Unsteady flow regulation in open channel by using inverse explicit method

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By

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NATIONAL INSTITUTE OF TECHNOLOGY, ROURKELA

DECLARATION

I hereby declare that this submission is my own work and that, to the best of my knowledge and belief, it contains no material previously published or written by another person nor material which to a substantial extent has been accepted for the award of any other degree or diploma of the university or other institute of higher learning, except where due acknowledgement has been made in the text.

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CERTIFICATE

This is to certify that the thesis entitled, **"Unsteady flow regulation in open channel by using inverse explicit method"** submitted by **Bhabani Shankar Das** in partial fulfilment of the requirements for the award of Master of Technology Degree in **CIVIL ENGINEERING** with specialization in **"WATER RESOURCE ENGINEERING"** at the National Institute of Technology, Rourkela is an authentic work carried out by her under my supervision and guidance.

To the best of my knowledge, the matter embodied in the thesis has not been submitted to any other University / Institute for the award of any Degree or Diploma.

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ABSTRACT

Flood routing and operation-type problems are two major problems which are required to be solved frequently in unsteady flow problems in open channels. Now-a-days for routing and regulating problem, generally the Saint-Venant equation is used to predict the discharge and water stage at the study area in the channel. Routing of flood calculates the discharge and flow depth at future time series. On the other hand the operation problem is mainly used to compute the inflow at required upstream section for regulating structure of the delivery system to get a predefined water demand at required section at downstream end of the channel. So it is the inverse computational problem from downstream to the study section at upstream. So an explicit finite difference scheme which is solved from downstream to upstream also known as inverse explicit scheme is presented to solve the operation type problem in open channels. The finite difference inverse explicit scheme is applied to solve the Saint-Venant equations based on the discretization of the Preissmann implicit scheme. In this research the finite difference inverse explicit method is applied to a rectangular canal. The computation is performed by proceeding first backward in time and then backward in space from downstream. The method is numerically stable and the computed upstream discharge hydrograph by the inverse explicit scheme, when used as upstream boundary condition in the routing problem by the HEC-RAS commercial computer model, reproduce downstream flow hydrographs very close to the predefined outflow. The suitable value of weighting coefficient in terms of time ($\emptyset$) and time step (Δt) is determined.

Key words: Operating problems, Saint-Venant equations, explicit finite difference method, inverse computational problem, Preissmann scheme, discretization, HECRAS

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NOTATIONS

A = wetted cross-sectional area

b = wetted top width

B= bottom width of the channel

g = gravitational acceleration

Q = discharge (through Area A)

q= Q/B

V= velocity of flow

y = depth of flow

Δt = time interval

Δx = space interval

i= grid size foe time

j= grid size for space

S_0 = bottom slope of the channel

S_f = friction slope.

$\emptyset$ = a weighting coefficient for distributing term in space

θ = a weighting coefficient for distributing term in time

C_n=Courant number

V_n= Vedernikov number

C = Celerity

CFL-Courant-Friedrichs-Lewy

HECRAS-Hydrologic Engineering Centre and River Analysis System

IE Method- Inverse Explicit Method

CHAPTER 1

INTRODUCTION

1.1 OVERVIEW

Most natural flows in a streams and rivers change slowly with time. Also, man-made channels and canal have gates that permit a greater or lesser flow through their structures in response to changing demand in water requirement. An important problem that has not been solved and will probably never be completely solved is how control are to be operated in time to optimize benefits to the water users, while minimizing waste, and anticipating change in demand caused by weather conditions, crop requirements , and supply limitations. Thus the flow in manmade channel is often controlled so as to be unsteady. The fact is that the real world, most open channel flows are not steady state, but unsteady and often only modestly so. However, the design of most channels has been for some steady state flow rate, generally the largest that the channel is expected to carry. Thus the study of unsteady flows in open channels is directed more towards the analysis of "what if" questions, than in the design of channel systems. As computers are able to quickly perform the numerous calculations associated with the unsteady open channel hydraulics, there will be greater emphasis on unsteady flow, and the design of channel systems, that might possibly occur in their operation. The intent of this thesis is to provide the introduction to the concepts associated with the unsteady open channel hydraulics and provided minimum back ground into currently used method for solving such problems.

Modelling unsteady flow in open channels using computers is an essential part of water-resources engineering. Expression of the principles of conservation of mass and momentum are required for analysis of the unsteady flow which change its flow characteristic with time. By numerical method the unsteady flow is solved very accurately so that the results obtained are practically applied in any problem case of water resource. Mathematical models of unsteady flow in open channels applied the fully dynamic Saint-Venant equation for analysis. These equations are typically used to predict canal and river flows to both analyse water-delivery schemes and determine possible flood conditions. In addition to modelling the conditions in unregulated channels, an open-channel network model should include hydraulic structures, such as gates, weirs, and flumes. Using such a model, the opening and closing of gates under flood conditions could be simulated and efficient flood-control measures determined. In operational type problem, the main work is to regulate the flow in various problems such flood defence design, navigation, forecasting of flood, darn-break analysis, and irrigation scheme control. In irrigation system operational type problems are focused now-a-days. Precise control of water is becoming very important due to the increasing of water demand for every purpose. Any control should be done in the upstream inflow so that losses of water or shortage of water could not be found at required demand area. The main objective of operation along irrigation canals aims for regulating structures at upstream to maintain a required water demand at downstream. Mathematical model is used to predict the upstream inflow according to the required downstream flow precisely. The partial differential Saint-Venant equations, which express the both principles of conservation of mass and momentum, can be solved at a finite number of grid points in the rectangular channel. The model developed by writing computer programs for operating type of problem like irrigation control. Some models are developed for simulation type of problems to know the upstream hydrograph for the demand of the downstream. The prediction of the changes between two hydrograph are very complex as the solution includes the nonlinear hyperbolic equations for building a numerical model the complex methods are to be assumed which are very time consuming. Proper solution cannot be easily obtained if the procedures are not accurate for exact solution.

For explicit method the characteristics curves are used for gate stroking ,Wylie (1969) to set the gate for proper control. For predetermined demand of downstream hydrograph the proper movement of gate at upstream is necessary. So by using the characteristic the proper inflow at upstream can be known by setting the proper arrangement of gate. But application of the methods of characteristics is very much complex (Liu et al. 1992, M.T. Shama, 2003). The modified method of characteristics are also developed now a days. For gate operation the inverse implicit method then used in simplified approach.The inverse explicit method presented in this book, being an explicit finite difference method based on preissmann scheme, can provide an easier solution.

1.2. OPEN CHANNEL FLOW

Open channel flow is distinguished from closed conduit flow or pipe flow by the presence of a free surface, or interface, between the two different fluids of different densities. The two most common fluids involved are water and air. The presence of free surface makes the subject of

Open channel flow, more complex and more difficult to compute commonly needed information about the flow, than closed conduit flow or pipe flow. A free surface is a surface having constant pressure such as atmospheric pressure. The flow of water through pipes at atmospheric pressure or when the water level in pipe is below the top of the pipe is also classified as open channel flow. In this flow, as the pressure is the atmospheric pressure, the flow takes place under the force of gravity which means the flow is due to the bed slope of the channel only. The hydraulic gradient line coincides with the free surface of water. In pipe flow problem, the cross sectional area of the flow is known to equal the area of pipe. In open channel flow, the area depends upon the depth of flow, which is generally unknown, and must be determine as part of the solution processes. Coupling this added complexity with the fact that there are more open channel flows around us than there are pipe flows, emphasizes the need for engineers, who plan work in water related fields, to acquire proficiency open channel hydraulics. The wide use of computers in engineering practice reduces the need for graphical, table look up, and the other techniques learned by the engineers who received their training a decade ago. The examples of open channel flow are Flow in natural rivers, streams, rivulets, drains, flow in irrigation canal, sewers, and flow in culverts with a free surface, flow in pipes not running full, and flow over streets after heavy rain fall.

1.3. ONE-, TWO-, THREE- DIMENSIONAL FLOWS

The dimensionality of a flow is defined as the number of independent space variables that are needed to describe the flow mathematically. If the variable of a flow change only in the direction of one space variable, e.g. in the direction along the channel x, then the flow is described as one dimensional flow. For such flows, variables such as depth Y and velocity V are only function of x, i.e. Y(x) and V(x). If the variables of flow change in two directions, such as the position along the channel x, and the position from the bottom of the channel y, or the position across the channel z, then the flow is described as two dimensional. For two dimensional flows the mathematical notation of variables contains two arguments such as Y(x, y) and V(x, y), or Y(x, z) and V(x, z). If the variables of flow change in three directions, such as the position along the channel x, with the vertical position within the flow, and the horizontal position across the flow, then the flow is three dimensional. If a Cartesian coordinate system with axes x, y, z is used , then the three dimensional flows are described mathematically by noting that the variables of the flow are a function of all three of these independent variables, or the velocity, for example ,is denoted as V(x, y, z), to indicate that its magnitude varies with respect to x, y, and z in space, and since velocity is a vector , its direction also depends on x, y, z, and t. If the velocity also changes with time, the additional dependency can be denoted by V(x, y, z, t).

A two dimensional flow that changes in time would have its velocity described as V(x, y, t) or V(x, z, t), and thus depend upon the three independent variables, and, from a mathematical point of view, is three dimensional. However in fluid mechanics such flows are called two dimensional, unsteady. A flow with V(x, z) is called two dimensional, steady. The notation for the variables of one-dimensional, unsteady flow consists of Y(x, t), V(x, t).The flow considered in this research work is one-dimensional.

1.4. CLASSIFICATION OF FLOW IN CHANNELS

Open channel flow can be classified and described in various ways based on the change in flow depth with respect to time and space. The flow in open channel is classified into the following types:

1. Steady flow and unsteady flow (Time as the criterion)
2. Uniform and non-uniform flow (Space as the criterion)
3. Laminar, transitional and turbulent flow (Reynold's number as the criterion)
4. Subcritical, critical and super critical flow (Frouds's number as the criterion)

1.4.1. Unsteady flow

In this research, we consider about the unsteady flow in open channel. If the flow characteristics such as depth of flow, velocity of flow, rate of flow at any point in the open channel, changes with respect to time, the flow is said to be unsteady flow .

Mathematically unsteady flow means,

$$\frac{\partial V}{\partial t} \neq 0 \text{ or } \frac{\partial y}{\partial t} \neq 0 \text{ or } \frac{\partial Q}{\partial t} \neq 0 \qquad (1.1)$$

1.5. TYPES OF CHANNELS

Channels are generally two types prismatic or non-prismatic. A prismatic channel has the same geometry throughout its length i.e. it has an unvarying cross section, a constant bottom slope, and other properties such as the wall roughness that does not change with position. This may consists of a trapezoidal section, a rectangular section, a circular section or any other fixed section. If the shape and/or size of the section changes with the position along the channel, the channel is referred to as a non-prismatic channel. Thus most man-made channels that are made from building materials are generally prismatic channel, but portions, such as channel transitions, will be non-prismatic. In theory, (while rare over a long distance) it is possible for a natural channel created by nature to be prismatic. However, in practice natural channel are non-prismatic. In this research work prismatic channel is consider which one is of rectangular type.

1.5.1. Rectangular Channel

A rectangular channel has vertical sides and a bottom width b. The cross sectional area is obtained from

$$A=bY. \tag{1.2}$$

And the wetted perimeter, P is computed from

$$P= b+2Y. \tag{1.3}$$

The top width of a rectangular channel is same as its bottom width, or T = b. For use with the momentum function in open channel flow, the first moment of area about the water surface will be denoted by Ah_c, and for a rectangle equal the area times, the distance from the water surface to centroid of rectangle, or one-half the depth and is given by

$$Ah_c = A\frac{y}{2} = \frac{bY^2}{2} \tag{1.4}$$

1.6. NUMERICAL METHOD

In Free surface flow, the unsteady flow is generally described by a set of quasi linear hyperbolic partial differential equations. The dependent variables are mainly velocity v and flow depth y of the Saint Venant equations are presented in terms of the partial derivatives of time t and distance x. But for practical applications in field, it is required to know the value of these variables rather the values of their derivatives. Except for some simplified case, a closed form solution of these equations is not available. Therefore the governing equations are integrated numerically for which several numerical methods can be possible for its solution. Various numerical methods are there with advantages and disadvantages. They are Finite-difference methods, Finite-element method, Finite-volume method, Method of characteristics, Spectral method, and Boundary-element method etc. Here in the research-work finite difference method is used.

1.6.1. Finite Difference Method

Out of all the numerical methods, finite difference methods have been used extensively by the investigators not only in Saint-Venant equations but also in some other non-linear partial difference equations as well. The principle of FDM is derivatives in the partial differential equation are approximated by linear combinations of function values at the grid points. The basic philosophy of FDM is replacement of derivative of governing equation with algebraic difference quotients results in a system of algebraic equations solvable for dependent variables at discrete grid points. Here the analytical solutions provide closed form expressions and variation of dependent variables within the domain. As for example unsteady non-linear partial difference equations in water hammer situations in pipe, flood simulation, regulation of water by control structures and unsteady equation in surge tanks have been suitably used. According to Cunge, et al. equations in conservative forms give better results than non-conservative form. Finite difference scheme is of two types-explicit and implicit finite schemes.

1.6.1.1 Explicit scheme

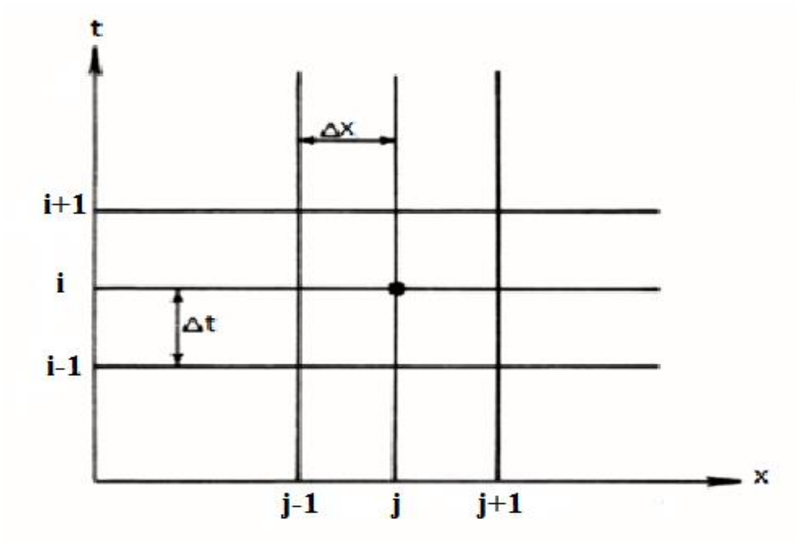

Fig.1.1. Grid of Finite Difference Scheme

It calculates the state of a system at a later time from the state of the system at the current time. Mathematically, If Y (t) is the current system state and Y (t+Δt) is a state at the later time (Δt is a small time step) then for an explicit method, Y (t+Δt) =F(Y (t)). If the n level, flow parameters are known then explicit method computes for n+1 time level. The basic principle of this method is to divide the flow domain in x-t plane in small grid of time and space as shown in Fig.1.Accuracy of finite difference method depends on the smaller value of Δx and Δt. There are various explicit finite difference schemes namely Lax diffusive Scheme, MacCormack Scheme, Lambda Scheme, Predictor scheme, Gabutti scheme etc. Accuracy of finite difference method depends on the smaller value of Δx and Δt.The reach of river or canal is divided into a number of small Δx and along the vertical time is divided into number of Δt. After generating regular grids i.e. Δx and Δt the main factor for the approximate solution is to follow the principle of finite difference method. That is the derivatives in the partial differential equation are linearly solved by approximating the values for every grid points with combinations of function values. If 'V' is the dependent variable and x and t independent variables (often space and time) then approximating the first-order derivatives for the space variables there are three methods for explicit scheme. The Special derivatives of finite difference schemes are forward difference method, central difference method and backward difference method. Mathematically for spatial derivatives at the grid point (j, i) in explicit FD scheme are given below:

