LECTURE _ 3

FLOOD ROUTING BY MUSKINGUM METHOD

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OBJECTIVE

The objective of the lecture is to introduce the participants with the various aspects involved in the application of Muskingum method, one of the most widely used method of flood routing. The latest developments and éxperiences of various researchers are also discussed. This will be useful in indegious use of the method and help in providing better understanding of limitations of the method.

1.0 INTRODUCTION

As man continues to build on the flood plain, an increased understanding of the nature of floods remains an ever present challange. For Hydraulic engineers, an assessment of the characteristics of flood waves is the logical starting point at the planning and design stages of flood control works. This lecture presents an overview of flood propagation phenomena in open channels. Some fundamental concepts on the nature of flood waves are introduced. This leads naturally into flood routing and a classification of flood routing methods. Further the lecture introduces general aspects of hydrologic flood routing methods. The conventional Muskingum method which is most commonly used in practice is discussed in detail.

2.0 NATURE OF FLOOD WAVE

It is perhaps an oddity that the nature of the flood waves is most readily grasped by looking at the system rather than at the flood wave itself. The system could be either a channel, a reservoir (or lake), or a channel-reservoir combination. This distinction is of fundamental importance, as will be shown here. The channel case is usually associated with the existence of a finite (nonzero) water surface slope. The reservoir case is normally taken to imply a zero water surface slope.

Flood waves travelling downstream in a channel or reservoir, in general, are subject to attenuation. The rate of travel (flood wave velocity) and the rate of attenuation depend on the system in which the flood waves are moving.

2.1 Flood waves in Stream Channels

The attenuation rate of waves is a function of the magnitude of the various forces involved in the motion. Kinamatic waves do not attenuate, deffusive waves attenuate at a small to moderate rate, and dynaamic waves are subject to very strong attenuation. Strictly speaking, St. Venant equations are valid only for flood waves which do not attenuate, i.e. kinamatic waves. However, it can also be used as an approximation for flood waves subject to moderate attenuation, i.e. diffusion waves.

2.2 Flood waves in Lakes and Reservoirs

Flood waves in lakes and reservoirs travel at an infinite velocity i.e., there is an instantaneous response (outflow hydrograph) to the excitation (inflow hydrograph). However, the system exerts a diffusive effect on the flood wave with the result that the peak of the outflow hydrograph is attenuated and delayed. A significant characteristics of flood routing through reservoirs is that when the inflow and outflow coincide, the outflow is a maximum (Fig. 1).

3.0 FLOOD ROUTING

Although flood waves appear to have well-defined properties, in practice it is often necessary to carry-out elaborate calculations in order to determine these properties. The reason for this is the variability of the natural environment, menifested in the need to handle large amount of data. Flood routing is defined as the process of tracking by calculating the movement of a flood wave. Chow (1964) defined 'Flood Routing' as the procedure whereby the time and magnitude of flood wave at a point on a stream is determined from the known or assumed data at one or more points upstream. The calculations can proceed along one of the following two lines, either by considering only temporal variations (lumped case), or by considering both temporal and spatial variations (distributed case). The lumped case is the classical reservoir routing situation, formulated in terms of one first order ordinary differential equation (the starage equation). The distributed case corresponds to stream channel routing, formulated in terms of two first order partial differential equations. The latter are commonly referred to as the equations of gradually varied unsteady open channel flow, or also, as the St. Venant equations.

4.0 CLASSIFICATION OF FLOOD ROUTING METHODS

A knowledge of the nature of flood waves provides a good basis on which to develop a classification of flood routing methods. The most general classification is that of (1) reservoir (or lake) routing and (2) stream channel routing. The essential difference betwween these two is that in the reservoir routing the water surface slope is zero; while in stream channel routing it is a non-zero value. Several other criteria could be used to classify flood routing methods. Among these, the following are readily identified: (1) the equations used to formulate the problem; (2) the overall approach to data collection and (3) the approach to obtain a solution.

4.1 Classification Based on Equations Used

According to the equations used to formulate the problem, flood routing methods can be classified as (1) Mass-balance methods and (2) Mass-and-momentum-balance-methods. The mass-balance methods use the ordinary differential equation of storage plus an auxiliary storage-outflow relationship. The mass- and momentum-balance methods use the Saint Venant equations or appropriate simplifications. The use of the complete St. Venant equations leads to the dynamic wave, while the simplified forms lead to kinamatic and diffusive waves.

