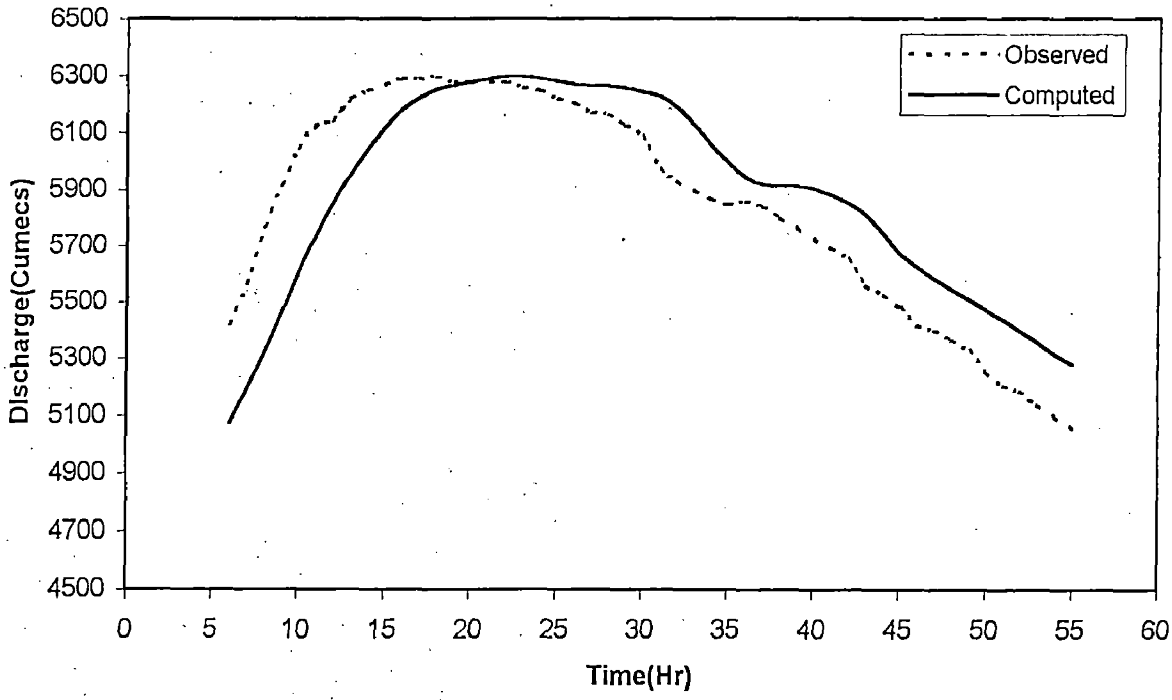


Figure 4.11 Outflow Hydrograph at Mandleswar (Case-2)

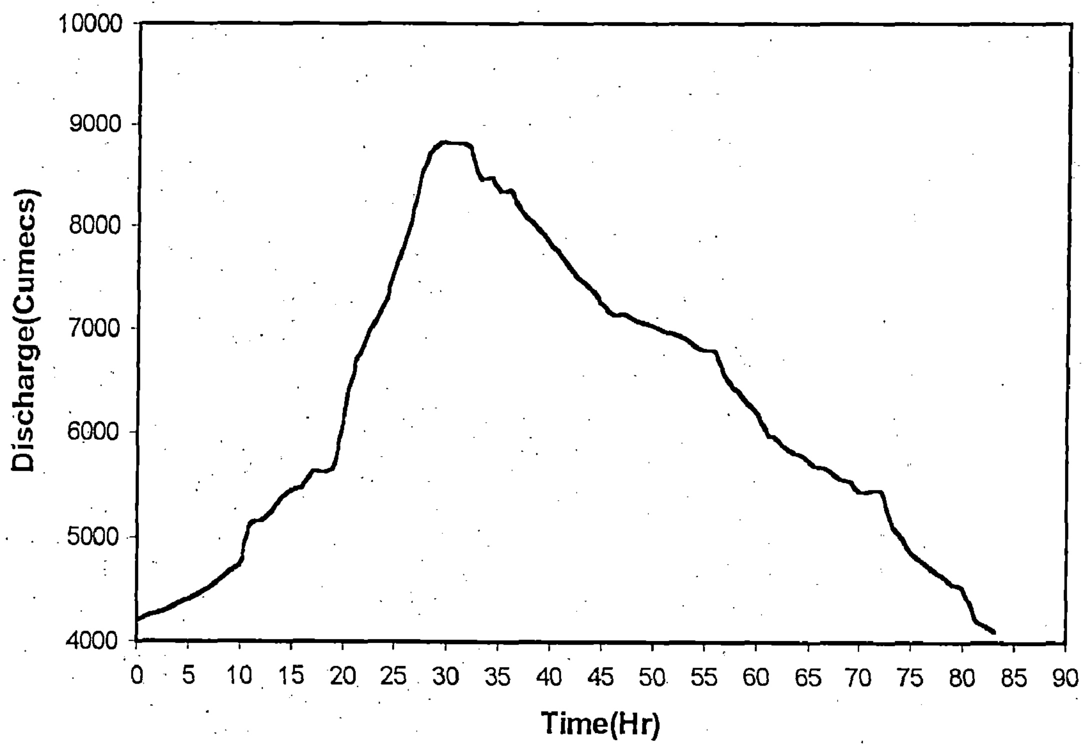


Figure 4.12 : Inflow Hydrograph at Mortakka (Case-3)

