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EFFECT OF TRIBUTARY JUNCTION
ON
ROUTING CHARACTERISTICS

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PREFACE

In river systems confluences are typical features and of a complex nature. The behaviour of the confluences is largely determined by the specific characteristics of the rivers involved. The junction imposes a computational difficulty in accurate routing of floods in river networks because of the mutual backwater effects of the channels joining at the junction.

In the present report various methods developed to predict the progress of a flood wave along a junction have been briefly described and the overlapping method using four-point implicit finite difference scheme has been described. The overlapping segment method faithfully simulates the flood flow in the network and the computer results agree well with those obtained by the time consuming solutions of all the flow equations of the entire network. Conversely, the commonly used sequential cascading channel method produces inaccurate results, particularly when the down-stream backwater effect is important and reverse flow occurs.

This report entitled "Effect of Tributory Junction on Routing Characteristics" is a part of the work programme of "Flood Studies" division of the Institute. The study has been carried out by Sh. Surendra Kumar, Scientist 'B' under the guidance of Dr. S.M. Seth, Scientist 'F'.

(SATISH CHANDRA)

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LIST OF SYMBOLS

1.	A	=	The flow cross-sectional area measured normal to x ;
2.	β	=	Width of Channel;
3.	BSS	=	Off-Channel storage width for middle cross-section at elevation E;
4.	$C_{i+0.5}$	=	The finite difference approximate equation of continuity written for the channel reach between station i and i+1;
5.	F	=	Grid function;
6.	g	=	Gravitational acceleration;
7.	H	=	Depth of water;
8.	h_f	=	Loss of energy head;
9.	i	=	i-th inflow channel;
10.	j	=	Time stage of computation;
11.	L	=	Distance between first and third cross-section;
12.	M	=	Total number of branches;
13.	$M_{i+0.5}$	=	The finite difference approximate equation of momentum written for the channel reach between station i and i-th.
14.	n	=	Roughness coefficient;
15.	N_i	=	Number of grid points in the i-th branch;
16.	0	=	Outflow channel;
17.	q	=	The time rate of lateral flow through unit length of σ ;
18.	Q	=	Discharge through A;
19.	$Q_i(t)$	=	Discharge in river 1 at time 't' ;
20.	$Q_2(t)$	=	Discharge in river 2 at time 't' ;
21.	R	=	Hydraulic radius ;
22.	S	=	Water stored in the junction;

23. SA = Surface area of Off-channel storage at elevation E;

24. S_f = Friction slope of the flow;

25. S_o = Channel bottom slope;

26. Δt = constant time interval;

27. U_x = The x-component of the velocity of the lateral flow when joining or leaving the main flow;

28. V = Velocity of the flow;

29. Δ_{xi} = Length of the flow reach measured in horizontal direction and centred at the i-th station;

30. y = Depth of flow;

31. y_{ic} = Critical depth corresponding to the instantaneous flow rate Q_i;

32. Y_o = Depth of the outflow channel;

33. Y_s = Water surface elevation above a reference horizontal datum;

34. Z = Elevation of channel bed at the junction;

35. Zo = Drop of the outflow channel;

36. β = Momentum correction factor;

37. σ = Perimeter bounding A;

38. θ = Weighting factor;

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ABSTRACT

In accurate routing of floods in river networks using the St. Venant's equations, the junction imposes a computational difficulty because of the mutual backwater effects of the channels joining at the junction. Various methods have been developed to predict the progress of a flood wave along a junction. These have been briefly discussed and the overlapping method using four-point implicit finite difference scheme has been described in detail in this report. Having surveyed the different methodologies and experiences of various authors/implementors, the data requirement needed for finding the solutions to such problems faced by Water Resources Engineers has also been given seperately. Conventionally, the downstream backwater effect is simply ignored and the computations for the network are proceeded in a cascading manner towards the downstream, leading to unrealistic results. The overlapping segment method faithfully simulates the flood flow in the network and the computer results agree well with those obtained by the time consuming solution of all the flow equations of the entire network. Conversely, the commonly used sequential cascading channel method produces in-accurate results, particularly when the downstream backwater effect is important and reverse flow occurs.

1.0 GENERAL

Simons and Gessler (1971) contended that "Theory on hydraulic processes is years ahead of theory on geomorphic processes and there is a pool of knowledge in the field of boundary layer theory which could be tapped for answers in relation to geomorphic problem". Progress has been realized with the increased use of theory in geomorphology since 1965, but such theoritical developments have concentrated upon river channel cross-sections or reaches because networks are necessarily more complex. In fact networks have been studied in at least three principal ways by geomorphologists. First, have been studies of network topology which have only recently been directed towards the problem of the relationship between network topology and stream flow hydrograph formation as attempted by Surkan (1974). Second, have been studies of drainage network densities in relation to climatic characteristics and streamflow (Gregory, 1976 b), but only since 1968 has the nature of the relationships become apparent so that we are in a position to model change in the manner previously attempted for river channel metamorphosis. Thirdly, there has been great progress in the study of network extension by gullying (Cooke and Reeves, 1976 b) but only recently has it become usual to include ancillary consideration of associated geometry changes downstream. Unfortunately, studies of network contraction by the producation of dry valleys, have not usually been investigated for recent short periods of time but have been placed in a longer time scale context.

Reversal of cause and effect at different scales complicates the relationship between a river and its catchment. For example, in the long term the spatial patterns of runoff and sediment production determine the evolution of drainage network structure. However , over the intermediate time-scale of the river - channel steady state the catchment water and sediment delivery represent direct environmental controls of the river channel which are focused on to the river by the drainage network. The essential underlying control of the drainage network is considered in terms of both its structural arrangement, which controls exploitation of the spatially-distributed runoff and sediment sources, and its density, which influences the intensity of runoff and sediment yield to the channel. Slope in hydrological processes, which vary regionally in relation to climate, relief, soils and vegetation, determine the form of the quickflow hydrograph which is then transformed as it is routed through the network. Thus downstream reaches are influenced by flood hydrographs which reflect the yeild of storm runoff from the hill-slopes modulated by network properties.

The density of the net of tributaries has a considerable impact on the density of the river net itself (i.e. the length of all the water course in km/km^2). It depends primarily upon the amount of precipitation, the hydrological attributes of the rocks, and the type of ground cover. For example, in similar conditions of precipitation and evaporation, river density is less on lime stone than on metamorphic rock. The tendency of limestone to faulting does not offer the right conditions for the formation of a dense surface network of run-off channels.

In river systems confluences are typical features, be it of a complex nature. There is no general treatise the existing literature is usually restricted to specific cases. Some typical confluences of Indian rivers are shown in figure (1). From the figures it is seen that some rivers are joining the main river making acute angle (in most of the cases), some are joining at right angle and some (very few) at obtuse angle.

This is quite logical that the behaviour of the confluences is largely determined by the specific characteristics of the rivers involved. Some general remarks about this can be made with reference to Figure (2) as below:

(a) At the confluence, the difference between the regimes $Q_1(t)$ and $Q_2(t)$ seems to be a dominating factor. For $Q_1 > Q_2$ strong backwater effects can be found in river 2. Hence the supply of sediment from river 2 to the main river 0 can be rather irregular. This means that in river 2 there is no direct link between Q_2 and S_2 near the confluence.

(b) At a confluence, the grain size distribution of the river 0 may vary greatly with time if the grain