4.2 Classification Based on Approach to Data Collection

According to the approach to data collection, flood routing methods can be classified as : (1) hydrologic, in which the parameter estimation is based on hydrologic observations for individual reaches; and (2) hydraulic, in which the parameter estimation is based on actual measurements of channel characteristics at individual cross-sections.

4.3 Classification Based on the Solution Technique

According to the approach based on the solution technique, flood routing methods can be classified as: (1) analytical and (2) numerical. The analytical methods are based on the solution of differntial equations specified on a continuous domain of space and time. The numerical methods are based on the algebraic equations on a discrete domain. Analytical solutions use the tools of classical mathematics such as linear analysis and Laplace transforms, while numerical solutions use characteristics or finite difference methods.

Table-1 provides a summary of the classification of flood routing methods presented herein.

TABLE-1 CLASSIFICATION OF FLOOD ROUTING METHODS

Based on equations used	)	Mass-balance : Storage equation and an auxiliary storage-outflow relationship
	)	Mass-and-momentum-balance : Saint Venant equations (dynamic wave) or appropriate simplifications (kinamatic and diffusive waves)
Based on Approach to Data Collection	)	Hydrologic Routing: observations for channel reaches.
	)	Hydraulic Routing: Measurements of channel characteristics at individual X-sections.
Base on Solutoion Technique	)	Analytical Routing: differential equations; continuous domain.
		Numerical routing: Algebraic equations; discrete domain.

5.0 HYDROLOLOGIC FLOOD ROUTING METHODS

The hydrologic routing method of routing can be broadly classified as (1) storage routing method; . and (2) complete linearized method and its simplifications. As complete linearized models are not very much used in practice only storage routing methods have been dealt with. The storage routing method may deal with linear, quasi-linear and non-linear flood routing problems. In linear routing the parameters of the model are kept constant throughout the routing operation. Examples are the conventional Muskingum flood routing method, Lag and route method etc. In quasi-linear routing some or all the parameters of the model change from one time step to another.

5.1 Storage Routing Models

All the storage routing models are based on the continuity equation in the lumped form which can be written for a channel reach as:

$$\frac{dS}{dt} = I(t)-Q(t) \qquad ..(1)$$

where, I(t) and Q(t) are inflow and outflow respectively, and S(t); the storage in the reach under study at time 't'. Since there are two unknowns viz. Q(t) and S(t) and only one equation, the solution for Q(t) can not be obtained. In order to eliminate one of the unknowns, expression for storage S(t) in terms of I(t) and $\dot{Q}(t)$ or Q(t) is used. The storage equation may be linear or non-linear in form. The following are the commonly used forms of storage equations in flood routing.

$$S(t) = K\ Q(t) \qquad ..(2)$$

$$S(t) = K\ Q(t+\tau\ ) \qquad ..(3)$$

$$S(t) = K\ [\ XI(t) + (1-X)\ Q] \qquad ..(4)$$

$$S(t) = a_0\ Q(t) + a_1 dQ(t)/dt + a_2\ d\ Q(t)/dt \qquad ..(5)$$

$$S(t) = a_0\ Q(t) + a_1 dQ(t)/dt + b_1 I(t) \qquad ..(6)$$

$$S(t) = a_0\ Q(t) + a_1 dQ(t)/dt + b_1 I(t) + b_2 dI(t)/dt \qquad ..(7)$$

$$S(t) = K\ (Q(t))^m \qquad ..(8)$$

For the sake of brevity the time functions attached with the notations for inflow, outflow and storages would be dropped here afterwards. Equation (2) represents the storage of a single linear reservoir (SLR) model proposed by Zoch (1934). Using a series of n-SLRs Nash (1957) conceptualised the catchment behaviour for a unit impulse input and derived the Instantaneous hydrograph (IUH) for the catchment. Dooge (1973) pointed out the same can also be used for modelling the flood in a river reach. Equation (3) forms the basis of the Lag and Route model proposed by Meyer (1941). It relates the outflow of time ($t+\tau$) to the storage at time 't'. The term represents the response delay time or the time taken for the leading edge of the flood wave to reach the outflow section. Equation (4) forms the basis of the classical Muskingum flood routing method proposed by McCarthy (1938). Equation (5) to (8) were studied by Kulandaiswamy et. al. (1957) as particular cases of general storage routing model applied to route floods in channels and river reaches. Equation

(8) represents the non-linear relationship between storage and discharge and it has been employed by Rockwood (1958), and Mein et. al (1974) for channel routing.