Forward $\frac{\partial V}{\partial x} = \frac{V_{j+1}^{i} - V_{j}^{i}}{\Delta x}$, $\frac{\partial y}{\partial x} = \frac{y_{j+1}^{i} - y_{j}^{i}}{\Delta x}$ (1.5)

Central $\frac{\partial V}{\partial x} = \frac{V_{j+1}^{i} - V_{j-1}^{i}}{\Delta x}$, $\frac{\partial y}{\partial x} = \frac{y_{j+1}^{i} - y_{j-1}^{i}}{\Delta x}$

Backward $\frac{\partial V}{\partial x} = \frac{V_{j}^{i} - V_{j-1}^{i}}{\Delta x}$, $\frac{\partial y}{\partial x} = \frac{y_{j}^{i} - y_{j-1}^{i}}{\Delta x}$

Similarly for time derivatives

Forward: $\frac{\partial V}{\partial t} = \frac{V_{j}^{i+1} - V_{j}^{i}}{\Delta t}$, $\frac{\partial y}{\partial t} = \frac{y_{j}^{i+1} - y_{j}^{i}}{\Delta t}$ (1.6)

Central: $\frac{\partial V}{\partial t} = \frac{V_{j}^{i+1} - V_{j}^{i-1}}{\Delta t}$, $\frac{\partial y}{\partial t} = \frac{y_{j}^{i+1} - y_{j}^{i-1}}{\Delta t}$

Backward: $\frac{\partial V}{\partial t} = \frac{V_{j}^{i} - V_{j}^{i-1}}{\Delta t}$, $\frac{\partial y}{\partial t} = \frac{y_{j}^{i} - y_{j}^{i-1}}{\Delta t}$

Where i, j refer to the variables for the present time and time space respectively. And i+1, j+1 refer to the variables for the next time and space level, where i-1and j-1 refers to the variables for the previous time and space level respectively. There is also approximation for second-order and mixed derivatives. But in this research we are dealing with first order approximation only.

1.6.1.2. Explicit scheme Stability

In every numerical scheme, stability analysis is essential as error in solution may grow without limit depending on the sizes or values of Δx and Δt. Less error is encountered if Δx and Δt is very small, but computation time will increase. The stability of the explicit scheme is determined by the Courant-Fredriches-Lewy (or Courant) condition. For the stability solution of the explicit scheme any one of the following Courant condition must be maintained:

$$\Delta t \leq \frac{\Delta x}{V_0 + c} \tag{1.7a}$$

i.e. $$\Delta t \leq \frac{\Delta x}{V_0 + \sqrt{gD_0}} \tag{1.7b}$$

Or $$\Delta t = \frac{\sqrt{1 + 2\left|\frac{V_0}{C_0}\right|} - 1}{\left|\frac{V_0}{C_0}\right| \frac{gs_b}{V_u}} \tag{1.8}$$

Koren while working on Lax diffusive scheme concluded that Courant criteria given in Eq. (1.8) gives better additional stability. Velocity V_0, celerity C_0 in the above two equations (1.7a), (1.7b), and (1.8) are the values at initial conditions.

French worked in this unstable scheme and concluded that the scheme is weak and inherently unstable. Care must be taken to choose very small Δx if the scheme is used for solution. The Courant number (CFL), which is the ratio of the physical speed of the wave to the speed of the numerical signal. It should be less than unity so the grid size will be obtained from that condition. Updated calculations of the water stage is used to evaluate the celerity of the wave, c and for the water velocity V, the discharge value is used. Hence CFL condition is applied at each time step. This condition is implemented at each time step to evaluate the value of flow at the advanced time step. Numerically by this method the time step must be kept small enough so that information will be most accurate. Courant number expressed as C_n

$$C_n = \frac{V \pm \sqrt{gy}}{\Delta x / \Delta t} \tag{1.9}$$

Where $0 < C_n \leq 1$

V is the flow velocity, Δx is grid size along the length, Δt is grid size of the time.

1.6.1.3. Implicit scheme

It finds a solution by solving an equation involving both the current state of the system and later one. Mathematically, If Y (t) is the current system state and Y (t+Δt) is a state at the later time (Δt is a small time step) then for an implicit method,

F (Y (t), Y (t+Δt)) = 0 (1.10)

To find from Eq. (1.10) Y (t+Δt) is the implicit method. It is clear that implicit methods require an extra computation (solving the above equation), than explicit one and they can be much harder to implement in the computation process. The phrase implicit difference scheme refers to a computational scheme in which the value of the parameters like depth and velocity are determined by solving a system of simultaneous equations. In this method, unknown values with increasing time steps occur implicitly in the finite difference form. Various investigators working on this explicit scheme are Preissmann and Cunge, Cunge, et al, Abbot, Choudhry, Liggett and Cunge, fenemma and Chaudhry, Strelkoff, Amein and Fang, Terzdis and Sterlkoff, Amein and

Chu, Abbot and Ionesq, Ligette and Woolhier, Prince, Mahmood and Yevjech, Vasiliev et al, Anderson Et al and others. The various implicit schemes are Preissmann scheme, Beam and Warming Scheme, Abbott -Ionescu scheme, Vasiliev Scheme etc.

Implicit methods are used because many problems arising in practice are stiff, for which the use of an explicit method requires impractically small time steps Δt to keep the error in the result bounded due to stability. For such problems, to achieve given accuracy, it takes much less computational time to use an implicit method with larger time steps, even taking into account that one needs to solve an equation of the form Eq.(1.10) at each time step. That said, whether one should use an explicit or implicit method depends upon the problem to be solved.

As explicit method, there are also three categories in implicit scheme. They are forward difference method, backward difference method and central difference method. These schemes are only differentiated by the time level. In explicit method, the known variables are in present time (i) level and the unknown variables are of next time (i+1) level. But for implicit method, the known variables are of forward time level i.e. (i+1) level and the parameters which are to be finding out are of current time (i) level. In the present work preissmann implicit scheme is used. The detail is discussed in Chapter 3 of this book.

1.6.1.4. Implicit scheme Stability

Regarding stability, scheme is unconditionally stable if the weighting coefficient in space is greater than 0.5, i.e. flow variables are weighted towards (i+1) time level. But it better to maintain courant criteria close to unity. Samuels and Skeels, Evan and Yen and Lin investigated the Vedernikov number for stability in their numerical experimentation. In 1945, Vedernikov employing a certain approximation in Saint Venant equation, developed a criterion for stability which is called Vedernikov number V_n, and this number is expressed as $V_n = \frac{x\, r\, V}{V_w - V}$

(1.11)

Where x is exponent of hydraulic radius r in uniform flow formula, being x=1/2 for Chezy's formula, 2/3 for Manning's equation and x=2 for laminar low. V is the average velocity, V_w is the absolute velocity of disturbance of wave in channel or canal. R is the shape factor of the channel section, defined by

$$r = 1 - R\frac{dP}{dA} \quad (1.12)$$

Here P is wetted perimeter, A is the water area. Thus r =1 for very wide channel, r=0 for narrow channel. $V_w - V$ is the celerity C or t the critical velocity V_c. Since Froude Number $F_r = \frac{V}{V_c}$,

Equation reduce to $$V_n = xrF_r \quad (1.13)$$

When $V_n < 1$, any wave in the channel or canal is stable and wave will be depressed. When $V_n \geq 1$, it becomes unstable.

1.7. BOUNDARY AND INITIAL CONDITIONS

All specification regarding geometry of interest is to be provided and appropriate boundary condition is to be given. Within the domain conservation equations are to be placed. Number of boundary condition is decided by the order of highest derivative appearing in each independent variable in equation. In unsteady equations governing by a first derivative in time require initial condition to carry out the time integration. The data required for unsteady flow analysis are boundary conditions, for both outer boundaries and for the initial condition. Initial flow and water depth at zero time level are applied as the initial condition at the start of the simulation. There are different types of conditions can be specified for the both upstream and downstream boundaries such as, Flow hydrograph, Stage hydrograph, Stage and Flow hydrograph, Single valued rating curve, Normal depth from manning's depth, Critical flow depth, Lateral Inflow hydrograph, Uniform lateral inflow, Ground water inflow, T.S. gate Openings, Elevated Controlled Gates, Navigation Dams, IB stage/flow etc.

1.7.1. Hydrograph

Hydrograph is graphical representation of flow of a river at a location with time. It is the total response or the output of a watershed beginning with the precipitation as the hydrological exciting agent or input. The systems represent the catchment physiographic, geologic and hydrometeorologic effect is complex and is difficult to model it accurately due to high variability of these parameters in space and time. The rate of flow is expressed as a cubic meters per second (cumec or m^3/s) or cubic feet per second (cusec or ft^3/s). A hydrograph comprises three phases, namely, surface, subsurface, base flow. The factor affecting hydrograph at a place being complex and interrelated, a basin may not produce two exact flood hydrographs with two similar precipitations as input, nor can two basin of the same drainage area produce the same flood hydrograph with similar precipitation. In this work the hydrograph used are flow hydrograph, stage hydrograph and flow/stage hydrograph as boundary condition.

1.7.1.1. Flow hydrograph- It is a graphical representation of flow of a river or canal at a location with time. Similarly Stage hydrograph is a graphical representation of stage of a river or canal at a location with time. Both the hydrographs are used as the upstream or downstream boundary condition simultaneously or singly.

1.7.1.2. Normal depth from Manning's equation-The normal depth obtained from manning's equation (for uniform flow) as downstream boundary condition for an open ended reach where the river is too long. From manning's *n* the stage value can be estimated for each computed flow. The friction slope is required for the evaluating of normal depth. For friction slope (S_f) the Manning's coefficient or roughness coefficient (*n*) is required.

$$S_f = \frac{n^2 v^2}{R^{4/3}} \tag{1.14}$$

$S_f =$ Frictional slope, n = manning's roughness coefficient, v = velocity of the flow, R = hydraulic radius = A/P; where *A*=area of the flow and *P*= wetted perimeter of the channel.

1.8. OBJECTIVES OF RESEARCH:

- Discretization of Saint-Venant equation (both continuity and momentum) by finite difference method (inverse explicit method) by using CFD (computational fluid dynamics) tool.
- To demonstrate the potential of the methods used in this research, models are applied to predict the upstream values for desired downstream demand by approaching this inverse explicit finite difference method to a test canal
- To write the Program in MATLAB software for simulating the flood hydrograph (both flow hydrograph and depth hydrograph) in upstream study section.
- Comparison of the results obtained from present approach and HEC-RAS Computer model to find out the suitable value of weighting co-efficient ($\emptyset$) and time step(Δt).

CHAPTER 2

LITERATURE REVIEW

2.1. OVERVIEW

In this chapter an attempt has been made to draw together various aspects of past research in open channel hydraulics concerning the regulation of flow in the channel or canal system using different numerical approaches. Many researchers proposed many different ways for numerical method for water resources. Some methods are easy to understand and some are difficult and some are more difficult. Scientists and researchers are developing new methods or modifying the developed one to make them simple and reliable. Prior to early twentieth century, nobody knows about the numerical model on water resources using computational fluid dynamics. In twentieth century, some researchers work on it. But now it is a booming area in water resource engineering worldwide. Regulation of flow is a very important aspect because of the discharge and depth requirement for various engineering purpose. In a natural river or in a canal to get water throughout the year as per demand is very necessary. In 1969, Weily first proposed idea of regulation of flow in the channel. Then many researchers work on it and give various numerical approaches to the world and criticizing the existing approaches, developed new methods for regulation of control structures. The Literature review conducted as part of the present study is divided into two segments i.e. the method used for operation type problem and the stability of that method. These two sections briefly explain the necessary information for regulation of flow using numerical method. Generally stability played a very important role in the numerical approximations. So every method used in analysis must go for stability check. Some of the extension literature are studied and presented below.

2.2. PREVIOUS WORK ON REGULATION OF UNSTEADY FLOW

E.B. Weily (1969) first gives the idea of transient control technique i.e. known as gate stroking for regulation of free surface flow. He used the characteristic method for solution of Saint-Venant equations and written these partial differential equations into four particular total differential equation and named the as C^+ and C^- characteristic equation. By first order and second order characteristic method he solved the equations. It helps in the site, if an online computer is used in a canal system, input data regarding the instantaneous condition in the channel together with the input or stored data pertaining to normal operational methods and responses to emergency conditions can be analysed in few seconds. The computed results are then avail for real-time computer model.

Eli et al. (1974) worked on operation type problem. Instead of method of characteristic, they used implicit finite difference method. They found some good agreement. However, they noted that “some problems were encountered, particularly at low flows. When the actual discharge was routed backward the program failed to provide stable results.”

Falvey (1987) worked on gate stroking concept and noted: “The gate motions determined by this method can be quite irregular if the flow changes are large”. His studies also showed a need for additional check gates since some pools were too long for adequate control of the transients. He advocated a hybrid system of control, where for "discharge variations which are small relative to the capacity of the canal, the canal would be controlled with a local control algorithm ... large flow changes would be accomplished using gate stroking concepts".

Fennema and Chaudhry (1990) carried out the research on two dimensional transient free surface flows by explicit method. They introduced MacCormack and Gabutti explicit finite difference schemes to integrate the equations describing unsteady gradually varied 2-D flow. Both the methods are second-order accurate in time and space; allow initial conditions of sharp discontinuous, and bore isolation is not required. Partial dam breach and passage of a flood wave through a channel contraction – these two typical hydraulic flow problems are analyzed by this model. By the selective addition of artificial viscosity, some numerical oscillations which occur near the sharp fronted wave could be controlled. Since the shock is spread only over a few mesh points, explicit tracking of the bore is not necessary by this scheme. They also discuss about the stability conditions and inclusion of boundaries and artificial viscosity addition to the smooth high frequency oscillations.

Paul et al. (1990) studied the stability limits for Preissmann scheme. They investigated the linear stability of Preissmann box-scheme applied to free surface flow, using the standard technique of Fourier analysis including the general form of the friction term. They found analytically that, when equations are weighted towards the new time level, the linearized, numerical equations are stable and also when the Vedernikov number magnitude is less than one. The first condition is necessary for preissmann scheme as applied to simplifications of the Saint-Venant equations and the second one is the limit of hydrodynamic stability at which roll waves form and the numerical scheme reflects this physical stability limit. They reported that the difficulties that occur in practical applications of this computational method must have their origin in factors omitted from the Fourier analysis, such as the treatment of boundary conditions and the nonlinear terms in the flow equations.

Chaw and Lee (1991) developed a mathematical model on Shing Mun River network, Chaina**.** They used Preissmann point implicit scheme because allowable time step does not depend on grid size like all explicit schemes. The program is written on FORTRAN version 4.0. Real hydraulic features, including branched channels and tidal flats are simulated. The tidal flats, uncovered at low water, are simulated by making the location of land boundary a function of water level.