Fig.- 1(a) GODAVARI BASIN

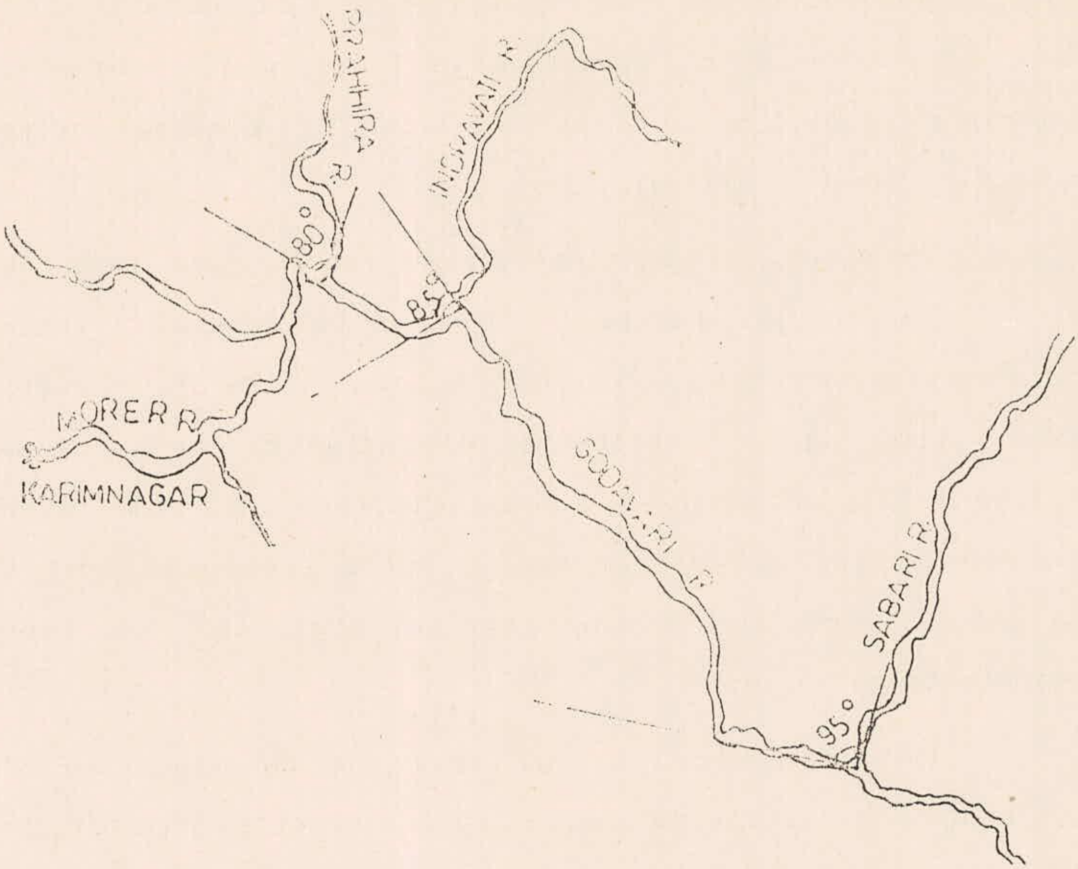

Fig. 1(b) MAHANADI BASIN

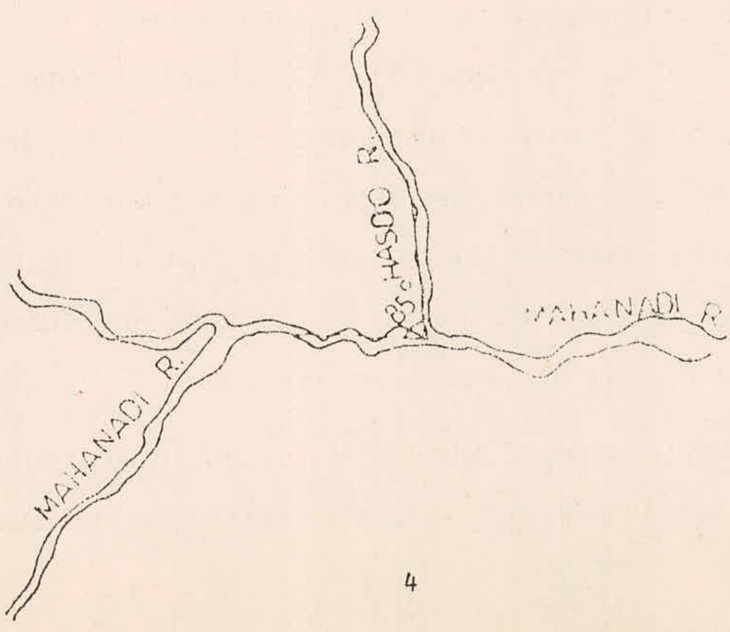

Fig. 1(c) PENNER BASIN

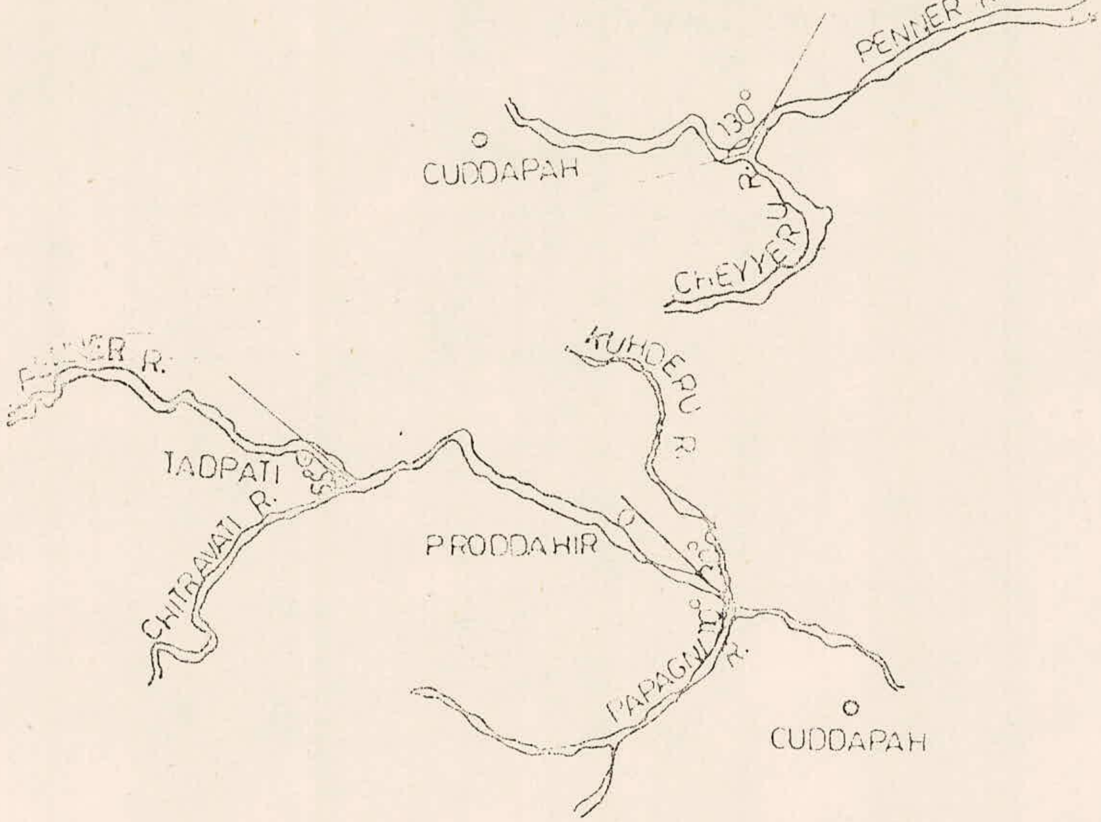

Fig. 1(d) NARMADA BASIN

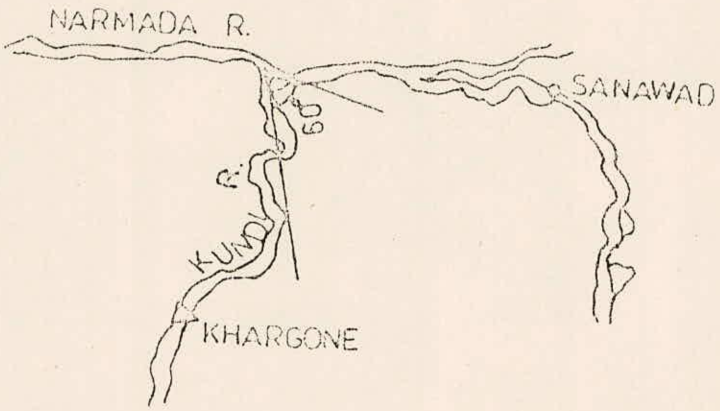

Fig. 1(e) GANGA BASIN

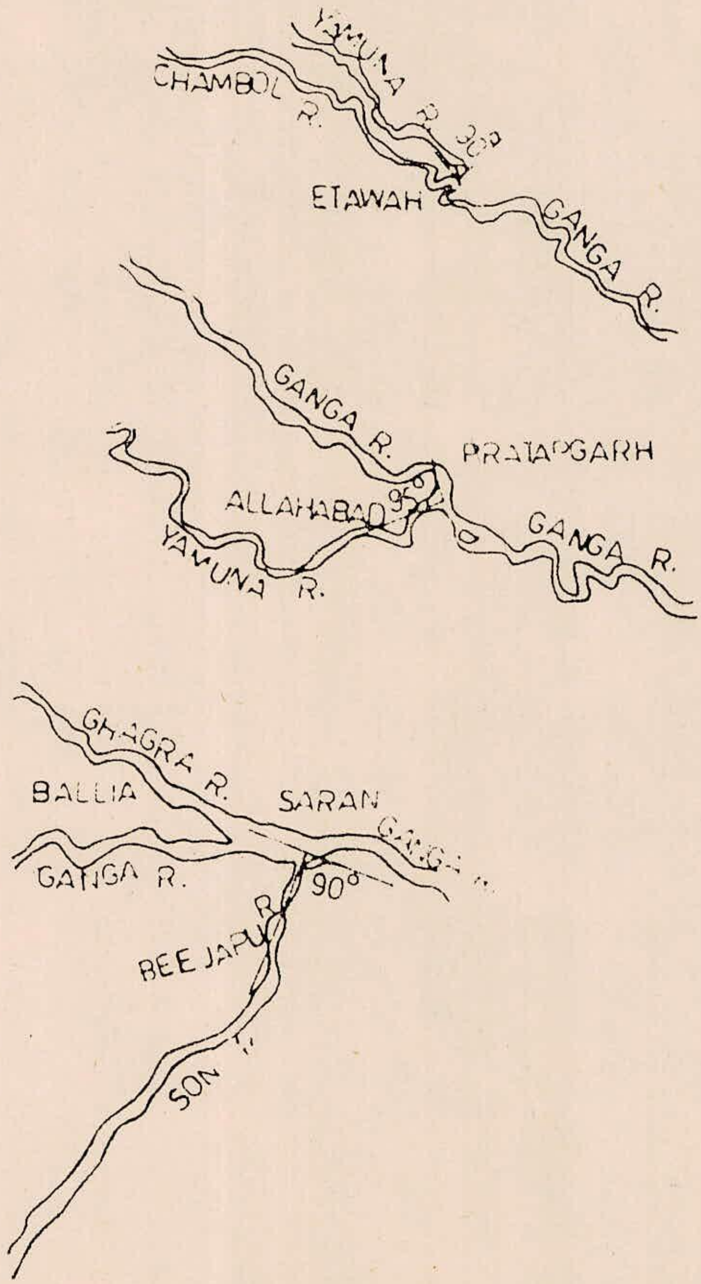

Fig. 1(f) KRISHNA BASIN

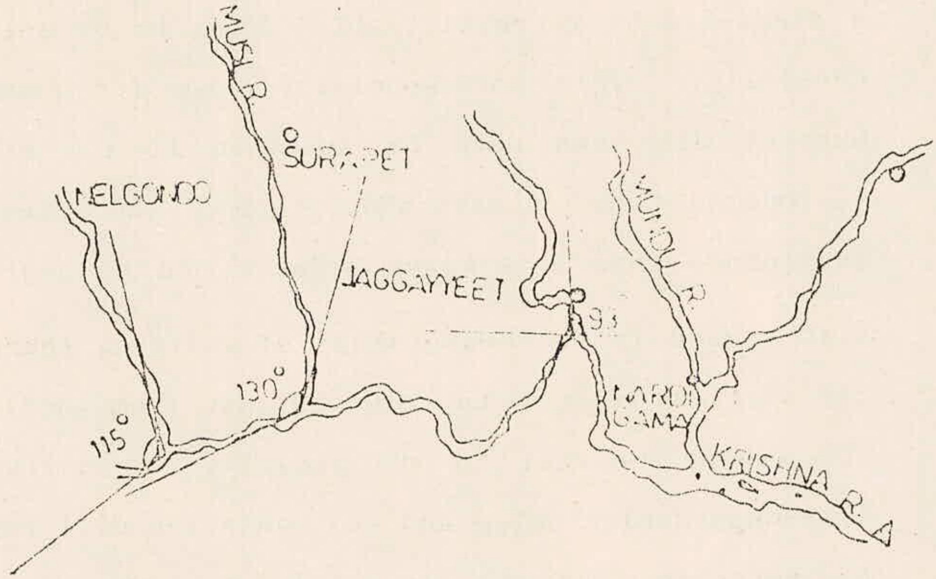

Fig. 1(g) - INDUS BASIN

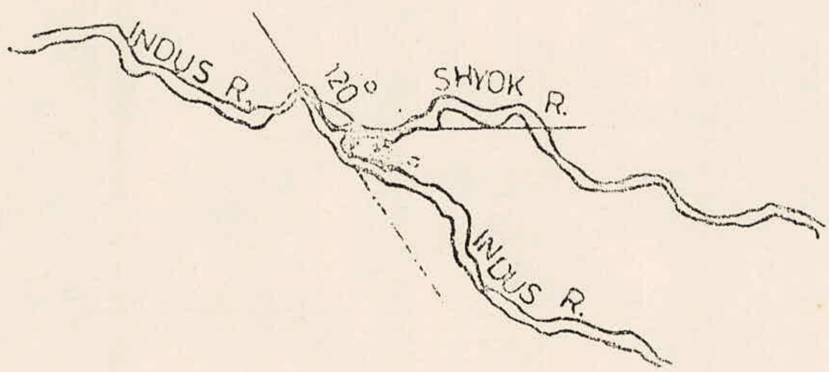

FIG. 1. TYPICAL CONFLUENCES OF INDIAN RIVERS

sizes of the rivers 1 and 2 differ substantially. These effects can also be attributed to differences in the regimes $Q_1(t)$ and $Q_2(t)$.

(c) The stations for discharge measurements (to establish a stage-discharge relationship) have to be selected carefully. This is especially true for river 2. Correct data can only be obtained if the station is located such a distance upstream of the actual confluence that back-water effects can be neglected.

(d) With regard to the measurement of sediment transport the stations have to be selected away from confluences. The reason is that in the vicinity of confluences time-dependent erosion and sedimentation will probably be relatively large (especially for the rivers 0 and 2).

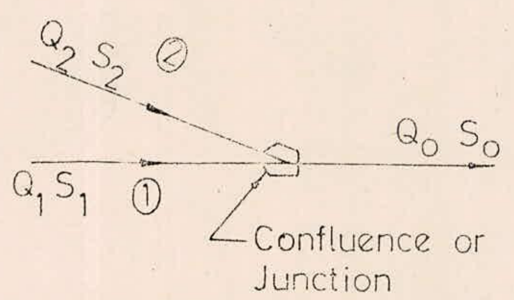

Fig. 2. A River junction

2.0 REVIEW:

The presence of junction in a river reach imposses complexity in routing the flood along the river. Several methods have been developed to route the flood along a river but somewhat limited number of studies exist for considering junction effect. The work done by different researchers dealing specifically with the junction problem in flood routing is discussed as follows:

Taylor (1944) in his study, established a relation between the flow characteristics and the geometry of the junction, for sub-critical flow passing through the channels at a junction, where all channels have the same breadth.

Stoker (1957) carried out a numerical computation for the passage of a flood wave originating in the Ohio River and passing through the junction with Mississippi River. In this study he assumed that the depths at the junction in all the three branches at any instant are the same. In essence this computation serves as the link between the behaviour of the flood wave in the different branches.

Stoker (1957) applied the fixed mesh explicit method the first and best known numerical method for the solution of the equations of unsteady flow.

Chow (1959) stated two basic methods of flood routing namely hydraulic method and hydrologic method. When a flood comes through a junction, backwater is usually produced. According to him this problem can accurately be evaluated only by the basic hydraulic equations employed in the hydraulic method but not by the hydrologic method.

Grushevskiy and Fedorov (1965) reported on field investigations by the State Hydrological Institute of unsteady flow on the Tvertsa, Oredeyh and Svirj Rivers, each with a different morphology. Stages and release wave travel were investigated. The authors made preliminary conclusions on the influence of flood plains, ice cover, wave celerities, and velocity distributions.

Meijer et.al. (1965) described an approximate method for computing discharges and water levels for unsteady flows in open channel networks. The system of channels has been schematized into a network of nodes and branches; storage capacity is concerned in the nodes and resistance in the middle of tne branches. In the derivation of equation it is assumed that the difference in velocity head between the end points of a branch is negligible, that the slope of the channel bottom is small, and that the pressure distribution is hydrostatic. An implicit-finite difference scheme and iteration process has been used to solve the equations. Comparison between computed results and corresponding field observations showed this method could yield good results in many cases.

Baltzer and Lai (1968) developed computer-oriented simulation techniques and applied these techniques in selected field reaches. They used one dimensional theoritical approach for the treatment of transient open-channel flow.

Garrison et.al. (1969) used a digital computer program for solving the basic equations of unstea flows in reservoirs and natural rivers.

Tsuchiya and Takahashi (1969) developed an analog computer in which the hydraulic variables are transformed to electric potential or current, considering the magnitude of each term in the equation of motion of unsteady flow in rivers.

The analog computer made it possible to calculate the flow in rivers with tributaries and diversions under various boundary conditions. Reliability of the computer was confirmed by an experiment on flood propagation in a experimental flume.

Amein and Fang (1970) presented a generalized implicit method for application to irregular channels in which the cross-sectional geometry and bottom elevation could vary from section to section. The method was found to be stable for large time steps, it was fast and was well suited for engineering applications involving flows of long duration and long channel reaches of complex geometry.

Ellis (1970) applied the method of characteristics to examine the possibility of producing a computational system for use in the conditions where glacial activities produces changes in river cross-section.

Fread (1972) showed that the interaction of storage and dynamic effects between the two river could be simulated efficiently by a mathematical model consisting of (1) the conservation form of the two unsteady flow differential equations, and (2) known stage-time and/or stage discharge relationships at the extremities of the rivers.

Henry (1972) presented a paper in which the system has been divided into overlapping segments and the motion has been computed for each segment seperately over a time increment short enough to ensure that errors due to neglect of neighbouring segments have been confined to the regions of overlap. The solution for the whole system at the end of each time increment has been found by discarding erroneous portion of the solutions for the various segments and piecing together the remaining parts. The difficulties of programming a single large difference scheme to cover a whole inlet or river have been avoided and the problem has been reduced to linking standard subroutines representing commonly encountered features such as bifurcating and confluences to test the method in a known physical situation and to compare various methods of linking adjacent segments.

Pinkayan (1972) solved the unsteady flow equations, describing the problem of routing storm water through storm drainage systems with lateral inflow, by the method of characteristics on digital computers. The storm drain consists of a single continuous line of circular channel with constant slope. The main inflow to the drain is at the upstream end. The lateral inflow comes through a circular conduit at the junction box being normal to the direction of the main drain. The outflow is a free fall at the downstream. A good agreement has been found in comparing computed hydrographs and observed hydrographs in the experiment at various locations along the drain.

Quinn & Wylie (1972) developed a hydraulic transient model of the Detroit River by using the implicit method to solve the complete equations of continuity and motion. In this, the river has been modeled in the shape of a Y and has one main channel and two branching channels. The stability of the numerical solution, which uses the Newton-Raphson algorithm, has been found to be dependent on the selection of a weighting co-efficient. This co-efficient determines the position at which the equations are evaluated on the x-t grid. The model input consists of water surface hydrographs at the head and mouth of the river. The output consists of flows at each end of the three channels and water surface elevations at the junction of the Y. Transient flows of the Detroit River induced by a severe wind tide or Lake Erie were simulated to illustrate the model. Good aggreement was obtained between measured and computed water surface elevations at the junction of the Y.

Sevuk (1973) applied the overlapping Y-segment method to sewer networks with prismatic channels, an in view of the relatively short duration of the floods and the range of Froude number of the flows, he used a first-order characteristic method to solve the St. Venant equations. The concept of overlapping segment has been adopted in this study and extended to fork junctions and non-prismatic channels using three upstream branches and one downstream branch as an example. A four-point finite difference implicit numerical scheme has been used to solve the St. Venant equts. because of its computational efficiency and stability for relatively slow, long duration river floods (Amein and Fang, 1969, Sevuk and Yen, 1973 a, Fread, 1974, Price, 1974).

Sevuk and Yen (1973) in their study presented a comparison of four different approaches used in routing flood waves through open channel junctions.

Sevuk and Yen (1973) considered the attenuation of a flood wave passing through a junction of a large river with a tributary of comparable size. They approximated the incoming flood wave to be a number of step waves satisfying the kinematic wave condition and the disturbances caused in the tributary to be dynamic waves, where the dynamic effects are large in comparison with frictional and gravitational effects. The authors presented a theory based on this model and obtained explicit expressions for the depths and discharges in the different branches.

Price (1974) found that the four point implicit method was the most efficient and maintained stability under severe test conditions.

Amein and Chu (1975) extended the work on the four point implicit method by Amein and Fang (1970). They modified the numerical scheme to provide a versatile technique that would embrace large changes in channel geometry, large fluctuations in discharge ranging from abrupt to gradually varied and a variety of boundary conditions. By applying to field problems they demonstrated the versatility of the implicit method.

Wood, et.al. (1975) presented the formulation of a mathematical model to predict transient flows in hydraulic networks. The network formulation consists of breaking the network in to a series of connected reaches; reducing the finite difference equations for each reach in to two reach equations, forming exterior matrix consisting of the reach equations, external boundary conditions, and interior compatibility conditions; solving the external matrix for the end values of discharge and water surface elevation for all reaches and back substituting for all interior values.

Soliman (1976), studied the Blue-white Nile confluence in Republic of Sudan and the Tigris-Diyala river confluence in Iraq for the purpose of arrangements required for flood protection. This case he studied theoritically using computer program which lead to useful recommendations for the flood control arrangements due to the heading up caused by the river junction.

In the study by Yen and Akan (1976), It has been shown that the overlapping segment method can be extended to route unsteady flow through fork-type dentritic networks of non-prismatic channels using a four-point implicit finite difference scheme. According to them it requires much less computer time than simulataneous solution of the flow equations for all the branches of the network, and it is considerably more accurate than the sequential method which neglects the downstream junction backwater effect. They concluded that the over-lapping segment method simulated the flood flow faithfully and its results aggreed well with the simultaneous solution method.

Akan & Yen (1981) developed a non-linear diffusion wave model for flood routing in dentritic-type open channel networks. This model accounts for the downstream backwater effects. The authors have applied overlapping-segment technique to the diffusion wave equations which are written in finite differences. The resulting set of non-linear algebraic equations for each segment has been solved by the Newton iteration process. The coefficient matrices obtained in the solution process are band and, therefore, the matrix equts. have been solved by a very efficient numerical technique. After comparision of this model with dynamic wave and non-linear kinematic wave models they concluded:

1. The diffusion-wave model could satisfactorily simulate the mutual backwater effects of channels joining at a junction.
2. The model was nearly as accurate as the dynamic wave model which might be classified among the most sophisticated one-dimensional routing techniques available in the literature.
3. The model was faster and cheaper in computation than a kinematic wave model which might be considered as one of the simplest hydraulic routing models known.

Sevuk and Yen (1982) investigated hydraulics of unsteady flow in storm sewer networks by using a dynamic wave simulation model. They adopted the Illinois storm sewer (ISS) Model which solves the complete dynamic equations using the method of characteristics. Two examples were presented. One was a 48-pipe sewer system to show that for unsteady open channel flow in sewer networks, the back water effect can not be ignored, under certain circumstances reversal flow may occur, the depth-discharge relationship is not unique and a loop-type rating curve exists, and the accurrence of maximum depth. The second example showed how a dynamic wave routing model could be used to evaluate and improve a sewer system.

The authors at last concluded that the junction effect could not be ignored for flow in sewer networks and the storage in the junctions and sewer pipes is an important factor for the attenuation and dispersion of the flood waves.

Li et.al. (1983) studied the methods for calculating flow in branch channels. In order to obtain the unsteady flow in the channels without branching, they provided the non-equidistant difference schemes on interior mesh points and the corresponding difference schemes on boundary mesh points.

Joliffe (1984) presented a numerical model for simulating flows in either looped or dentritic channel networks. The solution procedure adopted solves the full non-linear gradually varied unsteady flow equations using the generalized Newton-Raphson technique. A sparse matrix technique has been used to store and solve the resulting set of linear equations that has been solved to find the flow corrections during the simulation; use of this matrix technique allows the computer storage to be substantially reduced. Analytic differentiation has been used to evaluate the partial derivative terms of the linear flow correction equations. This type of differentiation permitted significant improvements in the computational efficiency of the model.

Lai (1986) in his paper described the computer modelling of unsteady flow. He included the development of the real-world algorithm for flow simulation in natural environments which deals with the computer science and engineering technology of building a simulation model involving the desciplines of model implementation, application, utilization.

Ramana Murthy et.al. (1989) have developed an electronic analogue computer for flood forecasting of Tapi Basin which is based on Muskingum storage equation of flood routing. This is the extension of the work done by the author earlier (Ramana Murthy, 1965, 1967). The presence of the tributary flows has been taken into account by adding the tributary flow to the routed flow in the main river upto the confluence. In case of the several tributaries the routing has been done upto the major tributary confluence.

3.0 METHODOLOGIES:

River models can be calssified according to the tools used as physical models, analog models and mathematical models. The mathematical models can again be classified as hydraulic models and hydrologic models; the former consider the mechanics of the flow, whereas the latter utilizes no more than the continuity relationship of the flow and usually treats the river as a lumped system.

Mathematical modeling of steady open-channel flow has been discussed extensively in the literature (e.g., see chow, 1959; Henderson, 1966). Three and two-dimensional modeling for rivers is still at the development stage. Therefore, in this report the discussion is centred on mathematical hydraulic, fixed boundary, one-dimensional, unsteady flow models.

Unsteady Flow Equations:

Hydraulically, the flow equations required to route floods through junctions can be divided into two groups. One group is the equations describing the junction conditions and the other group for the channels joining to the junction. In seeking numerical solution the former serves as the boundary conditions of the latter.

The flow in a channel can be described mathematically by a set of one dimensional shallow water wave equations commonly known as the St. Venant equations. By using a gravity oriented coordinate system with x measured horizontally along the channel i.e. longitudinal direction and vertically as shown in the figure (3), the St. Venant equations in discharge-area form are:

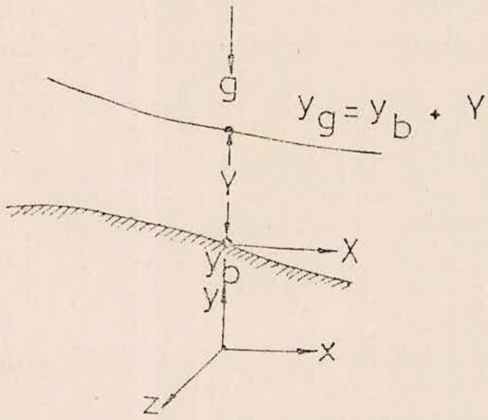

Fig. 3. Gravity Oriented Coordinates with Depth Measured Vertically

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = \int_{\sigma} q \, . \, d\sigma \qquad \ldots(1)$$

$$\frac{1}{gA} \frac{\partial Q}{\partial t} + \frac{1}{gA} \frac{\partial}{\partial x} \left(\rho \frac{Q^2}{A} \right) + \frac{\partial Y_s}{\partial x} - S_o + S_f = \frac{1}{gA} \int_{\sigma} U_x . q . d\sigma \qquad \ldots(2)$$

in which,

- A = the flow cross-sectional area measured normal to x (i.e. a vertical direction);
- Q = discharge through A;
- σ = the perimeter bounding A;
- q = the time rate of lateral flow through unit length of σ and being positive for inflow and negative for outflow;
- U_x = the x-component of the velocity of the lateral flow when joining or leaving the main flow;

Ys = water surface elevation above a reference horizontal datum;

β = momentum correction factor;

S_f = friction slope of the flow;

t = time;

g = gravitational acceleration;

S = channel bottom slope.

The friction slope S_f, can be estimated by using the Manning's, Darcy-Weisbach's or similar formulas. The first equation is the continuity equation whereas the second is the momentum equation. The derivation of these formulas is given by Yen (1975). The assumptions involved in deriving these equations include.

(a) the fluid is incompressible and homogeneous

(b) the piezometric pressure distribution on the cross-section A is uniform.

(c) the spatial rate of change of the internal flow stresses w.r.t. x is not appreciable.

The gravity oriented coordinates and the discharge area form of the equations are chosen because of their relative convenience in routing flows in channel of irregular cross-sections and alignments.

Boundary and Initial conditions:

Solution of an unsteady river flow problem obviously depends on the initial and boundary conditions imposed on the river flow. In other words, when using the St. Venant equations to model river flow, two initial conditions and two boundary conditions must be specified in order to obtain a unique solution that describes the flow.

Initial Conditions:

The initial conditions specify the flow conditions at the initial time (t=0) of computation of the unsteady flow. The two initial conditions for the St.Venant equations are the velocity, V (x,o), or discharge, Q (x,o), paired with the area, A(x,o), or depth, Y (x,o) or h (x,o), specified for the entire channel length at initial time t=0. However, if an initial dry bed condition is specified with V or Q=0 and A or h=0 at t=0, a numerical singularity is generated. For such a case one can assume a non-zero but small and negligible initial depth so that the computations can proceed. After all, the St. Venant equations are unreliable for dry bed because under such conditions the Weber number (interfacial or surface tension) effect is important, which is not accounted for in St. Venant equations.

Boundary Conditions:

The boundary conditions specify the time variations of discharge, velocity, depth, or area of the boundary locations. For a subcritical flow, both boundary conditions must be specified at the upstream boundary of the channel. For a subcritical flow, one boundary condition must be specified at the upstream end of the channel, whereas the other must be at the downstream end.

Channel Junctions:

The precise hydraulic description of the flow at channel junctions is rather complicated and difficult because of the high degree of flow mixing, seperation, turbulence, and energy loss. Yet correct representation of the junction hydraulics is important in realistic and reliable computation of flow in river networks. In addition to the continuity relationship, the dynamic relationship can be represented by either the energy or the momentum equations. In applications, the momentum equations are rarely used because they are vector equations and the pressure acting on the junction boundaries is usually difficult to describe. The energy equation is usually expressed in a simplified form of one of the following equations:

$$Q_i \left(\frac{V_i^2}{2g} + Y_i + Z_i\right) = Q_o \left(\frac{V_o^2}{2g} + Y_o + Z_o + h_f\right) \quad ..(3)$$

$$\frac{V_i^2}{2g} + Y_i + Z_i = \frac{V_o^2}{2g} + Y_o + Z_o + h_{fi} \quad ...3(a)$$

in which,

Q = Flow in to or from the junction;

v = velocity of flow in to or from the junction;

y = depth of flow;

Z = elevation of channel bed at the junction;

h_f = loss of energy head.

The subscript 'i' indicates the ith inflow channel at the junction and 'o' represents the outflow channel. The head loss h_f depends on the characteristics of the flow in junction and channel and is not easy to be determined. In general there are six possible junction flow conditions as follows:

(a) sub-critical flow in all the upstream branches and also in the downstream branch;

(b) subcritical flow in at least one but not all of the upstream branches whereas subcritical flows in all the rest of the upstream branches as well as in the downstream branch.

(c) super critical flow in all of the upstream branches and subcritical flow in the downstream branch.

(d), (e) and (f) are same as (a), (b) and (c) except that the downstream branch flow is super critical. In small mountain streams any one of the above six cases is possible. However, for rivers usually the flow is subcritical in all the branches.

Type of Junctions;

For practical purposes, the junctions can be classified in to point type and reservoir type depending upon whether the junction storage capacity is negligible relative to the volume of the flow.

(a) Point-type Junction:

For junctions with insignificant storage capacity, the junctions can be considered as a point-type junction which is assumed to be represented by a single confluence point without storage. The net discharge into the junction is therefore zero at all times. Hence;

$$\sum Qi = Qo \qquad ...(5)$$

A typical point-type junction with two inflow channels and one outflow channel is shown schematically in Figure 4, for which;

$$Q_1 + Q_2 = Qo \qquad ...(6)$$

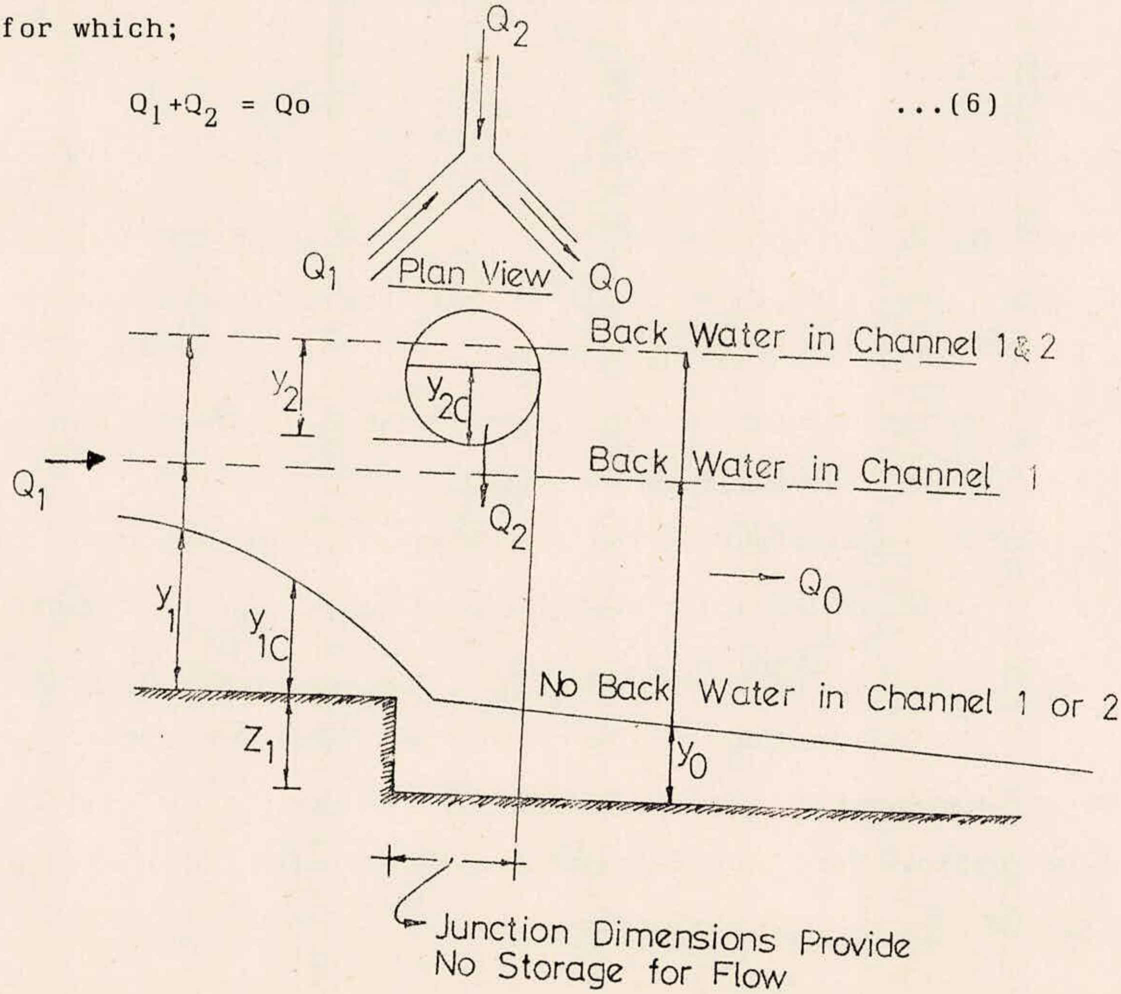

Fig. 4. POINT TYPE JUNCTION

For subcritical flow in the inflowing channels, the flow discharges freely into the junction only when a free fall exists over a non-submerged drop at the end of the channel. Otherwise, the subcritical flow in the inflowing channel is subject to back-water effect from the junction. Since, the junction is considered as a point, the energy compatibility condition can be represented by a common water surface at the junction for all the joining channels. Thus by referring to the Figure 4.

$$Yi = Yic \quad \text{if } Zi + Yic > Yo + Zo \qquad \ldots (7)$$

otherwise,

$$Yi + Zi = Yo + Zo \qquad \ldots (8)$$

in which,

Yi = depth of flow of the i-th inflowing channel at the junction;

Yic = critical depth corresponding to the instantaneous flow rate Qi;

Zi = height of the i-th inflowing channel;

Yo&Zo = depth and drop, respectively, of the outflowing channel.

Flow in the outflow channel may be either sub-critical or super-critical. In the latter case, Yo is equal to the critical flow depth, Yoc, corresponding to the instantaneous flow rate Qo.

Flow in the inflow channels can also be supercritical, discharging freely in to the junction, provided the flow at the downstream of the channel is not submerged by the back-water in the junction. For such case, the discharge of the inflowing channels in to the junction can be computed without considering the flow condition in the junction.

(b) Reservoir-type Junction:

The reservoir-type junction has a relatively large storage capacity in comparision to the flow. Consequently, it can be assumed to behave like a reservoir with a horizontal water surface and capable of obsorbing and dissipating all the kinematic energy of the inflows. The net discharge in to the junction is equal to the time rate of change of storage in the junction i.e.

$$\sum Qi - Qo = \frac{dS}{dt} \qquad \text{... (9)}$$

in which,

S = Water stored in the junction.

The depth of water, H, in the junction is assumed equal to the specific energy of the flow at the entrance of the outflow channel, i.e.

$$H + Yo + \frac{Vo^2}{2g} + Zo \qquad \text{... (10)}$$

A typical three-way reservoir junction is shown schematically in Figure 5.

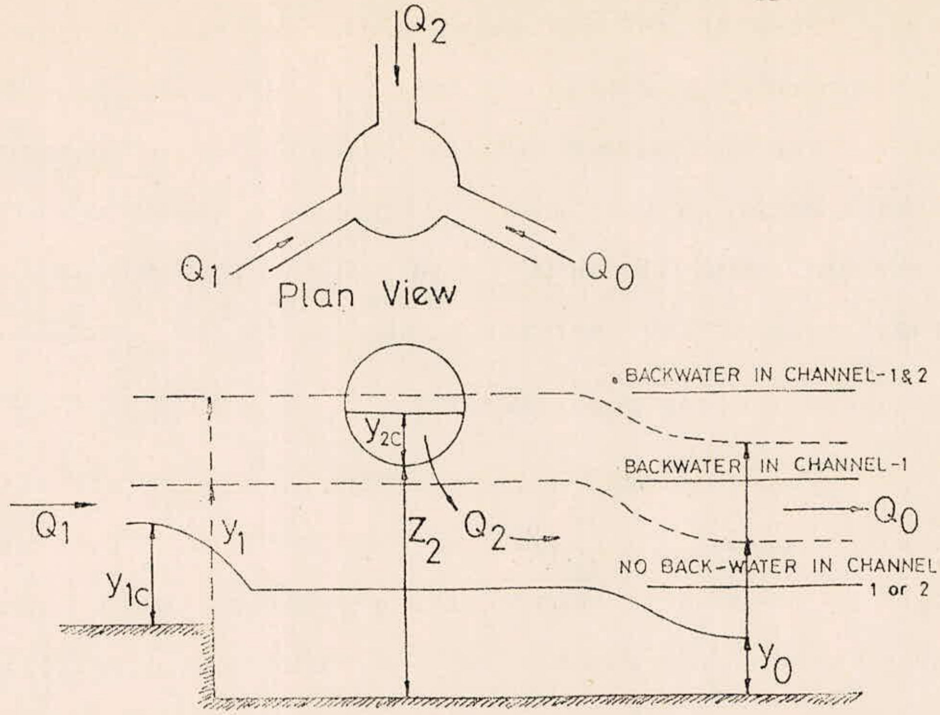

Fig. 5. Reservoir - type Junction

Since the kinetic energy of the inflows is assumed lost at the junction, for subcritical flow in the inflow channels,

$$Zi + Yi = H \quad \text{if} \quad Zi + Yic < H \qquad \ldots (11)$$

otherwise,

$$Yi = Yic \qquad \ldots (12)$$

If the flow in the outflow channel is super-critical, critical flow condition exists at its entrance and hence H in eq. 12 should be replaced by minimum specific energy corresponding to the instantaneous flow rate Qo.

As in the case of point-type junctions, supercritical flow in the inflowing channels discharges freely in to the reservoir-type junction provided the inflow is not sumberged by the back-water from the junction, abnd the discharge form the inflowing channels into the junction can be computed without considering the existing flow conditions in the junction or overflowing channel.

But the methodologies discussed previously for point type junction and Reservoir type junction (Figure 4 and 5) more relevant in urban drainage systems than in natural river systems. In natural situations the backwater effect is possible only in one channel and not in two channels simultaneously unless and until their outfall confluences with another large sized outfall/channel at a short distance. For example, the Sabari and its tributary Yerabagu experience backwater effect if and only if the main river Godavari is in spate. Otherwise only the tributary Yeravagu experiences backwater effect. The point that needs consideration in a case like this is that while assuming the initial conditions at the junction points, it is quite important to assess the initial conditions prevailing in the main river (e.g. Godavari as cited in the example), if the results of the routing studies are to be reasonably accurate.

Approximations to St. Venant's Equations:

Because St. Venant equations are rather complicated and it is not an easy task to obtain their solutions for unsteady river flows, various approximations to these equations have been proposed to provide simple but acceptable solutions. From a hydraulic view point, these approximations can be classified, according to the terms of the momentum equation considered, as quasi-steady dynamic wave, diffusion wave, and kinamatic wave approximations. If only the continuity equation is considered and the momentum equation is ignored, the approximation is a hydrologic routing model (e.g. see chow, 1964) and is not discussed here.

Kinamatic Wave Approximation;

The Kinamatic wave approximation is the simplest but also the least accurate model of the three approximations to the St. Venant equations. It retains only the two slope terms of the momentum equations and ignores the inertial and pressure terms, i.e.

$$So = S_f \qquad \ldots (13)$$

Equation 13 together with the continuity equation forms a non-linear kinamtic wave model, for which, except for special cases, solutions are obtained numerically. However, because of the simplification of the momentum equation, only one boundary condition is required instead of two, as in the case of the St. Venant equations. Since the downstream backwater effect can not be accounted for, the non-linear kinamatic wave model is unreliable for subcritical flow when the downstream back-water effect is important.

Diffusion Wave Approximation:

The next higher level approximation is the diffusion wave approximation, which incorporates the pressure term in addition to the two slope terms in the momentum equation, i.e.

$$\frac{y}{x} = S_o - S_f \qquad \ldots (14)$$

for gravity-oriented co-ordinates. The simplified momentum equation is combined with the continuity equation to form the non-linear diffusion wave model.

Inclusion of the pressure term in the diffusion wave model substantially improves the solution accuracy. It permits peak attenuation in addition to distortion and translation of the hydrograph. It provides a means of accounting for the downstream backwater effect , if any. However, to obtain a unique numerical solution, it requires that two boundary conditions be specified, as in the case of St. Venant equations. Therefore, it also requires simultaneous or iterative numerical solutions and is more complicated than the non-linear kinamatic wave approximation.

Quasi-Steady Dynamic Wave Approximation:

The quasi-Steady dynamic wave aproximation neglects only the local acceleration term and considers all other terms of the momentum equation. This simplified momentum-equation is coupled with the continuity equation to form the non-linear quasi-steady dynamic wave model. It accounts for the down-stream back-water effect and permits peak attenuation distortion and translation of the hydrograph. It also requires that two boundary conditions be specified for the solution to be unique. Its numerical solution procedure is nearly as complicated as that for the St. Venant equations.

Solution Scheme:

A non-linear flow routing models, each of which is formed by the continuity equation coupled with the momentum equation or its simplified form, are mathematically a set of first order quasi-linear hyperbolic partial differential equations. Usually, solutions can be obtained only numerically, with appropriately specified initial and boundary conditions. Many finite difference numerical schemes have been proposed to solve these equations. They can be classified in to the following three groups.

(a) Explicit Schemes:

The explicit schemes express the unknown parameters explicity as functions of known quantities, and solve them directly. They are relatively easy to understand, easy to formulate, easy to program, but they are also computationally highly efficient because of numerical instability problems. o minimize numerical instability, the computational grids are usually selected to satisfy the criterion,

$$\frac{\Delta x}{\Delta t} \geqslant V + (gA/B)^{\frac{1}{2}} \qquad \text{... (15)}$$

in which, x and t are computational space and time intervals respectively; other terms are as defined previously. Thus in order to ensure numerical stability, for a given reach

Δx, Δt are so small that the computation becomes very costly. Therefore, explicit schemes are only useful in flood routing of short duration events such as flash floods.

(b) Implicit Scheme:

The implicit schemes express the unknown parameters implicitly in simultaneous algebraic equations and then solve them using an appropriate solution technique. They are relatively much more difficult to formulate and program, but if done properly and carefully they can be computationally very efficient and stable. The finite difference computation grid sizes Δx and Δt can be chosen independently.

(c) Method of Characteristics:

The method of characteristics solved two sets of "Characteristics" equations, each set consisting of a pair of ordinary differential equations. These equations are transformed mathematically from the St. Venant equations. The characteristics equations are usually solved numerically using finite differences to approximate differentials. They may be expressed explicitly or implicitly using rectangular space-time grids or characteristics grids.

River Network Solution Technique:

There are four methods to facilitate solution of unsteady flow in river network as follows:

(a) Simultaneous solution technique:

When the diffusion wave, quasi-steady dynamic

wave (St. Venant equts.) models are applied to a river channel two boundary conditions in addition to the initial conditions are required to yield the unique solution. For a subcritical flow, one boundary condition shold be at the upstream end of the channel, whereas the other should be at the downstream end. The latter which provides through the junction the upstream boundary condition for the immediately following channel, is actually an unknown and a part of the solution being sought. In using the implicit numerical scheme, the flow in an interior channel can only be expressed in terms of the unknown boundary conditions at its upstream and downstream ends. This situation repeats until the last channel of the network is considered, for which the downstream boundary condition at the outlet of the network is specified. Thus, the solution can be obtained only by solving all of the flow equations at the channel and junctions simultaneously.
The simultaneous solution method is exact in the sense of solving all of the difference equtions for the entire network. But it is also very costly and requires a large digital computer when the network is large making the method impracticable.

One-sweep explicit solution method:

In this method, the unknown junction and channel flow parameters are expressed explicitly in terms of known quantities of the network at a previous time and of upstream points at the present time. Thus, simultaneous solution for the entire network is avoided. The solution proceeds starting from the upstream end towards the downstream end for the entire network for the present time step in sequence in a one-sweep manner before the computation advances to the next time step. This method bears the draw-back of explicit schemes concerning computational stability problems.

Overlapping segment method:

The overlapping segment method ; a single step, successive iteration technique is demonstrated schematically in Figure 6.

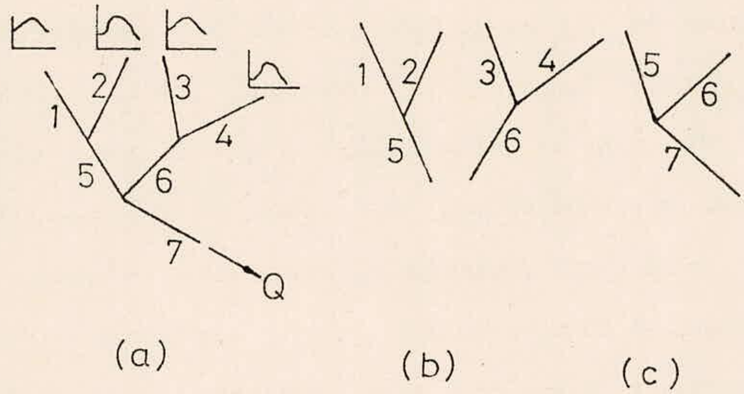

Fig. 6. Method of over-lapping segments

The river network is considered to be formed by a number of overlapping segments. Each segment is formed by a junction together with all the channels, each (interior) channel belongs to two segment - as a downstream channel for one segment and then as an upstream channel for the other segment, i.e. overlapped. Each segment is solved as a unit. The flow equations are first applied to each of the branches of the most upstream segment for which the upstream boundary condition is known, and solved numerically with appropriate junction equations. If the flow is subcritical and the boundary condition at the lower and of the downstream channel of the segment is unknown, the forward or backward differences, depending on the numerical scheme, are used as an approximate substitution. Simultaneous numerical solution is obtained for all the channels and junctions of the segment for each time step, repeating until the entire flow duration is completed. For example, for the network shown in Figure 6, solutions are first obtained for the two segments shown in Figure 6 (b). Since the downstream boundary condition of the segment is assumed, the solution for the downstream channel is discarded, whereas the solutions for the upstream channels are retained. The computation now proceeds to the next immediate downstream segment (e.g. the segment, shown in Figure 6 (c). The upstream channels of this new segment were the down stream channels of each of the proceeding segments for which solutions have already been obtained. The inflows into this new segment are given by the outflow from the junctions of the proceeding segments, with the inflows known, the solutions for this new segment can be obtained. This procedure is repeated successively, segment by segment, going downstream until the entire river network is solved. For the last (most downstream) segment of the network, the presented boundary condition at its downstream end is used.

The method of overlapping segments greatly reduces the necessary computer size and time requirements when solving for large river networks. It accounts for downstream backwater effect and simulates reversal flow, if it occurs. Its accuracy and practical usefulness have been demonstrated by Sevuk (1973) and Yen and Akan (1976).

Solution by the overlapping segment method accounts for the downstream backwater effects of subcritical flow only for the adjacent upstream channels of the junction, but can not reflect the backwater effect from the junction to channels farther upstream if such case occurs. However, by considering the length to depth ratio of natural river branches, the effect of backwater beyond the immediate upstream branches is small, and hence imposes problem in routing of river flows. For the rare case of two junctions being closely located, the overlapping segment method can be modified to include the short branch between the junctions as the internal branch and to use longer branches as the upstream and downstream channels. It should also be mentioned that the overlapping segment method, as well as the two previously described methods, can be modified to account for divided channels, i.e. loop networks in addition to tree-type networks.

Cascade Method:

In the non-linear kinematic wave approximation, only one boundary condition is required for routing the flow through a channel. Usually the upstream boundary condition is specified. Since the downstream boundary condition is not required and the downstream back-water effect is not accounted for the solution of the upstream channel, it is not effected by the downstream channel. At the junctions, only the continuity relationship is needed, and the dynamic condition is ignored. Consequently, solution of the flow can be obtained in a cascading manner, solving first for the most upstream channels individually over the entire flow hydrograph duration. Computation then proceeds and is repeated for the next downstream channel. Thus, the solution is obtained channel by channel individically and sequentially, moving downstream until the entire network is solved. No simultaneous or network iterative solution is required. This method is relatively simple and in-expensive, but it is in-accurate if the downstream backwater effect is important.

The cascading method can be used for diffusion wave, quasi-steady dynamic wave, and dynamic wave models without causing additional error if the flow is entirely in super-critical regime. However, such a case rarely occurs in natural rivers.

Using example networks, Yen and Akan (1976) compared the reliability of the methods of simultaneous solution, overlapping segments, and cascade. A typical example for the network shown in Figure 7 is reproduced in Figure 8, which shows the former two methods giving identical results, whereas the sequential cascading method gives significant errors.

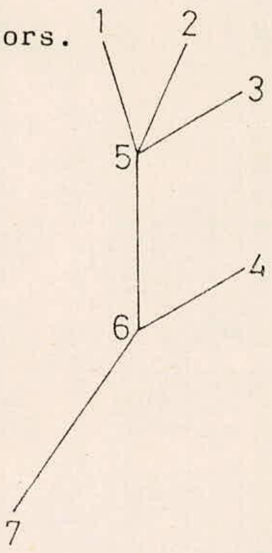

Fig. 7 Example Network

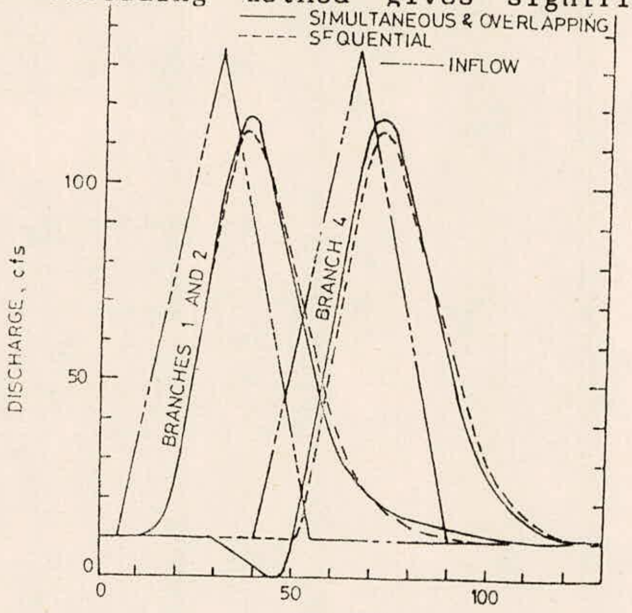

FIG. 8. OUTFLOW HYDROGRAPHS FOR BRANCHES 1, 2 & 4

There exists a number of appropriate models and numerical schemes for mathematical modeling. Which model and scheme is the best for a particular problem depends on the nature of the problem, the accuracy required, the computational facilities available, and the computational costs. There is no universally superior model or scheme. However, usually for gradually varied river flow, the implicit schemes and method of characteristics are superior to the explicit schemes, and the deffusion wave or dynamic wave models are preferred if the down-stream back-water effect is important, and the overlapping segment method if the river network is large. Looking at the importance of implicit scheme (Yen and Akan, 1976), it has been explained as below.

Four-point finite-difference-implicit scheme-

Eq. 2 can be re-written for the bed slope to be

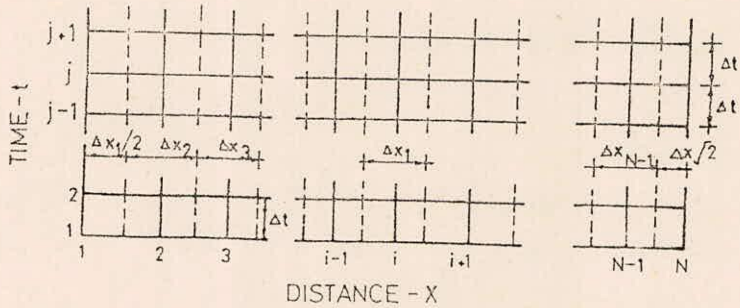

Fig. 9. Computational Grid for Four-Point Implicit Finite-Difference Scheme.

horizontal (i.e. So=0)

$$\frac{1}{gA}\frac{\partial Q}{\partial t} + \frac{1}{gA}\frac{\partial}{\partial x}\left(\beta \frac{Q^2}{A}\right) + \frac{\partial Y_s}{\partial x} = -S_f + \frac{1}{gA}\int_{\sigma} U_x . q . d\sigma \quad ...(16)$$

Now, with reference to the computational grid shown in Figure 9, the coefficients and p rtail differentials of Eqs. 1 & 16 are approximated by the following finite difference quotations.

$$F \approx \frac{1}{2}\left(Q\left(F_i^{j+1} \pm F_{i+1}^{j+1}\right) + (1-\theta)\left(F_i^j + F_{i+1}^j\right)\right) \quad ...(17)$$

$$\frac{\partial F}{\partial x} \approx 2\,\theta\left(\frac{F_{i+1}^{j+1} - F_i^{j+1}}{\Delta x_{i+1} + \Delta x_i}\right) + 2\,(1-\theta)\left(\frac{F_{i+1}^j - F_i^j}{x_{i+1} + x_i}\right) \quad ...(18)$$

$$\frac{\partial F}{\partial t} = \frac{1}{2\Delta t} (F_i^{j+1} + F_{i+1}^{j+1} - F_i^j - F_{i+1}^j) \quad ...(19)$$

in which,

F = any grid function;

ΔX_i = length of the flow reach measured in horizontal direction and centred at the i-th station;

Δt = constant time interval;

j = time stage of computation,

θ = weighting factor.

It can be noted that θ = 1 yields a forward-time implicit scheme used by Baltzer and Lai (1968) and θ = 0.5 produces a central implicit scheme used by Amein & Fang (1969). According to Fread (1974) the implicit scheme described by Equs. (17), (18) and (19) is unconditionally stable for $0.5 \leq \theta \leq 1$.

By assuming q=o and β=1, Substitution of Eqs. 17, 18 and 19 in to equt. 1 and 16 yields:

$$C_{i+0.5} \frac{A_i^{j+1} + A_{i+1}^{j+1} - A_i^j - A_{i+1}^j}{2\Delta t} + \frac{2\theta}{\Delta x_i + \Delta x_{i+1}} \quad ..(20)$$

$$(Q_{i+1}^{j+1} - Q_i^{j+1}) + \frac{2(1-\theta)}{\Delta x_i + \Delta x_{i+1}} (Q_{i+1}^j - Q_i^j) = o$$

and

$$M_{i+0.5} = \frac{1}{2\Delta t}(Q_i^{j+1} + Q_{i+1}^{j+1} - Q_i^j - Q_{i+1}^i) + \frac{20}{\Delta x_i + \Delta x_{i+1}} \left[\frac{(Q_{i+1}^{j+1})^2}{A_{i+1}^{j+1}} - \frac{(Q_1^{j+1})^2}{A_i^{j+1}} \right] + \frac{2\,(1-\theta)}{\Delta x_i + \Delta X_{i+1}} \left[\frac{(Q_{i+1}^j)^2}{A_{1+1}^j} - \frac{(Q_i^j)^2}{A_i^j} \right]$$

$$+ \overset{\theta}{Q} - \frac{g}{2}\,(A_{i+1}^{j+1} + A_i^{j+1}) \left[\frac{2(Y_{si+1}^{j+1} - Y_{si}^{j+1})}{\Delta x_i + \Delta x_{i+1}} + \frac{S_{fi+1}^{j} + S_{fi}^{j+1}}{2} \right]$$

$$+ (1-\theta)\ \frac{g\,(A_{i+1}^i + A_i^j)}{2} \left[\frac{2\,(Y_{si+1}^j - Y_{si}^j)}{\Delta x_1 + \Delta x_{i+1}} \right.$$

$$\left. + \frac{S_{f\ i+1}^j + S_{fi}^j}{2} \right] = 0 \qquad \text{... (21)}$$

where;

$C_{i+0.5}$ and $M_{i+0.5}$ represent respectively the finite difference approximate equations of continuity and momentum written for the channel reach between stations i and i+1. The unknown parameters in Eqs. 20 and 21 are Q_i^{j+1} , Q_{i+1}^{j+1} y_{si}^{j+1} and y_{si+1}^{j+1} . The quantities Q_i^j, Q_{i+1}^j, Y_{si}^j, Y_{si+1}^j are known either from previous computations or from the initial condition. The flow cross-sectional area A is a prescribed function of the water surface elevation Ys as determined by the geometry of the channel. The friction slope is computed by using Manning's formula as stated previously-

$$S_f = \frac{n^2\,Q^2}{2.21}\,A^{-2}\,R^{-4/3} \qquad \text{...(22)}$$

in which n is the roughness factor and R is the hydraulic radius. Both A & R are prescribed functions of Ys.

If overlapping segment scheme is used, the upstream condition is specified by the upstream inflow hydrograph which is obtained through previous computations for each branch. For fork-type junctions (i.e.. storage effect is negligible) with horizontal bottom Eq. 8 can be re-written as:

$$y_1 = y_2 = y_3 = y_o \qquad \text{... (23)}$$

The downstream boundary condition is the unknown junction kinamatic computability condition, Eq. 23, which also serves as the upstream boundary condition for the downstream branch of the overlapping segment. The downstream boundary condition of the downstream branch is usually unknown except for the last branch of the entire network, for which the outlet condition may be prescribed by the outflow stage-time relationship (such as flow into a reservoir) or the discharge-stage relationship (such as for a control section), or the discharge-time relationship (such as regulated outflow). As mentioned previously, Sevuk (1973) used backward differences applied to the momentum and continuity equations as the substitute for the downstream boundary condition and the last forward characteristic. The counter part of this substitution for the implicit finite difference scheme is to utilize the forward characteristic passing through the last grid point. However, because in the implicit scheme the space and time increments are selected independently and the time increment is usually much greater than that allowed by the Courant stability criterion, this forward characteristic starts at the preceding time level from upstream far beyond the last distance interval, making this substitution impractical.

After a number of trails, Yen and Akan (1976) found that approximating S_{fN} by linear or quadratic extrapolation form the immediate upstream grid stations provided reliable results with little computation represent the forward difference condition. They gave the downstream boundary condition for the downstream branch of the overlapping segment as:

$$S_{fN} = \left(1+\frac{\Delta x_N + \Delta x_{N-1}}{\Delta x_{N-1} + \Delta x_{N-2}}\right) S_{fN-1} - \left(\frac{\Delta x_N + \Delta x_{N-1}}{\Delta x_{N-1} + \Delta x_{N-2}}\right) S_{fN-2} \quad \ldots (24)$$

Eqs. 20 and 21 applied to each branch of an overlapping segment provide $\sum_{i=1}^{M} 2(N_i -1)$ equations where M is the total number of branches in the segment and N_i X is the number of grid points used in the -th branch, and there are $\sum_{i=1}^{M} 2 N_i$ unknowns. The additional 2 M conditions are provided by the prescribed upstream boundary conditions of the overlapping segment (M-1 inflow hydrographs), downstream boundary conditions for each branch (M equations from Eqs. 23 and 24) and the continuity equation at the junction (Eq. 6 for fork type junction). This set of $\sum_{i=1}^{M} 2N_i$ non-linear algebraic equations is solved symultaneously using the generalized Newton's iteration method.

Since the matrix of the equation coefficients does not process banded or other special properties, the Gaussian inversion technique is adopted.

4.0 DATA REQUIREMENT:

Data required for the analysis of the flow through junction are grouped as below:

(A) Cross-Section:

i. Location - Location of the section is specified in terms of river miles from some obligatory point (gauging station). Cross-sections should be positioned so as to best characterize the geometry of the anticipated flow paths.

ii. Alignment - Cross-Section alignment should always be perpendicular to the anticipated flow lines, which may require a dog-leg or curvilinear alignment.

iii. Cross-Sectionl Area - It is defined in terms of elevation for a cross-section, two width and an 'inactive' (or off-channel storage) width. It is presumed that flow through the active flow portion of a cross-section is normal to the plain of the cross-section with a velocity that can be appropriately represented with Manning's equation. Only the active flow portion of a cross-section is considered in defining terms in the momentum equation.

The in-active portion of a cross-section is intended to account for an area where water ponds does not have a significant velocity component in the direction of flow. Characteristics of the total cross-section, active plus inactive are reflected in terms of the continuity equation. Figure (10) ilustrates representation of a cross-section with elevation and widths that has both active and inactive areas.

Off-channel storage, for example on a tributary, are modelled by locating three cross-sections as shown in Figure (11), and developing off-channel storage widths to reflect storage in the tributary. The widths are calculated for the middle cross-section as follows:

$$BSS = \frac{2\ CSM}{L} \qquad \ldots (25)$$

in which;

BSS = Off-channel storage width in m for middle at elevation E-m.

SA = Surface area of off-channel storage in square M at elevation E-m,

L = Distance in m between first and third cross-sections.

(iv) Spacing - Theoretically, the distance between the cross-sections should be equal to the distance travelled by the flood wave during a computation interval. Because both the flood wave velocity and the computation interval vary during a simulation, the theoritical criteria can only be approximately satisfied. Rela-

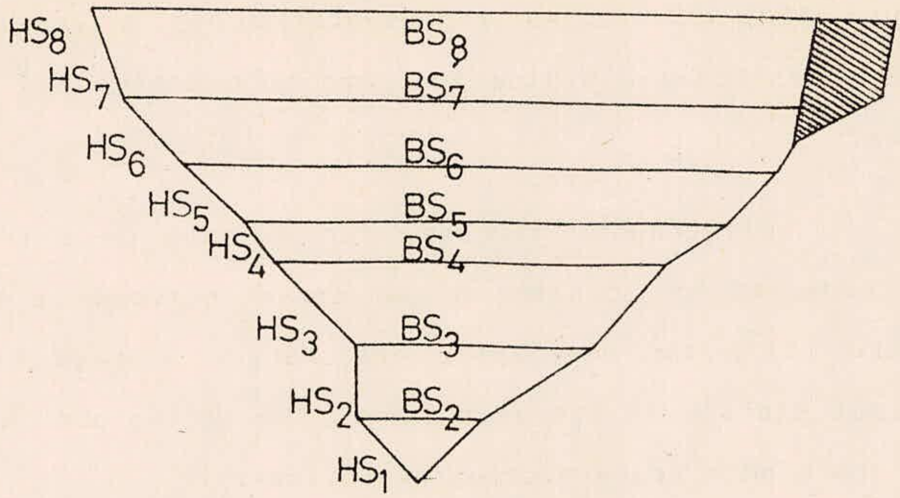

Fig. 10. Cross-Section Representation

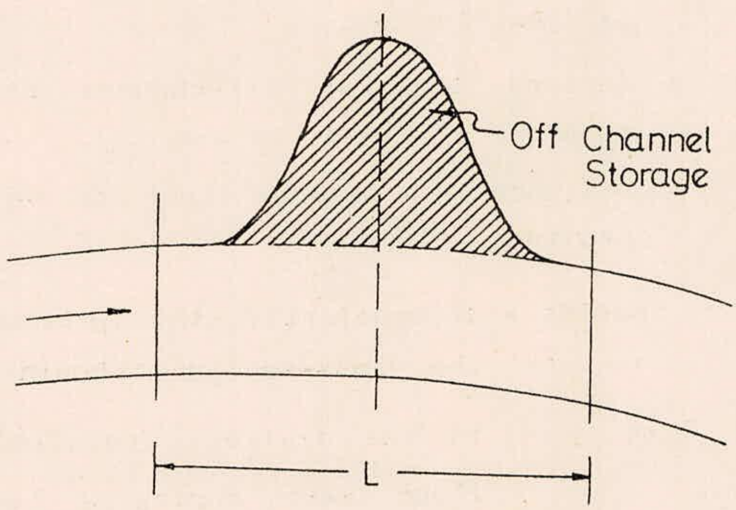

Fig. 11. Off-Channel Storage (Plan View)

tively short distance steps are required near the junction and steps can be lengthened with increasing distance from the junction in either direction.

(B) Roughness Coefficient:

(i) Application: Boundary resistance is reflected in the equation of motion through the friction slope, which is defined with Manning's equation. Friction slope is determined for a reach in terms of an arithmatic average of the hydraulic radii for the 'effective flow portions of cross-sections at each end of the reach. It is assumed that wetted perimeter is equal to the effective water surface width. This assumption results in negligible error if the width to depth ratio for effective flow is greater than a value of about 10. For narrow, deep cross-sections, the wetted perimeter assumption should be accommodated by employing appropriately larger n-values.

In case the roughness data are not available, the same can be calculated approximately if the median sediment size is known. Hence in absence of roughness coefficient data, median sediment size should be known at all cross sections' along the reach.

(ii) Composite roughness: Mannings' roughness coefficients (n-values) should be specified for reaches containing two or more cross-sections as a set of composite n-values which vary with elevation or discharge. A composite n-value is an equivalent n-value associated with the entire (effective) wetted perimeter of a cross-section. Typically, n-values should be specified with a lateral variation across the cross-section as shown in Figure (12).

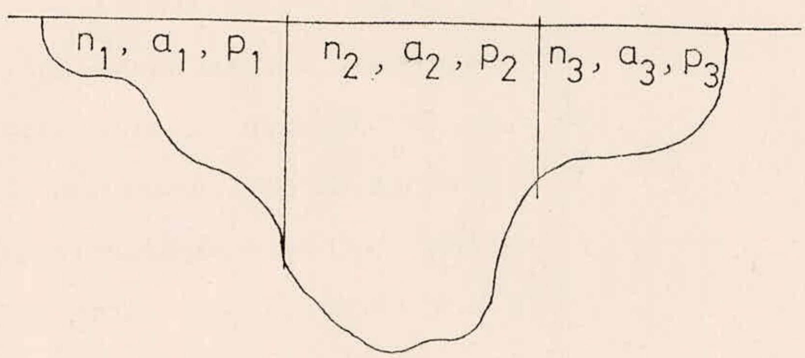

Fig. 12. Lateral Variation of n - Values Across a Cross-Section.

(C) Type of Flow:

The flow is characterized as to be sub-critical, critical or super-critical. If Froude number is less than 1, the flow is said to be in sub-critical regime and equal to 1 then the flow is critical and if greater than 1 the flow is in super-critical regime.

(D) Stage-Discharge Relation:

Rating surveys at the gauging stations just upstream and downstream of the junctions should be available. Gauging stations should be so located as they best represent the flows characteristics.

Generally, the gauging sites are located far upstream of the junction. In this case the ratings at just u/s of the junction can be calculated by routing process.

(E) Bed Slope of the Channels:

Bed slopes of all of the tributaries joining and of main river should be known. For alluvial rivers where silting and erosion takes place, a mean slope of the channels can be taken. In addition to this the bed slope at the junction should be known.

(F) Velocity Distribution of the Flow:

Velocity distribution of the flow w.r.t. depth should be known. As the flow velocity is changing with depth of flow, this will affect the momentum equation. The velocity distribution for tributary as well as main river should be known.

(G) Map of he Basin:

The basin map is required for acquiring knowledge of plan view of the junction and also to know the angle of inter-section of the tributary flow with the main river flow.

5.0 REMARKS

Yen and Akan (1976) in their paper presented a hypothetical study to demonstrate the validity of applying the overlapping segment method for routing of floods through fork-type junctions. Thy applied all the three methods discussed previously and made following remarks.

(i) Theoritically, the overlapping segment method would fail when the junction effect propagates significantly beyond the immediate upstream branches. From the examples tested it appears that such case would rarely occur in field conditions. Although how far upstream the backwater effect from a junction is felt depends on the network geometry as well as on the flood characteristics and required accuracy, some rough indication can be obtained from steady flow backwater effects. For instance, if at the upstream end of a branch the steady flow depth with backwater from the junction at its downstream end is 10% greater than the normal depth, most likely in unsteady flow routing the corresponding junction effect to the preceding overlapping segment would be much less than 10%.

(ii) The sequential method which is oftenly used, can produce erroneous results for sewer flows. However, for river networks with relatively wide and nearly rectangular channels, the error in discharge is not exceedingly large and in fact under favourable conditions the sequential method may be useful because of its relative simplicity, provided that simulation of the reduced or reversal flow such as that at the d/s of branch 3 (Figure 6) is not important. Better approximation by the sequential method for river networks than for sewer networks is due mainly to the fact that most sewers have circular cross-section for which the depth changes rapidly at low discharges and that there usually is a discontinuity in invert or crown of the joining sewers at the junction. However, even for river networks with favourable conditions, the sequential method can not give reliable depth at the downstream end of the branches. Only the computed depth at thier upstream end can be considered as approximately acceptable.

They further concluded that the overlapping segment method can be used to route unsteady flow through fork-type dendritic networks of non-prismatic channels using a four-point finite diff. implicit scheme. This method requires much less computer time than simultaneous solution of the flow equations for all the branches of the network, and it is considerably more accurate than the sequential method which neglects the downstream back-water effect. The overlapping segment method simulates the flood flow faithfully and its result agrees well with that by the simultaneous solution method. The sequential method produces less erroneous results for river networks than for sewer networks because the former usually have wider channel cross-sections and don't have the bottom discontinuity as the latter. However, the sequential method should not be used when the downstream backwater effect is important and when reverse flow occurs.

REFERENCES

1. Akan, A.O. and Yen, B.C. (1981), "Diffusion Wave Flood Routing in Channel Networks", JL. of Hyd. Div., ASCE, Vol. 107, No. 6, pp. 719-732.

2. Amein, M. and Chu, H.L. (1975), "Implicit Numerical Modelling of Unsteady Flows", Jl. of Hyd. Div., ASCE Vol. 101, Hy 6, pp. 717-732.

3. Amein, M. and Fang, C.S. (1969),"Streamflow Routing with applications to North Carolina Rivers",Water Resources Res. Inst. Rep. No. 17, Univ. of North Carolina.

4. Amein, M. and Fang, C.S. (1970), "Implicit Numerical Routing in Natural Channels", Jl. of Hyd. Div,A.S.C.E. Vol. 96, No. HY 12, pp 2481-2500.

5. Baltzer, R.A. and Lai, C. (1968), "Computer Simulation of Unsteady Flow in Waterways", Jl. of Hyd. Div., ASCE, Vol. 94, NO. HY4, pp. 1083-1117.

6. Chow, V.T. (1959), "Open Channel Hydraulics", Mc. Graw-Hill Book Co., New York.

7. Cooke, R.V. and Reeves, R.W. (1976), "Arroyas and Environmental Change", in the American South West, Clerendon Press, Oxford.

8. Ellis, J. (1970), "Unsteady Flow in Channel of variable Cross Section", Jl. of Hyd. Div., ASCE, Vol. 96, No. HY 10, pp, 1927-1945.

9. Fread, D.L. (1972), "Dynamic Flood Routing in Rivers with Major Tributaries", Paper pres. to the section of Hydrology, AGU Spring National Meeting, Washington, D.C., April 17-21, P.29.

10. Fread, D.L. (1974), "Numerical Properties of Implicit Four-Point Finite Difference Equations of Unsteady

Flow", NOAA Tech. Memo. NWS HYDRO-18, U.S. National Weather Service, Silver Spring, Md.

11. Garrison, J.M. Granju, J.P.P. and Price, J.T.(1969), "Unsteady Flow Simulation in Rivers and Reservoirs", Jl. of Hyd. Div., ASCE, Vol. 95, No. HY5, pp. 1559-1576.

12. Gregory, K.J. (1976 b), "Drainage Networks and Climate" In Derbyshire, E. (Ed.), Climate and Land Forms, Willy, London, pp. 289-315.

13. Grushevskiy, M.S. and Fedorov, N.N. (1965), "Field Investigations of Unsteady Flow in Natural Channels and First Results of the Analysis of the Material Obtained", Proc. 11th Congress IAHR, Leningrad, Paper 3.22, p.15.

14. Henderson, F.A. (1966), "Open Channel Flow", Mac Millian Co. New York.

15. Henry, R.F. (1972), 'Simulation of Long Waves in Branching Water Ways", Jl. of the Hyd. Div., ABCE, Vol. 98, No. HY 4, Proc. Paper, 8831, April, pp. 609-629.

16. Joliffe, I.B. (1984), "Computation of Dynamic Waves in Channel Networks", Vol. 110, No. 10, Jl. of Hyd. Engg. , ASCE, pp-1358-70.

17. Lai, C. (1986), "Numerical Modelling of Unsteady Open Channel Flow", Advances in Hydro-sciences, ed. V.T. Chow, Vol. 14, Academic Press, pp. 162-323.

18. Li, Z.C., Zhan, L.J. and Wang, H.L. (1983), "Difference Methods of Flow in Branch Channel", Jl. of Hydr. Engg. ASCE, Vol. 109, No. 3, pp. 424-446.

19. Meijer, Th. J.G.P., Vreugdenhi, C.B., and De Vries, M, (1965), "A Method of Computation for Non-stationary Flow in open Channel Networks", Proc. 11th Congress IAHR, Leningrad, Paper 3.28.

20. Pinkayan, S. (1972), "Routing Storm Water through a Drainage System", Jl. of the Hyd. Div., ASCE, Vol. 98, No. HY 1, Jan., pp. 123-135.

21. Price, R.K. (1974), "Comparison of Four Numerical Methods for Flood Routing" Jl. of Hyd. Div. ASCE, Vol. 100, HY 7, Proc. Paper 10659, pp. 879-899.

22. Quinn, F.H. and Wylie, E.B. (1972), "Transient Analysis of the Detroit River by the Implicit Method", Water Res. Research, Vol. 8, No. 6, pp-1461-1469.

23. Ramana Murthy, K.V. (1965), "Electronic Analogue for Flood Routing Based on Muskingum Storage Equation", Jl. of CBIP, Vol. 4, p. 443.

24. Ramana Murthy, K.V. (1967), "A Direct Discrete Analogue for Flood Routing", Jl. of the Institution of Engineers (India), Electronics and Telecommunication Engineering Division, NO. 5, p. 66.

25. Ramana Murthy, K.V., Gupta, P.K., Kshirsagar, M.M. and Tongaonkar, S.A. (1989), "Electronic Analogue Computer for Flood Forecasting for Tapi Basin", Jl. of the Institution of Engineers (India), Civil Engineering Div. Vol. 70, pp. 5-15.

26. Sevuk, A.S. (1973), "Unsteady Flow in Sewer Networks, Ph. D. Thesis, Deptt. of Civil Engg. Univ. of Illinois at Urbano-Champaign, III.

27. Sevuk, A.S. and Yen, B.C. (1973a), "A comparative study on Flood Routing Computation", Proc. IAHR, Internat. Symp. on River Mechanics, Bangkok, Thailand, Vol. 3, pp. 275-290.

28. Sevuk, A.S. and Yen, B.C. (1973), "A comparative study on Flood Routing Computation", IAHR 9-12 Jan, Bangkok, Thailand.

29. Sevuk, A.S. and Yen, B.C. (1973), "Comparison of Four approaches in Routing Flood wave through Junction",

Proc. 15th Congress, IAHR, Istanbul, Vol. 5, pp. 169-172.

30. Sevuk, A.S. and Yen, B.C. (1982), "Sewer Network Routing by Dynamic Wave Characteristics", Jl. of Hyd. Div. Vol. 108, No. HY-3, pp. 379-398.

31. Simons, D.B. and Gessler, J. (1971), "Research needs in Fluvial Process", In Shen, H.W. (Ed.), River Mechanics, Vol. II, Colorado State Univ., Fort Collins, pp. 33-1 to 33-13.

32. Soliman, M.M. (1976), "Flood Control Arrangements For River Junctions", Vol. II, Third Annual Symposium of the Waterways, Harbours and Coastal Engg. Div. of ASCE, Colorado State Univ. Fort Collins, Colo.

33. Stoker, J.J. (1957), "Water Waves", Interscience Publishing Co., New York.

34. Surkan, A.J. (1974), "Simulation of Storm Velocity Effects on Flow from Distributed Channel Networks", WRR, 10, pp. 1149-1160.

35. Taylor, E.H. (1944), "Flow Characteristics at Rectangular Open-channel Junctions", Trans. ASCE, Vol. 109, pp. 893-903.

36. Tsuchiya, A. and Takaheshi, A. (1969), "The Analysis of Unsteady Flow in Rivers by an Analog Computer", Proc. 13th Cong. IAHR, Kyoto, Vol. 1, pp. 439-446, Vol. 5-2, pp. 111-112, 123-125.

37. Wood, Enc. F., Harley, Brendan, M., and Perkins, Frank, G. (1975), "Transient Flow Routing in Channel Networks", Res. Rep. RR-75-1, Inter, Inst. for App. Sys. Analysis, Scloss Laxenburg, Austria, January.

38. Yen, B.C. (1975), "Further Study on Open-Channel Flow Equations", Sonder-forschung-sbereich 80 publication No. SFB 80/T/49 Uni. of Karlsruhe, Karlsruhe, West Germany.

39. Yen, B.C. and Akan, A.O. (1976), "Flood Routing through River Junction", River '76, Third Annual Symposium of the Waterways, Harbours and Coastal Engineering Div. of ASCE, Colarado State Univ., For Collins, Colo.

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