6.0 CONVENTIONAL MUSKINGUM METHOD

The Muskingum method falls under the category of hydrologic methods based upon storage routing approach. The Muskingum method of flood routing was developed in 1930s in connection with the design of flood protection schemes in the Muskingum River Basin, Ohio. It is the most widely used method of hydrologic stream channel routing, with numerous applications throughout the world.

The Muskingum method is based on the differential equation of storage as below;

$$I - Q = dS/dt \qquad ..(1)$$

The method involves the concept of wedge and prism storages (Fig. 2). Storage volume can be correctly related to outflow with a simple linear function only when inflow and outflow are equal, that is, when a steady flow exists. During the advance of a flood wave, however, inflow always exceeds outflow, thus producing a wedge of storage, called wedge storage. Conversely, during the recession, outflow exceeds inflow, resulting in a negative wedge storage. The wedge can be related to the difference between the instantaneous values of inflow and outflow. In Fig. 2 the wedge storage is represented by $KX(I-Q)$. In addition, there is a storage of prism, or prism storage, as represented by KQ. The total storage is therefore

$$S = K[XI + (1-X)Q)] \qquad .(4)$$

in which S = storage volume, I = inflow, Q = outflow, K = a time constant or storage coefficient, and X a dimensionless weighting factor. with inflow and outflow in cubic metre per second, K in hours, storage volume in (cubic metre per second)-hour, K could be expressed in seconds, in which case storage volume is in cubic metres.

Equation (4) was developed in 1938 and has been widely used since then. It is essentially a generalisation of the linear reservoir concept. In fact, for X=0, Equation(4) reduces to Eq. (2). In other words, linear reservoir routing is a special case of Muskingum channel routing for which X=0.

To derive the Muskingum routing equation, Eq. (1) is discretised on the xt plane (Fig. 3) to yield Eq. (9) repeated here:

$$\frac{I_1 + I_2}{2} - \frac{Q_1 + Q_2}{2} = \frac{S_2 - S_1}{\Delta t} \quad ..(9)$$

Eq. (4) is expressed at time levels 1 and 2 :

$$S_1 = K[XI_1 + (1-X)Q_1)] \quad ..(10)$$

$$S_2 = K[XI_2 + (1-X)Q_2)] \quad ..(11)$$

Substituting Eqs. (10) and (11) into Eq. (9) and solving for Q yields Eq. (12) as below :

$$Q_2 = C_1 I_1 + C_2 I_2 + C_3 Q_1 \quad ..(12)$$

in which C_1 , C_2 and C_3 are routing coefficients defined in terms of t, K and X as follows:

$$C_1 = \frac{(\Delta t/K) - 2X}{2(1-X) + (\Delta t/K)} \quad ..(13)$$

$$C_2 = \frac{(\Delta t/K) + 2X}{2(1-X) + (\Delta t/K)} \quad ..(14)$$

$$C_3 = \frac{2(1-X) - (\Delta t/K)}{2(1-X) + (\Delta t/K)} \quad ..(15)$$

Since $(C_1 + C_2 + C_3) = 1$, the routing coefficients can be interpreted as weighting coefficients.

Given an inflow hydrograph, an initial flow condition, a chosen time interval Δt, and routing parameters K and X, the routing coefficients can be calculated with Eqs. (13) to (15) and the outflow hydrograph, with Eq. (12). The routing parameters K and X are related to flow and channel characteristics, K being interpreted as the travel time of the flood wave from upstream end to downstream end of the channel reach. Therefore, K accounts for translation (or concentration) portion of routing (Fig.4).

The parameter X accounts for the storage portion of the routing. For a given flood event, there is a value of X for which the storage in the calculated outflow hydrograph matches that of the measured outflow hydrograph. The effect of storage is to reduce the peak flow and spread the hydrograph in time (Fig. 4). Therefore, it is often used interchangeably with the term diffusion and peak attenuation.

The routing parameter K is a function of the flow and channel characteristics that cause runoff diffusion. In the Muskingum method, X is interpreted as a weighting factor and restricted in the range 0.0 to 0.5. Values of X greater than 0.5 produce hydrograph amplification (i.e. negative diffusion), which does not correspond with reality. With K= Δt and X =0.0, Muskingum routing reduces to linear reservoir routing.

In the Muskingum method, parameters K and X are determined by calibration using stream flow records. Simultaneous inflow-outflow discharge measurements for a given channel reach are coupled with a trial-and-error procedure, leading to the determination of K and X. The procedure is time-consuming and lacks predictive capability. Values of K and X determined in this way are valid only for the given reach and flood event used in the calibration. Extrapolation to other reaches or to other flood events (of different magnuitude) within the same reach is usually unwarranted.