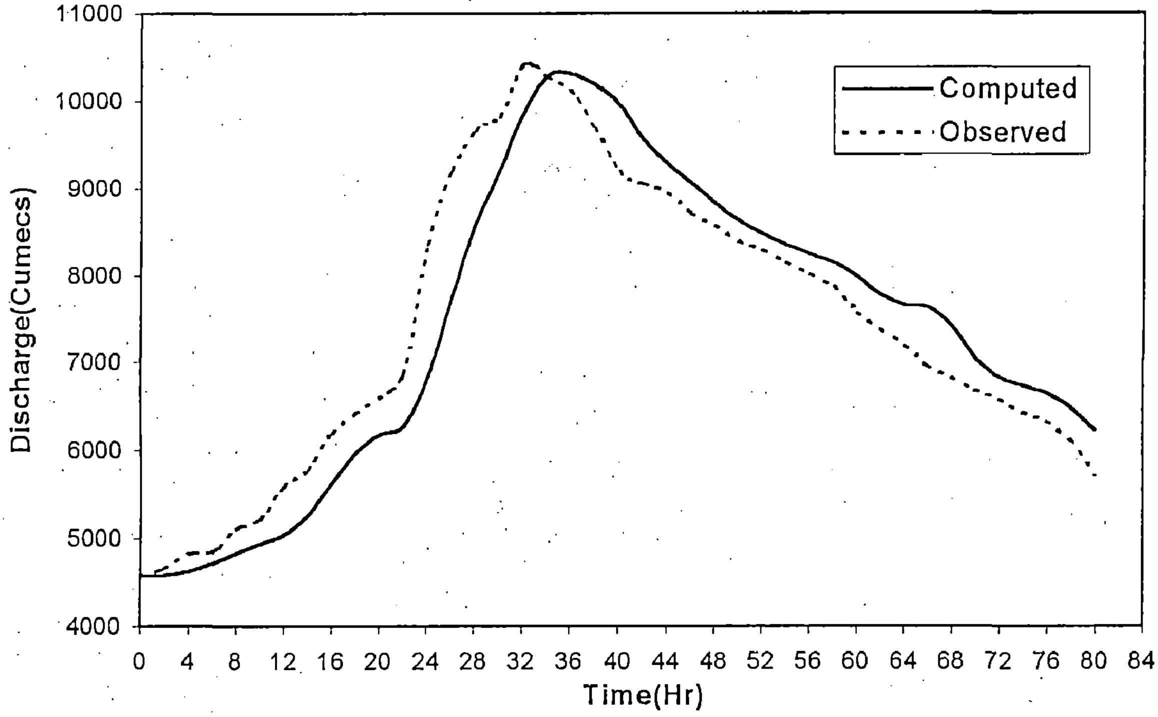


Figure 4.13 Outflow Hydrograph at Mandleswar (Case-3)

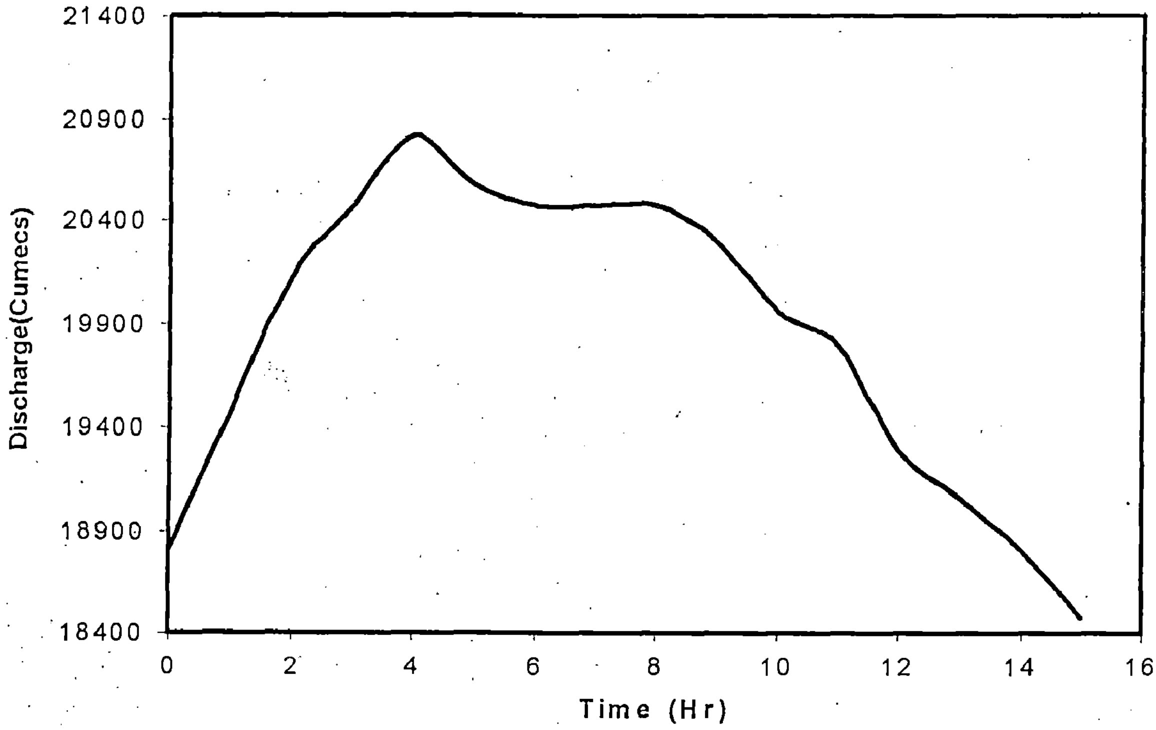


Figure 4.14 Inflow Hydrograph at Mortakka (Case-4)

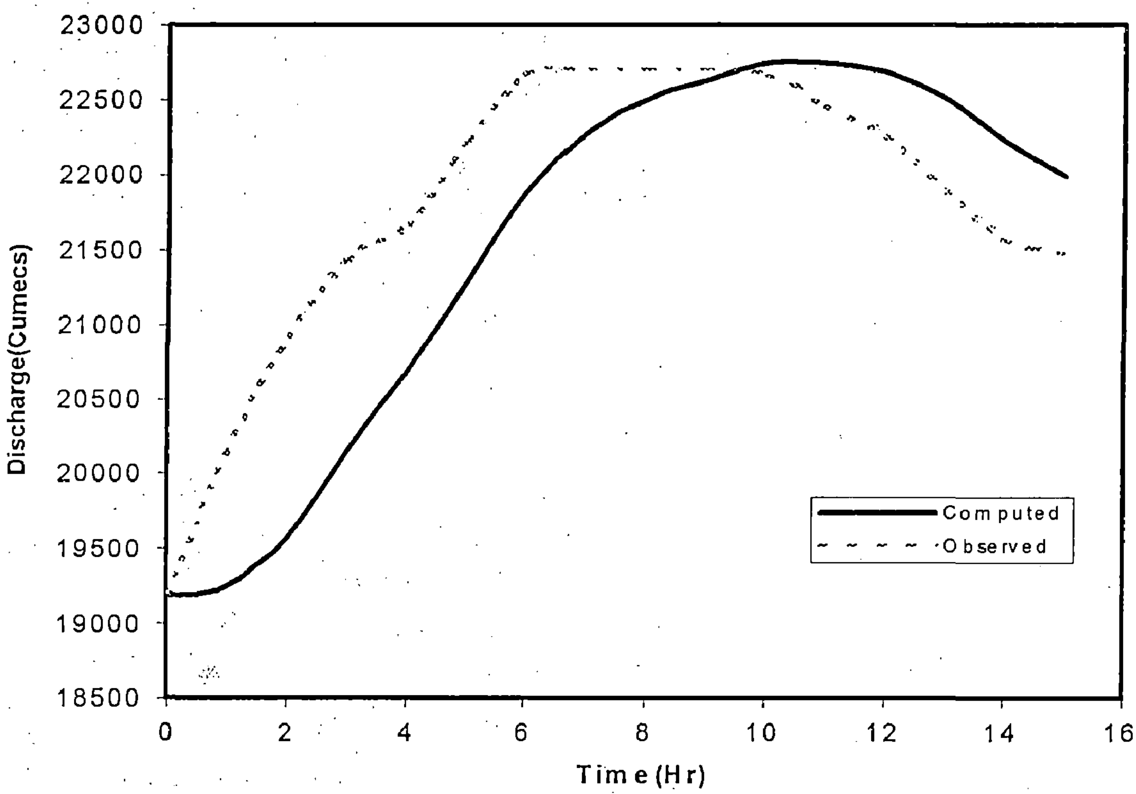


Figure 4.15 Outflow hydrograph at Mandleswar (Case-4)

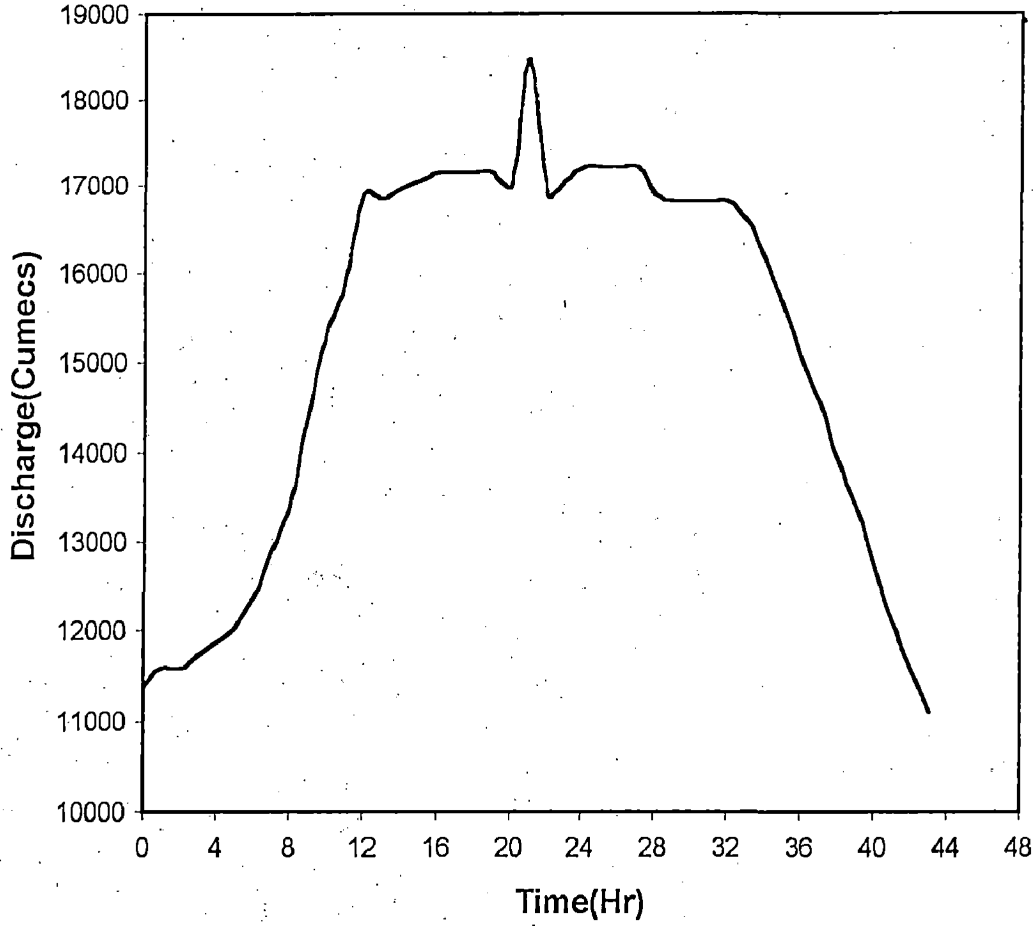


Figure 4.16 Inflow Hydrograph at Mortakka (Case-5)

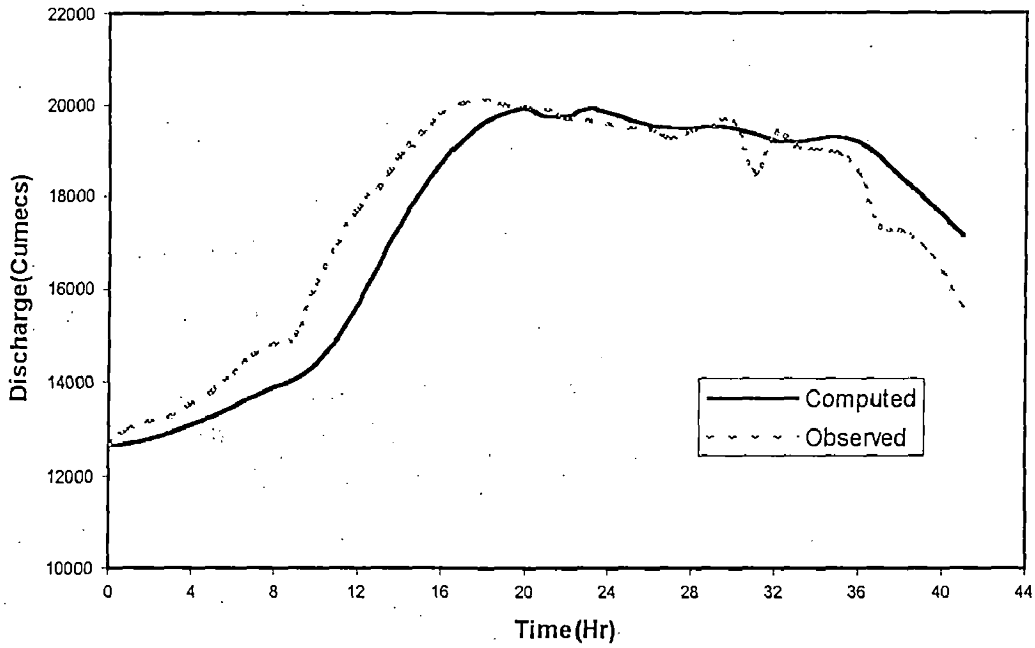


Figure 4.17 Outflow Hydrograph at Mandleswar (Case-5)

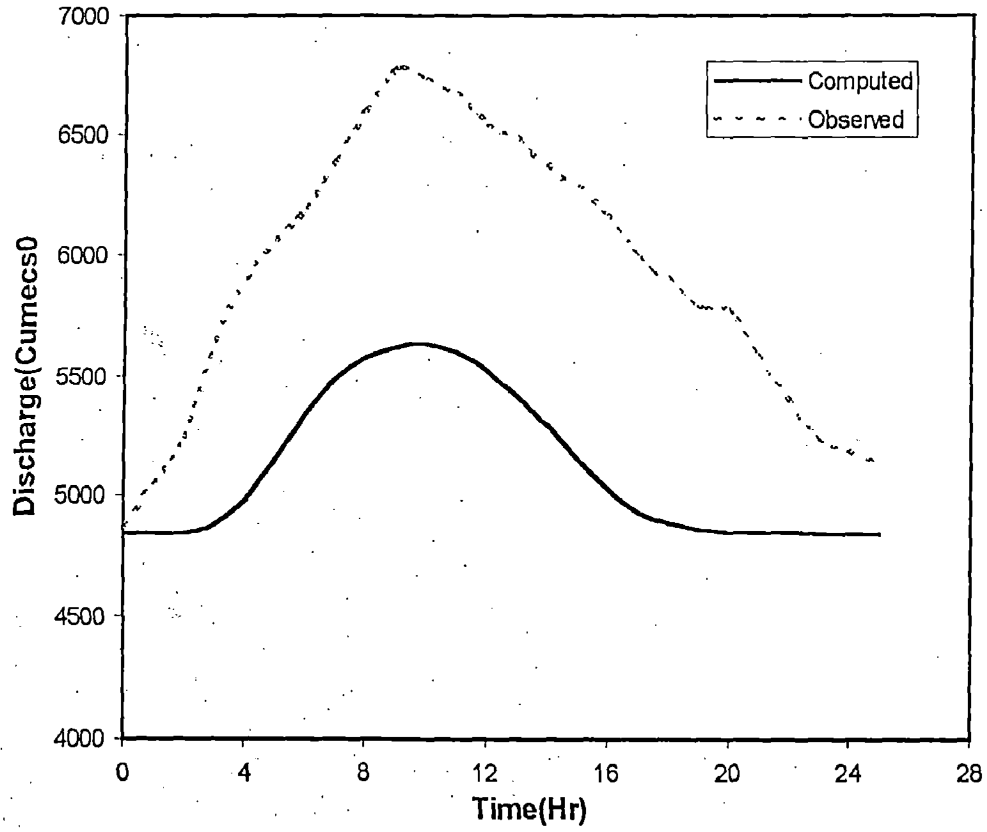


Figure 4.18 Outflow hydrograph at Mandleswar without lateral inflow (Case-1)

Table 4.8 : Computed and Observed Peak Outflow and Time

Cases	Computed peak discharge(Cumecs)	Computed peak time(Hr)	Observed peak discharge(Cumecs)	Observed peak time (Hr)
Case1	6800	12.6	6776	9
Case2	6296	22.80	6295	18
Case3	10335	36.55	10406	32
Case4	22767	10.4	22706	7
Case5	19931	23.10	20090	18

4.3 GENERALIZED EQUATIONS:

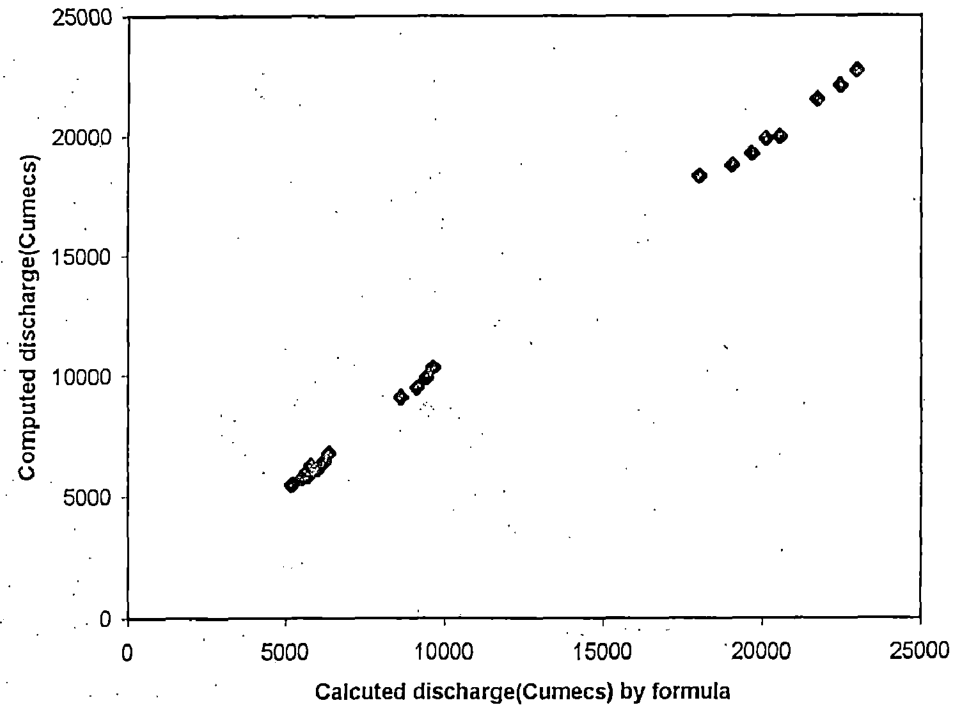
As the flood wave moves downstream in a channel, the flood peak and time to flood peak are changed. These values are complicated functions of distance, channel roughness, bed slope and inflow characteristics. These values can be accurately found out using a good flood routing model. However, it is easier to use some simple equations. Also, these relationships are helpful to field engineers. In the present study, simple equations for peak discharge and time to attain this value, as a function of inflow characteristics and distance, are derived for flows from Mortakka to Mandleswar. These generalized equations are based on non-linear regression analysis of the result obtained in the previous section. For this purpose three intermediate sections are considered at 10 Km., 20Km. and 30 Km., respectively. The Cross-sections at these places are obtained using linear interpolation between cross-sections at Mortakka and Mandleswar. Outflow hydrographs have been computed at Mandleswar and all the intermediate sections. The derived equations are given below.

$$\frac{Q2}{Q1} = 1.106(L_r)^{0.08}(t_r)^{-0.009} \tag{9}$$

$$\frac{t2 - t1}{t1} = 0.823(L_r)^{0.54}(Q_r)^{-1} \tag{10}$$

Where, L_r= L/40, t_r=(t1*Q1)/(40)^3 and Q_r=(t1*Q1)/(40)^3

L is the distance from Mortakka, in Km. Q1 is the peak discharge in cumecs at Mortakka. and t1 is the time to peak discharge at Mortakka. Q2 and t2 is the future peak discharges and time to peak to be finding out using the quation.



Figures 4.19 Shows the comparison of discharge between values Obtained using equation and mathematical model

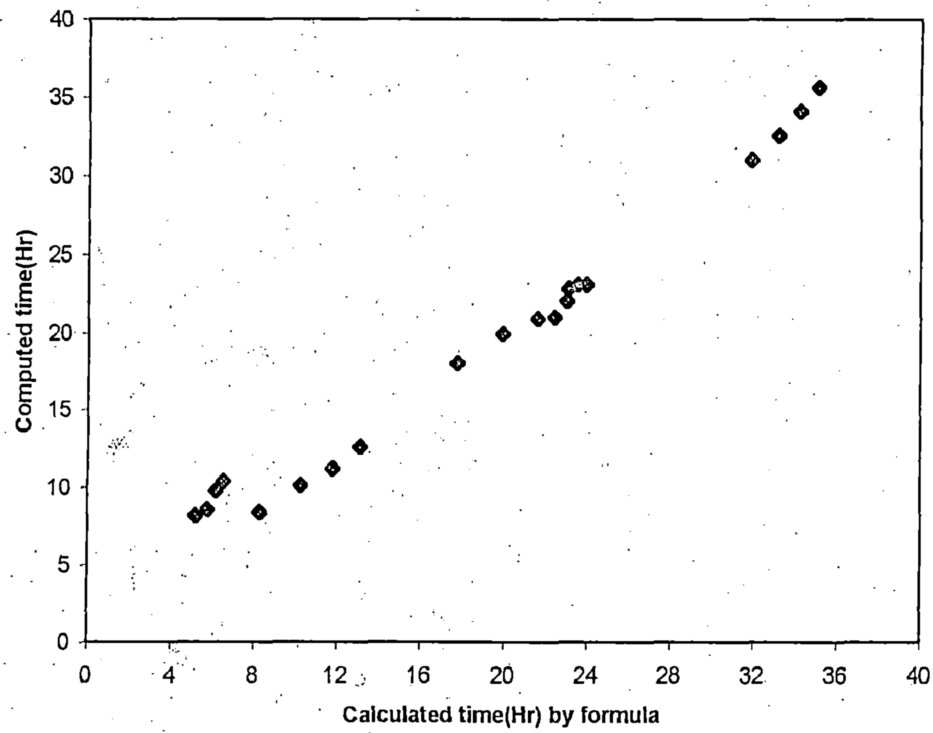


Figure 4.20 Shows the comparison of peak time between the values obtained using equation and mathematical model

A comparison between the values obtained using equations (9) and (10) are presented against those obtained from the mathematical model (Figures 4.19,4.20). The comparison shows that the equations can predict peak discharge and time to attain this value at any place between Mortakka to Mandleswar for a known hydrograph at Mortakka. Here it may be noted that the effects of cross-sections, bed slope and roughness are included in the constant terms in the right hand side of the equations (9) and (10). It is important to mention here that these equations are limited for flows confined to the main rivers as very high flood values are not considered in the derivation of the equations.

4.3 ESTMATION OF DISCHARGE FROMA TRIBUTARY

Unlike the reach from Mortakka to Mandleswar, the reach between Jamtara and Bermanghat has two major tributaries (Sher and Hiran) of Narmada River. Generally, the measurement sites are established on the main river and the flows from the tributaries remain ungauged. In this section, an attempt has been made to estimate discharge hydrograph for Hiran and Sher using the observed hydrograph at Jamtara and Bermanghat. The procedure for this purpose is described below.

The cross sections at Jamtara and Bermanghat are given in the figures 4.21(a) and (b). The Manning's roughness coefficient used in this reach is 0.060. First, the volume contribution from both the tributaries (Hiran and Sher) is calculated by deducting the upstream hydrograph (Jamtara) from downstream hydrograph (Bermanghat). This total volume is shared in the ratio of the areas for the Catchments of the tributaries. The shape of the hydrograph for a tributary is calculated based on the weitage factors for each time interval. The weitage factor is determined using the inverse square law for distances from the observed sites(Jamtara and Bermanghat). The procedure is explained systematically through on the following steps.

Flood rouing taking the hydrographs from both the tributaries (Hiran and Sher) produces a better match of hydrographs (Observed versus Computed) at the downstream station on the main river Narmada shown in the figure 4.24.

The observed discharge is 4056 cumecs and time to peak 26 hr. and computed discharge is 4071 cumecs and time to peak is 26 hr. The computed tributaris peak discharge for Hiran and Sher is 719.86 and 574.06 cumecs and time to peak 7 hr. (both). The hydrographs for Jamtara and Bermanghat (observed) and hydrographs for Hiran and Sher are given in figure 4.22 and 4.23 respectively.

STEP:

1) V1 and V2 is the volume computed at Jamtara (station 1) and Bermanghat (station 2).
2) Volume from tributaries, (VT) = V2-V1
3) Distribution of VT to the each tributary in the ratio of individual catchment area of Hiran(AT1) and Sher(AT2).

   VT1=(AT1*VT)/(AT1+AT2)

   VT2=(AT2*VT)/(AT1+AT2)

   Where, VT1 and VT2 is the volume of tributaries Hiran and Sher. and AT1 and AT2 is the catchment area of Hiran and Sher.

4) From the Observed hydrograph at station 1 and 2, compute percentage of area, O1 and O2.for Jamtara and Bermanghat. i.e. $O1 = Oi_1 / \Sigma Oin_1$, and $O2 = Oi_2 / \Sigma Oin_2$, where Oi_1, Oi_2 is the value(each) of ordinate of hydrograph at Jamtara.and Bermanghat and ΣOin_1, ΣOin_2 is the total value of ordinates for Jamtara and Bermanghat.

5) Distribute the percentage of area (O1 and O2) inversely proportional to the square of distance of tributaries from the station 1 and 2.for each ordinates.

$$OT1=(D_1^2 *O1)/(D_1^2 +D_2^2) +(D_2^2 *O2)/(D_1^2 +D_2^2)$$

$$OT2=(D_1^2 *O1)/(D_1^2 +D_2^2) +(D_2^2 *O2)/(D_1^2 +D_2^2)$$

Where D1 and D2 is the confluence distances of tributaries from Jamtara and Bermanghat. OT1 and OT2 is the distribution of observed hydrographs ordinates (each) for Jamtara and Bermanghat

6) Compute the hydrograph ordinates of tributaries Hiran(QTH) and Sher(QTS) i.e.

$$QTH= (OT1*VT1)/\Sigma OT1$$

$$QTS= (OT2*VT2)/\Sigma OT2$$

Where OT1 and OT2 is the distribution of observed hydrograph ordinates (each) , VT1 and VT2 is the volume of tributaries Hiran and Sher and ΣOT1 and ΣOT2 is the total distribution of observed hydrograph ordinates.

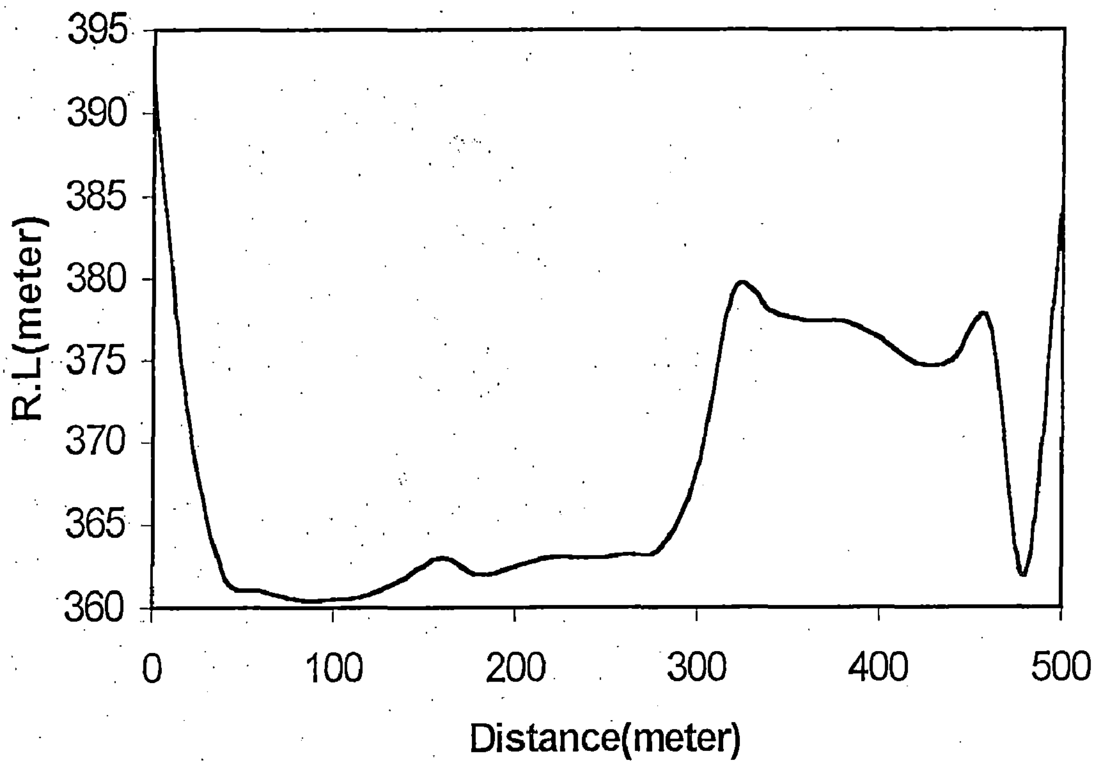


Figure 4.21(a) Cross section at Jamtara

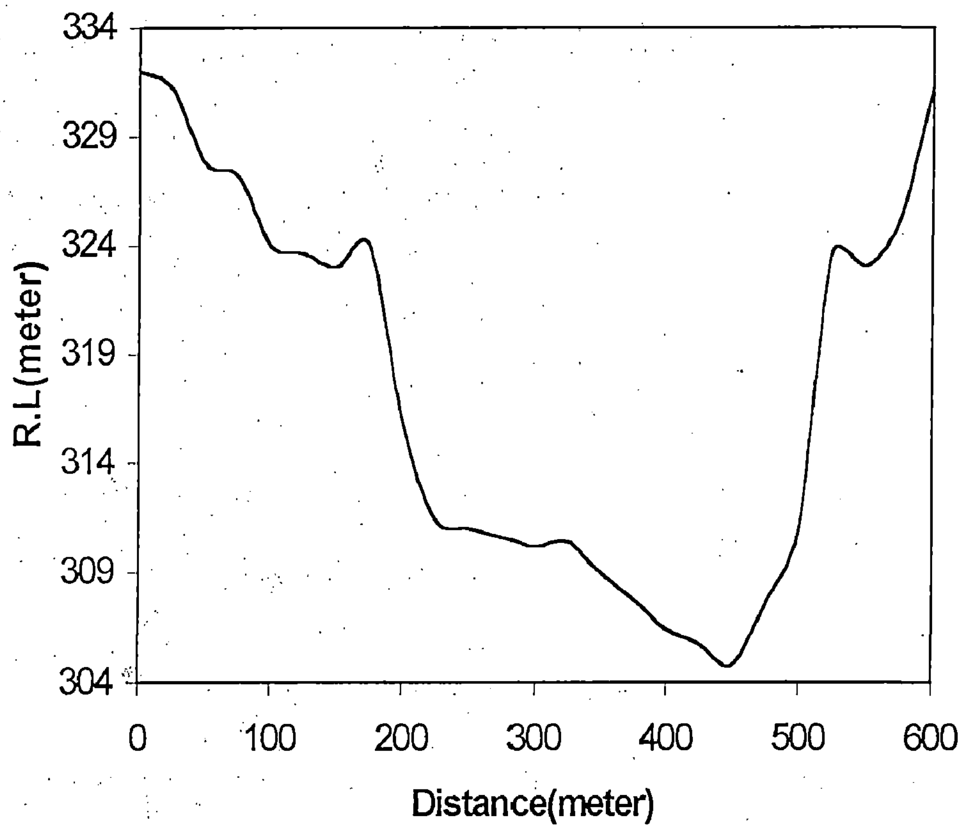


Figure 4.21(b) Cross section at Bermanghat

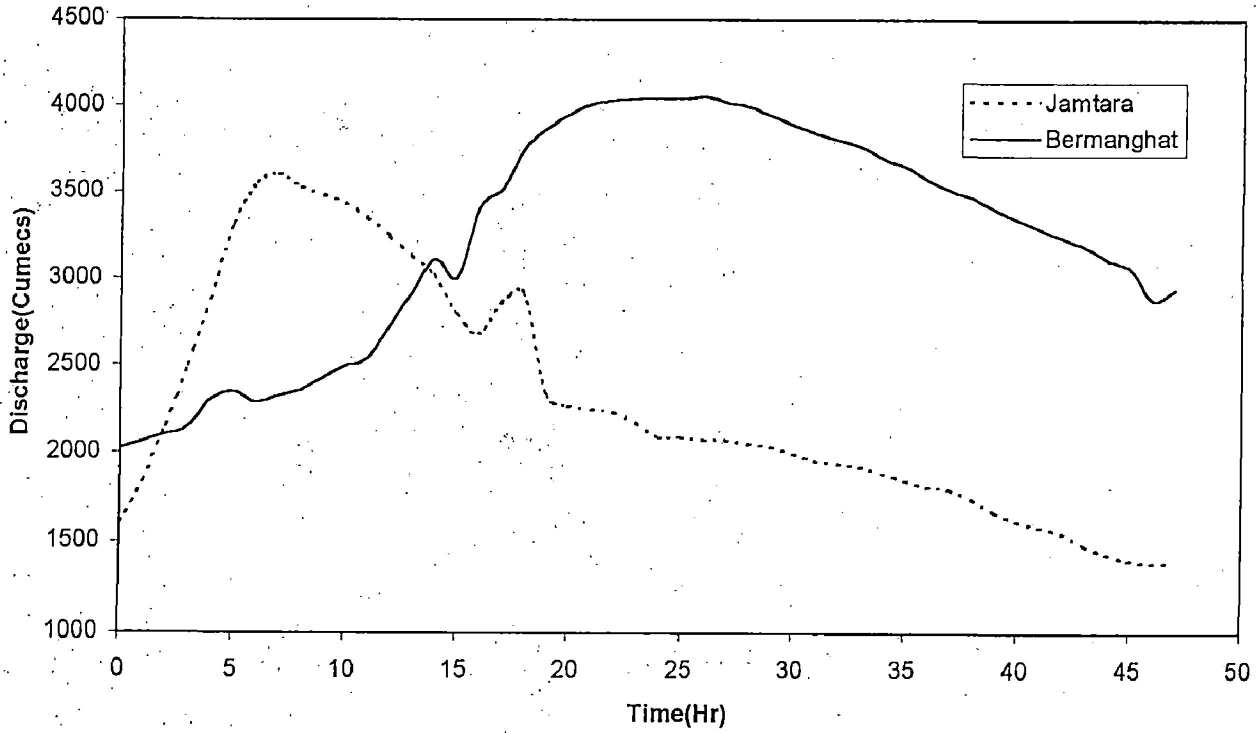


Figure 4.22 Hydrographs at Jamtara and Bermanghat(Observed)

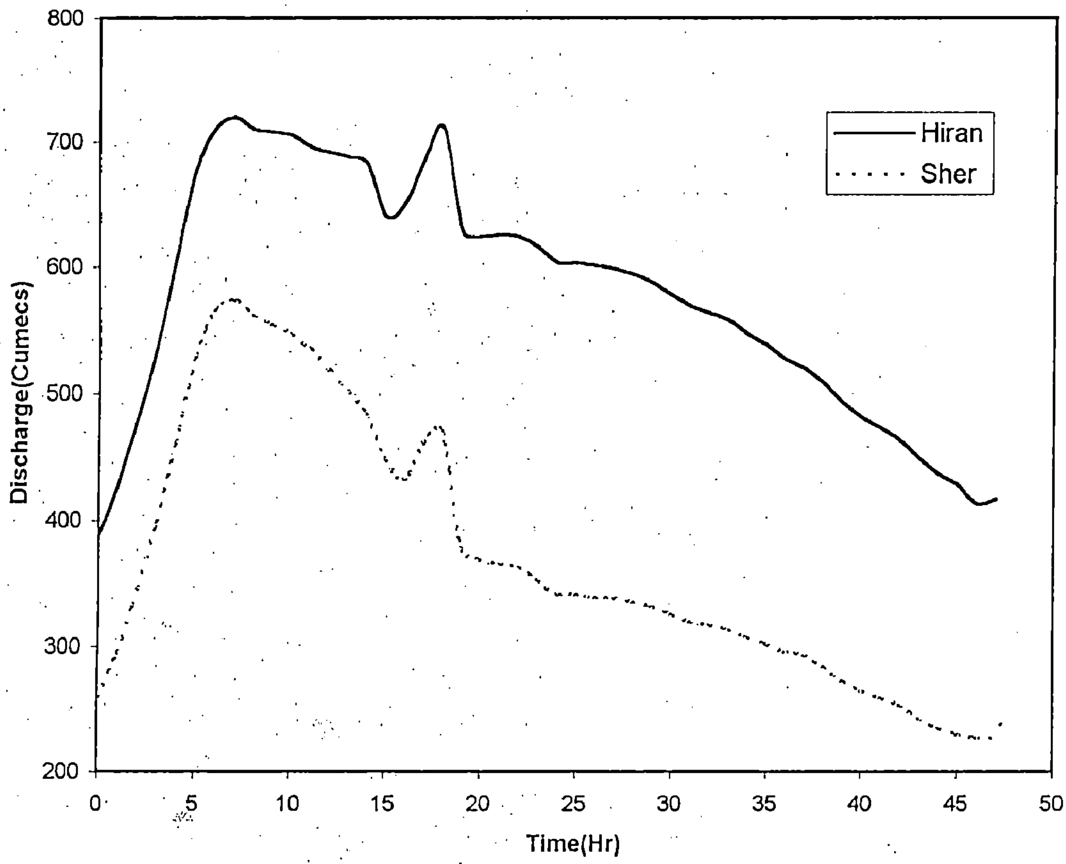


Figure 4.23 Hydrograph of Sher and Hiran

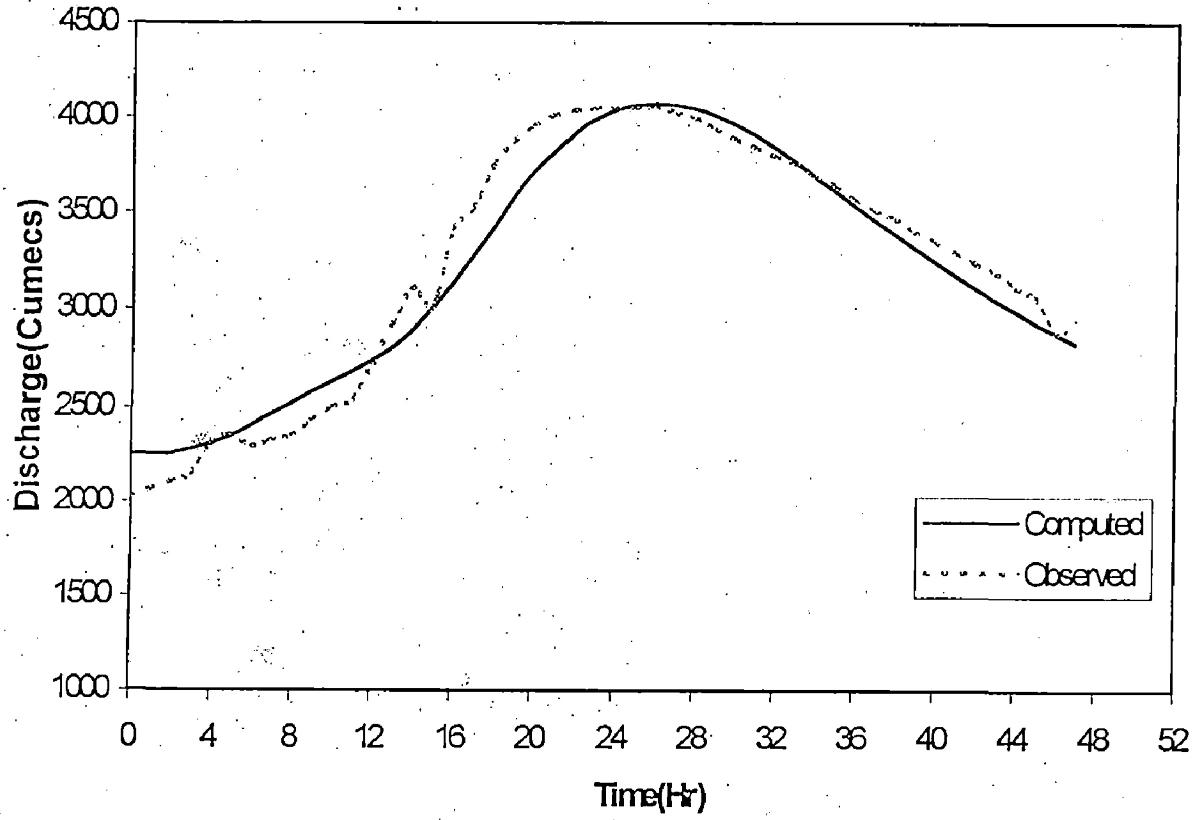


Figure 4.24 Outflow hydrograph at Bermanghat (Computed Vs. Observed)

Chapter 5

CONCLUSION

In this study, specific reaches in the river Narmada was attempted for flood routing. For the above purpose, a computer model for channel routing (Fread, 1988) using one – dimensional Saint Venant equations was used. Different model parameters were calibrated using an indirect technique for water flow conservation.

The important conclusion of the present study are given below :

1. In the flood routing study from Mortakka to Mandleswar, the computer hydrograph matches well with the observed hydrograph at Mandleswar. Although the peak discharges are approximately equal, the time to peak values obtained by the computational model are always higher.
2. The considerations for the intermediate flows are important. These can not be neglected.
3. Derived generalized equations can predict the flood peak and time to attain these values at any intermediate location, using the information at Mortakka.
4. The derived generalized equations are not valid for very high flows.
5. In the flood routing studies from Jamtara to Bermanghat, a procedure is developed to estimate the discharges in the two tributaries present between these two stations.
6. Taking the tributary flows into account the computer hydrograph at Bermanghat matches satisfactory with the observed hydrograph at that place.

RECOMMENDATIONS

The following recommendations as listed below for future studies :

1. An inverse technique, using optimization method, to determine the contribution from the intermediate catchment area, should be development.
2. A criteria, to choose whether the intermediate contributions are significant or not, have to be developed.
3. Effect of sediment flows in flood routing studies for specific reaches in Narmada needs to be performed.
4. A methodology to extent the results obtained with limited data, for higher flood values is to be developed.

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APPENDIX – I

```
Narmada river
b.saha
   9      0      0      3     48      0      0      1
   1     48
1603   1832   2151   2493   2880   3314   3553   3602
3602   3529   3480   3432   3353   3243   3132   3014
2792   2683   2860   2917   2329   2269   2249   2230
2170   2093   2093   2074   2074   2055   2036   1998
1960   1942   1923   1886   1850   1813   1796   1742
1671   1620   1586   1552   1485   1436   1404   1387
   2      6      2      0      0      0      2      0
   1      2
 0.0
364.0  368.0  372.0  376.0  380.0  381.8
251.3  275.6  288.1  340.5  462.1  472.9
  0.0    0.0    0.0    0.0    0.0    0.0
111.0
318.7  322.0  324.0  326.0  328.0  331.3
315.78 328.4  404.2  433.6  528.8  568.4
  0.0    0.0    0.0    0.0    0.0    0.0
0.060  0.060  0.060  0.060  0.060  0.060
 1.0
   0      0
 0.0    0.0    0.0
  63     96
 388    426    479    535    606    677    710    720
 710    708    705    696    692    688    682    640
 651    686    712    628    624    625    623    615
 603    603    601    598    594    587    577    568
 563    557    547    538    527    520    509    493
 482    472    464    450    437    428    413    417
 258    294    345    399    460    528    566    574
 562    555    547    535    518    502    484    448
 432    461    470    378    369    366    363    354
 342    342    339    339    336    332    326    320
 317    314    308    302    296    293    285    273
 265    260    254    243    236    230    227    227
```