FLOOD ROUTING

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(Literature review)

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TABLE OF CONTENTS

1. INTRODUCTION

A common problem facing engineers and hydrologists is predicting the rise and fall of a river at a particular point during a flood event. The problem can be solved by the technique of flood routing (Shaw, 1994). The purpose of flood routing in most engineering work is to learn what stages or rates of flow occur, without actually measuring them, at specific locations in streams or structures during passages of floods. The stages or rates may be used in evaluating or designing a water-control structure or project (Styner, 1972).

As described by Smithers and Caldecott (1995), determining the degree of lagging and attenuation of a wave traversing a reservoir or reach is important when any of the following operations are performed: Design of dam spillways, design of retarding or detention basins in urban areas, flow forecasting and warning systems especially for floods, evaluation of flood reducing or protection schemes. The routing process shows how a flood wave may be reduced in magnitude and attenuated by the use of storage in the reach between the upstream and downstream points (Wilson, 1969).

As explained by Styner (1972), the routing need not be only downstream because the process can be reversed for upstream routing, which is often done to determine upstream hydrographs from gauged downstream hydrographs. Nor is routing confined to streams and rivers; it is regularly used in obtaining inflow or out flow hydrographs, mass curves, or peak flow rates in reservoirs, farm ponds, tanks, swamps, and lakes. low flows are routed, as well as floods.

The term "flood routing" covers all of the above practices. In the subsequent chapters the catchment morphology and its effect on flood routing, the methods of routing in reservoir, and river reaches, and the procedure of determining Muskingum parameters of routing are discussed.

2. CATCHMENT GEOMORPHOLOGY

It is appropriate to consider how various properties of the catchments area affect the rate and quantity of discharge from it. A few descriptions, which seem to have special relevance in hydrology, are discussed in the following paragraphs.

2.1 Physical Descriptors of Catchment Form

If basin forms and hydrologic characteristics are to be related, the basin form must be represented by quantitative descriptors (Linsley *et al.* 1982). Many such measures have been proposed; only limited progress has been made in relating the physical and hydrologic features for a number of reasons (Wilson, 1969**)**.

Because of different land use activities and different catchment characteristics, flood generated from catchments differ in its quantity as well as flow stage, hence relating the physical characteristics of a catchment with the flood generated from it is important to come out with a reliable flood estimate(Wilson, 1969).

2.1.1 Stream order

A classification of stream order used as a measure of the amount of branching with in a basin. A first order stream is a small-unbranched tributary; a second order stream has only first order tributaries; a third order stream has only first and second order tributaries. Using this system the order of the main stream is the highest (Horton, 1945).

To determine completely the composition of a stream it is necessary to know, drainage area, A, the orders of the main stream, the bifurcation ratio, the stream length ratio and the length of the main stream or preferably the average length of the first order streams (Horton, 1945). The bifurcation ratios with in a basin tend to be about the same magnitude (Linsley *et al.*1982). Generally, bifurcation ratios are found to be between 2 and 4 with a mean value near 3.5. This observation led to the law of stream numbers.

$N_u = R_b^{\ k-u}$ Eq 2.1

Where: -

N_u is the number of streams of order u, R_b is the bifurcation ratio, and k is the order of the main stream. This shows that the bifurcation ratio is equal to the number of streams of the next to the highest order for the given drainage basin (Horton, 1945)

The stream length ratio can be obtained by dividing the average stream length of any order by the average stream length of the next lower order,.

$L_u = L_1 * R_e^{u-1}$

Eq.2.2

Where L is the average length of streams of order u and R_e is the length ratio. An equivalent equation also applies to the average area (A) of basins of order u.

$A = A_1\ R_e^{u-1}$ Eq.2.3

The relationships have been confirmed under a wide range of conditions (Linslay *et al.*1982). The above equations can be used by measuring stream number N, stream length L, and catchment area A, for the two highest orders in the basin and then estimating these values for all lower orders. Some geomorphologists favor reversing the number system so that the main stream is order one and tributaries of increasingly higher order. Both approaches are arbitrary, but Horton's system seems to be the most widely used procedure (Linsley *et al.*1982)

2.1.2 Drainage densities

The total length of streams within a catchment divided by the drainage area, defines the drainage density, the length of channels per unit area. A higher drainage density reflects a highly dissected basin, which should respond relatively rapidly to a rainfall input, while low drainage density reflects a poorly drained basin with slow hydrologic responses. Low drainage densities are observed where soil materials are resistant to erosion or very permeable and where the relief is small. High values may be expected where soils are easily eroded or relatively impermeable, slops are steep, and vegetal cover is scanty (Linsley *et al.* 1982). Horton (1945) express drainage density as :-

$$D_d = \frac{\sum L}{A}$$

where:-

D_d is drainage density or average length of streams per unit area, $\sum L$ is the total length of streams and A is the area, both in units of the same system. The poorly drained basin has a drainage density of 2.74, the well-drained one, 0.73, or one fourth as great (Horton, 1945).

2.1.3 Length of overland flow

The average length of overland flow L_0 may be approximated by

$$L_0 = \frac{1}{2} * D \qquad \text{Eq. 2.4}$$

Where D is the drainage density, this approximation ignores the effect of ground and channel slope, which make the actual overland path longer than the estimate. The error is probably of little significance. Horton (1945) suggested that the denominator be multiplied by $\sqrt{1 - S_c / S_g}$ where S_c and S_g are the average channel and ground slopes, respectively.

2.1.4 Area relation

The topographic water divide usually, but not necessarily, bound a catchment area. Because of the underlying geology it is perfectly possible for areas beyond the divide to contribute to the catchments. The true boundary is indeterminate (Wilson, 1969).

Data for a number of the larger rivers of the world seems to conform to the equation.

$$L = 1.27 A^{0.6} \qquad \text{Eq. 2.5}$$

Where L is main channel length (km) and A is drainage area (km^2) the coefficient becomes 1.4 with dimensions in mile.

2.1.5 Basin shapes,

The shape of a catchment affects the stream flow hydrograph and peak–flow rates (Linsley *et al.*1982). The effect of shape can best be demonstrated by considering the hydrographs of discharge from three differently shaped catchments of the same area subjected to rainfall of the same intensity (Wilson, 1969).

2.2 Descriptors of Catchment Relief

The topography of relief of a basin may have more influence on its hydrologic response than catchment shape, and various authors have advanced numerous descriptors of relief, some of the more useful descriptors are discussed in this section.

2.2.1 Channel slope

The slope of a channel affects velocity of flow and must play a role in hydrograph shape. Commonly, only the main stream is considered in describing the channel slope of a catchment. Taylor and Schwarz (1952, cited by Linsley *et al.* 1969) calculated the slope of a uniform channel having the same length and time of flow as the main channel. Since the velocity is proportional to the square root of slope, the procedure used by Taylor and Schwarz is equivalent to weighting channel segments by the square root of their slope, which gives relatively less weight to the steep upstream reaches of the stream. Thus, if the channel were dividing into n equal segments, each of slope s_i, a simple index of slope would be

$$R_s = \left(\frac{\sum_{i=1}^{1=n} \sqrt{s_i}}{n} \right)^2 \qquad \text{Eq.2.6}$$

Where: - R_s=slope index

2.2.3 Land slope

The more steeply the ground surface is sloping the more rapidly will surface runoff travel, so that concentration time will be shorter and flood peaks higher. Infiltration capacity tends to be lower as slopes get steeper, since vegetation is less dense and soil more easily eroded, thus accentuating runoff (Wilson, 1969). The distribution of land–surface slope can be determining by establishing a grid or set of randomly located points over a map of the catchment. The slope of a short segment of line normal to the contour is determined at each grid intersection or random point. The frequency distribution of these numbers may be plotted (Linsley *et al.* 1982).

2.2.4 Area–elevation data

Generally, precipitation increases with altitude though individual catchments show wide variation from the general rule (Wilson, 1969). When one or more of the factors of interest in a hydrologic study vary with elevation, it is useful to know how the catchment area is distributed with elevation. An area-elevation (or hypsometric) curve can be constructed by planimetering the area between contours on a topographic map and plotting the cumulative area above (or below) a given versus that elevation. In some cases it is convenient to use percentage of area instead of actual area, particularly if a comparison between basins is desired. If a grid with about 100 or more intersection is in each elevation range is noted, an area-elevation curve can be constructed which will be about as accurate as one derived by planimetry but requires less effort (Linsley *et al.* 1982).

2.2.5 Aspect

Aspect of a slope is the direction toward which the slope faces. Orientation is important with respect to the meteorology of the area in which the catchment lies (Wilson, 1969). Precipitation amounts are often influenced by the aspect of a slope relative to the direction of the wind. Customarily, aspect is used as a characteristic of a particular point or at most a specific hillside. The distribution of aspect may be determined in manner similar to that described for land slope (Linsley *et al.*1969).

2.3 Hydraulic Geometry

Hydraulic geometry describes the character of the channel of basin: the variation of mean depth, top width, and velocity at a particular cross section and between cross sections. These relations apply to alluvial channels, where the cross section is readily adapted to the flows, which occur, but are less reliable, where rock out crops controls the channel characteristics.

2.4 Stream Pattern

The pattern of stream development in a catchment can have a marked effect on the rate of runoff. A well-drained catchment will have comparatively short time of concentration and hence a steeper flood-rise hydrograph than a catchment with many surface depressions marshy ground and minor lakes (Wilson, 1969).

When viewed in plan, stream channels may be described as meandering, braided or strait. The important question for the engineering hydrologist is to explain why a channel adopts one of the patterns described above. Both braided and meandering can be explained as means of energy dissipation (Linsley *et al.* 1982).

2.5 Other Factors

Other factors which influence the rate and total quantity of surface runoff and so affect the magnitude of flood peaks and hydrographs shape are (Wilson, 1969):-

- Condition of stream channels: clean or weedy and over grown.
- The presence of reservoirs, lakes flood plains, swamps, etc.
- Land use: for example arable land, grassland or forest; artificial drainage.
- Subsurface conditions including initial soil moisture, height of the phreatic surface, depth and permeability of aquifers and infiltration capacities.

3. FLOOD ROUTING

Flood routing methods may be divided into two main categories differing in their fundamental approaches to the problem. One category of methods uses the principle of continuity and a relationship between discharge and the temporary storage of excess volumes of water during the flood period. The second category of methods favored by hydraulicians, adopts the more rigorous equations of motion for unsteady flow in open channel. The choice of methods depends very much on the nature of the problem and the data available (Shaw, 1994)

3.1 Flood Routing Techniques

Reservoirs have the characteristic that their storage is closely related to their outflow rate. In reservoir routing methods the storage-discharge relation is used for repeatedly solving the continuity equation, each solution being a step in delineating the outflow hydrograph (Caldecott, 1989).

For a given time interval, the volume of inflow minus the volume of outflow equals the change in volume of storage. The equation is often written in the simple form:

$$\frac{\Delta S}{\Delta t} = \text{I-O} \qquad \text{Eq. 3.1}$$

Where:-

ΔS=change in storage during time interval (m^3), Δt,

I= average inflow during Δt in m^3/s, and

O=average outflow during Δt in m^3/s.

For more convenient solution may be rewritten as

$$(\frac{I_n + I_{n+1}}{2})\ \Delta t\ -(\frac{O_n + O_{n+1}}{2})\ \Delta t\ \ = S_{n+1} - S_n$$

Eq3.2

where:-

S_n = channel or temporary storage (m^3) at time increments equal to n

I_n = inflow rate ($m^3.s^{-1}$)at time increment equal to n

O_n= outflow rate ($m^3.s^{-1}$) at time increment equal to n

Δt=routing period (seconds)

Where subscripts *(n)* and *(n+*1*)* refer to the number of increments in time interval, Δt. It is assumed that the mean of the flows at the beginning and end of a time period are equal to the average flow in the time period. At the beginning of any time step, all values are known except for $O_{(n+1)}$ and $S_{(n+1)}$. Thus with two unknowns, a second equation is needed to solve for $O_{(n+1)}$ at the end of the time step (Smithers and Caldecott, 1995).

Eq 3.2 may be transformed in to form

$$\frac{2S_2}{\Delta t} + O_2 = I_1 + I_2 + \frac{2S_1}{\Delta t} - O_1 \qquad \text{Eq. 3.3}$$

3.1.1 Reservoir routing

According to Viessman *et al.* (1989, cited by Smithers and Caldecott 1995) For a reservoir or river reach where a natural or in artificial structure forms an approximately level pool, it can be assumed that the temporary storage (S_n) is uniquely related to the head (H) of water over the crest of the control section water stored in a reservoir is released as pipe flow through turbines or outlet works and during flood events, over an emergency spillway, can described for gate uncontrolled ogee spillway by

$$O_s = C_d\ W_s\ H^{(3/2)} \qquad \text{Eq. 3.4}$$

Where:-

O_s=flow rate ($m^3.s^{-1}$) over an uncontrolled spillway.

C_d = coefficient of discharge, input by the user (Theoretically =3.0)

W = width (m) of spillway, input by the user.

H=depth (m) of storage above spillway crest.

Storage values for various pool elevations may be computed from topographic maps. Since storage and outflow both depend only on water elevation above full supply level, the resulting storage–elevation curve and outflow-elevation curve can be combined to from storage–outflow graph (Smithers and Caldecott, 1995).

The two methods of reservoir routing are mass-curve method and storage-indication methods. The mass curve method can be applied numerically or graphically. The storage indication method is a widely used method for both reservoir and channel routings (Styner, 1972).

3.1.1.1 Storage indication method

Where the terms on the right of equation 3.3, S_2 and O_2, are unknown and can be solved by employing a second relationship between storage and outflow. The storage outflow relationship is derived, by plotting $2S/\Delta t + O$ versus O, given the elevation storage relationship and the elevation discharge relationship for the reservoir. For each successive routing period, Δt, a value of $\frac{2S}{\Delta t} + O_2$ may be computed from Equation, 3.3, since all the terms on the left of the equation are known. The corresponding outflow value, O_2, may then be read off from the storage–outflow relationship. The routing period Δt, must be selected to be short relative to the curvature of the hydrograph so that the assumption that the change in flow rate (I and O) are linear over time interval Δt is an acceptable approximation. A reasonable practical compromise between accuracy and computational efficiency is to select Δt to be fall within the range of a 1/4 to 1/8 of the time to peak discharge (Smithers and Caldecott, 1995).

In channel routing the Storage-Indication method has the defect that outflow begins at the same time inflow begins so that presumably the in-flow at the head of the reach passes instantaneously through the reach, regardless of its length. Styner (1972) explained that, this defect is not serious if the ratio T_t/T_p is about l/2 or less, where T_p is the inflow hydrograph time to peak and T_t is a travel time defined as:-

$$\mathbf{T_t} = LA/3600*q = L/3600*V \qquad \text{Eq.3.5}$$

Where:-

T_t = reach travel time in hours; the time it takes a selected steady-flow discharge to pass through the reach

L = reach length in feet.

A = average end-area for discharge q in square m

q = selected steady-flow discharge is cumecs

$V = q/A$ average velocity of discharge q in m/s

In determining T_t the discharge q is usually the bank-full discharge under steady flow conditions (Styner, 1972).

3.1.1.1.2 Linear reservoirs

If the storage: outflow relationship is found to be linear, and the slope of the storage: outflow graph is defined as K_1, then

$$S=K_1O \qquad \text{Eq.3.6}$$

And the reservoir is called linear reservoir. Flow routing through a linear reservoir is accomplished by first dividing time into a series of number of equal increments and then substituting $S_{(n+1)}=K_1O_{(n+1)}$ into Equation, 3.3, and solving for $O_{(n+1)}$, which is the only remaining unknown for each time increment.

$$\frac{I_n + I_{n+1}}{2})\Delta t - (\frac{O_n + O_{n+1}}{2})\Delta t = K_1 O_{n+1} - S_n$$

Rearranging similar variables the equation becomes:-

$$O_{(n+1)} = \frac{0.5(I_n + I_{n+1})\Delta t - 0.5\Delta t O_n + S_n}{(K_1 + 0.5\Delta t)} \qquad \text{Eq.3.7}$$

3.1.1.1.3 Non-linear reservoir

To route a hydrograph through a non –linear reservoir, the storage: outflow relationship and the continuity equation are combined to determine the outflow and storage at the end of every time step. Equation, 3.3 can be rewritten as

$$\frac{2S_{n+1}}{\Delta t} + O_{n+1} = I_n + I_{n+1} + \frac{2S_n}{\Delta t} - O_n \qquad \text{Eq. 3.8}$$

In which the only unknown for any time increments the term on the right hand side of the equation. Rather than solving equation–by trial and error procedure, a value of Δt is selected and a storage indication curve is used to aid the analysis. Storage for various depths of outflow above the spillway level (full supply level) may be determined from topographic maps. The relationship between storage and depth of outflow is calculated by hydraulic computation such as given in Equation, 3.4. Thus O_{n+1} can be determined from the storage indication curve once a value of ($2Sn+1/\Delta t + O_{n+1}$) has been determined from equation, and the solution of the outflow hydrograph can proceed in a step wise manner.

3.1.2 River reach routing

Flow routing in natural river channels is complicated by the fact that storage is not a function of out flow alone. Routing in streams requires a storage relationship, which adequately represents the wedge storage (Smithers and Caldecott, 1995).

Muskingum method is a commonly use hydrologic routing method for handling a variable discharge-storage relationship (Chow *et al.* 1988). This method models the storage volume of flooding in a river channel by a combination of wedge and prism storages. During the recession, out flow exceeds inflow, resulting a negative wedge. In addition, there is a prism of storage, which is formed by a volume of constant cross section along the length of prismatic channel. Assuming that the cross-sectional area of the flood flow is directly proportional to the discharge at the section, the volume of prism storage is equal to KO where K is a proportionality coefficient, and the volume of wedge storage is equal to KX (I-O), where X is a weighting factor having the range $0 \leq X \leq 0.5$ (Chow *et al.* 1988). Then total storage is expressed as:-

$$S = K[XI + (1 - X)O] \qquad \text{Eq. 3.9}$$

Equation 3.9 represents a linear model for routing flow in streams.

The value of X depends on the shape of the modelled wedge storage. The value of X ranges from 0 for a reservoir type storage to 0.5 for a full edge. When X=0, there is no wedge and hence no backwater: this is the case for reservoir model, S=KO. In natural Streams, X is between 0 and 0.3 with a mean value near 0.2. Great accuracy in determining X may not be necessary because the results of the method are relatively insensitive to the value of this parameter. The parameter K is the time of travel of the flood wave through the channel reach. For hydrologic routing, the values of K and X are assumed to be specified and constant throughout the range of flow. The storage change in the river reach during the routing interval from Equation 3.10 is expressed as:

$$S_2 - S_1 = K[X(I_1 - I_2) + (1 - X)(O_2 - O_1)] = 0.5(I_1 + I_2)\Delta t - 0.5(O_1 + O_2)\Delta t$$

Eq.3.10

If C_0, C_1, C_2 are substituted in the equation.

$O_2 = I_1 C_0 + I_2 C_1 + O_1 C_2$

Where:

$$C_0 = \frac{-KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad \text{Eq.3.11}$$

$$C_1 = \frac{KX + 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad \text{Eq.3.12}$$

$$C_2 = \frac{K - KX - 0.5\Delta t}{K - KX + 0.5\Delta t} \qquad \text{Eq.3.13}$$

Where: the sum of, $C_0 + C_1 + C_2 = 1$

The theoretical stability of the numerical method is accomplished if,

$$2KX \le \Delta t < 2K(1 - X) \qquad \text{Eq.3.14}$$

3.2 Estimation of Muskingum K and X Parameters

The accuracy as well as the success of applying the Muskingum method relies on the accurate estimation of parameters relating channel storage volume to inflow and outflow rates (Shaw, 1988; Wilson and Ruffin, 1988, cited by Smithers and Caldecott, 1995). A number of methods may be used and are dependent on the availability of observed data with which the parameters may be calculated.

3.2.1 Reverse routing procedure

If observed inflow and outflow data are available for the reach, a reverse routing procedure may be implemented which solves Eq.3.9, for K by assuming different values of X (Chow *et al.* 1988). Since S_n and $(XI_n+(1-X)O_n)$ are assumed to be linearly related via Eq 3.9, the accepted value of X will be that which gives the best linear plot (narrowest loop). The value of K is then determined as the reciprocal of the slope through the narrowest plot.

3.2.2 Muskingum-Cunge method

Equation, 3.15- is used in the physically based Muskingum-Cunge method to estimate the X parameter:

$$X = \frac{1}{2} * \frac{(1-q_0)}{S_0 c_w \Delta L} \qquad \text{Eq. 3.15}$$

where:-

q_0 = reference discharge per unit channel width ($m^3.s^{-1}.m^{-1}$)

S_0 = dimensionless channel bottom slope ($m.m^{-1}$)

c_w = wave celerity ($m.s^{-1}$) ~ $c_{cs}V_{av}$

c_{cs} = kinematic wave velocity constant, depending on channel shape (Table 3.1)

V_{av} = average velocity ($m.s^{-1}$) and

ΔL = routing reach length (m).

The average velocity (V_{av}) may be calculated from a reference flow rate and a typical channel area. The approximate wave velocity for different channel shapes may then be estimated from the information contained in Table 3.1 (Viessman *et al.* 1989, cited by Smithers and Caldecott, 1995).

A dilemma in implementing Equation 3.15 is deciding on the reference flow rate to be used in the Calculation of the average velocity. Wilson and Ruffin (1988, cited by Smithers and Caldecott, 1995) suggest using Eq3.161 to calculate the representative flow rate.

$$O_m = q_{bm} + 1/2(q_{pm} - q_{bm}) \qquad \text{Eq. 3.16}$$

Where:-

O_m= representative flow rate ($m^3.s^{-1}$)

q_m = reference flow rate ($m^3.s^{-1}$) in the Muskingum-Cunge method

q_{pm} = peak flow rate ($m^3.s^{-1}$) in the Muskingum-Cunge method and

q_{bm} = base flow rate ($m^3.s^{-1}$) in the Muskingum-Cunge method.

Table 3.1, Kinematic wave velocities for various channel shapes (after Viessman *et al* 1989, cited by Smithers and Caldecott, 1995).

Channel shape	Manning equation	Chezy equation
Wide rectangular	5/3 V_{av}	3/2V_{av}
Triangular	4/3V_{av}	5/4V_{av}
Wide parabolic	11/9V_{av}	7/6V_{av}

In the absence of sufficient information to calculate X using Equation 3.2. or of not having available a better estimate for X, a typical value of 0.2 may be used for a river reach. The wave travel time (and hence K) down a reach of known length may be estimated using Equation 3.17:-

$$K=\Delta l/c_w \qquad \text{Eq. 3.17}$$

The wave celerity (c_w) may be estimated from a typical flow rate and an average channel cross-sectional area, or may be estimated by using the Manning formula to calculate the average velocity (V_{av}), which is then multiplied by an appropriate value of c_{cs} from Table3.2.2.1.

$$V=\frac{1}{n}R^{2/3}S_0^{0.5} \qquad \text{Eq. 3.18}$$

where:-

v = velocity (m.s-1)

n = Manning roughness coefficient (Table 3.2)

R = hydraulic radius (m) and

S_0 = dimensionless channel bottom slope ($m.m^{-1}$).

Table 3.2 Typical values of Manning's roughness coefficient, n (After Chow, 1959)

Type of channel and description		N(maximum value)
Minor natural streams (Top width at flood stage <30m)	Clean, straight stream	0.025-0.030
	Clean, winding stream	0.033-0.045
	Winding with weeds and pools	0.035-0.050
	With heavy bush and timber	0.075-0.150
Major natural streams (Top width at flood stage >30m)	Regular section, no boulders or bush	0.025-0.060
	Irregular and rough section	0.035-0.100

The hydraulic radius (R) is calculated as

$R=A/p$ Eq. 3.19

Where:-

A = cross-sectional area of channel (m^2) and

P = wetted perimeter of the channel cross-section (m)

4. DISCUSSION AND CONCLUSION

Hydrological flood routing techniques are widely accepted and have been extensively used in engineering practice. Some of the advantages and limitations of hydrologic methods are discussed below: -

Advantages of hydrologic flood routing method includes: -

- A detailed knowledge of the geometrical characteristics of the river reach is not required i.e. does not require data to define the complex geometric and characteristic of a river reach (Caldecott, 1989).

- More efficient in terms of computer operating time and storage (Caldecott, 1989).

The limitations of hydrologic methods includes:-

- The hydrologic method is in general simpler but fails to give entirely satisfactory results in problems other than that of determining the progress of a flood down a long river. For example, when a flood comes through a junction, backwater is usually produced. When a dam regulates a flood, surges are generally evolved. The backwater effect and the effect of surges in these problems can be accurately evaluated only by the basic hydraulic equations employed in the hydraulic method, but not by the hydrologic method (Chow, 1959).

- Flow data are not readily available.

- The data’s are historical.

- In channel routing the Storage-Indication method has the defect that outflow begins at the same time inflow begins so that presumably the in-flow at the head of the reach passes instantaneously through the reach, regardless of its length. This defect is not serious if the ratio T_t /T_p is about l/2 or less, where T_p is the inflow hydrograph time to peak and T_t is a travel time (Styner, 1972).

- Another limitation of the Storage-Indication method, for both channel and reservoir routing, is that there is no rule for selecting the proper size of routing interval. Trial routings show that negative outflows will occur during recession periods of outflow whenever Δt is greater than 2 * S_2/O_2 (or whenever $O_2/2$ is greater than $S_2/\Delta t$). This also means that rising portions of hydrographs are being distorted. In practice, to avoid these possibilities, the working curve can be plotted and; if any part of the working curve falls above the line of equal values then the entire curve should be discarded and made a new one using a smaller value of Δt. For channel routing the possibility of negative outflows is usually excluded by taking Δt less than T_t (Styner, 1972). A reasonable practical compromise between accuracy and computational efficiency is to select Δt to be fall within the range of a 1/4 to 1/8 of the time to peak discharge (Smithers and Caldecott, 1995).

- In the Muskingun method of flood routing technique, invariable relationships between stage and discharge at the ends of the routing reach are assumed, i.e. The K and X parameters are assumed to be the same throughout the reach and flow is assumed to change slowly with time. The effect of abnormal surface slope and changing channel storage are neglected together with factors such as kinetic energy and acceleration energy.

The catchment form and relief affect low or rapid rainfall response of a catchment, water harvest volume from a catchment, rainfall type and amount, the amount of water contributed from a catchment. Hence representing a catchment in its quantitative descriptors and relating with the rate of flood helps to come up with a reliable and full knowledge of incoming flood. In general, good knowledge of flood volume and discharge increase the awareness of decision makers to take appropriate flood protection measures, building flood detention dams with efficient flood release mechanism, and build cost effective structures.

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