When sufficient data are available, a calibration can be performed for several flood events, each of different magnitude, to cover a wide range of flood levels. In this way, K is more sensitive to flood level than X. A sketch of the variation of K with stage and discharge is shown in Fig. 5.

Unlike reservoir routing, stream channel-routing calculations exhibit a definite (time) lag between inflow and outflow. Furthermore, in the general case (X not equal to zero) maximum outflow does not occur at the time when inflow and outflow coincide (Fig. 4).

6.1 Accounting of Lateral Inflow

One of the factors which affects the accuracy of flood routing is the magnitude of lateral inflows to the channel. If lateral inflows to the channel are ignored in routing the flows from upstream to downstream of the reach, then the computed flows at the downstream point would be smaller than the observed flows. The errors propagate as floods are routed down the stream. Consequently, estimating lateral inflows would improve the routing accuracy. Estimation of lateral inflows has not received much attention. In fact, the development of different flood routing techniques have been investigated more than the lateral inflow estimation. Nevertheless attempts have been made to estimate the lateral inflow in a meaningful manner. In HEC-1 flood hydrograph pasckage (HEC, 1981) the lateral inflow is estimated based on the supplied pattern of lateral inflow hydrograph and the volume difference between the outflow and inflow hydrograph of the given reach. To determine the lateral inflow contribution in a more rational manner, Mimikov and Rao (1976) have proposed a model in which the lateral inflow volume of each storm is distributed over time equal to storm duration in a manner similar to the time distribution of rainfall of the storm within the intervening catchment of the reach. They have applied their model for daily flow and daily storm and therefore the application of such a concept to the estimated lateral inflow based on hourly data is questionable due to its dynamic nature than that of daily data. Slocum and Dandekar (1975) have estimated the lateral inflow hydrograph by subtracting the outflow hydrograph estimated using different K and X values of the Muskingum Method, from the observed outflow hydrograph. The lateral inflow hydrograph which does not contain even a single negative ordinate has been considered as the correct one. Using Muskingum-Cunge model, Price (1973) has taken into account the lateral inflow assuming a uniform contribution along the channel reach. However, such assumptions are not valid for the urban storm sewer system rather than to natural river system.

6.2 Parameters Estimation

It can be seen from Equations (13), (14) and (15) that the constants C_1, C_2 and C_3 are functions of two parameters viz. K and X. These parameters can be estimated by :

a) Graphical method,
b) Method of moments,
c) Method of Optimisation

using inflow(I) and outflow(Q) hydrograph for the reach under consideration. It is important to note that before going to the parameters estimation it is necessary to correct the inflow and/or outflow hydrographs for lateral inflow so as to make them equal volumetrically.

a) Graphical Method

The computational steps are as follows:

(i) Compute I-Q and subsequently $(I_1 + I_2)/2 - (Q_1 + Q_2)/2$ by taking average of two consecutive values of (I-Q)

(ii) Compute change in storage (ΔS)

(iii) Compute storage (S) incrementing the storage at one time step earlier by change in storage;

(iv) Assume trial value of X within the range of 0.0 to 0.5 and compute the values of [XI+(1-X)Q];

(v) Plot storage (S) versus [(XI+(1-X)Q] for diferent values of X;

(vi) From the above referred plots, identify the one which is closer to straight line and then the slope of the straight line provides an estimate of the parameter K and the value of X corresponding to that plot is the required value of the parameter X. Typical plots between storage and XI+(1-X)Q (weighted discharge for three different values of X viz. X=0.3, 0.2, and 0.1) are shown in Fig. 6. From the Figure it is observed that the plot corresponding to X=0.1 shows the narrowest loop in comparison to the other two plots.

b) Method of Moments

The computational steps are given as below:

(i) Subtract the base flow from the inflow hydrograph and outflow hydrograph in order to compute direct inflow hydrograph (Q);