Hussain et al (1991) worked in a channel to develop a network model to operate the unsteady flow. The simulation of the hydrograph with respect to time is carried out under the network model based on unsteady flow analysis. They developed a dynamic node for easier data calculation for the user. It is a numbering system to update the related data quickly. For the simulation process of unsteady flow at irrigation system, they also introduce a system break up feature. It is also used in minicomputer as well as in microcomputer. The change of discharge coefficient with respect to flow parameter occurs at gate at low differential head. The calibration of result shows this coefficient higher at low head also. But when the differential head is more than the discharge coefficient is also decreased and get stable. They also reported that the stability of computation of the model depends on the distance between nodes, Δx and the time step $\Delta \mathrm{t}$. The ranges of $\Delta x$and Δt should be determined using wave velocity and time of rise of the inflow hydrograph.

J. Sinha, et al. (1993) presented a spectral method for solving 1-D shallow water wave equations and compared it with the finite difference method. They consider a wide rectangular open channel for routing a log-Pearson Type III hydrograph. Preissmann finite difference and Chebyshev collocation spectral methods are used for routing.

R. Mishra (1994) worked on operation control of canal systems using inverse method. A Non-linear explicit inverse solution method is used for solving the St. Venant equations. Finite difference approximation for operational problem gives a system of two non-linear equations with two unknowns which are solved by Newton’s method. The model results are verified with the unsteady flow simulation model based on non-linear Preissmann scheme.

D.D. Franz and C.S. Melching (1997) introduced a full equation (FEQ) model for the solution of full, dynamic equation of motion for 1-D unsteady open channels and through control structures. By application of FEQ a stream is subdivided into stream reaches, parts of the stream system for which complete information of flow and depth are not required (dummy branches), and level pool reservoirs. This model can be applied in the simulation of a wide range of stream configurations, lateral inflow conditions and special features.

E. Bautista et al. (1997) developed a nonlinear implicit finite difference gate stroking method. They worked on single pool canal systems and compared the accuracy of results and robustness of the methods –method of characteristics (MOC) and inverse implicit finite difference method.

Strelkoff et al. (1998) gives a lot of evidence regarding gate stroking. There are three important aspect for which over the past several decades much attention is given to the methods. First one is for controlling canal downstream water levels or volume with feedback control. Second one is routing flow changes through canals with open loop or feed forward control and last one is to utilizing local structures for controlling either water level or flows. But the success of any of these methods is largely dependent on the properties and characteristics of canal not the method used for its regulation. However there is little in the literature examining the limitations that canal properties place on controllability. “Finally, at the end of the conclusion they observe “Our analyses suggest that not all flow changes in a canal pool can be accommodated by feedback alone. The amount of flow change that can be handled just by feedback is dependent upon the pool properties, the amount of allowable depth or pool volume change, and the properties of the feedback controller. This result emphasizes the need to include both feedback and feed forward components into canal control systems.”

Fenton, Oakes and Aughton (1999) worked under gate stroking and gave a clear idea about forward control. They used the nonlinear unsteady flow equation i.e. Saint- Venant equations for canals. They approximated a low Froude number in derivation of that long wave governing equation for simulating the waves in a canal system. The approached this method in such a way that it becomes simpler than the existing methods. They have written simple programs by solving that equation for finding out the accurate stage and discharge value in that canal. Lager time steps are allowed in their program. For mild slope i.e. almost smooth gradient in a irrigation canal, they have clearly shown the diffusion i.e. occurs due to friction. They got a result for gate stroking that the diffusion process makes the solution more complex. For the case of feed forward control also the same results are found. Due to fluctuation of the waves at upstream section, the disturbances are found in downstream section. They also describe the programs written by the previous researchers in gate stroking and feed forward control. They concluded that proper control should be given in upstream section for finding the required quantity of downstream discharge.

F. R. Fiedler and J. A. Ramirez (2000) presented a computational method for simulating discontinuous shallow flow over an infiltrating surface. They used MacCormack finite difference scheme for simulating 2-D overland flow with spatially variable infiltration and micro topography using the hydrodynamic flow equations. The developed method is useful for simulating irrigation, tidal flat and wetland circulation, and floods.

S. A. Yost and P. Rao (2000) introduced a multiple grid algorithm for one-dimensional transient open channel flow equations. They coupled this algorithm to a second-order accurate MacCormack scheme, and demonstrated that the solution can be accelerated to the desired transient state. And they tested their algorithm to simulate shocks arising from sudden closure of a sluice gate and for flows accompanied with a hydraulic jump.

M. T. Shamaa (2003) worked on regulation of unsteady flow by numerical method. A finite difference algorithm is presented for regulating the unsteady open channel flow. They consider a test canal model, simulate the Saint-Venant equations for different depth and discharge, then applied this to an actual canal. Downstream demand was taken, by inverse explicit method based on Preissmann scheme, the upstream hydrograph was found. Applying this hydrograph to Preissmann implicit flood routing scheme, they got the downstream hydrograph which reasonably matches to the required demand. Then they compare the both the results for different distance and time interval. Also consider the different weighting coefficient of time and space and the corresponding results were discussed for the canal. The method is explicit and numerically stable and can be used to compute the upstream inflow and setting of the controlling structure according to required downstream outflow.

L. Kranjcevic, B. Crnkovic and N. CrnjaricZic (2006) carried out research by applying implicit method for solving 1-D open channel flow. They used upwind implicit scheme for resolution of the one-dimensional shallow water equation equations by accurate and efficient numerical modeling of non-homogeneous hyperbolic system of conservation laws and emphasizes on space dependency of the flux and significant geometrical source term variation. They modified original finite volume Linearized Conservative Implicit (LCI) scheme in order to account for the spatially variable flux dependency, and consequently the source term was appropriately decomposed to balance the upwind flux decomposition. The use of implicit numerical scheme in modeling of the open channel flow equations involving non-prismatic channels with rectangular cross section geometry is enabled by these numerical modifications. They tested improved balanced implicit scheme and compared with simple non-balanced point wise version of the scheme on different open channel test problems which include friction, non-uniform bed slopes, strong channel width variations and dam break problem with analysis of transient solution.

M. Jin-bin and Z. Xiao-feng (2007) developed a real time flood forecasting method coupled with the1-D unsteady flow model with recursive least square method. The flow model was modified by using time variant parameter and dynamically revising it through introducing a variable weighted forgetting factor, so that the output of the model could be adjusted for real time flood forecasting. The model gives high accuracy of flood fore casting than the original 1-D unsteady flow model. First order upwind scheme for forward difference and central difference is used for discretization of Saint-Venant equation.

H.Elhanafy and G. J. M. Copeland (2007) modify the characteristic method for shallow water equations. The main idea behind the modified method of characteristics is to calculate the time interval accurately using the geometry. This method is able to simulate sharply rising flood events producing stable, mass conserving solutions and tracks the characteristics path carefully by means of a series of small increments to locate the correct values in the domain for interpolation to the boundary.

Guang Ming et. al. (2008) used characteristics method in 1-D unsteady flow numerical model for gate regulation. Simulate the process of water flow, taking different boundary condition. Analyse the influence of gate regulation speed and channel operation methods on flow transition process. And it is helpful for the scheme design of automatic operation control in water conveyance channels.

N. Grujovic, et al. (2009) modelled the 1-D unsteady free surface flow with reservoirs, dams and hydropower plant objects.By steady flow calculation the initial values of discharge and stage is determined for each node in the model. The model helps in decision making at a level of dispatcher and also at management level.

G. Akbari and B. Firoozi (2010) carried out implicit and explicit numerical solution of one dimensional shallow water equations for simulating flood wave in natural rivers. In explicit method and implicit method, they used Lax-diffusive scheme and Preissmann finite difference scheme respectively for solution of the equations. In preissmann scheme for solving non-linear system of equations they used Newton-Raphson method and compared the obtained result with HEC-RAS commercial software.

M.A. Moghaddam and B. Firoozi (2011) developed a numerical method for dynamic flood wave routing. They used preissmann implicit scheme for the discretization of Saint-Venant equations. They consider a hypothetical wide rectangular channel for the routing of flood.Upstream boundary is given by equations and downstream boundary is consider from manning's equation. Courant number stability check found that the model is numerically stable. For solving non-linear system of equation they used Newton-Raphson method and compared the obtained result with the result of HEC-RAS commercial software.

G. Akbari and B. Firoozi (2011)carried out the research on characteristics of flood in Persian Gulf catchment. They used two explicit methods -Lax method, MacCormack method, Method of characteristics (MOC) and Preissmann implicit finite difference method. They write and compile the program using MATLAB software and compared with the results obtained from the HEC series computer model.

M. T. Shamaa and H. M. Karkuri (2011) combinedly extended the work of M. T. Shamaa (2003) for regulation of unsteady flow by inverse implicit method and backward explicit method. They consider a test canal model with a given downstream value and simulate the Saint-Venant equations for different depth and discharge. From given downstream demand, by backward explicit method and inverse implicit method based on Preissmann scheme, the upstream hydrograph found. Applying this hydrograph in Preissmann implicit routing scheme, downstream hydrograph was determined which reasonably reproduced the prescribed demand. Then they compare the both the results for different space and time step. Also consider the different weighting coefficient of time and space and analyze them .The implicit method is numerically stable and can be used to compute the upstream inflow and setting of the controlling structure according to desired downstream outflow. They found that inverse implicit scheme is stable for all tested factor but backward explicit scheme is stable only for the appropriate weighting coefficient.

W. Artichowicz (2011) numerically analyzed the free surface steady gradually varied flow by simplified St. Venant equations. He discretized the dynamic wave equation and energy equation. The system of non-linear equations solved with modified Picard method which gives different result for dynamic wave and energy equations although they are same.

H. M Kalita and A. K Sarma (2012) carried out their research on the performance and efficiency of finite difference scheme on solution of Saint-Venant equations. For solving these nonlinear equations they used Lax-diffusive explicit scheme and Beam Warming implicit scheme. Validation of the model is done by comparing with the result of MIKE21 modeling tools. It was found that Beam Warming implicit scheme take very large computational time for simulation. They consider a prismatic trapezoidal channel for modeling of two dimensional open channel flows. It is shown that the computational time interval depends upon the spatial grid spacing, flow velocity and celerity which are the flow depth function. It is observed that due to lengthy procedure of implicit scheme it takes more time than explicit scheme.

S. L.Doiphode and R.A Oak (2012) worked on unsteady flow modeling and dynamic flood routing of upper Krishna River**.** They take the available survey data of Krishna and Koyna River, and put in HECRAS software model for simulation of stage and discharge. The rating curve obtained by the model is matching with the observed rating curve. The model gives the scenario of Koyna dam, Dhom dam and they also discussed the flood sensitivity.

CHAPTER 3

METHODOLOGY AND PROBLEM STATEMENT

3.1. OVERVIEW

Normally the experimental work based on river is sometime very difficult to conduct. In this computer era various mathematical model are developed in engineering fields. Now a-days in water resource engineering the numerical methods are widely used for water flow relate problem solving, software designing which are helpful for the engineers. In this chapter the methods used for this research work briefly described. There are various numerical methods for solving computational problem regarding fluid flow. Here Inverse explicit method based on Preissmann scheme is used for the solving operational problem. Saint-Venant equation is used as the governing equation. The detailed about the method used and procedure for solving the problem are given below.

3.2. GOVERNING EQUATION

In this study, the methods which is applied for simulating and predicting the unsteady flow mathematically is given. For Saint- Venant equation which is nonlinear hyperbolic equations are used for unsteady floe for regulation as it was deduced by A.J.C. Barri de Saint-Venant in 1871 and most popularly used till today almost in all situations of unsteady flow of open channel. There are many assumptions on which the solution for unsteady flow through Saint Venant equation depends upon. Many methods can be applied for the simulating the flow and stage in regulation work. If there is no lateral flow, then the governing equation is written as

For mass conservation

$$\frac{\partial y}{\partial t} + \frac{1}{b}\frac{\partial Q}{\partial x} = 0 \tag{3.1}$$

And for momentum equation

$$\frac{\partial Q}{\partial t} + \frac{\partial}{\partial x}\left(\frac{Q^2}{A}\right) + gA\left(\frac{\partial y}{\partial x} + S_f - S_0\right) = 0 \tag{3.2}$$

But for putting the equation the conservation form of these equation in matrix form is given by

as $X_t + Y_x + Z = 0$ (3.3)

Where

$$X = \begin{bmatrix} A \\ Q \end{bmatrix}$$

$$Y = \begin{bmatrix} Q \\ QV + gAy' \end{bmatrix}$$

$$Z = \begin{bmatrix} 0 \\ -gA(S_0 - S_f) \end{bmatrix} \tag{3.4}$$

And Ay'=moment of flow area about the free surface.

3.2.1 Preissmann Implicit Scheme

Large time steps are allowed in solution by implicit method which is the main advantages in this method. It is stable for any condition for which the stability check is not necessary. Among several implicit schemes the Preissmann implicit scheme is considered extensively for open channel flow.

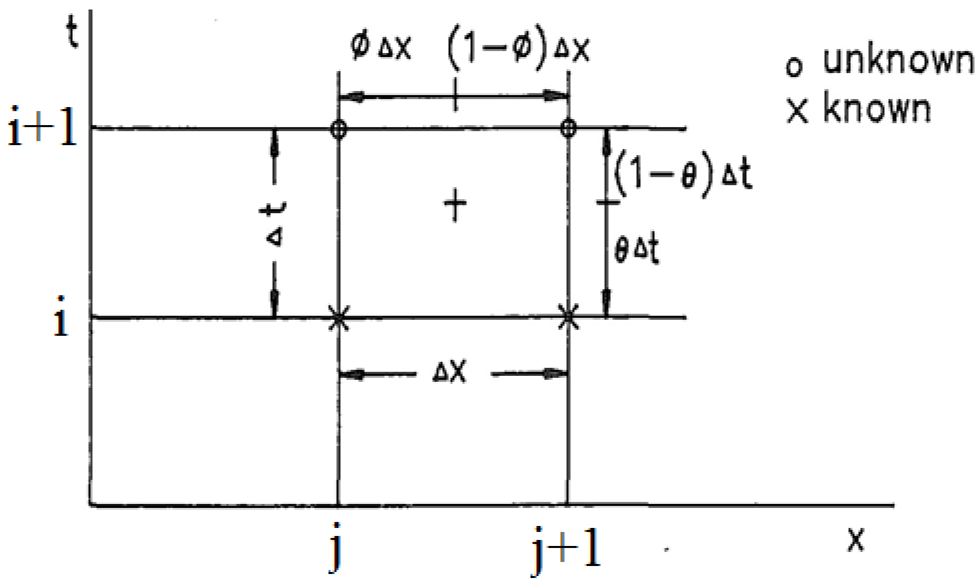

Fig. 3.1 Preissmann scheme computational grid

Due to the straightforward structure in generating the grids the Preissmann scheme is most advantageous one for the numerical examination. This intimates a straightforward medication of limit conditions and a basic consolidation in structural geometry. In this method a weighting coefficient is considered. For proper simulation the flow and stage in open channel the weighting coefficients are varying. As this method is unconditionally stable so any time steps of higher value is allowed at the time of generating the grids for computing discharge and stage at various points. So the formulae of Preissmann scheme is given by

$$\frac{\partial f}{\partial t} = \emptyset \frac{f_{j+1,i+1} - f_{j+1,i}}{\Delta t} + (1-\emptyset)\frac{f_{j,i+1} - f_{j,i}}{\Delta t} \tag{3.5}$$

$$\frac{\partial f}{\partial x} = \theta \ \frac{f_{j+1,i+1} - f_{j,i+1}}{\Delta x} + (1-\theta)\frac{f_{j+1,i} - f_{j,i}}{\Delta x}$$

$$F(x,t) = \theta\left[\emptyset f_{j+1,i+1} + (1-\emptyset) f_{j,i+1}\right] + (1-\theta)\left[\emptyset f_{j+1,i} + (1-\emptyset) f_{j,i}\right]$$

In which $f_{j,i} = f(j\Delta x, i\Delta t)$ $\emptyset$ and θ are the weighting coefficients, $0 \le \theta \le 1$; f refers to both *V* and *y* i.e. the discharge *Q*, and *F* stands for S_f, grid of size (j,j), j for distance and i for time .

Introducing Preissmann scheme into Eq. (3.1) and Eq. (3.2) gives the linear algebraic equations for each two adjacent grid points (Ligette and Cunge 1975; Hromadka II et al. 1985) which are given below

$$P_1 q_j + Q_1 q_{j+1} + R_1 y_j + S_1 y_{j+1} + T_1 = 0 \tag{3.6}$$

$$P_2 q_j + Q_2 q_{j+1} + R_2 y_j + S_2 y_{j+1} + T_2 = 0 \tag{3.7}$$

Where q_j and y_j = discharge and water level increment from time level *I to (I+1)* at grid point *j;* q_{j+1} and y_{j+1} are these at grid point (*j+1*) and P_1, Q_1, R_1, S_1, T_1, P_2, Q_2, R_2, S_2, and T_2 are coefficients computed with known values at time level *I.*

There are four unknowns in the linear algebraic equations (3.6) and (3.7). Two unknowns for each grid point i.e. q and y. On the row I+1 as there are J points, so j-1 rectangular grid and j-1 cell in the channel. Consequently there are 2(J-1) equations for the assessment of 2 j unknowns. Thus for the evaluation of 2J unknowns there are 2(j-1) equations. To close the system the necessary two additional equations are provided by two boundary condition i.e. expected discharge and depth. Any numerical method like Newton-Raphson method or double sweep method or any standard method can be applied to get the solution of 2(*J*-1) equations. These procedure help for the routing problem however for operational problem reverse technique is used. Here in this research inverse explicit method is used which given below.

3.2.2 Inverse Explicit Scheme

Based on the same principle of Preissmann implicit method an inverse explicit method is introduced. Two boundary conditions are needed for simulation process by this scheme i.e. stage and discharge are required at downstream boundary at the time of solution. To define the unknown parameters at observation point with respect to time a set of equations are simultaneously solved in this scheme.

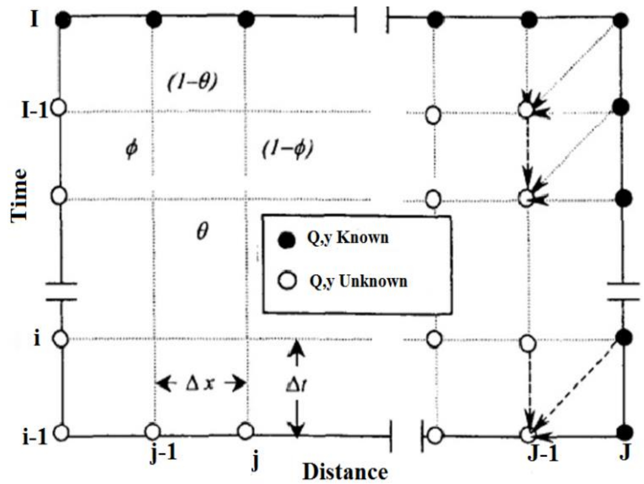

Fig. 3.2 Inverse explicit computational grid

However for operation issues, there are two boundary conditions, i.e. expected discharge and water level at downstream. Considering the time level 'I' as the final condition, Fig. 2, and knowing q_j and y_j, between any two time levels at the downstream section, the discharge and water depth profile at the time level (I-1) can be computed by proceeding first backward in time and then backward in space. In this approach, the Preissmann scheme is written as:

$$\frac{\partial f}{\partial t} = \emptyset \frac{f_{j-1,i} - f_{j-1,i-1}}{\Delta t} + (1-\emptyset)\frac{f_{j,i} - f_{j,i-1}}{\Delta t} \tag{3.8}$$

$$\frac{\partial f}{\partial x} = \theta \, \frac{f_{j,i-1} - f_{j-1,i-1}}{\Delta x} + (1-\theta)\frac{f_{j,i} - f_{j-1,i}}{\Delta x}$$

$$F(x,t) = \theta\left[\emptyset f_{j-1,i-1} + (1-\emptyset) f_{j,i-1}]\right] + (1-\theta)\left[\emptyset f_{j-1,i} + (1-\emptyset) f_{j,i}\right]$$

The solution begins at the top-right corner of the time-distance plane, Fig. 3.2. The application of the finite difference equations gives an algebraic system of two equations and two unknowns. The solution obtained cell by cell, moving first regressive in time and after that retrograde in space. Introducing inverse explicit scheme based on Preissmann scheme into (1) and (2) yields the following algebraic equations for every two neighboring grid points

$$P_1 q_{j-1} + Q_1 q_j + R_1 y_{j-1} + S_1 y_j + T_1 = 0 \tag{3.9}$$

$P_2 q_{j-1} + Q_2 q_j + R_2 y_{j-1} + S_2 y_j + T_2 = 0$ (3.10)

Where q_{j-1} *and* y_{j-1} = discharge and water level increment from time level (*I*-1) to *I* at grid point (*J-1*); q_j *and* y_j are these at grid point (*J*) and P_1, Q_1, R_1, S_1, T_1, P_2, Q_2, R_2, S_2, and T_2 are

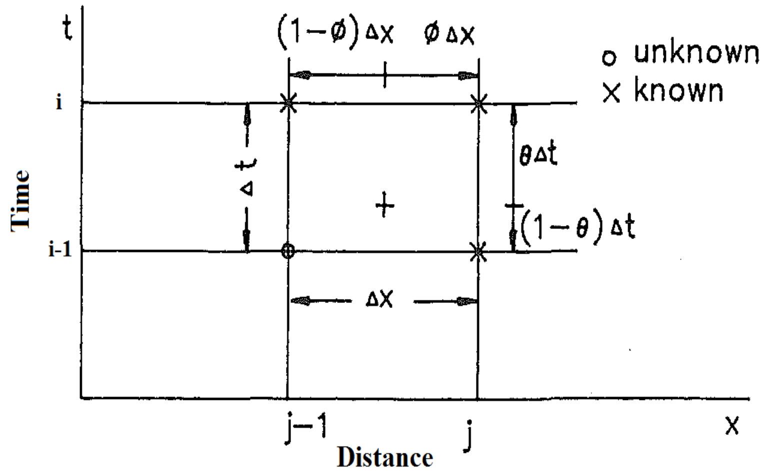

Fig. 3.3 Preissmann scheme used in Inverse explicit (IE) method

-coefficients computed with known values at time level *I*. These coefficients are computed by applying Inverse Explicit equations to Saint-Venant equations i.e. Eq. (3.8) in Eq. (3.1) and Eq. (3.2), we have the discretized form as follows:

$$\frac{\theta}{\Delta x}\left(q_j^{i-1} - q_{j-1}^{i-1}\right) + \frac{(1-\theta)}{\Delta x}\left(q_j^{i} - q_{j-1}^{i}\right) + \frac{\emptyset}{\Delta t}\left(A_{j-1}^{i} - A_{j-1}^{i-1}\right) + \frac{(1-\emptyset)}{\Delta t}\left(A_j^{i} - A_j^{i-1}\right) = 0 \quad (3.11)$$

And

$$\frac{\emptyset}{\Delta t}\left(q_{j-1}^{i} - q_{j-1}^{i-1}\right) + \frac{(1-\emptyset)}{\Delta t}\left(q_j^{i} - q_j^{i-1}\right) + \frac{2Q}{A}\left[\frac{\theta}{\Delta x}\left(q_j^{i-1} - q_{j-1}^{i-1}\right) + \frac{(1-\theta)}{\Delta x}\left(q_j^{i} - q_{j-1}^{i}\right)\right] -$$
$$\frac{Q^2}{A^2}\left[\frac{\theta}{\Delta x}\left(y_j^{i-1} - Y_{j-1}^{i-1}\right) + \frac{(1-\theta)}{\Delta x}\left(y_j^{i} - y_{j-1}^{i}\right)\right] + gy\left[\left\{\frac{\theta}{\Delta x}\left(y_j^{i-1} - y_{j-1}^{i-1}\right) + \frac{(1-\theta)}{\Delta x}\left(y_j^{i} - y_{j-1}^{i}\right)\right\} - S_0 +\right.$$
$$\left. S_f\right] = 0 \quad (3.12)$$

Comparing the coefficient of Eq.(3.9) with Eq.(3.11), and , Eq.(3.10) with Eq.(3.12), we have P_1, Q_1, R_1, S_1, T_1, P_2, Q_2, R_2, S_2, and $T_2$are as follows

$P_1 = \frac{-\theta}{\Delta x}$ (3.13)

$Q_1 = \frac{\theta}{\Delta x}$ (3.14)

$$R_1 = \frac{-\emptyset}{\Delta t} \tag{3.15}$$

$$S_1 = \frac{-(1-\emptyset)}{\Delta t} \tag{3.16}$$

$$T_1 = \frac{(1-\theta)}{\Delta x}\left(q_j^i - q_{j-1}^i\right) + \frac{\emptyset}{\Delta t} y_{j-1}^i + \frac{(1-\emptyset)}{\Delta t} y_j^i \tag{3.17}$$

$$P_2 = \frac{-\emptyset}{\Delta t} - \frac{\theta}{\Delta x}\frac{2Q}{A} \tag{3.18}$$

$$Q_2 = \frac{-(1-\emptyset)}{\Delta t} + \frac{\theta}{\Delta x}\frac{2Q}{A} \tag{3.19}$$

$$R_2 = \frac{-\theta}{\Delta x}(gy) + \frac{\theta}{\Delta x}\frac{Q^2}{A^2} \tag{3.20}$$

$$S_2 = \frac{\theta}{\Delta x}(gy) - \frac{\theta}{\Delta x}\frac{Q^2}{A^2} \tag{3.21}$$

$$T_2 = \frac{(1-\emptyset)}{\Delta t}\left(q_j^i\right) + \frac{\emptyset}{\Delta t}\left(q_{j-1}^i\right) + \frac{2Q}{A}\left[\frac{(1-\theta)}{\Delta x}\left(q_j^i - q_{j-1}^i\right)\right] - \frac{Q^2}{A^2}\left[\frac{(1-\theta)}{\Delta x}\left(y_j^i - y_{j-1}^i\right)\right] + \mathrm{gy}\left[\left\{\frac{(1-\theta)}{\Delta x}\left(y_j^i - y_{j-1}^i\right)\right\} + S_f - S_0\right] \tag{3.22}$$

Since there is J grid point, there will be $2J$ variables. Knowing q_j and y_j between any two time levels at the last section of the channel, one can apply (6) and (7) for the last two sections J-1 and J (Fig. 2), and solve $q_{j-1}$and y_{j-1} explicitly:

$$q_{j-1} = \frac{R_1(Q_2 q_j + S_2 y_j + T_2) - R_2(Q_1 q_j + S_1 y_j + T_1)}{P_1 R_2 - R_1 P_2} \tag{3.23}$$

$$y_{j-1} = \frac{P_1(Q_2 q_j + S_2 y_j + T_2) - P_2(Q_1 q_j + S_1 y_j + T_1)}{R_1 P_2 - P_1 R_2} \tag{3.24}$$

With the calculated value of $q_{j-1}$and y_{j-1} the value of $q_{j-2}$and y_{j-2} can be computed. This computation process is continued until the upstream boundary is reached, as shown in Fig.2. The foregoing is a backward or inverse computation in space. This computing procedure cannot be applied easily because the changes of flow at the downstream end of the channel are caused by the changes upstream (Fig. 3.4).

If the time is T, taken by the first propagation to travel from upstream to the downstream end of the channel, the discharge and water depth at the downstream-end section remain the same before the time level (T_0+T), and the process between $T_0$and (T_0+T),cannot be computed. Since the flow state at the time level (T_0+T) is needed for the computation after that time level, the problem cannot be solved. Luckily, the previously stated difficulty can be overcome by

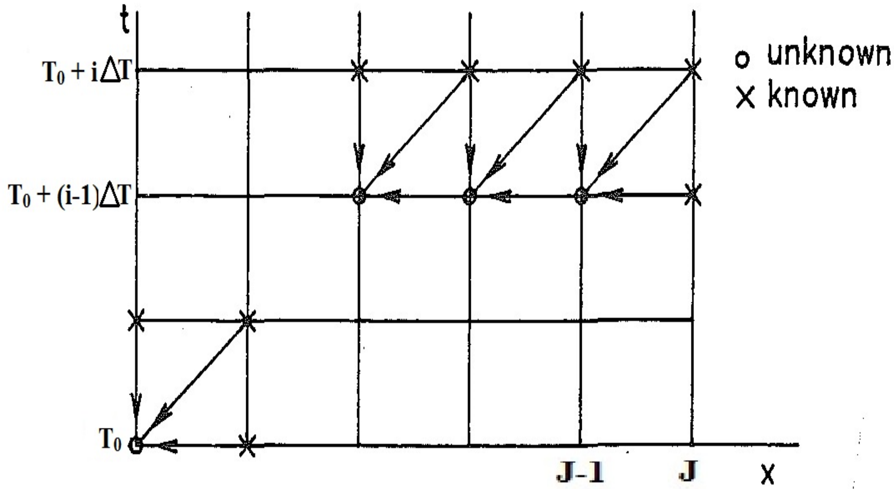

Fig.3.4. Sketch of backward computation in space and time

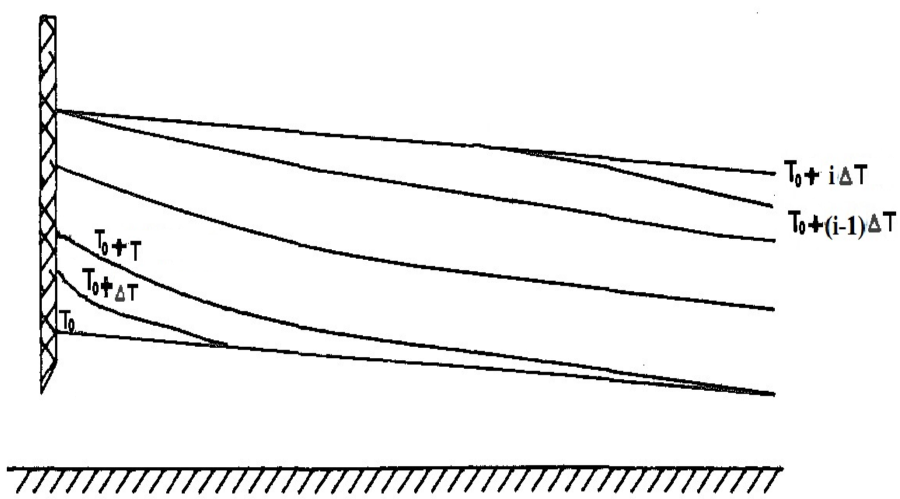

Fig.3.5.Changes of flow transferred from upstream to downstream

-backward computation in time. Considering the time level $(T_0+i\Delta)$,as the final condition, and knowing q_j and y_j between any two time levels at the downstream-end section, one may proceed first backward in time and then backward in space to compute the discharge and water-level profile at the time level $[T_0+(i-1)\Delta T]$, as shown in Fig. 4.

This computation process is continued until time level T_0. The solution gives the discharge and water level in the channel at each section, and characterizes the flow pattern at the upstream intake required to match the specification of the downstream discharge. The physical significance of the backward-computation method is clear. Knowing the expected outflow at the downstream outlet, one has to look backward, in both space and time, for the necessary upstream inflow.

