HYDRODYNAMIC MODELING OF RIVER CHENAB FOR FLOOD ROUTING (MARALA TO QADIRABAD REACH)

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ABSTRACT

The flood flow in a natural channel is purely unsteady, and should be analyzed by using full Saint Venant equations of continuity and momentum conservation. The mathematical modelling is the most commonly used tool to analyze the unsteady flow. The study was meant for the development of a (1-d) hydrodynamic flood routing model based on full Saint Venant equations. The governing equations were discretized by an implicit finite difference scheme (Pressimann Scheme). The system of equations was solved by the Newton Raphson method. The model was applied to river Chenab from Marala to Qadirabad reach for 1992 flood. The flood hydrograph of 1992 was routed through the selected reach by the hydrodynamic model. The series of hydrographs showing attenuation in peak flow at ten-km interval was calculated. A single representative uniform cross section (wide rectangular) was used as channel geometry in the model. It has been found that an average rectangular cross section can be adequately used to simulate flow conditions in a natural river. The developed scheme was also validated for the flood event of 1988 for Khanki to Qadirabad reach. The results of the model reveal that flood flow phenomena in the river Chenab from Marala to Qadirabad reach can be simulated well.

INTRODUCTION

The Indus Basin River System consists of five major rivers. The most of the rainfall occurs in monsoon period causing the flood condition in the rivers. The brief description of main rivers is as under.

Sutlej: The river Sutlej enters Pakistan at Ganda-Singh-Wala. The total length of upper catchment is 720 km. The floods in River Sutlej are rare due to construction of reservoirs such as Ponah Dam and Bhakra Dam having live storage 6166 MCM (5MAF) and 7030 MCM (5.7 MAF), respectively. However the flood may occur in late monsoon season when these reservoirs are filled and have no significant impact on peak flows.

Ravi: The river Ravi enters the Pakistan upstream of Jassar bridge. The river ravi has catchment area of about 11520 km^2 with the total length of the catchment is 224 km. India have completed Thein dam, with live storage over 3700 MCM (3 MAF) located about 70 miles upstream of Jassar bridge, the

Madhopur barrage is about 40 km upstream of Jassar. In 1988 the flood in river Ravi caused damages in the Khupura district.

Chenab: The Chenab enters Pakistan near Marala Barrage about 480 km from its origin. The Salal dam in India is located 64 km upstream of Marala, which runoff river power station. The flood damages due to river Chenab are frequent.

Jhelum: The Jhelum river has catchment area about 24600 km^2, the most of catchment area is in Kashmir. It has Mangla reservoir of 6166 MCM (5 MAF) capacity. The Punch river also enter the Mangla reservoir, which chment area about 4100 km^2 and peak flows in the order of 11325 cumecs (400,000 cusecs). The Jhelum river's flood in 1992 caused heavy damages to life & property.

Floods are natural phenomena in many countries and Pakistan is no exception, where rivers are flooded frequently. Devastating floods occurred in river Chenab during 1988 and 1992. Marala to Qadirabad reach experiences serious problems of embankments, erosion, bank sloughing, and flooding. The floods in the joining nallas are often sudden and have sharp peaks, which usually cause extensive damage and casualties. This sudden flood cause extensive damages to the nearby cities. The use of flood protection structure cannot always prevent flood damages as expected because i) the flood protection structures and levees are not designed for all the possible floods. ii) The river aggradations reduce the discharge carrying capacity of the channel. iii) Extensive use of river valley for agricultural production, industrial use and urbanization. Mathematical modelling is the most commonly used tool to analyze the unsteady flow. Numerical simulation requires less time as well as expenses as compared to physical models, which also involve distortion of scale. Once a system has been modeled it is an easy exercise to foresee the consequences of such a system, this only involves the change in data or minor modifications in model it self.

Flood routing is a procedure to predict the changing magnitude, speed, and shape of flood wave as it propagates through the waterways, such as a canal, river, reservoir, or estuary. It is the process of calculating flow conditions (discharge, depth) at certain section in a channel from the known initial and boundary conditions in the selected reach. The flood hydrograph is modified in two ways. Firstly the time to peak rate of flow occur later at the downstream point this is known as translation. Secondly the magnitude of the peak rate of flow diminishes at downstream point, the shape of the hydrograph flattens out, so the volume of the water takes longer time to pass a lower section. This second modification to the hydrograph is called attenuation

The objective of this article is to present calibration and validation results of 1-d hydrodynamic flood routing model.