(ii) Compute normalised first and second moments of direct inflow hydrograph about the origin using:

$$XM1O = \sum_{i=1}^{m} \frac{I_1 + I_2}{2} t \Big/ \sum_{i=1}^{m} \frac{I_1 + I_2}{2} \quad ..(16)$$

$$XM2O = \sum_{i=1}^{m} \frac{I_1 + I_2}{2} t^2 \Big/ \sum_{i=1}^{m} \frac{I_1 + I_2}{2} \quad ..(17)$$

where, m is the number of ordinates of direct inflow hydrograph

(iii) compute normalised first and second moments of outflow hydrograph using :

$$YM1O = \sum_{i=1}^{n} \frac{Q_1 + Q_2}{2} t \Big/ \sum_{i=1}^{n} \frac{Q_1 + Q_2}{2} \quad ..(18)$$

$$YM2O = \sum_{i=1}^{n} \frac{Q_1 + Q_2}{2} t^2 \Big/ \sum_{i=1}^{n} \frac{Q_1 + Q_2}{2} \quad ..(19)$$

Where, n is the number of outflow hydrograph ordinates;

(iv) Compute the normalised second moments of direct inflow and outflow hydrographs about the centroid using the following equations:

$$XM2C = XM2O - (XM1O)^2 \quad ..(20)$$

$$YM2C = YM2O - (YM1O)^2 \quad ..(21)$$

(v) Estimate the parameters using following relationships for the moments:

$$K = YM1O - XM1O \quad ..(22)$$

$$X = 0.5 * [\, 1 - (YM2C - XM2C)/ K^2 \,] \quad ..(23)$$

c) Method of Optimisation

The direct optimization method (Stephenson, 1979; O'Donnell, 1985) is based on directly determining the coefficients C_1 , C_2

and C in Eq. 12 without first estimating K and X in Eq.s 13, 14 an 15. This would involve minimizing the error function in comparing observed hydrograph with computed hydrograph. Again, the error function can be defined in a least squares sense or differently. There are, in fact, only two unknowns, since the third can be obtained from

$$C_1 + C_2 + C_3 = 1 \qquad ..(24)$$

Let us consider C_2 and C_3 to be the unknowns. then the the Eq. 12 can be reduces to

$$Q_2 - I_2 = C_2 (I_2 - I_1) + C_3 (I_2 - Q_1) \qquad ..(4a)$$

The coefficients C_2 and C_3 can be derived by comparing flows of a known inflow hydrograph with flows of the corresponding outflow hydrograph. This comparison can be done in a least square sense or differently. By employing the least squares-type error function, the C_2 and C_3 can be estimated as below :

Let us define

$$R = Q_2 - I_2 \qquad ..(25)$$

$$F = I_2 - I_1 \qquad ..(26)$$

$$G = I_2 - Q_1 \qquad ..(27)$$

Eq. 4a can be rewritten as

$$R = C_2 F + C_3 G. \qquad ..(28)$$

which is identical in form to Eq. 12 with $C_1 = 0.0$. Therefore $_2$

$$C_2 = \frac{\Sigma R_o F \Sigma G - \Sigma R \Sigma G_o \Sigma FG}{DET} \qquad ..(29)$$

$$C_3 = \frac{\Sigma R_o G \Sigma F^2 - \Sigma R_o F \Sigma FG}{DET} \quad ..(30)$$

Where

$$DET = \Sigma F^2 \Sigma G^2 - (\Sigma FG)^2$$

and R is observed R. Then the C can be computed from Eq. 24. further, a little algebraic manipulation shows that

$$X = \frac{C_2 + 0.5C_3 - 0.5}{C_2 + C_3} \quad ..(34)$$

$$K = \frac{\Delta t(C_2 + C_3)}{1 - C_3} \quad ..(35)$$

Thus K and X can be objectively determined. The value of X may take any value ranging from negative infinite to 0.5. Though, physically the value of X should lie within the range of zero to 0.5 but it has been established that mathematically the negative values are correct.The amount of computation involved is quite small and can be undertaken on a desk calculator.

6.2.1 Comparison of the parameter estimation techniques

Singh (1980) presented a brief of the works carried-out by different hydrologists on comparison of techniques utilised for estimating the Muskingum parameters. It is stated that all the methods are comparable on the whole. Any of the method, depending upon the availability of the computational facilities, can be utilised.

6.3 Routing Procedure

After estimating the parameters K and X, the observed direct inflow hydrograph can be routed through the channel using Eq. (12) as given below:

(i) Compute routing coefficients C_1, C_2 and C_3 using Equations (13),(14) and (15) respectively;

(ii) Compute the direct outflow hydrograph ordinates Qi using the recursive form of relationship given as in Eq. (12) for known initial outflow which is used as initial condition. the initial outflow is assumed to be equal to the initial inflow.

(iii) Add the base flow to the computed direct outflow hydrograph to obtain outflow hydrograph.