3.2.3 DESCRIPTION OF TEST CANAL

For the application of the introduced new scheme i.e. the inverse explicit scheme was tested using the example presented in Liu et al. (1992). A rectangular channel with a bottom slope 0.001 is taken for knowing the performance of the scheme. For geometric cross section the width of the channel is taken as 5 metre. The length is considered as 2.5 km for forgoing test. The roughness coefficient is taken as 0.025 by watching the field condition. As the downstream out, a fixed overflow weir with free flow condition is used. For the downstream hydrograph is taken as boundary condition at downstream. The flow is increased $5m^3$/sec to $10m^3$/sec in one hour. For constant flow $10m^3$/sec is assumed for the next two hour. Then the flow is decreased to $5m^3$/sec. This hydrograph is taken as demand line. The stage discharge relationship is used to determine the water depth at downstream end condition. The hydrographs of stage and discharge are to determine at upstream section. By knowing the downstream boundary condition the upstream boundary condition are to be determine. The fixed overflow weir with free flow condition is used at downstream boundary condition.

3.2.4. WEIGHTING COEFFICIENT

Weighting coefficient when used in the equation generally gives more weightage to the results. Two weighting coefficient is used here $\emptyset$ and θ in terms of space and time respectively. θ is taken as 1 and $\emptyset$ as 0.5, 0.7 and 1.0 in this mathematical model.

3.2.5 INITIAL AND BOUNADARY CONDITION

Depth (y) values and the discharge (Q) values at the beginning of time step are to be specified at all the nodes along the canal as initial conditions. Two boundary conditions are needed for the downstream section for the operation-type problem. This objective is can be achieved by giving the discharge hydrograph and depth hydrograph at the downstream sections. The initial condition is given as $5.0m^3/s$ for discharge and water depth as 1.6m.

3.2.6 PROCEDURE OF SOLVING

Considering the all geometrical parameter the program for regulation of unsteady using inverse explicit method based on Preissmann scheme flow written in MATLAB. The two boundary condition i.e. the expected discharge and depth are given as input for the end time level. Then the working formula is written in MATLAB to simulate the data in backward manner to find upstream discharge and depth hydrograph. This hydrograph is the input of HECRAS computer model as the upstream boundary condition. HECRAS software is based on Preissmann implicit scheme simulate the upstream hydrograph to get downstream discharge and depth hydrograph.

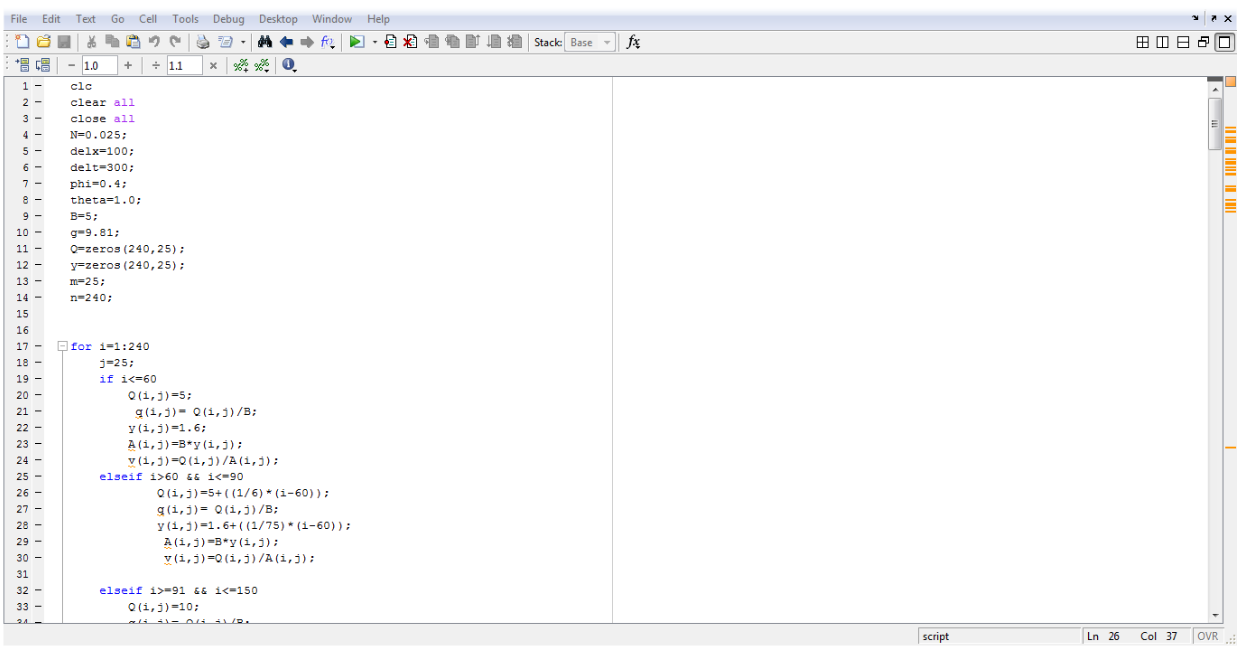

Fig.3.6 Image of programming in MATLAB

CHAPTER 4

RESULTS AND DISCUSSION

4.1. OVERVIEW

In the previous chapter, the methodology for solving the problem has been described. Mathematical model is developed by writing the codes in MATLAB and the result are simulate through HECRAS to get the accuracy and robustness of the model. Using different values of weighting coefficient in term of space and the different time step, the computation process is carried out and the corresponding results are shown in figures and tables. Both the discharge and depth hydrograph are obtained are compared with demand hydrograph and between them also. Effect of weighting coefficient (∅) and time interval (Δt) are compare for finding out the most suitable value of (∅) and (Δt), which can be used for the regulation of the flow problem giving accurate results. The detailed of the analysis is given below.

4.2. NUMERICAL TEST AND COMPARISION OF RESULTS

The computed discharge and depth hydrograph using inverse explicit method, the resulted hydrographs from HECRAS computer model and the comparison between them which are shown below from Fig.1 and Fig.10. Taking weighting coefficient ∅ as 0.5, 0.7, 1.0 and the weighting coefficient θ as 1.0 the computation is carried out. Different time intervals Δt as 120sec, 180sec,300sec are taken and space interval Δx is taken as 100m.

4.2.1. WEIGHTING COEFFICIENT (∅) COMPARISON

The computed discharge hydrograph using Inverse Explicit Method (IE Method) for ∅ = 0.5, 0.7, and 1.0 with time interval Δt=120sec are shown in Fig.4.1 to Fig4.3. It is clearly seen from the figure that the discharge hydrograph for ∅ = 0.5 has more oscillation than ∅ = 0.7 and 1.0 both in increasing and decreasing of flow rate. It is also observed that the ∅ = 0.7 has less oscillation where for ∅ = 1.0 has no oscillation. Oscillation is reduced with increase of ∅ value from 0.5 to 1.0. The computed depth hydrographs for ∅ = 0.5, 0.7, and 1.0 with time interval Δt=120sec are shown in Fig.4.6 to Fig4.8. In the depth hydrograph for ∅ = 0.5 the upstream section has oscillation as discharge hydrograph. For ∅ = 0.7 and 1.0 the computed depth hydrographs have less or no oscillation at the upstream. The flow and depth hydrograph are shown in Fig. 4.1 to

Fig.4.11 and the corresponding values are entered in Table 1, Table 2, Table-3 for Δt=120sec withdifferent Ø values 0.5, 0.7, and1.0 respectively.

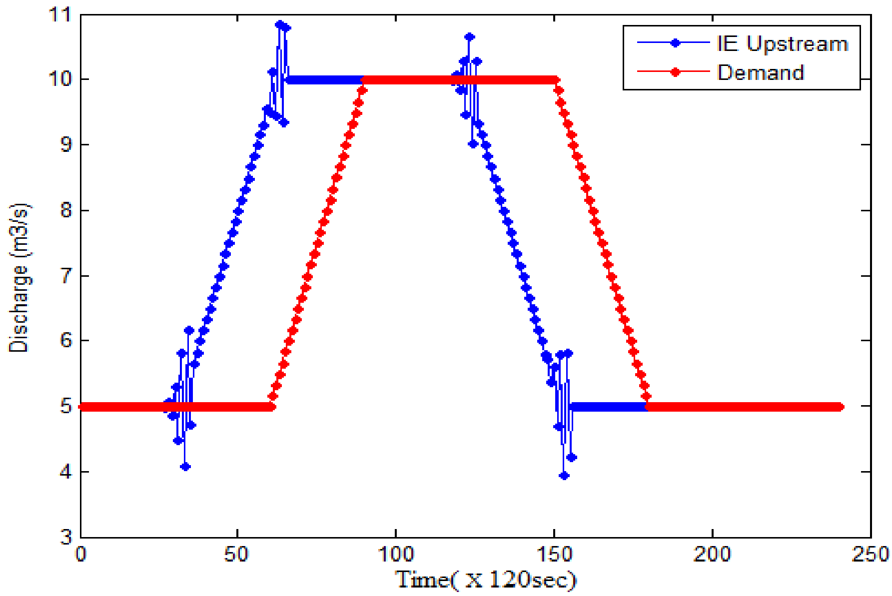

Fig.4.1. Computed Discharge hydrograph for Ø=0.5 and Δt=120sec

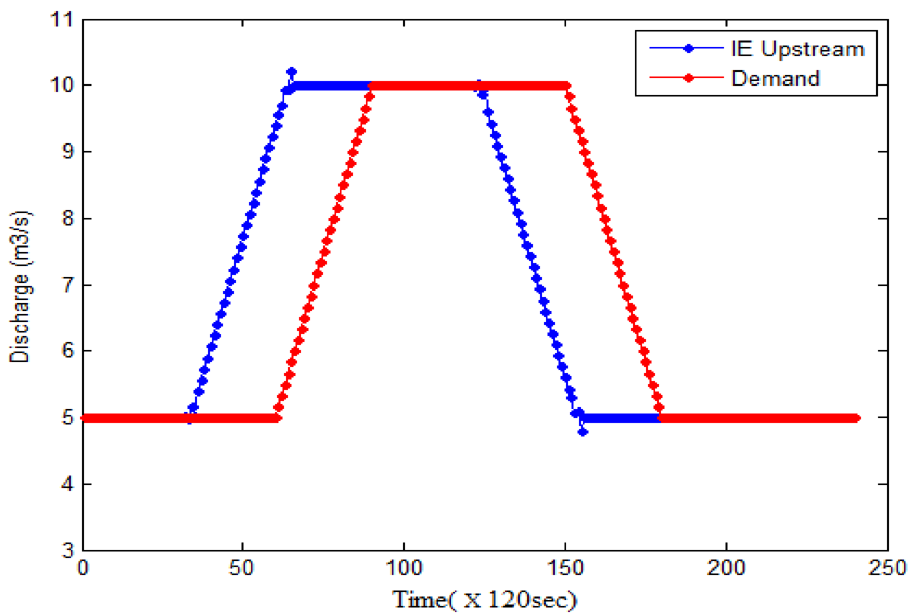

Fig.4.2.Computed Discharge hydrograph for Ø=0.7 and Δt=120sec

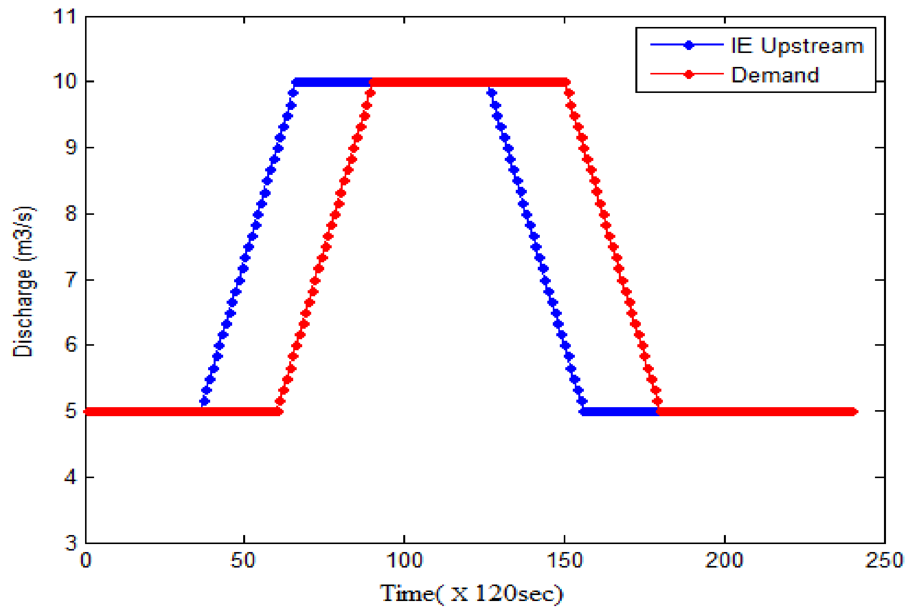

Fig.4.3.Computed Discharge hydrograph for Ø=1.0 and Δt=120sec

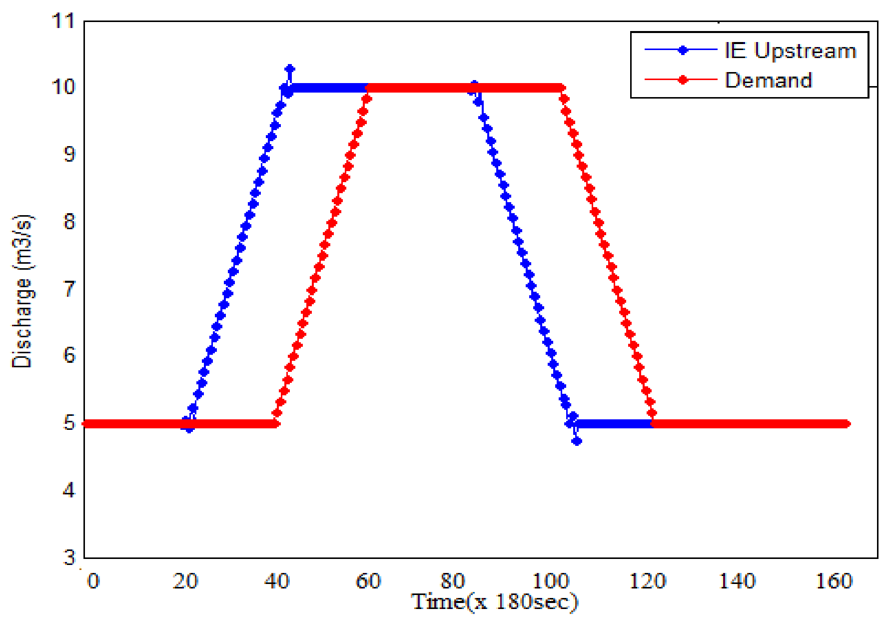

Fig.4.4.Computed Discharge hydrograph for Ø=0.5 and Δt=180sec

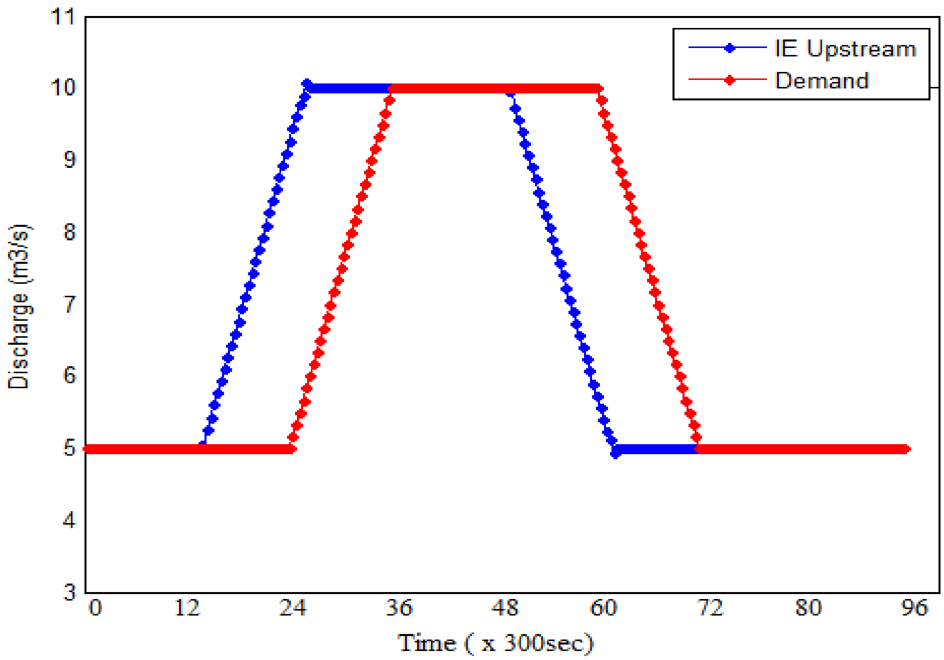

Fig.4.5. Computed Discharge hydrograph for Ø=0.5 and Δt=300sec

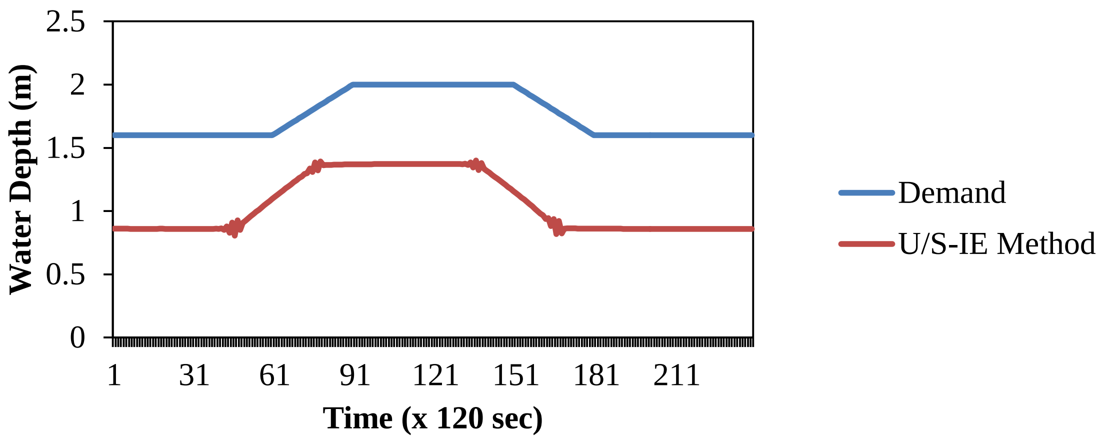

Fig.4.6. Computed Water Depth hydrograph for Ø=0.5 and Δt=120sec

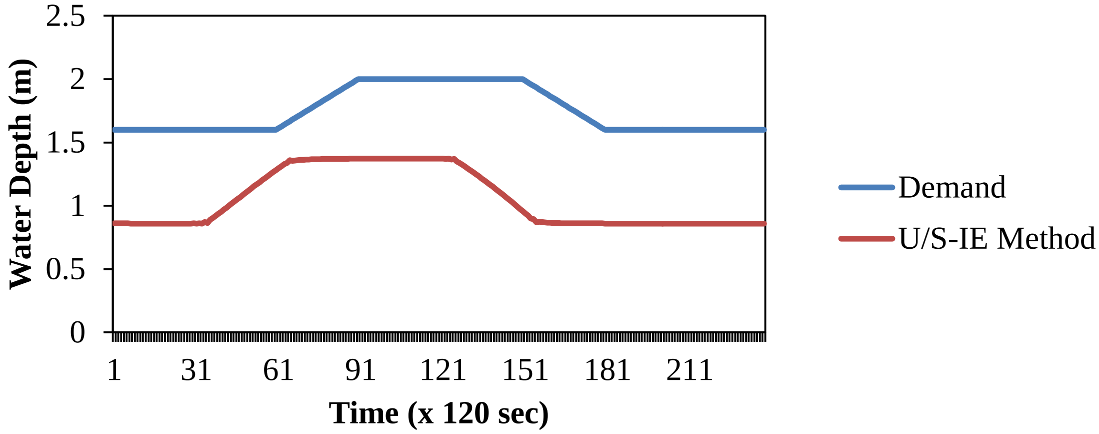

Fig.4.7. Computed Water Depth hydrograph for Ø=0.7 and Δt=120sec

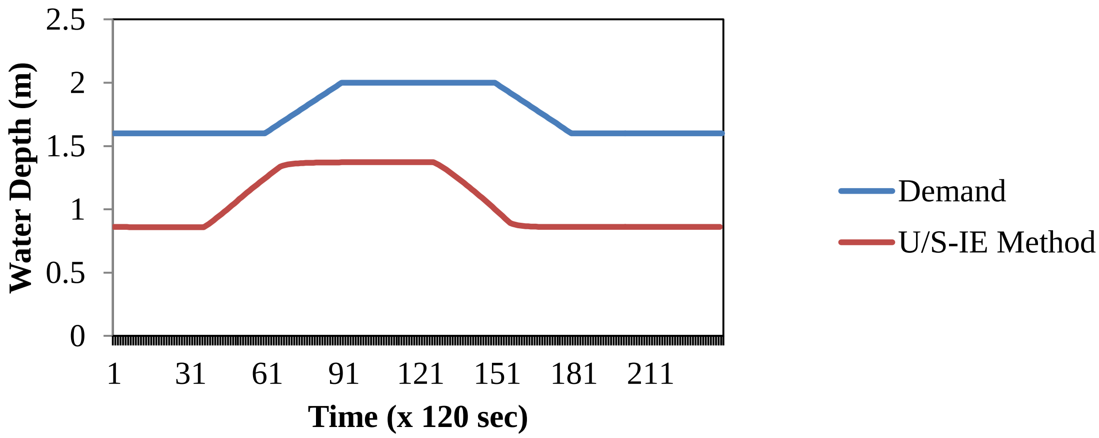

Fig.4.8. Computed Water Depth hydrograph for Ø=1.0 and Δt=120sec

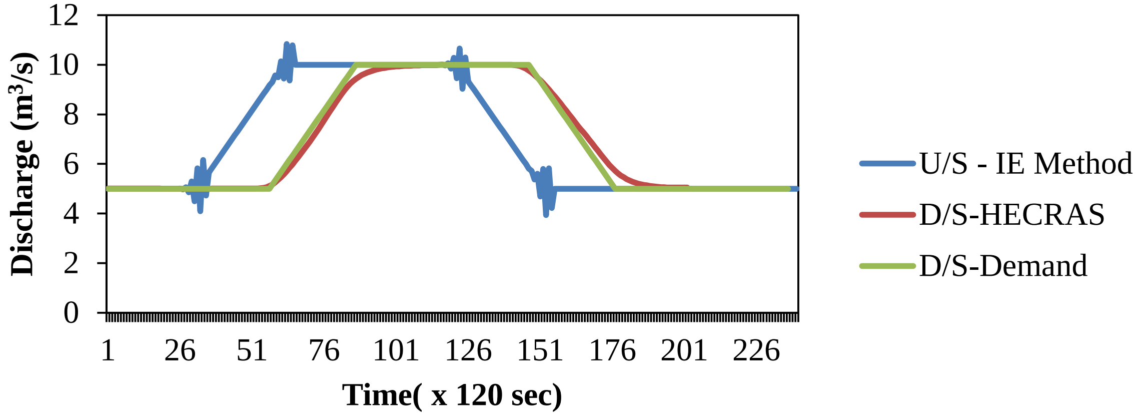

Fig.4.9. Computed Discharge hydrograph for Ø=0.5 and Δt=120sec

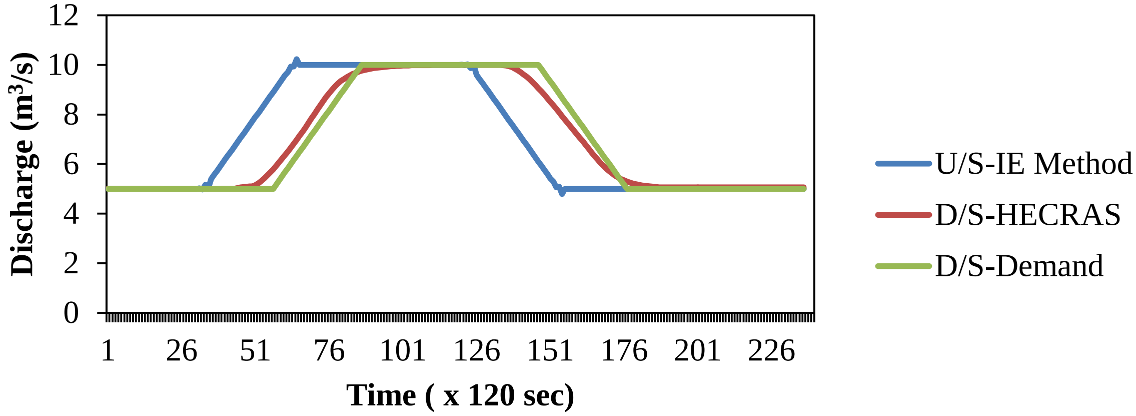

Fig.4.10. Computed Discharge hydrograph for Ø=0.7 and Δt=120sec

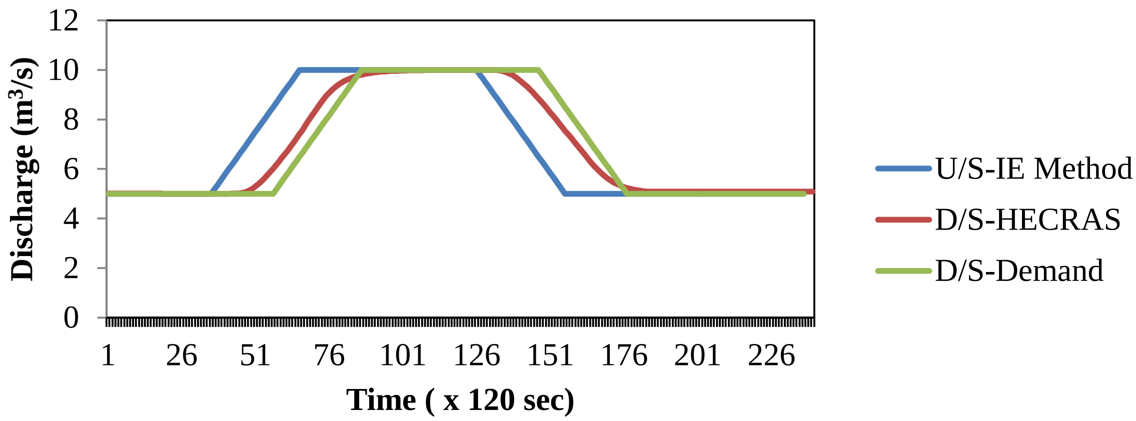

Fig.4.11. Computed Discharge hydrograph for Ø=1.0 and Δt=120sec

And the corresponding values of flow and depth are given in TABLE 1, TABLE 2, and in TABLE 3. The demand hydrograph at downstream is same as the downstream hydrograph obtained from HECRAS.

TABLE 1: Values of Discharge (Q) in m^3/s and water depth (y) in metre at D/S and U/S for Inverse explicit scheme and HECRAS (For $\emptyset=0.5$ and $\Delta t=120sec$) are given:

Time (minutes)	Inverse explicit method				HEC-RAS			
	At downstream end		At upstream end		At upstream end		At downstream end	
T	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)
0	5.000	1.600	5.000	0.861	5.000	0.860	5.010	1.591
20	5.000	1.600	5.000	0.860	5.000	0.860	5.009	1.598
40	5.000	1.600	5.000	0.860	5.297	0.860	5.003	1.599
60	5.000	1.600	5.297	0.860	6.328	0.864	5.001	1.600
80	5.000	1.600	6.328	0.864	7.996	0.943	5.010	1.600
100	5.000	1.600	7.996	0.943	9.498	1.113	5.009	1.600
120	5.000	1.600	9.498	1.113	10.000	1.273	5.003	1.600
140	5.000	1.600	10.000	1.273	10.000	1.365	5.001	1.600
160	5.000	1.600	10.000	1.365	10.000	1.370	5.005	1.601
180	5.000	1.600	10.000	1.370	10.000	1.372	5.005	1.642
200	5.500	1.613	10.000	1.372	10.000	1.372	5.353	1.685
220	7.167	1.747	10.000	1.372	9.847	1.372	6.742	1.813
240	8.833	1.880	9.847	1.372	8.663	1.370	8.471	1.947
260	10.000	2.000	8.663	1.370	6.996	1.303	9.639	1.992
280	10.000	2.000	6.996	1.303	5.601	1.141	9.916	1.999
300	10.000	2.000	5.601	1.141	5.000	0.968	9.980	2.000
320	10.000	2.000	5.000	0.968	5.000	0.863	9.995	2.000
340	10.000	2.000	5.000	0.863	5.000	0.861	9.999	2.000
360	10.000	2.000	5.000	0.861	5.000	0.860	9.997	1.996

380	9.500	1.987	5.000	0.860	5.000	0.860	9.485	1.909
400	7.833	1.853	5.000	0.860	5.000	0.860	8.147	1.787
420	6.167	1.720	5.000	0.860	5.000	0.860	6.680	1.662
440	5.000	1.600	5.000	0.860	5.000	0.860	5.456	1.611
460	5.000	1.600	5.000	0.860	5.000	0.860	5.103	1.602
480	5.000	1.600	5.000	0.860	5.000	0.860	5.043	1.600

TABLE 2: Values of Discharge (Q) in m^3/s and water depth (y) in metre at D/S and U/S for Inverse explicit scheme and HECRAS (For ∅=0.7 and Δt=120sec) are given:

Time (minutes)	Inverse explicit method				HEC-RAS			
	At downstream end		At upstream end		At upstream end		At downstream end	
T	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)
0	5.000	1.600	5.000	0.861	5.000	0.861	5.010	1.599
20	5.000	1.600	5.000	0.860	5.000	0.86	5.008	1.600
40	5.000	1.600	5.000	0.860	5.000	0.86	5.003	1.600
60	5.000	1.600	4.993	0.859	4.993	0.86	5.001	1.600
80	5.000	1.600	6.232	0.969	6.232	0.889	5.006	1.600
100	5.000	1.600	7.899	1.137	7.899	1.027	5.158	1.600
120	5.000	1.600	9.570	1.297	9.570	1.201	5.246	1.600
140	5.000	1.600	10.000	1.364	10.000	1.326	5.786	1.600
160	5.000	1.600	10.000	1.370	10.000	1.361	6.356	1.600
180	5.000	1.600	10.000	1.372	10.000	1.369	8.021	1.623
200	5.500	1.613	10.000	1.372	10.000	1.371	9.433	1.734
220	7.167	1.747	10.000	1.372	10.000	1.372	9.865	1.868
240	8.833	1.880	10.004	1.372	10.004	1.372	9.968	1.976
260	10.000	2.000	8.765	1.279	8.765	1.332	9.992	1.997
280	10.000	2.000	7.098	1.118	7.098	1.21	9.998	2.000

300	10.000	2.000	5.425	0.939	5.425	1.062	9.999	2.000
320	10.000	2.000	5.000	0.865	5.000	0.925	9.702	2.000
340	10.000	2.000	5.000	0.861	5.000	0.876	8.517	2.000
360	10.000	2.000	5.000	0.860	5.000	0.867	7.063	1.971
380	9.500	1.987	5.000	0.860	5.000	0.867	5.705	1.861
400	7.833	1.853	5.000	0.860	5.000	0.867	5.164	1.736
420	6.167	1.720	5.000	0.860	5.000	0.867	5.061	1.631
440	5.000	1.600	5.000	0.860	5.000	0.867	5.061	1.605
460	5.000	1.600	5.000	0.860	5.000	0.86	5.061	1.601
480	5.000	1.600	5.000	0.860	5.000	0.86	5.043	1.600

TABLE 3: Values of Discharge (Q) in m^3/s and water depth (y) in metre at D/S and U/S for Inverse explicit scheme and HECRAS (For ∅=1.0 and Δt=120sec) are given:

Time (minutes)	Inverse explicit method				HEC-RAS			
	At downstream end		At upstream end		At upstream end		At downstream end	
T	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)
0	5.000	1.600	5.000	0.861	5.000	0.861	5.010	1.599
20	5.000	1.600	5.000	0.860	5.000	0.860	5.008	1.600
40	5.000	1.600	5.000	0.860	5.000	0.860	5.003	1.600
60	5.000	1.600	5.000	0.860	5.000	0.860	5.001	1.600
80	5.000	1.600	5.833	0.931	5.833	0.889	5.001	1.600
100	5.000	1.600	7.500	1.097	7.500	1.027	5.001	1.600
120	5.000	1.600	9.167	1.260	9.167	1.201	5.001	1.600
140	5.000	1.600	10.000	1.360	10.000	1.326	5.001	1.600
160	5.000	1.600	10.000	1.370	10.000	1.361	5.296	1.600
180	5.000	1.600	10.000	1.371	10.000	1.369	6.603	1.623
200	5.500	1.613	10.000	1.372	10.000	1.371	8.305	1.734
220	7.167	1.747	10.000	1.372	10.000	1.372	9.540	1.868

240	8.833	1.880	10.000	1.372	10.000	1.372	9.890	1.976
260	10.000	2.000	9.167	1.314	9.167	1.332	9.974	1.997
280	10.000	2.000	7.500	1.158	7.500	1.210	9.994	2.000
300	10.000	2.000	5.833	0.983	5.833	1.062	9.999	2.000
320	10.000	2.000	5.000	0.869	5.000	0.925	9.999	2.000
340	10.000	2.000	5.000	0.861	5.000	0.876	9.549	2.000
360	10.000	2.000	5.000	0.860	5.000	0.867	8.284	1.971
380	9.500	1.987	5.000	0.860	5.000	0.867	6.829	1.861
400	7.833	1.853	5.000	0.860	5.000	0.867	5.570	1.736
420	6.167	1.720	5.000	0.860	5.000	0.867	5.134	1.631
440	5.000	1.600	5.000	0.860	5.000	0.867	5.088	1.605
460	5.000	1.600	5.000	0.860	5.000	0.860	5.088	1.601
480	5.000	1.600	5.000	0.860	5.000	0.860	5.043	1.600

The computed upstream hydrograph from inverse explicit method which is used as upstream hydrograph for HECRAS and that produce downstream discharge hydrograph and the downstream demand are compared for $\emptyset$ = 0.5, 0.7, and 1.0 with time interval Δt=120sec and the graphs are shown in Fig.4.9 , Fig.4.10 and Fig.4.11 respectively. From Fig.4.9 it is clearly seen that the downstream hydrograph which is obtained for $\emptyset$=0.5 satisfy the specification of downstream demand which is the very accurate result. In Fig.4.10 the computed downstream discharge hydrograph for $\emptyset$=0.7 moves towards left by nearly 10 minutes and in Fig.4.11 the computed downstream discharge hydrograph for $\emptyset$=1.0 moves towards left by nearly 15 minutes, which is very less and thus demand is little affected. But if we take some weighting co-efficient we would have accurate results.

4.4.2. TIME INTERVAL (Δt) COMPARISON

In this clause we discuss about the effect of time step on the computation of discharge and water depth hydrographs. When comparing weighting coefficient $\emptyset$= 0.5 gives good results, so taking $\emptyset$ as 0.5 and θ as 1.0 for different time steps the depth and discharge hydrograph are computed. Different time steps taken are Δt=120 sec, 180 sec, 300 sec. The discharge and depth hydrographs are plotted for different time level are shown in Fig. 4.12 to Fig. 4.17, in all the figures computed upstream hydrographs by present approach i.e. inverse explicit method and the upstream hydrograph for HECRAS computer model are same as it taken for the computation for downstream hydrograph. In Fig.4.4 and Fig 4.5 it is shown that for Δt= 180 sec and 300 has oscillation. But the oscillations gradually decrease with increase in time intervals. Δt=120sec having more oscillation in both discharge and depth hydrograph than other two time steps.

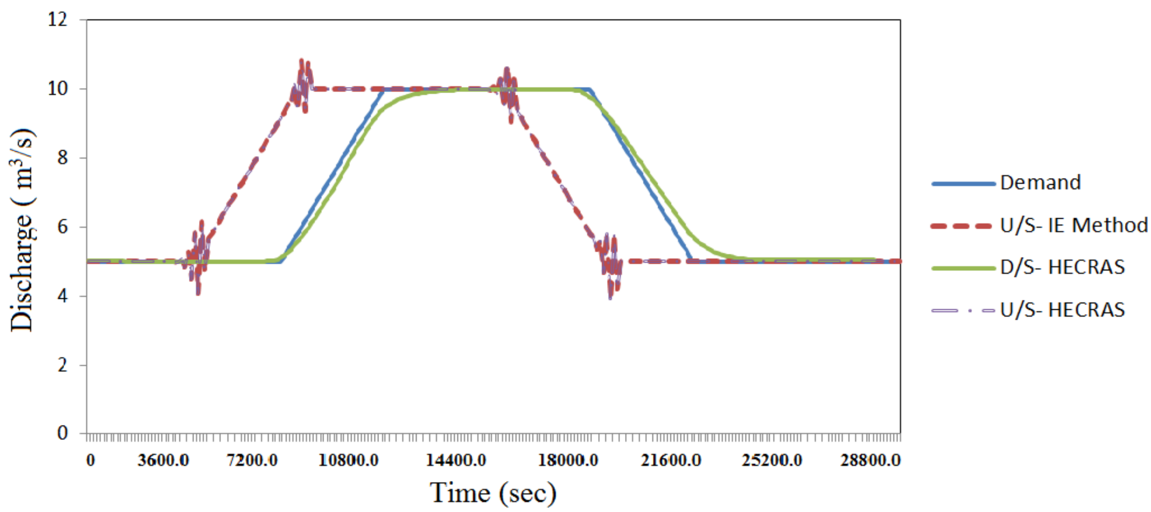

Fig.4.12 Computed Discharge hydrograph for Ø=0.5 and Δt=120sec

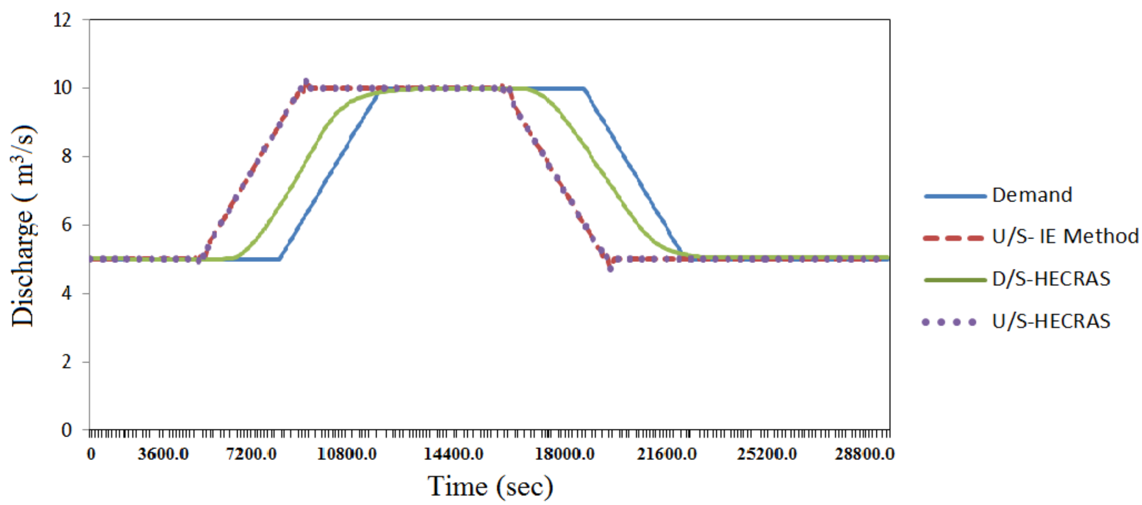

Fig.4.13 Computed Discharge hydrograph for Ø=0.5 and Δt=180sec

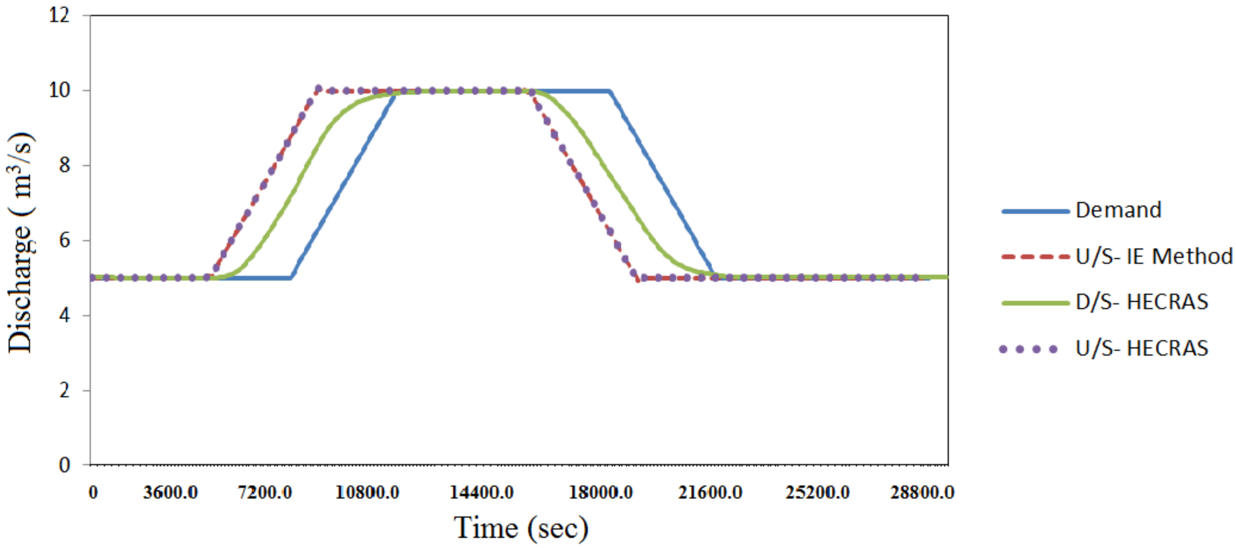

Fig.4.14 Computed Discharge (m^3/s) hydrograph for Ø=0.5 and Δt=300sec

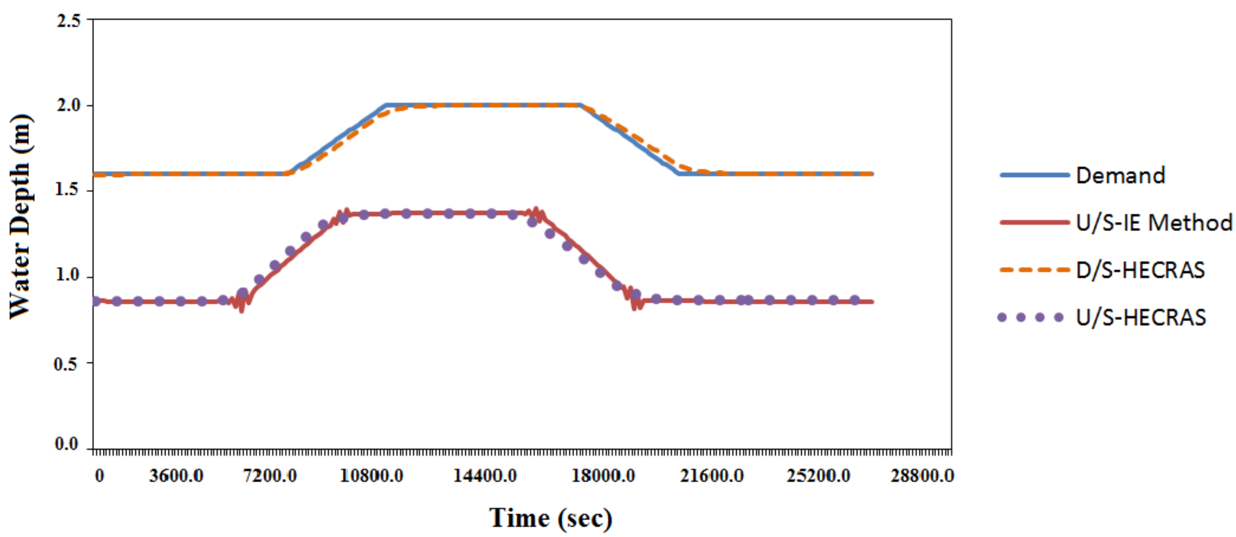

Fig.4.15 Computed Water Depth (m) hydrograph for Ø=0.5and Δt=120sec

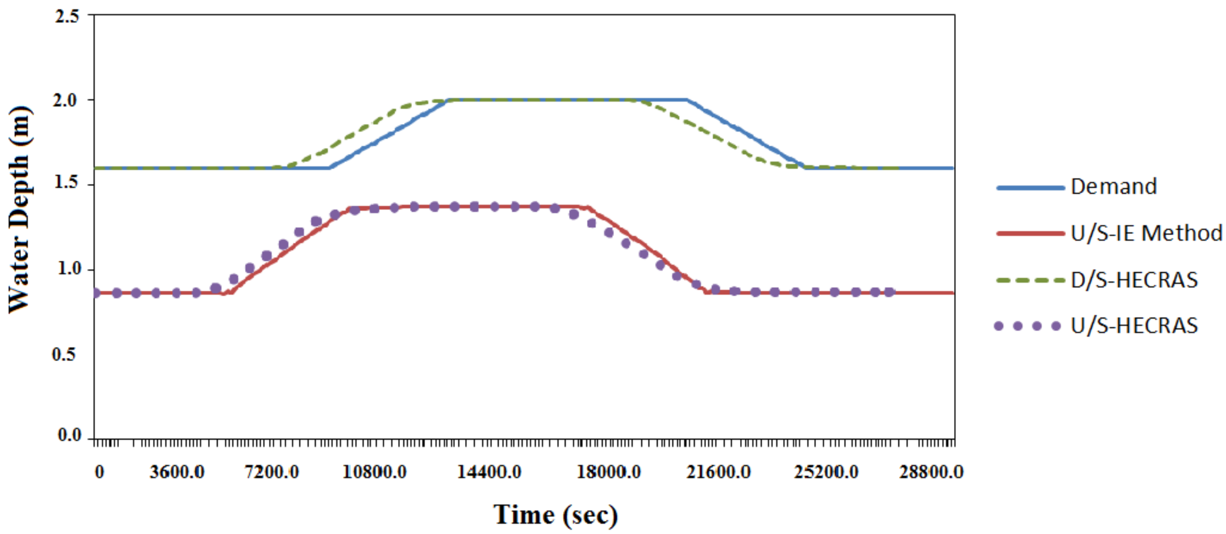

Fig.4.16 Computed Water Depth (m) hydrograph for Ø=0.5and Δt=180sec

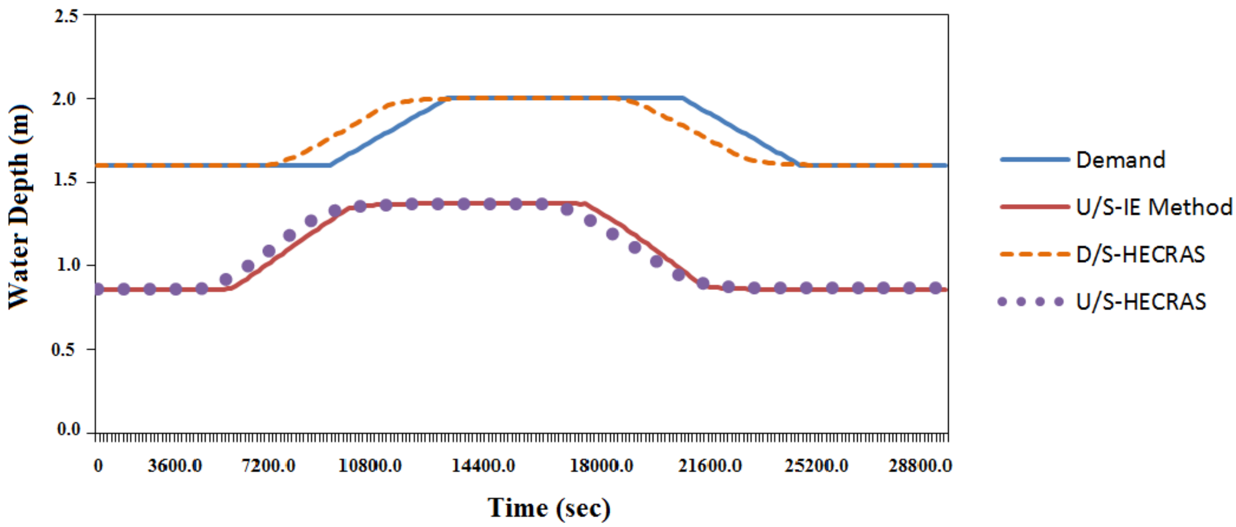

Fig.4.17. Computed Water Depth (m) hydrograph for Ø=0.5and Δt=300sec

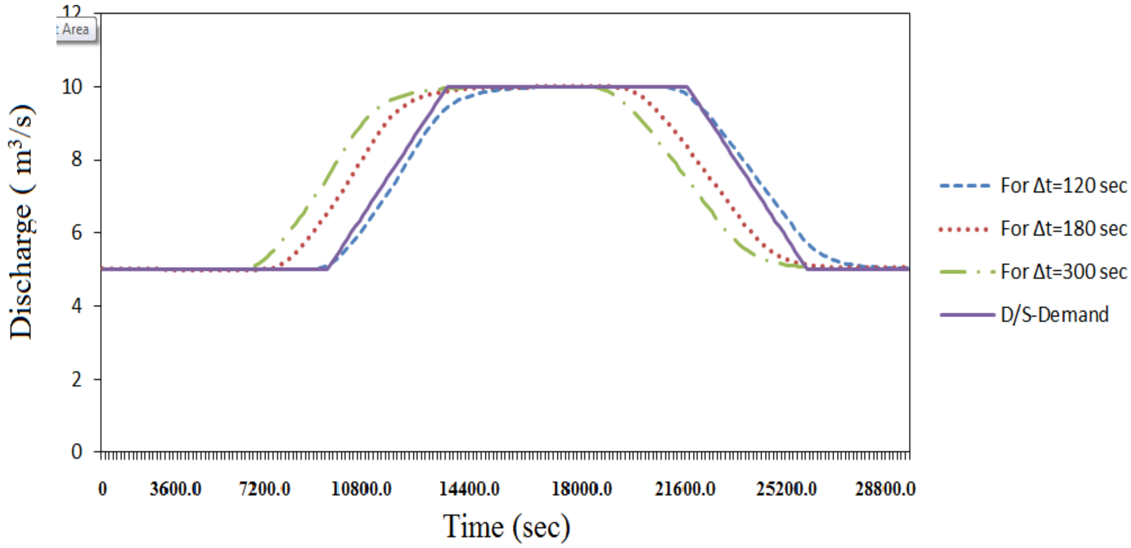

Fig.4.18. Computed Downstream Discharge (m^3/s) hydrograph for Ø=0.5 from HECRAS

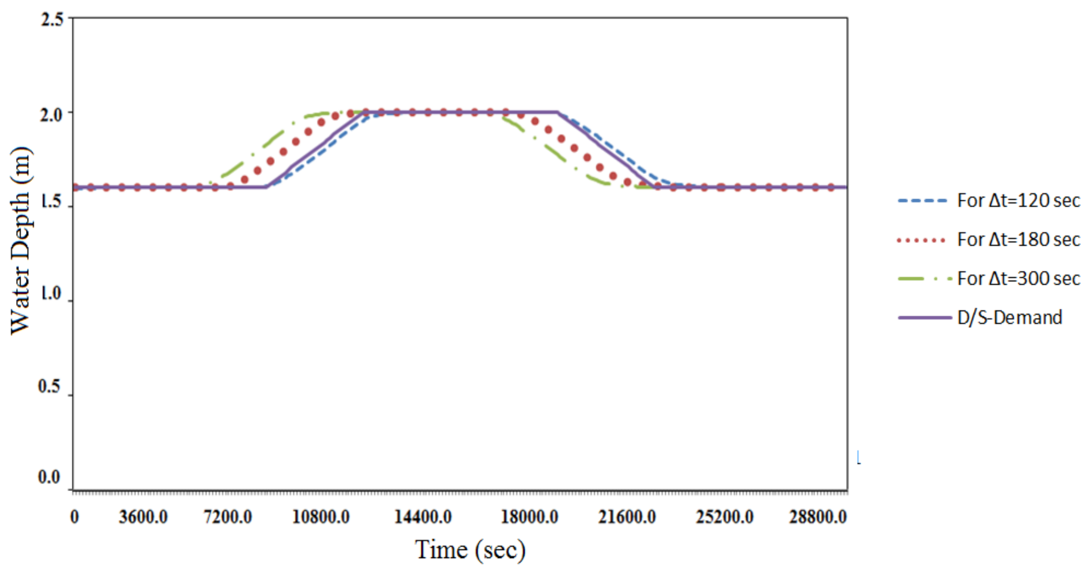

Fig.4.19. Computed Downstream Water Depth (m) hydrograph for Ø=0.5 from HECRAS

In Fig.4.15 for Δt=300sec the downstream hydrograph comes earlier than Δt=180sec and same things occurs for depth hydrograph also. With increase in time level the oscillation decreases and the depth and discharge hydrograph found is not matching specification of downstream demand.

Fig.4.18 and Fig. 4.19 shows the comparison of the depth hydrographs and discharge hydrographs for different time interval i.e. for Δt=120sec, 180 sec, and 300 sec. The result of the time interval Δt =120 sec shows more oscillation in the computed results during both the increasing and decreasing of the flow rate than that of the bigger values. Liu et al. 1992 mentioned that the computed upstream hydrographs using the backward operation explicit method with a smaller time interval has more oscillation which can cause the failure of the computation. It is known that the application of the explicit schemes in the routing problems required small time intervals to coincide with the Courant condition. In the contrary to this, Fig. 4.14 to 4.19 shows that the computed upstream hydrographs becomes smoother with larger time intervals. The accuracy of the reproduced downstream hydrographs is better for smaller time interval than those obtained bigger time intervals. So, with smaller time interval, the computed upstream inflow can be defined more accurately, but this will produce more oscillation to meet more specifications of the downstream outflow. The values for depth and discharge for different time interval i.e. Δt=120sec, Δt=180sec, and Δt=300sec are entered in TABLE 1, TABLE 4, and TABLE 5 respectively.

TABLE 4: Values of Discharge (Q) in m^3/s and water depth (y) in metre at D/S and U/S for Inverse explicit scheme and HECRAS (For ∅=0.5 and Δt=180sec) are given:

Time (minutes)	Inverse explicit method				HEC-RAS			
	At downstream end		At upstream end		At upstream end		At downstream end	
T	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)
0	5.000	1.600	5.000	0.861	5.000	0.861	5.010	1.599
20	5.000	1.600	5.000	0.860	5.000	0.860	5.008	1.600
40	5.000	1.600	5.000	0.860	5.000	0.860	5.003	1.600
60	5.000	1.600	4.984	0.859	4.984	0.860	5.001	1.600
80	5.000	1.600	6.276	0.974	6.276	0.889	5.008	1.600
100	5.000	1.600	7.943	1.142	7.943	1.027	5.692	1.623
120	5.000	1.600	9.620	1.302	9.620	1.201	7.184	1.734

140	5.000	1.600	10.000	1.365	10.000	1.326	8.906	1.868
160	5.000	1.600	10.000	1.370	10.000	1.361	9.327	1.976
180	5.000	1.600	10.000	1.372	10.000	1.369	9.564	1.997
200	5.500	1.613	10.000	1.372	10.000	1.371	9.737	2.000
220	7.167	1.747	10.000	1.372	10.000	1.372	9.937	2.000
240	8.833	1.880	10.010	1.372	10.010	1.372	9.985	2.000
260	10.000	2.000	8.721	1.275	8.721	1.332	9.996	2.000
280	10.000	2.000	7.054	1.114	7.054	1.210	9.999	1.971
300	10.000	2.000	5.372	0.933	5.372	1.062	9.966	1.861
320	10.000	2.000	5.000	0.864	5.000	0.925	9.152	1.736
340	10.000	2.000	5.000	0.861	5.000	0.876	7.743	1.631
360	10.000	2.000	5.000	0.860	5.000	0.867	6.285	1.605
380	9.500	1.987	5.000	0.860	5.000	0.867	5.323	1.601
400	7.833	1.853	5.000	0.860	5.000	0.867	5.077	1.600
420	6.167	1.720	5.000	0.860	5.000	0.867	5.068	1.600
440	5.000	1.600	5.000	0.860	5.000	0.867	5.068	1.600
460	5.000	1.600	5.000	0.860	5.000	0.860	6.680	1.600
480	5.000	1.600	5.000	0.860	5.000	0.860	5.043	1.600

TABLE 5: Values of Discharge (Q) in m^3/s and water depth (y) in metre at D/S and U/S for Inverse explicit scheme and HECRAS (For Ø=0.5 and Δt=300sec) are given:

Time (minutes)	Inverse explicit method				HEC-RAS			
	At downstream end		At upstream end		At upstream end		At downstream end	
T	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)	Q(m^3/s)	y(m)
0	5.000	1.600	5.000	0.861	5.000	0.860	5.012	1.599
20	5.000	1.600	5.000	0.860	5.000	0.860	5.005	1.600
40	5.000	1.600	5.000	0.860	5.000	0.860	5.002	1.600
60	5.000	1.600	5.000	0.859	5.000	0.889	5.000	1.600

80	5.000	1.600	6.099	0.974	6.099	1.027	5.117	1.600
100	5.000	1.600	7.766	1.142	7.766	1.201	6.242	1.623
120	5.000	1.600	9.433	1.302	9.433	1.326	7.878	1.734
140	5.000	1.600	10.000	1.365	10.000	1.361	9.356	1.868
160	5.000	1.600	10.000	1.370	10.000	1.369	9.846	1.912
180	5.000	1.600	10.000	1.372	10.000	1.371	9.903	1.976
200	5.500	1.613	10.000	1.372	10.000	1.372	9.963	1.997
220	7.167	1.747	10.000	1.372	10.000	1.372	9.991	2.000
240	8.833	1.880	10.000	1.372	10.000	1.332	9.998	2.000
260	10.000	2.000	8.899	1.275	8.899	1.210	10.00	2.000
280	10.000	2.000	7.233	1.114	7.233	1.062	9.756	2.000
300	10.000	2.000	5.566	0.933	5.566	0.925	8.627	1.971
320	10.000	2.000	5.000	0.864	5.000	0.876	7.179	1.861
340	10.000	2.000	5.000	0.861	5.000	0.867	5.794	1.736
360	10.000	2.000	5.000	0.860	5.000	0.867	5.184	1.631
380	9.500	1.987	5.000	0.860	5.000	0.867	5.012	1.605
400	7.833	1.853	5.000	0.860	5.000	0.867	5.012	1.600
420	6.167	1.720	5.000	0.860	5.000	0.867	6.680	1.600
440	5.000	1.600	5.000	0.860	5.000	0.867	5.456	1.600
460	5.000	1.600	5.000	0.860	5.000	0.860	5.103	1.600
480	5.000	1.600	5.000	0.860	5.000	0.860	5.043	1.600

In TABLE 1, TABLE 4 and TABLE 5, it is observed that the value of discharge coming earlier with increase of time interval. Only in TABLE 1, for $\emptyset = 0.5$ and Δt=120 the values are very nearly matching the specification of the demand.

CHAPTER 5

CONCLUSIONS

5.1 CONCLUSION

In solution of unsteady flow the operation type problem is used for computing the flow at upstream section for operating, to allow required quantity of inflow as per the demand at downstream end of a canal. It is also used for regulation of the structure to find the necessary magnitude of outflow at observation section at downstream locality. The regulation is done at the delivering system at upstream. Here, for regulating the unsteady flow in the canal finite difference algorithm of inverse explicit method based on Preissmann scheme has been presented. As the inverse explicit scheme is based on preissmann implicit scheme, for the space interval Δx, and time interval Δt , the method is unconditionally stable .

As the scheme is unconditionally stable the solution is made to determine the upstream flow to get predefined water demand at a distance of 2.5 km from upstream end. The performance of the scheme is tested by using Δt =120 sec for the given values of $\emptyset = 0.5$, 0.7 and 1.0. The result obtained from the scheme it is seen that, when the value of $\emptyset$ is less i.e. 0.5 the oscillation is high and it is decreases with the higher value i.e. 1.0. It is seen from the reproduce downstream hydrograph using HECRAS computer model that, when the value $\emptyset$ is 0.5 the result of this hydrograph is equal to specified demand. But for the higher values of $\emptyset$ the time taken by the inflow to meet the downstream demand is less than the time taken by the inflow for smaller value.

Taking the $\emptyset$ as 0.5, the different time interval (Δt = 180sec, 300sec) are taken. It is seen from the result of explicit scheme for more time intervals the oscillations are less than the smaller time interval at upstream. From the reproduce hydrograph obtained by HECRAS computer model, it is seen that for the higher values of time interval, the time taken for the downstream demand hydrograph is less than smaller values of time interval. It is concluded for the given problem the weighting coefficient and the time interval is 0.5 and 120 sec respectively.

5.2. SCOPE FOR FUTURE WORK

The present work leaves a broad spectrum for the investigator to analyze the regulation process by taking different parameter. The work can be extended by changing weighting coefficient in term of time and space interval for getting the discharge and depth at downstream area. Various other numerical method like method of characteristics, Beam-warming implicit method etc. can be used for getting the result. The discharge and depth hydrographs at downstream sections are found for straight rectangular channel. The algorithm developed may be improved by taking different geometry of the channel or canal.

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