Past Studies

Lai [1] (1986) studied different methods for numerical modelling of unsteady flow in open channel. He concluded that among different numerical methods the implicit finite difference method gave good results. Warwick and Kenneth.[2] (1995) made comparison of two hydrodynamic models (HYNHYD) and RIVMOD developed by U.S Environmental Protection Agency**.** Both models were based on explicit method. The explicit method requires relatively small computational time step, about five seconds time step was used to ensure model stability. A larger computational time step can be used if the continuity and the momentum equations were solved by an implicit method instead of an explicit method. The model results compared well with the observed data. Wurbs[3] (1987) studied the Dam Break flood wave models and applied only selected ones from several leading routing models. Dynamic routing model is preferred when a maximum level of accuracy is required. U.S. National Weather Service [4] (1970) developed a dynamic wave routing model, DWOPER (Dynamic Wave Operation Model). Saint-Venant equations were solved by an implicit method. The model was applied on various rivers satisfactorily. Greco[5] (1977) developed a model named as Flood Routing Generalized System (FROGS**)** to simulate flood wave propagation in natural or artificial channel. The complete Saint-Venant equations were solved by an implicit finite difference scheme. The model was simulated for flood event in 1951. Yar [6] developed a hydraulic model for simulation of unsteady flow in meandering rivers with flood plain. This 1-d model was based on the complete solution of Saint-Venant equations by implicit finite difference method. Huge data is required to simulate hydraulic and topographic conditions. Fread [7] developed the dynamic wave model (FLDWAV) for one-dimensional unsteady flow in a single or branched waterways. The Saint Venant equations were solved by four-point implicit finite difference scheme. This model was successfully applied to simulate downstream flood wave caused by failure of Tento Dam in Idaho. Tainaka and Kuwahara [8] (1995) developed a hydrodynamic model for sand spit flushing at river mouth during a flood.

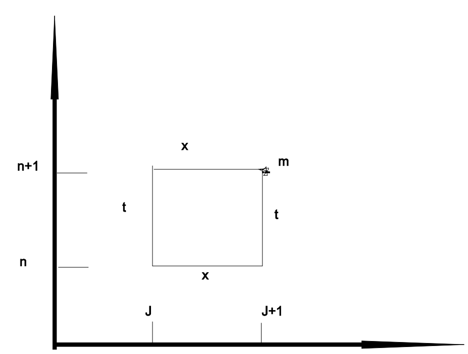


Figure 1: Generalize Pressimenn Scheme

The hydraulic equations were solved along with the sediment equation. A leapfrog finite difference scheme with uniform grid spacing was used to formulate the differential equations. The hydrodynamic and the morphological equations were coupled. The model computes water level, velocity, and sediment transport rate. This model was applied to Natori River in Japan. The model results were satisfactory. Amein and Fang[9] (1970) developed a hydrodynamic flood routing model solving the complete Saint Venant equation by an implicit method. The fixed mesh implicit method is more stable as compared to explicit method.

MODEL FORMULATION

The continuity and momentum equations were used for formulation of hydrodynamic model. The continuity and momentum equations are:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = ql \qquad (1)$$

$$\frac{\partial Q}{\partial t} + \beta \frac{\partial(\frac{Q^2}{A})}{\partial x} + g\frac{A}{T}\frac{\partial A}{\partial x} - gA(S_o - S_f) - ql\frac{Q}{A} = 0 \qquad (2)$$

Where Q is discharge or volume flow rate at distance x (m^3 s^{-1}), A is cross-sectional area (m^2), x is distance along the flow (m) and t is time (s), ql is the lateral inflow to the channel, S_f is the friction slope.

The equations (1) and (2) were numerically solved using implicit box scheme of Preissmann and Cunge (1961). It has some advantages such as variable spatial grid may be used and steep fronts may be properly simulated by varying the weighing coefficients θ and ϕ. This scheme gives better solution for linearized and non-linearized form of governing equations for a particular value of Δ_x and Δ_t. The detailed numerical solution can be found in Fread (1973). The Schematic diagram of Preissmann scheme is shown in Figure 1. The pressimann scheme in generalized form may be written as.

$$\int(x,t)_m = \theta \left\{(\phi \int_{j+1}^{n+1} + (1-\phi)\int_{j}^{n+1} \right\} + (1-\theta)\left\{(\phi \int_{j+1}^{n} + (1-\theta)\int_{j}^{n})\right\} \qquad (3)$$

$$\frac{\partial \int(m)}{\partial x} = \frac{1}{\Delta x}\left\{\theta \left(\int_{j+1}^{n+1} - \int_{j}^{n+1}\right) + (1-\theta)\left(\int_{j+1}^{n} - \int_{j}^{n}\right)\right\} \qquad (4)$$

$$\frac{\partial \int(m)}{\partial t} = \frac{1}{\Delta t}\left\{\phi \left(\int_{j+1}^{n+1} - \int_{j}^{n+1}\right) + (1-\phi)\left(\int_{j+1}^{n} - \int_{j}^{n}\right)\right\} \qquad (5)$$

In Equations (3) to (5) j and n are grid locations and θ and ϕ are the weighing factors for the time and space, respectively. As an example the value of Q at jth spatial grid point and nth time grid point will be denoted by Q_j^n. The known time level is denoted by superscript n, unknown time level by n+1. Applying finite difference approximation Equation (3) to (5) to the Equation (1) and (2) gave two equations for every box with four unknowns. This process is called discretization of the governing equations. A system of nonlinear algebraic equations was obtained. After discretization, the finite difference relationship made a system of nonlinear algebraic equations with four unknowns. As there were N nodle points on a reach, so there were (N-1) rectangular grids and (N-1) interior nodes so the equations applied at each node produced 2 (N-1) equations. However two unknowns were common to any two contiguous rectangular grids. So there are 2N unknowns and 2 (N-1) equations, two additional equations were required for the system to be determinate. These two equations were provided by boundary conditions, one at upstream end and second at down stream end. The resulting system of equation was solved by Newton-Raphson method, which was firstly applied by Amein and Fang (1970) to an implicit non-linear formulation of Saint Venant equations. The Newton-Raphson method was used to solve this system of equations. In this method trial values were assigned to the unknown variables Q and A in each node. These values were iterated to refine the solution. To determine the correction for each iteration, partial derivative of equation (1) and (2) and of boundary equations with respect to $Q_j^{n+1}, A_j^{n+1}, Q_{j+1}^{n+1}, A_{j+1}$ was required. After taking the partial derivative, the equations are arranged in matrix form.

$$A\,X = B \tag{6}$$

Where A is a matrix having the partial derivative and X is a column vector consist of correction, B is the column vector having known values called residuals. The detailed form of above equation is given in Amein and Fang (1970).

STUDY AREA

The study area is situated on the north eastern part of the Punjab province between longitude 32^o 30$'$ to 33^o 00$'$ E and latitude 73^o 40$'$ to 74^o,50$'$ as shown in Figure 2 . The length of the Chenab river reach selected for the study purpose is 88 km (from Marala to Qadirabad).

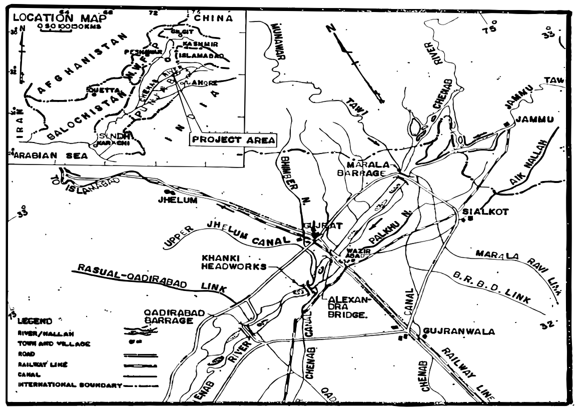


Figure 2: Location of the study area

The average slope is 0.38 meter per km. Two major tributaries, Jammu Tawi and Munawar Tawi join the Chenab river at upstream of the Marala barrage, which is located about 10 km Below Marala barrage the river flows in plain area, where it attains the flatter slope. The flow data consist of, flood hydrograph at Marala Headworks, flood hydrograph at Khanki Headworks and Flood hydrographs at Qadirabad Barrage.The collected data was processed before using in the hydrodynamic models. A representative cross-section was prepared to use in the model.

APPLICATION OF THE MODEL AND RESULTS

Marala to Khanki

The Marala to Khanki reach is 56 Km long. The basic data used for this reach is given below: The wide rectangular channel having width = 2050 m was used in hydrodynamic model. The bed slope calculated from data was 0.0004. The elevations are taken reference to mean sea level. The Manning's roughness coefficient is taken as n = 0.025. The steady state flow condition was taken as initial flow conditions. The initial discharge was 850 cumecs. The initial flow depth was assumed as normal depth. The initial depth calculated by Manning's formula was 0.75 m. Inflow flood hydrograph of 1992 flood from 06-09-1992, at 6.00 a.m to 20-09-1992, at 6.00 a.m at Marala barrage was used as upstream boundary condition. The Manning's equation was used as downstream boundary condition. Flood hydrograph at Marala barrage was input of hydrodynamic model and the output was outflow hydrograph at Khanki headworks.

The outflow hydrograph computed by the hydrodynamic model and observed hydrograph at Khanki Headwork compare well as shown in Figure 3. The flood wave attenuates as it propagates through waterway. The hydrodynamic model differs from simplified model as the kinematic model does not show wave attenuation. A series of flood hydrograph showing attenuation in peak at 10 km interval is shown in Figure 4. The distance between each hydrograph is same and therefore attenuation of peak is also equal.

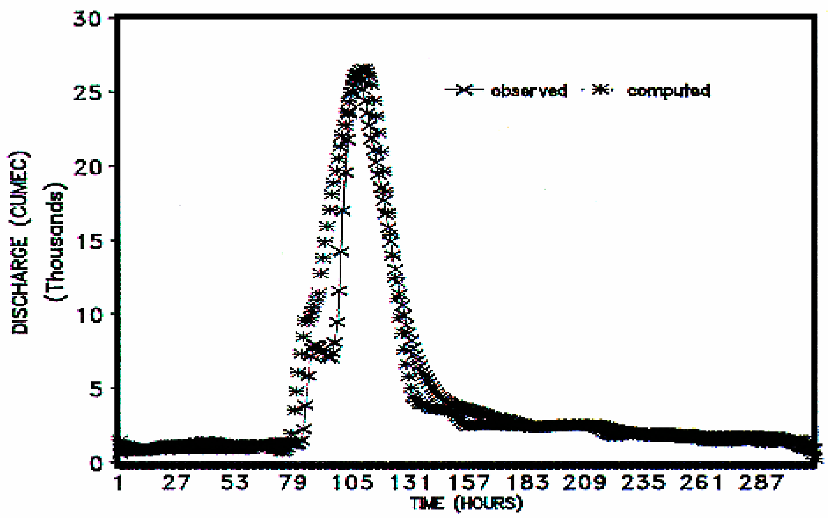


Figure 3. Observed and computed flood hydrograph at

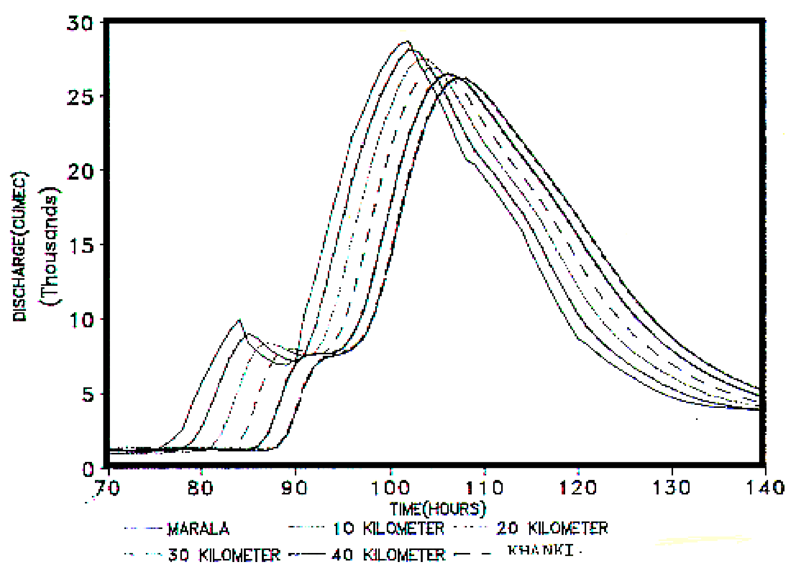


Figure 4. Series of hydrographs showing attenuation of

Khanki to Qadirabad reach

Khanki to Qadirabad reach is 29 km long. The channel width was 1750 m, the bed slope was 0.00041. The Manning's roughness coefficient n = 0.025. Normal flow condition was used as initial flow condition. The initial discharge was used 1026 cumecs. The initial flow depth calculated by Manning's formula was 0.75 m. The flood hydrograph of 1992 from 06-09-1992, at 6.00 a.m. to 20-09-1992, at 6.00 a.m was used as upstream boundary condition. The Manning's equation was used as downstream boundary condition. In Khanki to Qadirabad reach the inflow hydrograph is the flood hydrograph at Khanki headworks.The outflow hydrograph is computed at Qadirabad. The computed and the observed hydrograph compared well with each other as shown in Figure 5.

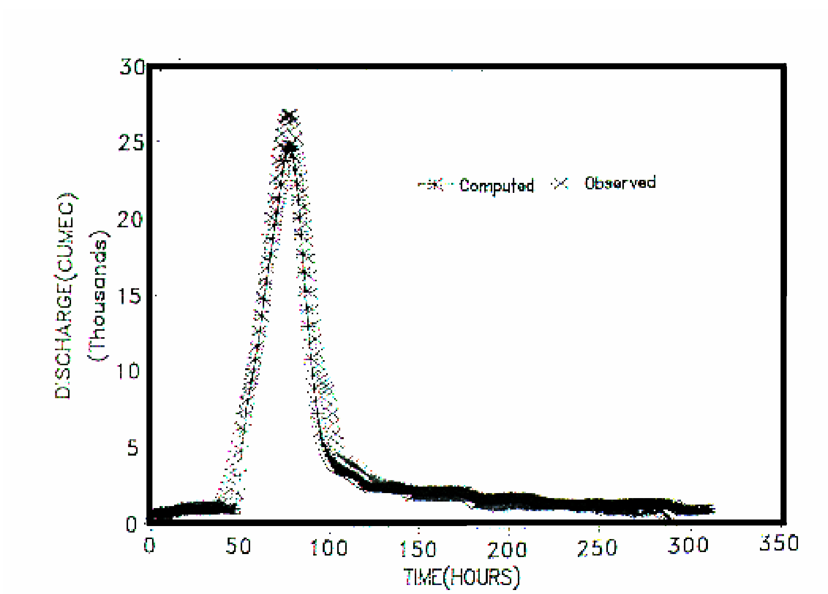
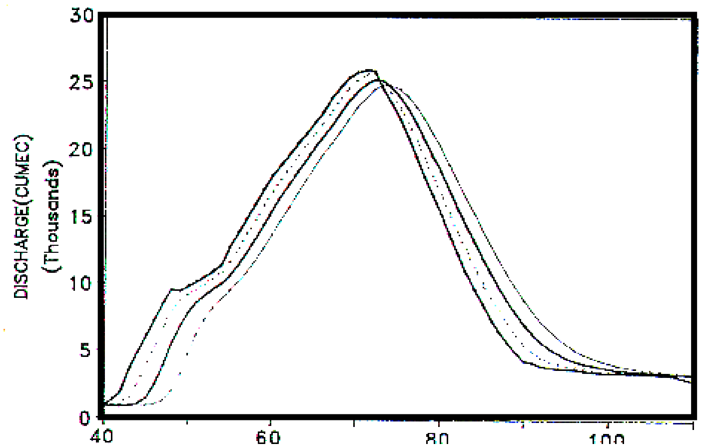


Figure 5. Comparison between computed and observed flood Hydrograph at Qadirabad



Time (hours)

Figure 6. Series of hydrograph showing attenuation of peak

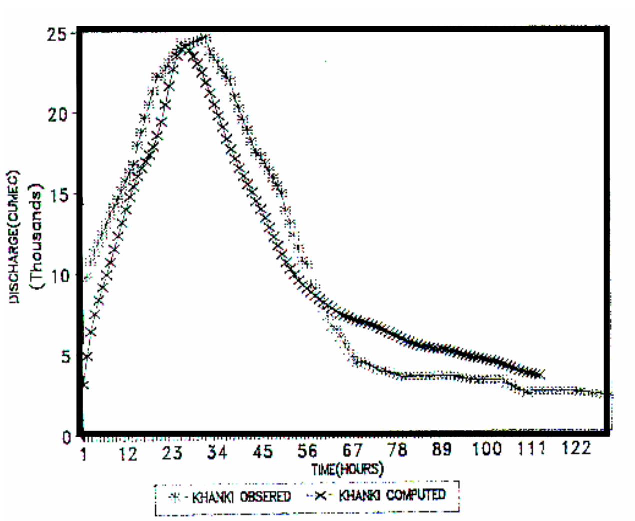


Figure 7. Comparison between observed and computed hydrographs at Khanki.

The attenuation in peak of flood hydrographs computed at 10 Kms intervals is shown in Fig.6. The distances are same so attenuation in peak is also same.

VALIDATION OF HYDRODYNAMIC MODEL

Marala to Khanki

The validation of hydrodynamic model was done for the flood event of 1988. The model was run for the both reach Marala to Khanki and Khanki to Qadirabad. The results observed in both reaches are shown in Figure 7. The same topographic and cross sectional data was used for the validation of the hydrodynamic model for the Marala to Khanki reach. The flood hydrograph of 1988 from 24-09-1988 to 30-09-1988 was used as input hydrograph at Marala. The output hydrograph was computed at Khanki. The Figure 7 shows the comparison between computed and observed hydrograph at Khanki.

There were small discrepancies between observed and computed values in the recession limb of hydrograph. This difference maybe due to the following reasons.

I. There may some initial storage after passing the peak discharge, the storage volume is also mixed with outflow discharge, so the discharge in recession is increased.
II. The river cross sections change every year specially in the flood season due to heavy sediment load.
III. The lateral inflow is not included in the hydrodynamic model. The Marala to Khanki reach is 56 km long and many Nallas are joining the river which increases the discharge.
IV. The observed hydrograph at Khanki and Qadirabad are almost similar to each other, but in real situation is that the hydrographs at upstream and downstream cannot be equal. There may be error in observed data.

As the hydrodynamic model is generalized model developed for the flood routing, in the flood study the peak flow is very important. The Figure 7 shows that model had successfully simulated the peak discharge. So the discharge in the recession limb are not so important as compared to peak discharge.

Khanki to Qadirabad

The flood hydrograph of 1988 was routed from Khanki to Qadirabad reach. The same topographic and cross sectional data was used. The inflow hydrograph at Khanki was used from 25-09-1988 to 01-10 1988. The outflow hydrograph was computed at Qadirabad. There is good agreement between computed and observed hydrographs at Qadirabad as ahown in Figure 8.

CONCLUSIONS

1. The steady state flow depth computed for an initial discharge provide

satisfactory initial conditions.

2. Because the hydrodynamic model is based on Implicit finite difference method (Pressimann Scheme), irregular mesh interval can be used along the river reach. Similarly unequal time step can be used.
3. The hydrodynamic model is capable of simulating the shape of output flood hydrograph as well as the time lag. The attenuation observed can be accurately predicted by the model.
4. The hydrodynamic model can be used successfully if large number of cross sections are not available in a channel reach.

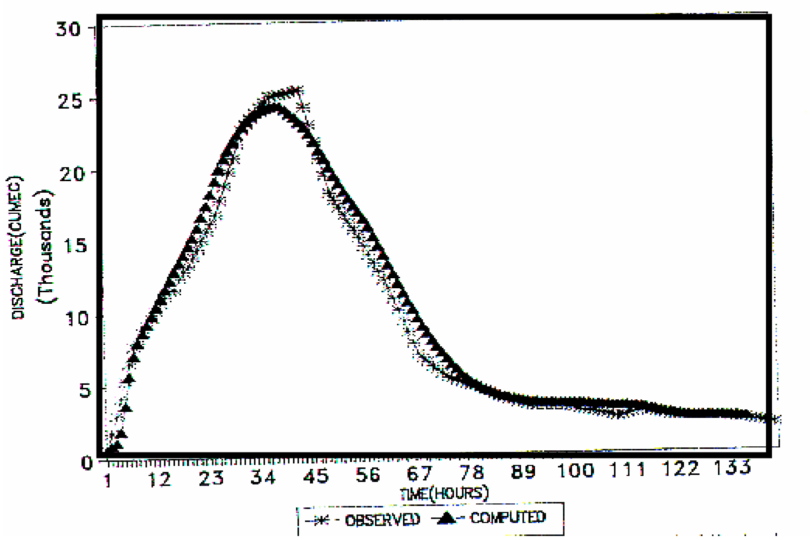


Figure 8. Comparison between observe and computed flood hydrographs at Qadirabad

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