In addition to this following points should be taken into account before the application of the Muskingum routing process:

(1) The reach selected should be uniform and prismatic. If the variation in the cross-sections of the upstream end, middle and downstream end, is significant, the reach may be sub-divided in such a way so as to satisfy the above condition up to the extent it can, as the routing accuracy depends upon the selection of the reach.

(2) From the map of the reach or basin it can be identified whether there is any major tributory joining the main stream reach. if so, the reach may be divided in to sub-reaches in a manner to incorporate the lateral inflow with the inflow of the reach downstream of the tributary junction without any serious error. If there are several small tributaries joining the reach with significant lateral inflow (the contribution of the lateral inflow can be estimated from the volumetric difference of outflow and inflow as discussed above), a judicious decision is required to be made about the way of incorporating the lateral inflow, as discussed in the following section, in routing process.

(3) A further consideration in this method is the routing period being used. If the ordinates of hydrographs are too far apart to define the hydrograph adequately, the storage curve (if graphical method of parameter estimation is used) will be in error, which will affect the values of K and X thus derived. The best results are obtained when the routing period is not less than 2KX or more than K. It is, thus, desirable to keep the routing period within these limits.

7.0 THREE PARAMETER MUSKINGUM TYPE PROCEDURE INCORPORATING LATERAL INFLOW

Recently O'Donnell (1985) has suggested the estimation of lateral inflow hydrograph from the given inflow and outflow data using the method of least squares technique based on the matrix method. The governing equations are

$$(1 + r)I - Q = dS/dt \quad ..(36)$$

$$S = K[(1+r)XI + (1-X)Q] \quad ..(37)$$

Where, r is a third parameter accounting for the lateral inflow assuming the lateral inflow to be of the shape of inflow and joining at the upstream end of the reach. There are three unknowns, namely; K, X and r and computation of three routing coefficients (C_1, C_2, and C_3) by matrix inversion procedure leads to the estimation of the three parameters which are in turn related with the routing coefficients.

In all these approaches the influence of addition of the lateral inflow on the wave speed characteristics of the flood has not been considered.

8.0 REMARKS

The hydrologic approach to flood routing is based on the consideration that in a large number of practical cases, the inertia terms of the equations of motion play an exceedingly small role. Such methods have advantages from the practical consideration. Miller and Cunge (1975) have listed the advantages and disadvantages in detail. Some of these are listed below:

Advantages

(1) Hydrologic methods may provide answers in much less time than the solution procedures based on the complete equations.
(2) The channel geometry need not be described in detail.
(3) Computation cost is less, programming for solution is simpler.
(4) Hydrologic flood routing models probably can be more easily integrated with rainfall-runoff models.
(5) The use of the results from mathematical modelling often do not require the accuracy provided by the complete model.

Disadvantages

(1) Hydrologic methods do not have the accuracy of a solution pocedure based on the complete equations. Probably a

hydrologic routing model may give sufficiently accurate results for a particular application but there is often considerable doubt as to how accurate the results are for any application.

(2) Considerable amount of past data, especially of inflow and outflow is required for reliable estimation of the parameters involved in the model.

(3) Backwater effect can not be accounted for by hydrologic methods as they are single characteristic passing only the upstream disturbance to downstream.

(4) The impact of lateral inflow on the parameters of the hydrologic models can not be taken into account due to longer length of reach usually considered.

(5) Solutions of simplified equations may lack desired generality.

Sometimes it happen that routing by Muskingum method may lead to negative values of the outflow generally occurring in the end ordinates of the outflow hydrograph. The negative values of the flow are physically impossible but mathematically correct. These negative values are taken as zero.

Although current interest in the field of flood routing seems to be in the methods utilising numerical solutions of the complete equations of continuity and momentum, hydrologic methods are still useful and may be preferable in some circumstances. The limitations of each technique must be thoroughly understood so that an intelligent choice of method, to be used, may be made.

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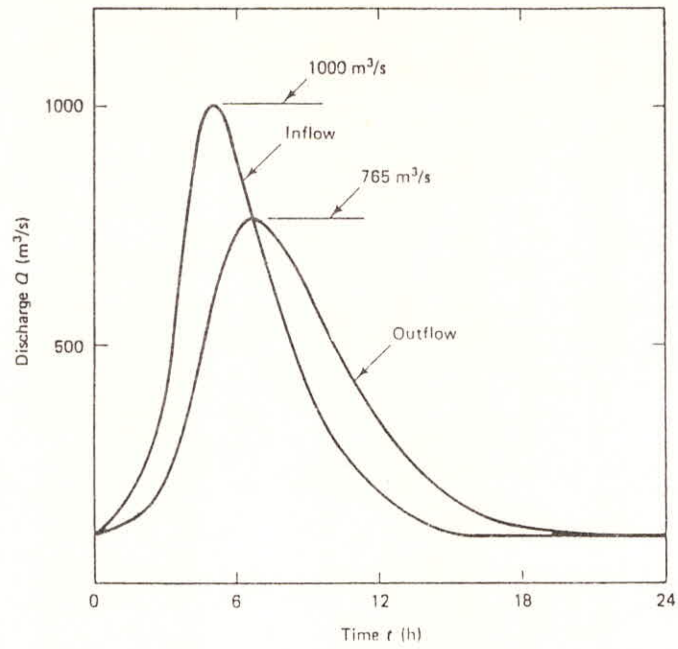

Figure 1 Linear reservoir routing:

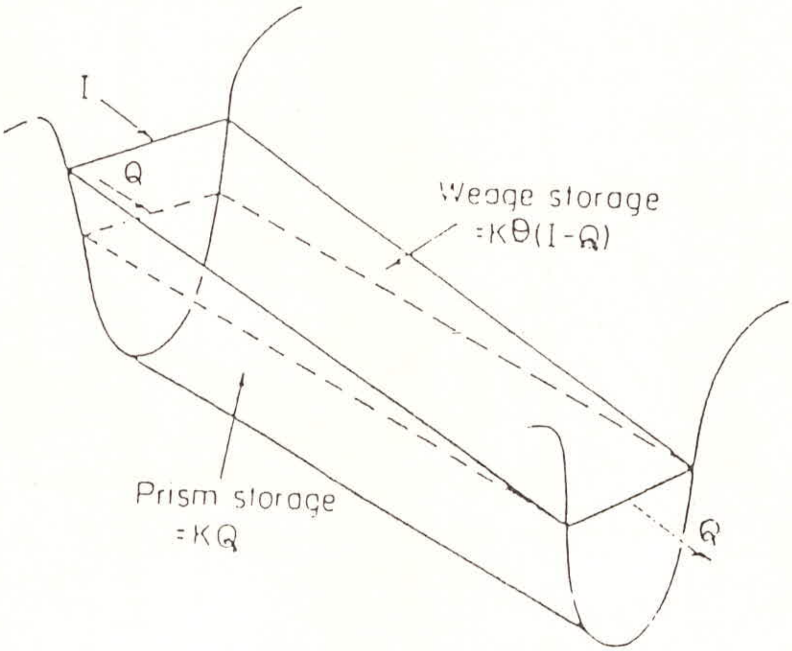

Fig. 2 - Prism and wedge storages in a river channel.

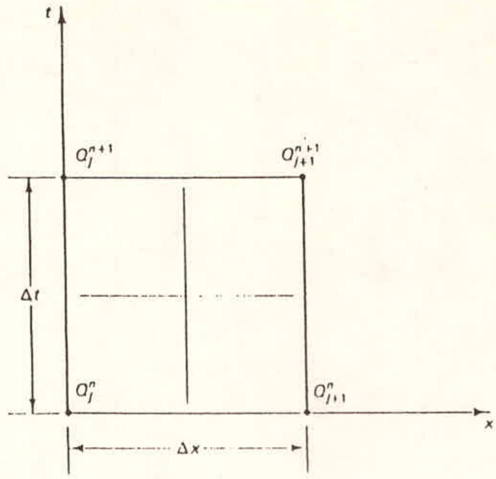

Fig. 3 : Space & Time Grid

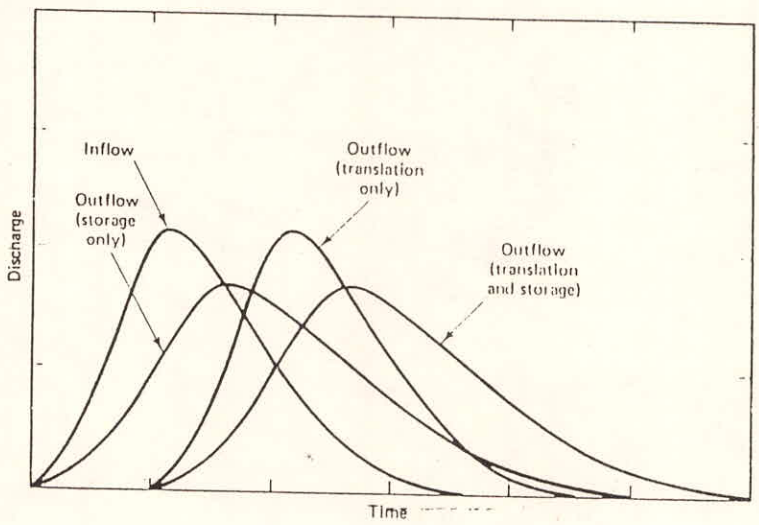

Figure 4 Translation and storage processes in stream channel routing.

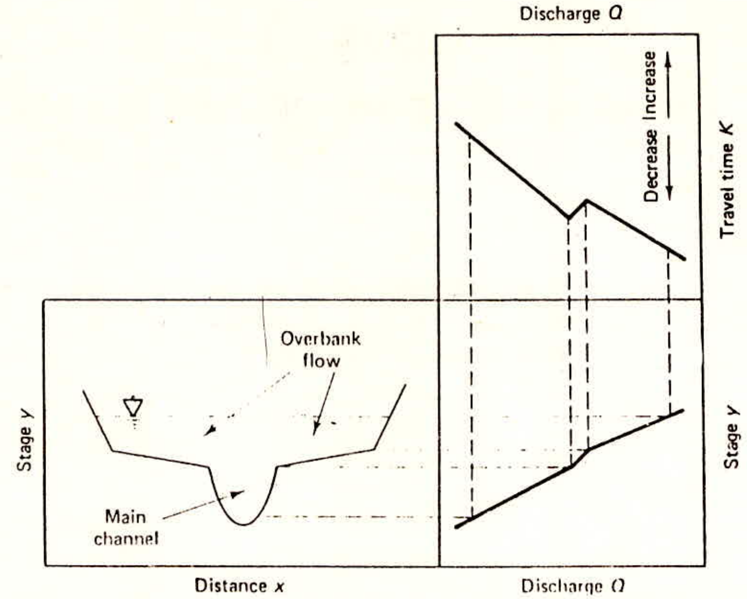

Figure 5 Sketch of travel time as a function of discharge and stage.

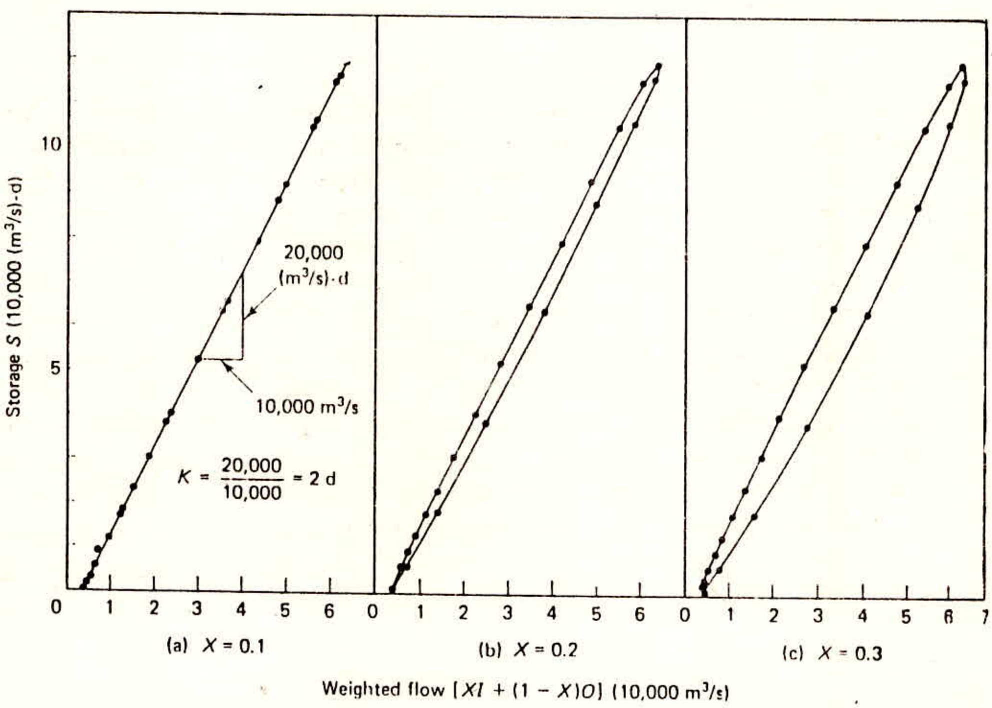

Figure 6 Calibration of Muskingum routing parameters: