Chapter 9
Streamflow and Reservoir Routing

9-1. General

a. Routing is a process used to predict the temporal and spatial variations of a flood hydrograph as it moves through a river reach or reservoir. The effects of storage and flow resistance within a river reach are reflected by changes in hydrograph shape and timing as the floodwave moves from upstream to downstream. Figure 9-1 shows the major changes that occur to a discharge hydrograph as a floodwave moves downstream.

b. In general, routing techniques may be classified into two categories: hydraulic routing, and hydrologic routing. Hydraulic routing techniques are based on the solution of the partial differential equations of unsteady open channel flow. These equations are often referred to as the St. Venant equations or the dynamic wave equations. Hydrologic routing employs the continuity equation and an analytical or an empirical relationship between storage within the reach and discharge at the outlet.

c. Flood forecasting, reservoir and channel design, floodplain studies, and watershed simulations generally utilize some form of routing. Typically, in watershed simulation studies, hydrologic routing is utilized on a reach-by-reach basis from upstream to downstream. For example, it is often necessary to obtain a discharge hydrograph at a point downstream from a location where a hydrograph has been observed or computed. For such purposes, the upstream hydrograph is routed through the reach with a hydrologic routing technique that predicts changes in hydrograph shape and timing. Local flows are then added at the downstream location to obtain the total flow hydrograph. This type of approach is adequate as long as there are no significant backwater effects or

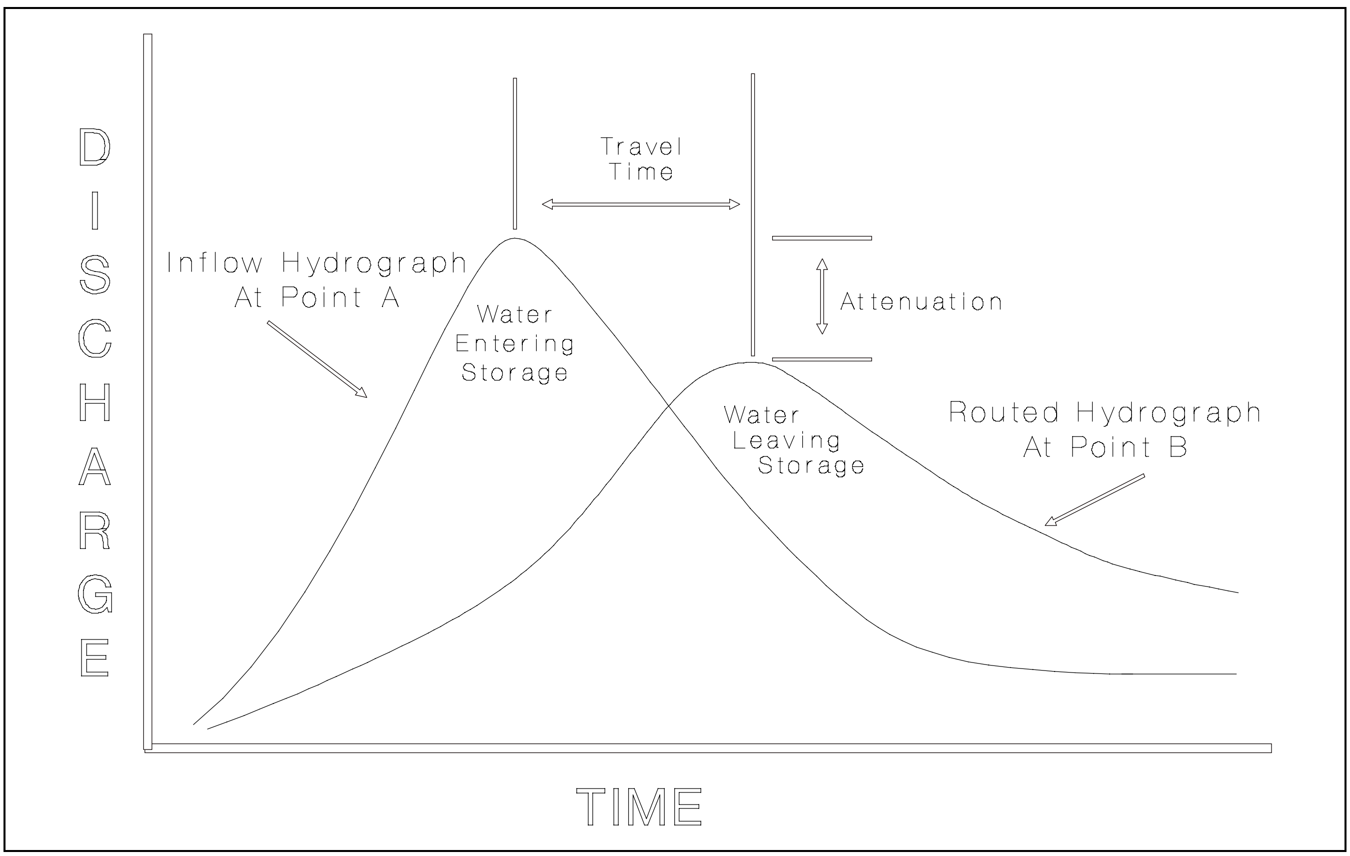

Figure 9-1. Discharge hydrograph routing effects

discontinuities in the water surface because of jumps or bores. When there are downstream controls that will have an effect on the routing process through an upstream reach, the channel configuration should be treated as one continuous system. This can only be accomplished with a hydraulic routing technique that can incorporate backwater effects as well as internal boundary conditions, such as those associated with culverts, bridges, and weirs.

d. This chapter describes several different hydraulic and hydrologic routing techniques. Assumptions, limitations, and data requirements are discussed for each. The basis for selection of a particular routing technique is reviewed, and general calibration methodologies are presented. This chapter is limited to discussions on 1-D flow routing techniques in the context of flood-runoff analysis. The focus of this chapter is on discharge (flow) rather than stage (water surface elevation). Detailed presentation of routing techniques and applications focused on stage calculations can be found in EM 1110-2-1416.

9-2. Hydraulic Routing Techniques

a. The equations of motion. The equations that describe 1-D unsteady flow in open channels, the Saint Venant equations, consist of the continuity equation, Equation 9-1, and the momentum equation, Equation 9-2. The solution of these equations defines the propagation of a floodwave with respect to distance along the channel and time.

$$A\frac{\partial V}{\partial x} + VB\frac{\partial y}{\partial x} + B\frac{\partial y}{\partial t} = q \tag{9-1}$$

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \tag{9-2}$$

where

A = cross-sectional flow area

V = average velocity of water

x = distance along channel

B = water surface width

y = depth of water

t = time

q = lateral inflow per unit length of channel

S_f = friction slope

S_o = channel bed slope

g = gravitational acceleration

Solved together with the proper boundary conditions, Equations 9-1 and 9-2 are the complete dynamic wave equations. The meaning of the various terms in the dynamic wave equations are as follows (Henderson 1966):

(1) Continuity equation.

$A\frac{\partial V}{\partial x}$ = *prism storage*

$VB\frac{\partial y}{\partial x}$ = *wedge storage*

$B\frac{\partial y}{\partial t}$ = *rate of rise*

q = *lateral inflow per unit length*

(2) Momentum equation.

S_f = *friction slope (frictional forces)*

S_o = *bed slope (gravitational effects)*

$\frac{\partial y}{\partial x}$ = *pressure differential*

$\frac{V}{g}\frac{\partial V}{\partial x}$ = *convective acceleration*

$\frac{1}{g}\frac{\partial V}{\partial t}$ = *local acceleration*

(3) Dynamic wave equations. The dynamic wave equations are considered to be the most accurate and comprehensive solution to 1-D unsteady flow problems in open channels. Nonetheless, these equations are based on specific assumptions, and therefore have limitations. The assumptions used in deriving the dynamic wave equations are as follows:

(a) Velocity is constant and the water surface is horizontal *across* any channel section.

(b) All flows are gradually varied with hydrostatic pressure prevailing at all points in the flow, such that vertical accelerations can be neglected.

(c) No lateral secondary circulation occurs.

(d) Channel boundaries are treated as fixed; therefore, no erosion or deposition occurs.

(e) Water is of uniform density, and resistance to flow can be described by empirical formulas, such as Manning's and Chezy's equation.

(f) The dynamic wave equations can be applied to a wide range of 1-D flow problems; such as, dam break floodwave routing, forecasting water surface elevations and velocities in a river system during a flood, evaluating flow conditions due to tidal fluctuations, and routing flows through irrigation and canal systems. Solution of the full equations is normally accomplished with an explicit or implicit finite difference technique. The equations are solved for incremental times (Δt) and incremental distances (Δx) along the waterway.

b. Approximations of the full equations. Depending on the relative importance of the various terms of the momentum Equation 9-2, the equation can be simplified for various applications. Approximations to the full dynamic wave equations are created by combining the continuity equation with various simplifications of the momentum equation. The most common approximations of the momentum equation are:

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} - \frac{1}{g}\frac{\partial V}{\partial t} \qquad (9\text{-}3)$$

Steady Uniform Flow
Kinematic Wave Approx.

Steady Nonuniform Flow
Diffusion Wave Approximation

Steady Nonuniform Flow
Quasi-Steady Dynamic Wave Approximation

Unsteady Nonuniform Flow
Full Dynamic Wave Equation

The use of approximations to the full equations for unsteady flow can be justified when specific terms in the momentum equation are small in comparison to the bed slope. This is best illustrated by an example taken from Henderson's book *Open Channel Flow* (1966). Henderson computed values for each of the terms on the right-hand side of the momentum equation for a steep alluvial stream:

Term:	S_o	$\frac{\partial y}{\partial x}$	$\frac{V}{g}\frac{\partial V}{\partial x}$	$\frac{1}{g}\frac{\partial V}{\partial t}$
Magnitude (ft/mi):	26	.5	.12-.25	.05

These figures relate to a very fast rising hydrograph in which the flow increased from 10,000 to 150,000 cfs and decreased again to 10,000 cfs within 24 hr. Even in this case, where changes in depth and velocity with respect to distance and time are relatively large, the last three terms are still small in comparison to the bed slope. For this type of flow situation (steep stream), an approximation of the full equations would be appropriate. For flatter slopes, the last three terms become increasingly more important.

(1) Kinematic wave approximation. Kinematic flow occurs when gravitational and frictional forces achieve a balance. In reality, a true balance between gravitational and frictional forces never occurs. However, there are flow situations in which gravitational and frictional forces approach an equilibrium. For such conditions, changes in depth and velocity with respect to time and distance are small in magnitude when compared to the bed slope of the channel. Therefore, the terms to the right of the bed slope in Equation 9-3 are assumed to be negligible. This assumption reduces the momentum equation to the following:

$$S_f = S_o \qquad (9\text{-}4)$$

Equation 9-4 essentially states that the momentum of the flow can be approximated with a uniform flow assumption as described by Manning's or Chezy's equation. Manning's equation can be written in the following form:

$$Q = \alpha A^{m} \qquad (9\text{-}5)$$

where α and m are related to flow geometry and surface roughness

Since the momentum equation has been reduced to a simple functional relationship between area and discharge, the movement of a floodwave is described solely by the continuity equation, written in the following form:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q \tag{9-6}$$

Then by combining Equations 9-5 and 9-6, the governing kinematic wave equation is obtained as:

$$\frac{\partial A}{\partial t} + \alpha m A^{(m-1)} \frac{\partial A}{\partial x} = q \tag{9-7}$$

Because of the steady uniform flow assumptions, the kinematic wave equations do not allow for hydrograph diffusion, just simple translation of the hydrograph in time. The kinematic wave equations are usually solved by explicit or implicit finite difference techniques. Any attenuation of the peak flow that is computed using the kinematic wave equations is due to errors inherent in the finite difference solution scheme.

(a) The application of the kinematic wave equation is limited to flow conditions that do not demonstrate appreciable hydrograph attenuation. In general, the kinematic wave approximation works best when applied to steep (10 ft/mile or greater), well defined channels, where the floodwave is gradually varied.

(b) The kinematic wave approach is often applied in urban areas because the routing reaches are generally short and well defined (i.e., circular pipes, concrete lined channels, etc.).

(c) The kinematic wave equations cannot handle backwater effects since, with a kinematic model flow, disturbances can only propagate in the downstream direction. All of the terms in the momentum equation that are used to describe the propagation of the floodwave upstream (backwater effects) have been excluded.

(2) Diffusion wave approximation. Another common approximation of the full dynamic wave equations is the diffusion wave analogy. The diffusion wave model utilizes the continuity Equation 9-1 and the following simplified form of the momentum equation:

$$S_f = S_o - \frac{\partial y}{\partial x} \tag{9-8}$$

The diffusion wave model is a significant improvement over the kinematic wave model because of the inclusion of the pressure differential term in Equation 9-8. This term allows the diffusion model to describe the attenuation (diffusion effect) of the floodwave. It also allows the specification of a boundary condition at the downstream extremity of the routing reach to account for backwater effects. It does not use the inertial terms (last two terms) from Equation 9-2 and, therefore, is limited to slow to moderately rising floodwaves (Fread 1982). However, most natural floodwaves can be described with the diffusion form of the equations.

(3) Quasi-steady dynamic wave approximation. The third simplification of the full dynamic wave equations is the quasi-steady dynamic wave approximation. This model utilizes the continuity equation, Equation 9-1, and the following simplification of the momentum equation:

$$S_f = S_o - \frac{\partial y}{\partial x} - \frac{V}{g}\frac{\partial V}{\partial x} \tag{9-9}$$

In general, this simplification of the dynamic wave equations is not used in flood routing. This form of the momentum equation is more commonly used in steady flow-water surface profile computations. In the case of flood routing, the last two terms on the momentum equation are often opposite in sign and tend to counteract each other (Fread 1982). By including the convective acceleration term and not the local acceleration term, an error is introduced. This error is of greater magnitude than the error that results when both terms are excluded, as in the diffusion wave model. For steady flow-water surface profiles, the last term of the momentum equation (changes in velocity with respect to time) is assumed to be zero. However, changes in velocity with respect to distance are still very important in the calculation of steady flow-water surface profiles.

c. Data requirements. In general, the data requirements of the various hydraulic routing techniques are virtually the same. However, the amount of detail that is required for each type of data will vary depending upon the routing technique being used and the situation it is being applied to. The basic data requirements for hydraulic routing techniques are the following:

(1) Flow data (hydrographs).

(2) Channel cross sections and reach lengths.

(3) Roughness coefficients.

(4) Initial and boundary conditions.

(a) Flow data consist of discharge hydrographs from upstream locations as well as lateral inflow and tributary flow for all points along the stream.

(b) Channel cross sections are typically surveyed sections that are perpendicular to the flow lines. Key issues in selecting cross sections are the accuracy of the surveyed data and the spacing of the sections along the stream. If the routing procedure is utilized to predict stages, then the accuracy of the cross-sectional dimensions will have a direct effect on the prediction of the stage. If the cross sections are used only to route discharge hydrographs, then it is only important to ensure that the cross section is an adequate representation of the discharge versus flow area of the section. Simplified cross-sectional shapes, such as 8-point cross sections or trapezoids and rectangles, are often used to fit the discharge versus flow area of a more detailed section. Cross-sectional spacing affects the level of detail of the results as well as the accuracy of the numerical solution to the routing equations. Detailed discussions on cross-sectional spacing can be found in the reference by the Hydrologic Engineering Center (HEC) (USACE 1986).

(c) Roughness coefficients for hydraulic routing models are typically in the form of Manning's n values. Manning's coefficients have a direct impact on the travel time and amount of diffusion that will occur when routing a flood hydrograph through a channel reach. Roughness coefficients will also have a direct impact on predicted stages.

(d) All hydraulic models require that initial and boundary conditions be established before the routing can commence. Initial conditions are simply stated as the conditions at all points in the stream at the beginning of the simulation. Initial conditions are established by specifying a base flow within the channel at the start of the simulation. Channel depths and velocities can be calculated through steady-state backwater computations or a normal depth equation (e.g., Manning's equation). Boundary conditions are known relationships between discharge and time and/or discharge and stage. Hydraulic routing computations require the specification of upstream, downstream, and internal boundary conditions to solve the equations. The upstream boundary condition is the discharge (or stage) versus time relationship of the hydrograph to be routed through the reach. Downstream boundary conditions are usually established with a steady-state rating curve (discharge versus depth relationship) or through normal depth calculations (Manning's equation). Internal boundary conditions consist of lateral inflow or tributary flow hydrographs, as well as depth versus discharge relationships for hydraulic structures within the river reach.

9-3. Hydrologic Routing Techniques

Hydrologic routing employs the use of the continuity equation and either an analytical or an empirical relationship between storage within the reach and discharge at the outlet. In its simplest form, the continuity equation can be written as inflow minus outflow equals the rate of change of storage within the reach:

$$I - O = \frac{\Delta S}{\Delta t} \tag{9-10}$$

where

I = the average inflow to the reach during Δt

O = the average outflow from the reach during Δt

S = storage within the reach

a. Modified puls reservoir routing.

(1) One of the simplest routing applications is the analysis of a floodwave that passes through an unregulated reservoir (Figure 9-2a). The inflow hydrograph is known, and it is desired to compute the outflow hydrograph from the reservoir. Assuming that all gate and spillway openings are fixed, a unique relationship between storage and outflow can be developed, as shown in Figure 9-2b.

(2) The equation defining storage routing, based on the principle of conservation of mass, can be written in approximate form for a routing interval Δt. Assuming the subscripts "1" and "2" denote the beginning and end of the routing interval, the equation is written as follows:

$$\frac{O_1 + O_2}{2} = \frac{I_1 + I_2}{2} - \frac{S_2 - S_1}{\Delta t} \tag{9-11}$$

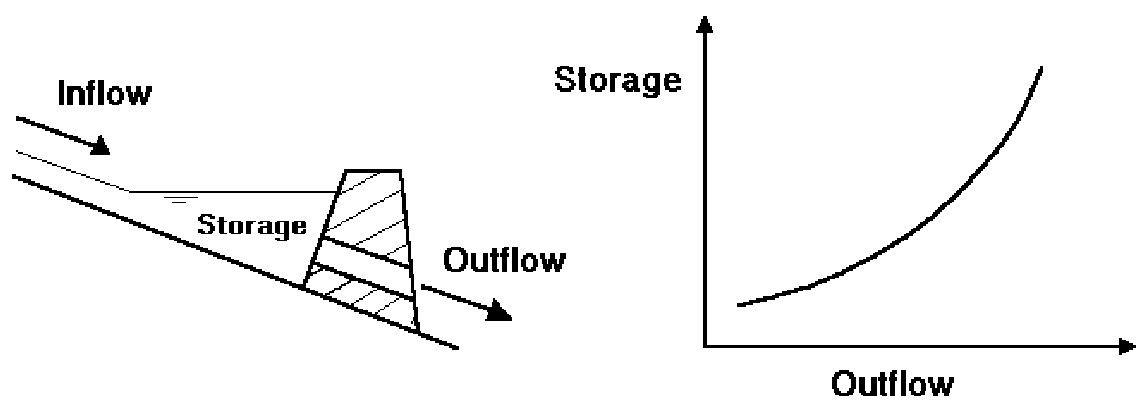

Figure 9-2. Reservoir storage routing

The known values in this equation are the inflow hydrograph and the storage and discharge at the beginning of the routing interval. The unknown values are the storage and discharge at the end of the routing interval. With two unknowns (O_2 and S_2) remaining, another relationship is required to obtain a solution. The storage-outflow relationship is normally used as the second equation. How that relationship is derived is what distinguishes various storage routing methods.

(3) For an uncontrolled reservoir, outflow and water in storage are both uniquely a function of lake elevation. The two functions can be combined to develop a storage-outflow relationship, as shown in Figure 9-3. Elevation-discharge relationships can be derived directly from hydraulic equations. Elevation-storage relationships are derived through the use of topographic maps. Elevation-area relationships are computed first, then either average end-area or conic methods are used to compute volumes.

(4) The storage-outflow relationship provides the outflow for any storage level. Starting with a nearly empty reservoir, the outflow capability would be minimal. If the inflow is less than the outflow capability, the water would flow through. During a flood, the inflow increases and eventually exceeds the outflow capability. The difference between inflow and outflow produces a change in storage. In Figure 9-4, the difference between the inflow and the outflow (on the rising side of the outflow hydrograph) represents the volume of water entering storage.

(5) As water enters storage, the outflow capability increases because the pool level increases. Therefore, the outflow increases. This increasing outflow with increasing water in storage continues until the reservoir reaches a maximum level. This will occur the moment that the outflow equals the inflow, as shown in Figure 9-4. Once the outflow becomes greater than the inflow, the storage level will begin dropping. The difference between the outflow and the inflow hydrograph on the recession side reflects water withdrawn from storage.

(6) The modified puls method applied to reservoirs consists of a repetitive solution of the continuity equation. It is assumed that the reservoir water surface remains horizontal, and therefore, outflow is a unique function of reservoir storage. The continuity equation, Equation 9-11, can be manipulated to get both of the unknown variables on the left-hand side of the equation:

$$\left(\frac{S_2}{\Delta t} + \frac{O_2}{2}\right) = \left(\frac{S_1}{\Delta t} + \frac{O_1}{2}\right) - O_1 + \frac{I_1 + I_2}{2} \qquad (9\text{-}12)$$

Since I is known for all time steps, and O_1 and S_1 are known for the first time step, the right-hand side of the equation can be calculated. The left-hand side of the equation can be solved by trial and error. This is accomplished by assuming a value for either S_2 or O_2, obtaining the corresponding value from the storage-outflow relationship, and then iterating until Equation 9-12 is satisfied.

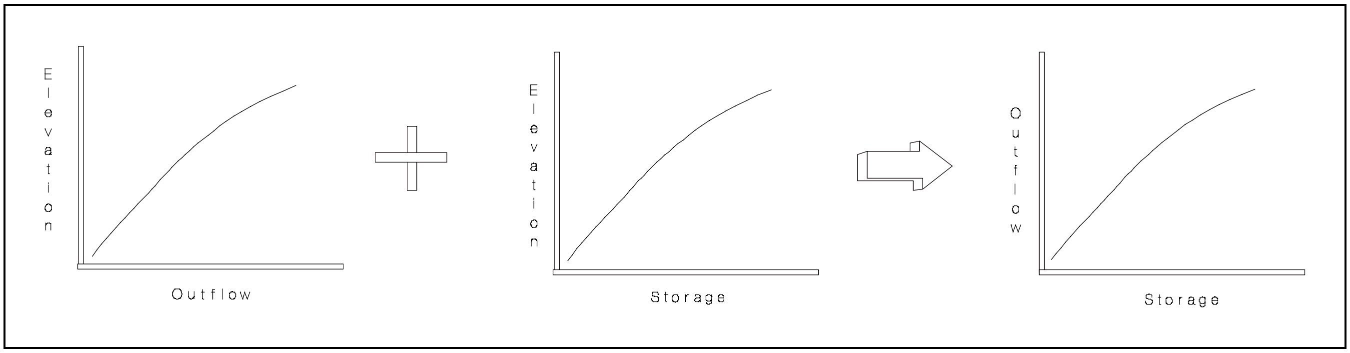

Figure 9-3. Reservoir storage-outflow curve

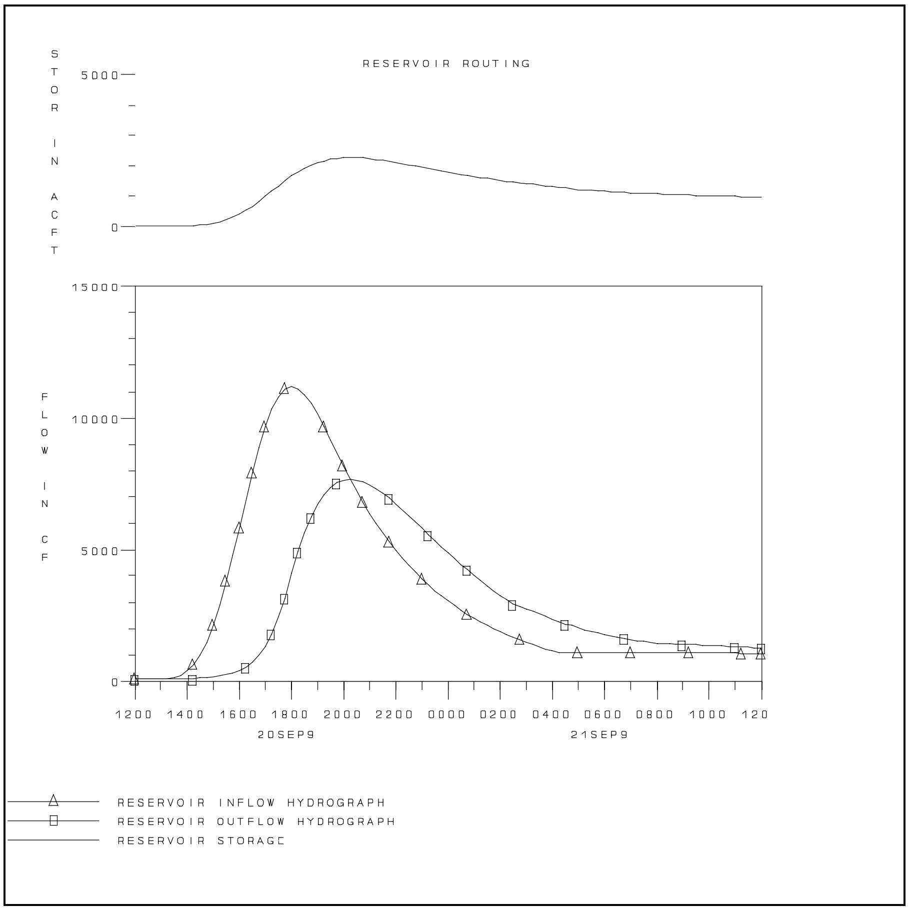

Figure 9-4. Reservoir routing example

Rather than resort to this iterative procedure, a value of Δt is selected and points on the storage-outflow curve are replotted as the "storage-indication" curve shown in Figure 9-5. This graph allows for a direct determination of the outflow (O_2) once a value of storage indication ($S_2/\Delta t + O_2/2$) has been calculated from Equation 9-12 (Viessman et al. 1977). The numerical integration of Equation 9-12 and Figure 9-5 is illustrated as an example in Table 9-1. The stepwise procedure for applying the modified puls method to reservoirs can be summarized as follows:

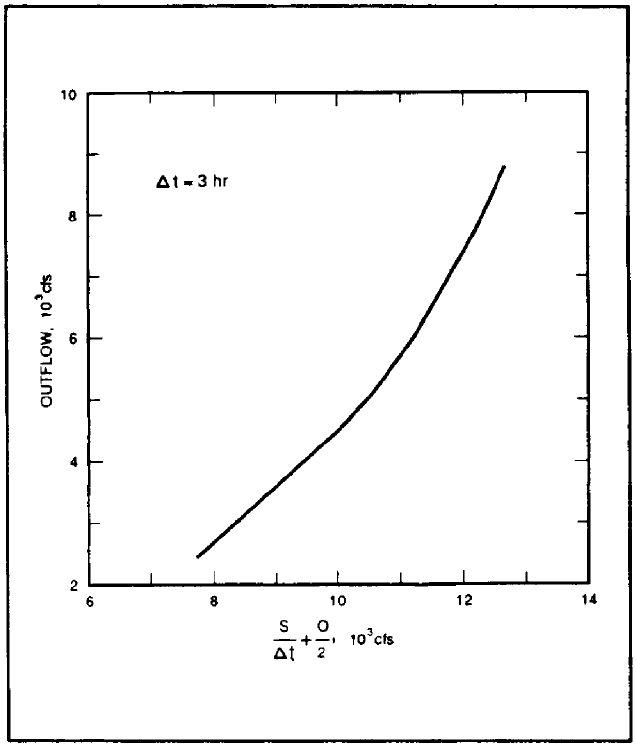

Figure 9-5. Storage-indication curve

(a) Determine a composite discharge rating curve for all of the reservoir outlet structures.

(b) Determine the reservoir storage that corresponds with each elevation on the rating curve for reservoir outflow.

(c) Select a time step and construct a storage-indication versus outflow curve $[(S/\Delta t) + (O/2)]$ versus O.

(d) Route the inflow hydrograph through the reservoir based on Equation 9-12 and the storage-indication curve.

(e) Compare the results with historical events to verify the model.

b. Modified puls channel routing. Routing in natural rivers is complicated by the fact that storage in a river reach is not a function of outflow alone. During the passing of a floodwave, the water surface in a channel is not uniform. The storage and water surface slope within a river reach, for a given outflow, is greater during the rising stages of a floodwave than during the falling (Figure 9-6). Therefore, the relationship between storage and discharge at the outlet of a channel is not a unique relationship, rather it is a looped relationship. An example storage-discharge function for a river is shown in Figure 9-7.

(1) Application of the modified puls method to rivers. To apply the modified puls method to a channel routing problem, the storage within the river reach is approximated with a series of "cascading reservoirs" (Figure 9-8). Each reservoir is assumed to have a level pool and, therefore, a unique storage-discharge relationship. The cascading reservoir approach is capable of approximating the looped storage-outflow effect when evaluating the river reach as a whole. The rising and falling floodwave is simulated with different storage levels in the cascade of reservoirs, thus producing a looped storage-outflow function for the total river reach. This is depicted graphically in Figure 9-9.

(2) Determination of the storage-outflow relationship.

(a) Determining the storage-outflow relationship for a river reach is a critical part of the modified puls procedure. In river reaches, storage-outflow relationships can be determined from one of the following:

- steady-flow profile computations,
- observed water surface profiles,
- normal-depth calculations,
- observed inflow and outflow hydrographs, and
- optimization techniques applied to observed inflow and outflow hydrographs.

(b) Steady-flow water surface profiles, computed over a range of discharges, can be used to determine storage-outflow relationships in a river reach (Figure 9-10). In this illustration, a known hydrograph at A is to be routed to location B. The storage-outflow relationship required for routing is determined by computing a series of water surface profiles, corresponding to a range of discharges. The range of discharges should encompass the range of flows that will be routed through the river reach. The storage volumes are computed by multiplying the cross-sectional area, under a specific flow profile, by the channel reach lengths. Volumes are calculated for each flow profile and then plotted against

Table 9-1
Storage Routing Calculation

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Time (hr)	Inflow (cfs)	Average Inflow (cfs)	$\frac{S}{\Delta t} + \frac{O}{2}$ (cfs)	Outflow (cfs)	$\frac{S}{\Delta t}$ (cfs)	S (acre-ft)
0	3,000		8,600	3,000	7,100	1,760
		3,130				
3	3,260		8,730	3,150	7,155	1,774
		3,445				
6	3,630		9,025	3,400	7,325	1,816
		3,825				
9	4,020		9,450	3,850	7,525	1,866
		4,250				
12	4,480		9,850	4,300	7,700	1,909
.						
.						
etc.						

the corresponding discharge at the outlet. If channel or levee modifications will have an effect on the routing through the reach,modifications can be made to the cross sections, water surface profiles recalculated, and a revised storage-outflow relationship can be developed. The impacts of the channel or levee modification can be approximated by routing floods with both pre- and post-project storage-outflow relationships.

(c) Observed water surface profiles, obtained from high water marks, can be used to compute storage-outflow relationships. Sufficient stage data over a range of floods are required for this type of calculation; however, it is not likely that enough data would be available over the range of discharges needed to compute an adequate storage discharge relationship. If a few observed profiles are available, they can be used to calibrate a steady-flow water surface profile model for the channel reach of interest. Then the water surface profile model could be used to calculate the appropriate range of values to calculate the storage-outflow relationship.

(d) Normal depth associated with uniform flow does not exist in natural streams; however, the concept can be used to estimate water depth and storage in natural rivers if uniform flow conditions can reasonably be assumed. With a typical cross section, Manning's equation is solved for a range of discharges, given appropriate "n" values and an estimated slope of the energy grade line. Under the assumption of uniform flow conditions, the energy slope is considered equal to the average channel bed slope; therefore, this approach should not be applied in backwater areas.

(e) Observed inflow and outflow hydrographs can be used to compute channel storage by an inverse process of flood routing. When both inflow and outflow are known, the change in storage can be computed, and from that a storage versus outflow function can be developed. Tributary inflow, if any, must also be accounted for in this calculation. The total storage is computed from some base level storage at the beginning or end of the routing sequence.

(f) Inflow and outflow hydrographs can also be used to compute routing criteria through a process of iteration in which an initial set of routing criteria is assumed, the inflow hydrograph is routed, and the results are evaluated. The process is repeated as necessary until a suitable fit of the routed and observed hydrograph is obtained.

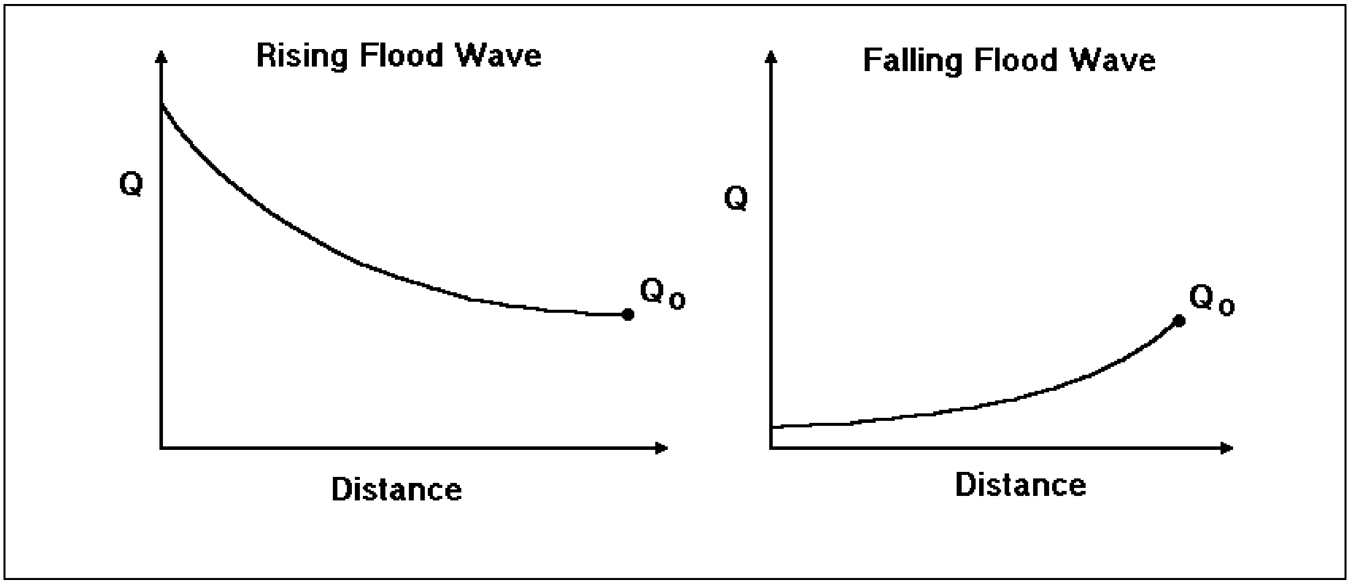

Figure 9-6. Rising and falling floodwave

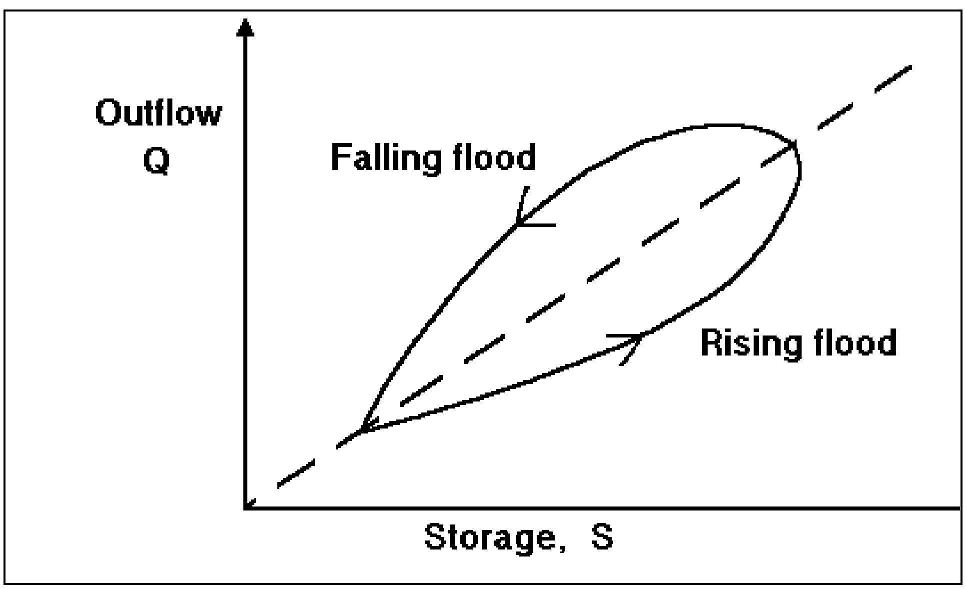

Figure 9-7. Looped storage-outflow relationship for a river reach

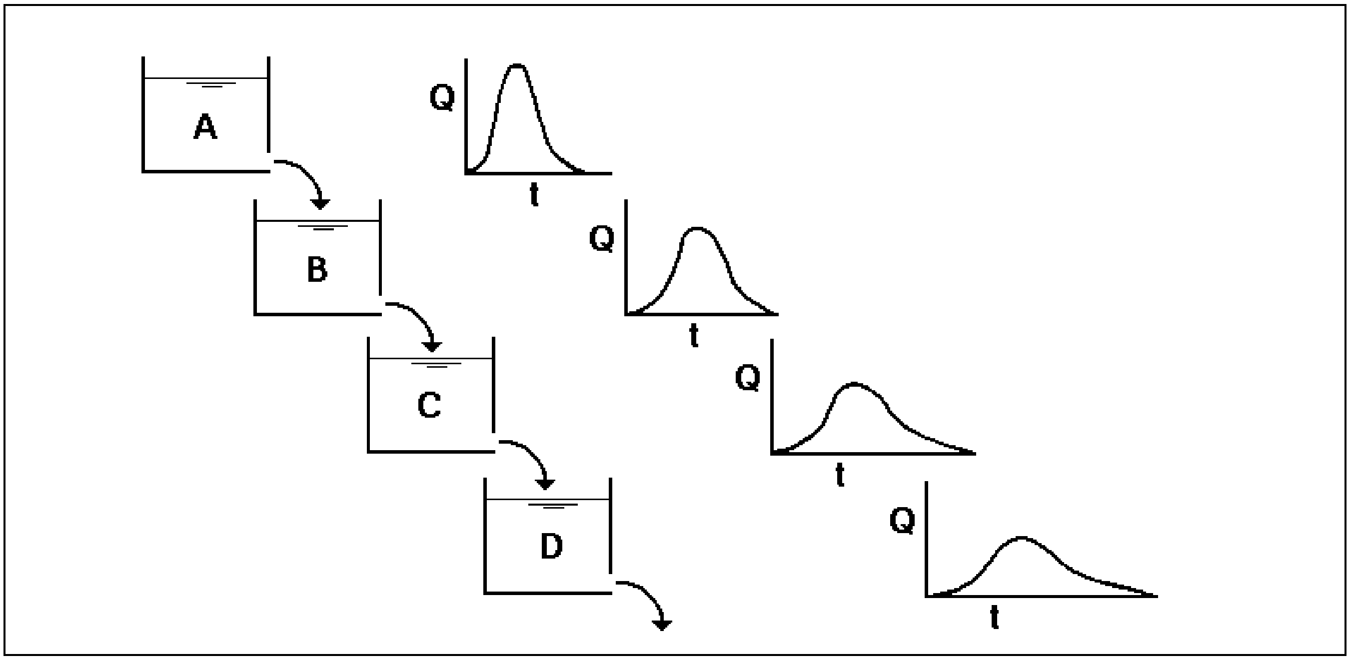

Figure 9-8. Cascade of reservoirs, depicting storage routing in a channel

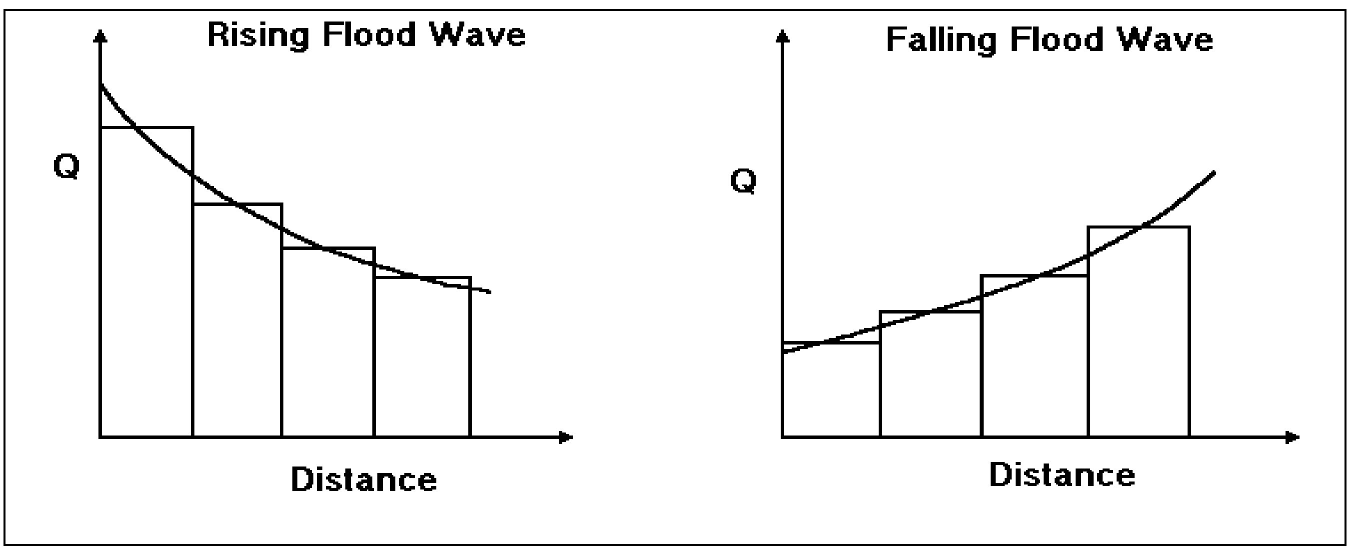

Figure 9-9. Modified puls approximation of the rising and falling floodwaves

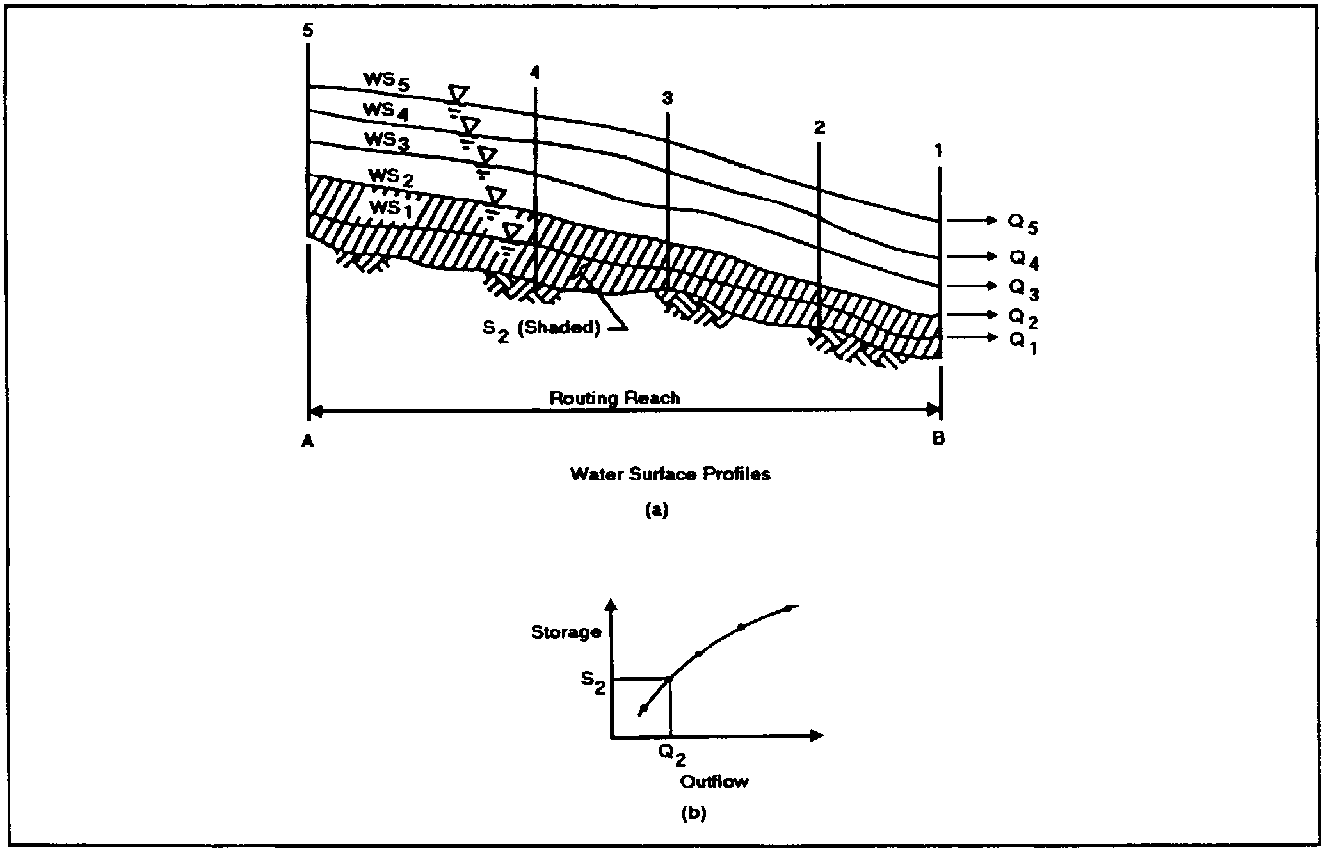

Figure 9-10. Storage-outflow relationships

(3) Determining the number of routing steps. In reservoir routing, the modified puls method is applied with one routing step. This is under the assumption that the travel time through the reservoir is smaller than the computation interval Δt. In channel routing, the travel time through the river reach is often greater than the computation interval. When this occurs, the channel must be broken down into smaller routing steps to simulate the floodwave movement and changes in hydrograph shape. The number of steps (or reach lengths) affects the attenuation of the hydrograph and should be obtained by calibration. The maximum amount of attenuation will occur when the channel routing computation is done in one step. As the number of routing steps increases, the amount of attenuation decreases. An initial estimate of the number of routing steps (NSTPS) can be obtained by dividing the total travel time (K) for the reach by the computation interval Δt.

$$K = \frac{L}{V_w}$$

$$NSTPS = \frac{K}{\Delta t} \tag{9-13}$$

where

K = floodwave travel time through the reach

L = channel reach length

V_w = velocity of the floodwave (not average velocity)

$NSTPS$ = number of routing steps

The time interval Δt is usually determined by ensuring that there is a sufficient number of points on the rising side of the inflow hydrograph. A general rule of thumb is that the computation interval should be less than 1/5 of the time of rise (t_r) of the inflow hydrograph.

$$\Delta t \le \frac{t_r}{5} \tag{9-14}$$

c. Muskingum method. The Muskingum method was developed to directly accommodate the looped relationship between storage and outflow that exists in rivers. With the Muskingum method, storage within a reach is visualized in two parts: prism storage and wedge storage. Prism storage is essentially the storage under the steady-flow water surface profile. Wedge storage is the additional storage under the actual water surface profile. As shown in Figure 9-11, during the rising stages of the floodwave the wedge storage is positive and added to the prism storage. During the falling stages of a floodwave, the wedge storage is negative and subtracted from the prism storage.

(1) Development of the Muskingum routing equation.

(a) Prism storage is computed as the outflow (O) times the travel time through the reach (K). Wedge storage is computed as the difference between inflow and outflow (I-O) times a weighting coefficient X and the travel time K. The coefficient K corresponds to the travel time of the floodwave through the reach. The parameter X is a dimensionless value expressing a weighting of the relative effects of inflow and outflow on the storage (S) within the reach. Thus, the Muskingum method defines the storage in the reach as a linear function of weighted inflow and outflow:

$$S = \text{prism storage} + \text{wedge storage}$$

$$S = KO + KX(I-O)$$

$$S = K\,[XI + (1-X)O] \tag{9-15}$$

where

S = total storage in the routing reach

O = rate of outflow from the routing reach

I = rate of inflow to the routing reach

K = travel time of the floodwave through the reach

X = dimensionless weighting factor, ranging from 0.0 to 0.5

(b) The quantity in the brackets of Equation 9-15 is considered an expression of weighted discharge. When $X = 0.0$, the equation reduces to $S = KO$, indicating that storage is only a function of outflow, which is equivalent to level-pool reservoir routing with storage as a linear function of outflow. When $X = 0.5$, equal weight is given to inflow and outflow, and the condition is equivalent to a uniformly progressive wave that does not attenuate. Thus, “0.0” and “0.5” are limits on the value of X, and within this range the value of X determines the degree of attenuation of the floodwave as it passes through the routing

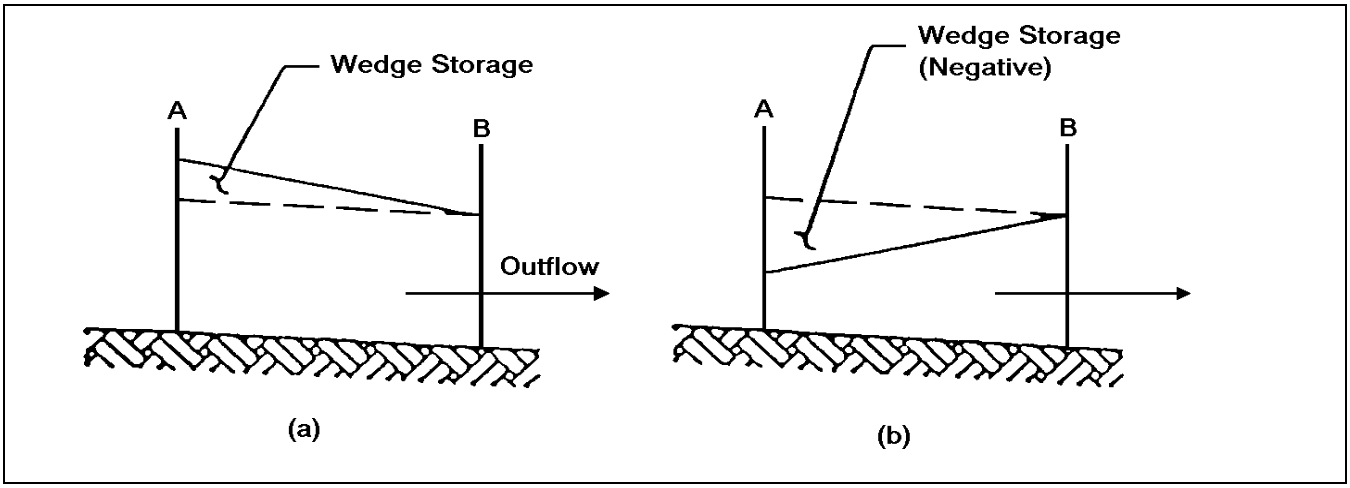

Figure 9-11. Muskingum prism and wedge storage concept

reach. A value of "0.0" produces maximum attenuation, and "0.5" produces pure translation with no attenuation.

(c) The Muskingum routing equation is obtained by combining Equation 9-15 with the continuity equation, Equation 9-11, and solving for O_2.

$$O_2 = C_1 I_2 + C_2 I_1 + C_3 O_1 \tag{9-16}$$

The subscripts 1 and 2 in this equation indicate the beginning and end, respectively, of a time interval Δt. The routing coefficients C_1, C_2, and C_3 are defined in terms of Δt, K, and X.

$$C_1 = \frac{\Delta t - 2KX}{2K(1 - X) + \Delta t} \tag{9-17}$$

$$C_2 = \frac{\Delta t + 2KX}{2K(1 - X) + \Delta t} \tag{9-18}$$

$$C_3 = \frac{2K(1 - X) - \Delta t}{2K(1 - X) + \Delta t} \tag{9-19}$$

Given an inflow hydrograph, a selected computation interval Δt, and estimates for the parameters K and X, the outflow hydrograph can be calculated.

(2) Determination of Muskingum K and X. In a gauged situation, the Muskingum K and X parameters can be calculated from observed inflow and outflow hydrographs. The travel time, K, can be estimated as the interval between similar points on the inflow and outflow hydrographs. The travel time of the routing reach can be calculated as the elapsed time between centroid of areas of the two hydrographs, between the hydrograph peaks, or between midpoints of the rising limbs. After K has been estimated, a value for X can be obtained through trial and error. Assume a value for X, and then route the inflow hydrograph with these parameters. Compare the routed hydrograph with the observed outflow hydrograph. Make adjustments to X to obtain the desired fit. Adjustments to the original estimate of K may also be necessary to obtain the best overall fit between computed and observed hydrographs. In an ungauged situation, a value for K can be estimated as the travel time of the floodwave through the routing reach. The floodwave velocity (V_w) is greater than the average velocity at a given cross section for a given discharge. The floodwave velocity can be estimated by a number of different techniques:

(a) Using Seddon's law, a floodwave velocity can be approximated from the discharge rating curve at a station whose cross section is representative of the routing reach. The slope of the discharge rating curve is equal to dQ/dy. The floodwave velocity, and therefore the travel time K, can be estimated as follows:

$$V_w = \frac{1}{B}\frac{dQ}{dy} \tag{9-20}$$

$$K = \frac{L}{V_w} \tag{9-21}$$

where

V_w = floodwave velocity, in feet/second

B = top width of the water surface

L = length of the routing reach, in feet

(b) Another means of estimating floodwave velocity is to estimate the average velocity (V) and multiply it by a ratio. The average velocity can be calculated from Manning's equation with a representative discharge and cross section for the routing reach. For various channel shapes, the floodwave velocity has been found to be a direct ratio of the average velocity.

Channel shape	Ratio V_w/V
Wide rectangular	1.67
Wide parabolic	1.44
Triangular	1.33

For natural channels, an average ratio of 1.5 is suggested. Once the wave speed has been estimated, the travel time (K) can be calculated with Equation 9-21.

(c) Estimating the Muskingum X parameter in an ungauged situation can be very difficult. X varies between 0.0 and 0.5, with 0.0 providing the maximum amount of hydrograph attenuation and 0.5 no attenuation. Experience has shown that for channels with mild slopes and flows that go out of bank, X will be closer to 0.0. For steeper streams, with well defined channels that do not have flows going out of bank, X will be closer to 0.5. Most natural channels lie somewhere in between these two limits, leaving a lot of room for "engineering judgment." One equation that can be used to estimate the Muskingum X coefficient in ungauged areas has been developed by Cunge (1969). This equation is taken from the Muskingum-Cunge channel routing method, which is described in paragraph 9-3*e*. The equation is written as follows:

$$X = \frac{1}{2}\left(1 - \frac{Q_o}{BS_o c\Delta x}\right) \tag{9-22}$$

where

Q_o = reference flow from the inflow hydrograph

c = floodwave speed

S_o = friction slope or bed slope

B = top width of the flow area

Δx = length of the routing subreach

The choice of which flow rate to use in this equation is not completely clear. Experience has shown that a reference flow based on average values (midway between the base flow and the peak flow) is in general the most suitable choice. Reference flows based on peak flow values tend to accelerate the wave much more than it would in nature, while the converse is true if base flow reference values are used (Ponce 1983).

(3) Selection of the number of subreaches. The Muskingum equation has a constraint related to the relationship between the parameter K and the computation interval Δt. Ideally, the two should be equal, but Δt should not be less than $2KX$ to avoid negative coefficients and instabilities in the routing procedure.

$$2KX < \Delta t \le K \tag{9-23}$$

A long routing reach should be subdivided into subreaches so that the travel time through each subreach is approximately equal to the routing interval Δt. That is:

$$Number\ of\ subreaches = \frac{K}{\Delta t}$$

This assumes that factors such as channel geometry and roughness have been taken into consideration in determining the length of the routing reach and the travel time K.

d. Working R&D routing procedure. The Working R&D procedure is a storage routing technique that accommodates the nonlinear nature of floodwave movement in natural channels. The method is useful in situations where the use of a variable K (reach travel time) would assist in obtaining accurate answers. A nonlinear storage-outflow relationship indicates that a variable K is necessary. The method is also useful in situations wherein the horizontal reservoir surface assumption of the modified puls procedure is not applicable, such as normally occurs in natural channels.

(1) The working R&D procedure could be termed "Muskingum with a variable K" or "modified puls with wedge storage." For a straight line storage-discharge (weighted discharge) relation, the procedure is the same solution as the Muskingum method. For $X = 0$, the procedure is identical to Modified Puls.

(2) The basis for the procedure derives from the concept of a "working discharge," which is a hypothetical steady flow that would result in the same natural channel storage that occurs with the passage of a floodwave. Figure 9-12 illustrates this concept.

where

I = reach inflow

O = reach outflow

D = working value discharge or simply working discharge

(3) The wedge storage (WS) may be computed in the following two ways: As in the Muskingum technique where X is a weighting factor and K is reach travel time:

$$WS = KX\ (I - O) \tag{9-24}$$

or using the working discharge (D) concept:

$$WS = K\ (D - O) \tag{9-25}$$

equating and solving for O:

$$K\ (D - O) = KX\ (I - O) \tag{9-26}$$

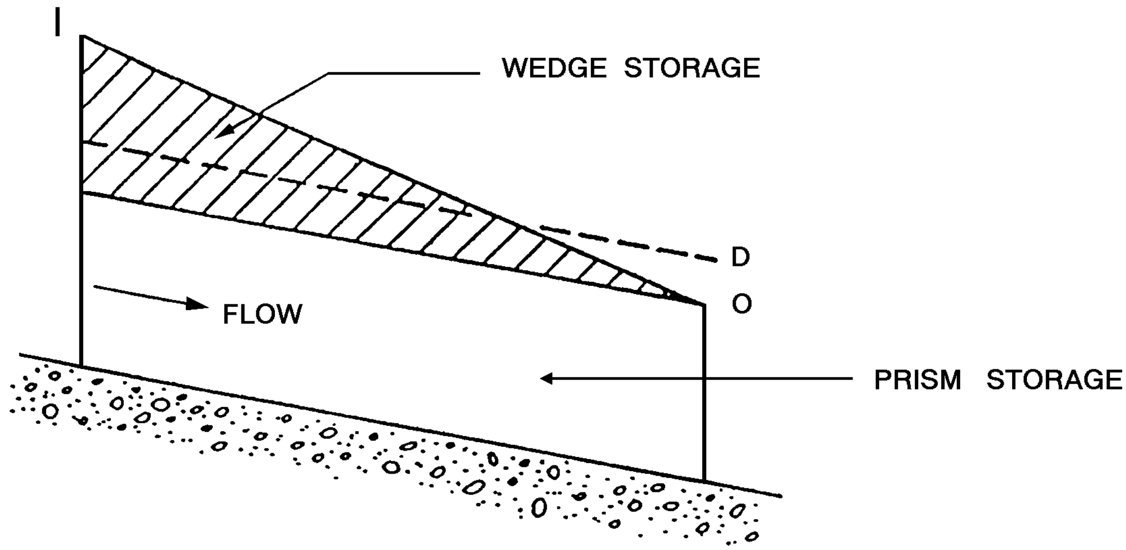

Figure 9-12. Illustration of the "working discharge" concept

or

$$O = D - \frac{X}{1-X}(I-D) \quad (9\text{-}27)$$

The continuity equation may be approximated by:

$$\frac{S_2 - S_1}{\Delta t} = 0.5\,(I_1 + I_2) - 0.5\,(O_1 + O_2) \quad (9\text{-}28)$$

where

S = storage

Δt = time increment

Substituting Equation 9-27 into 9-28 and appending the appropriate subscripts to denote beginning and end of period and performing the appropriate algebra yields:

$$0.5\Delta t(I_1 + I_2) + [S_1(1 - X) - 0.5D_1\Delta t] = [S_2(1 - X) + 0.5D_2\Delta t] \quad (9\text{-}29)$$

Let

$$R = S\,(1 - X) + 0.5D\Delta t \quad (9\text{-}30)$$

where R is termed the "working value of storage" or simply working storage and represents an index of the true natural storage. Equation 9-29 may therefore be written:

$$R_2 = R_1 + 0.5\Delta t\,(I_1 + I_2) - D_1\Delta t \quad (9\text{-}31)$$

transposing Δt results in the equation used in routing computations:

$$\frac{R_2}{\Delta t} = \frac{R_1}{\Delta t} + 0.5\,(I_1 + I_2) - D_1 \quad (9\text{-}32)$$

The form of the relationship for R (working discharge) is analogous to storage indication in the modified puls procedure. $R_2/\Delta t$ may be computed from information known at the beginning of a routing interval. The outflow at the end of the routing interval may then be determined from a rating curve of working storage versus working discharge. The cycle is then repeated stepping forward in time.

(4) The solution scheme using this concept requires development of a rating curve of working storage versus working discharge as stated above. The following column headings are helpful in developing the function when storage-outflow data are available.

1	2	3
Storage (S)	$\frac{S}{\Delta t}(1-X)$	*Working Discharge* (D)

4	5
$\frac{D}{2}$	$\frac{S}{\Delta t}(1-X) + \frac{D}{2}$

(5) Column 2 of the tabulation is obtained from column 1 by using an appropriate conversion factor and appropriate X. The conversion factor of 1 acre-ft/hour = 12.1 cfs is useful in this regard. Column 5 is the sum of columns 2 and 4. Column 3 is plotted against column 5 on cartesian coordinate paper and a curve drawn through the plotted points. This represents the working discharge-working outflow rating curve. An example curve is shown in Figure 9-13.

(6) The routing of a hydrograph can be performed as the one shown in Table 9-2. The procedure, in narrative form is:

- Conditions known at time 1: I_1, O_1, D_1, and $R_1/\Delta t$.

- At time 2, only I_2 is known, therefore:

$$\frac{R_2}{\Delta t} = \frac{R_1}{\Delta t} + 0.5\,(I_1 + I_2) - D_1$$

- Enter working storage, working discharge function, and read out D_2.

- Calculate O_2 as follows:

$$O_2 = D_2 - \frac{X}{1-X}\,(I_2 - D_2)$$

- Repeat process until finished.

e. Muskingum-Cunge channel routing. The Muskingum-Cunge channel routing technique is a nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are the parameters of the model are more physically based; the method has been shown to compare well against the full unsteady flow equations over a wide range of flow situations (Ponce 1983 and Brunner 1989); and the solution is independent of the user-specified computation interval. The major limitations of the Muskingum-Cunge technique are that it cannot account for backwater effects, and the method begins to diverge from the full unsteady flow solution when very rapidly rising hydrographs are routed through flat channel sections.

(1) Development of equations.

(a) The basic formulation of the equations is derived from the continuity Equation 9-33 and the diffusion form of the momentum Equation 9-34:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = q_l \tag{9-33}$$

$$S_f = S_o - \frac{\partial Y}{\partial x} \tag{9-34}$$

(b) By combining Equations 9-33 and 9-34 and linearizing, the following convective diffusion equation is formulated (Miller and Cunge 1975):

$$\frac{\partial Q}{\partial t} + c\frac{\partial Q}{\partial x} = \mu\frac{\partial^2 Q}{\partial x^2} + cq_L \tag{9-35}$$

where

Q = discharge, in cubic feet per second

A = flow area, in square feet

t = time, in seconds

x = distance along the channel, in feet

Y = depth of flow, in feet

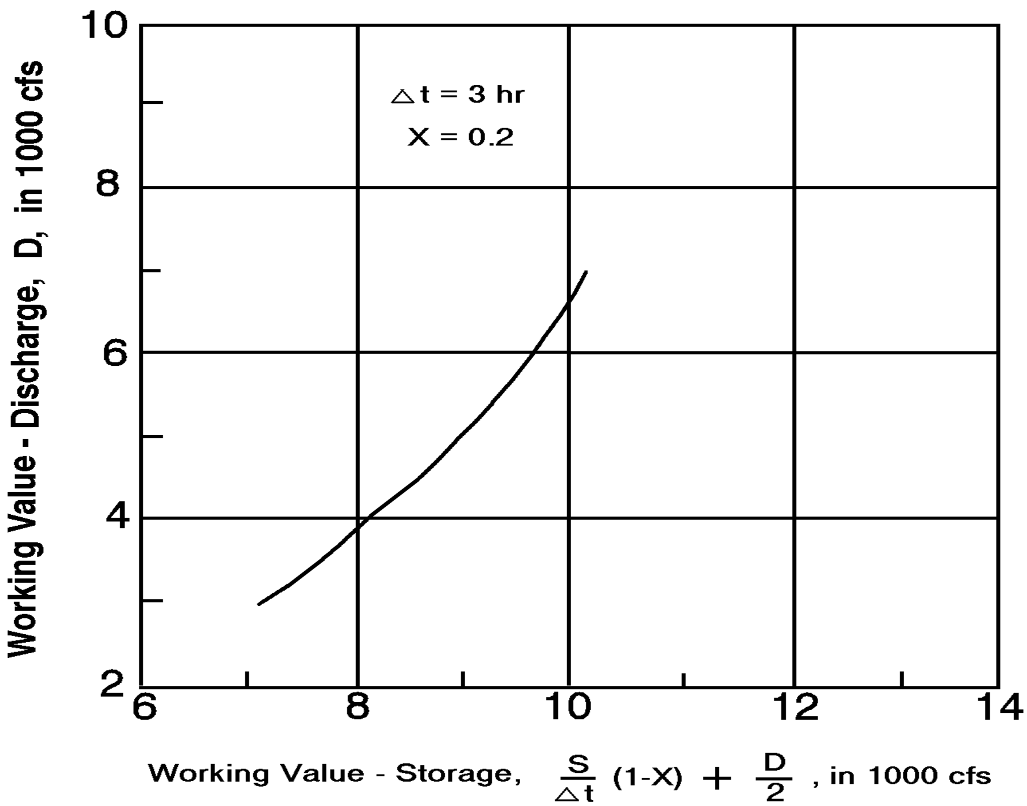

Figure 9-13. Rating curve for working R&D routing

q_L = lateral inflow per unit of channel length

S_f = friction slope

S_o = bed slope

c = the wave celerity in the x direction as defined below

The wave celerity (c) and the hydraulic diffusivity (μ) are expressed as follows:

$$c = \frac{dQ}{dA} \tag{9-36}$$

$$\mu = \frac{Q}{2BS_o} \tag{9-37}$$

where B is the top width of the water surface. The convective diffusion Equation 9-35 is the basis for the Muskingum-Cunge method.

(c) In the original Muskingum formulation, with lateral inflow, the continuity Equation 9-33) is discretized on the x-t plane (Figure 9-14) to yield:

$$Q_{j-1}^{n-1} = C_1 Q_j^{\,n} + C_2 Q_j^{\,n-1} + C_3 Q_{j-1}^{\,n} + C_4 Q_L \tag{9-38}$$

It is assumed that the storage in the reach is expressed as the classical Muskingum storage:

$$S = K\,[XI + (1-X)O] \tag{9-39}$$

Table 9-2
Working R&D Routing Example

Time hr	Inflow cfs	Average Inflow cfs	$\frac{K}{\Delta t} + 0.5(I_1 + I_2) - D_1$ cfs	D cfs	O cfs
	3,000		7,100	3,000	3,000
		3,130			
3	3,260		7,230	3,100	3,060
		3,445			
6	3,630		7,575	3,300	3,220
		3,825			
9	4,020		8,100	3,800	3,745
		4,250			
12	4,480		8,550	4,400	4,420

where

S = channel storage

K = cell travel time (seconds)

X = weighting factor

I = inflow

O = outflow

Therefore, the coefficients can be expressed as follows:

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

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

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

$$Q_L = q_L \Delta X$$

$$C_4 = \frac{2(\frac{\Delta t}{K})}{\frac{\Delta t}{K} + 2(1 - X)}$$

(d) In the Muskingum equation the amount of diffusion is based on the value of X, which varies between 0.0 and 0.5. The Muskingum X parameter is not directly related to physical channel properties. The diffusion obtained with the Muskingum technique is a function of how the equation is solved and is therefore considered numerical diffusion rather than physical. Cunge evaluated the diffusion that is produced in the Muskingum equation and analytically solved for the following diffusion coefficient:

$$\mu_n = c\ \Delta x \left(\frac{1}{2} - X \right) \tag{9-40}$$

In the Muskingum-Cunge formulation, the amount of diffusion is controlled by forcing the numerical diffusion to match the physical diffusion of the convective diffusion Equation 9-35. This is accomplished by setting Equations 9-37 and 9-40 equal to each other. The Muskingum-Cunge equation is therefore considered an approximation of the convective diffusion Equation 9-35. As a result, the parameters K and X are expressed as follows (Cunge 1969 and Ponce and Yevjevich 1978):

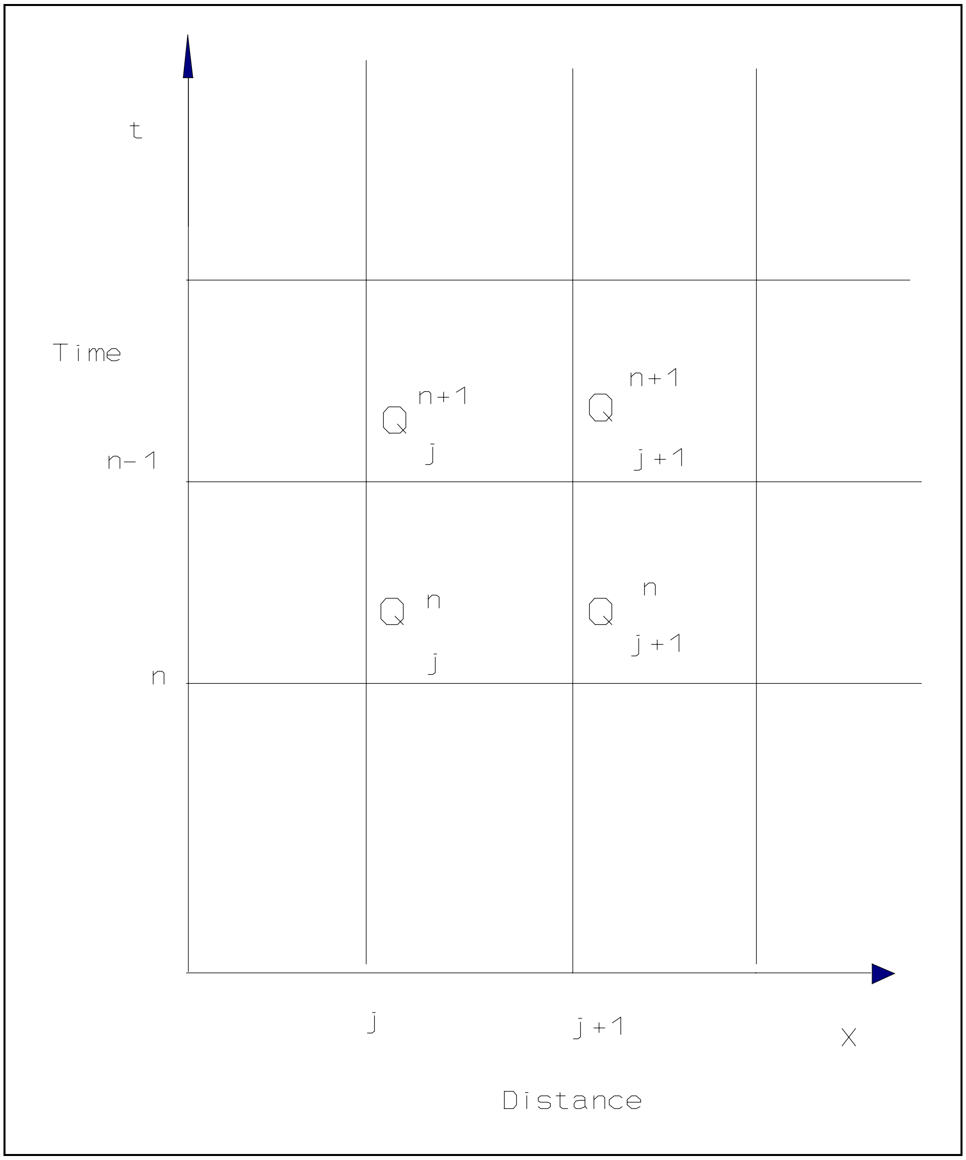

Figure 9-14. Discretization of the continuity equation on x-t plane

$$K = \frac{\Delta X}{c} \tag{9-41}$$

$$X = \frac{1}{2}\left(1 - \frac{Q}{BS_o c \Delta x}\right) \tag{9-42}$$

(2) Solution of the equations.

(a) The method is nonlinear in that the flow hydraulics (Q, B, c), and therefore the routing coefficients (C_1, C_2, C_3, and C_4) are recalculated for every Δx distance step and Δt time step. An iterative four-point averaging scheme is used to solve for c, B, and Q. This process has been described in detail by Ponce (1986).

(b) Values for Δt and Δx are chosen for accuracy and stability. First, Δt should be evaluated by looking at the following three criteria and selecting the smallest value: (1) the user-defined computation interval, (2) the time of rise of the inflow hydrograph divided by 20 ($^{Tr}/_{20}$), and (3) the travel time through the channel reach. Once Δt is chosen, Δx is defined as follows:

$$\Delta x = c\Delta t \tag{9-43}$$

but Δx must also meet the following criteria to preserve consistency in the method (Ponce 1983):

$$\Delta x < \frac{1}{2}\left(c\Delta t + \frac{Q_o}{BS_o c}\right) \tag{9-44}$$

where Q_o is the reference flow and Q_B is the baseflow taken from the inflow hydrograph as:

$$Q_o = Q_B + 0.50\ (Q_{peak} - Q_B)$$

(3) Data requirements.

(a) Data for the Muskingum-Cunge method consist of the following:

- Representative channel cross section.
- Reach length, L.
- Manning roughness coefficients, n (for main channel and overbanks).
- Friction slope (S_f) or channel bed slope (S_o).

(b) The method can be used with a simple cross section (i.e., trapezoid, rectangle, square, triangle, or circular pipe) or a more detailed cross section (i.e., cross sections with a left overbank, main channel, and a right overbank). The cross section is assumed to be representative of the entire routing reach. If this assumption is not adequate, the routing reach should be broken up into smaller subreaches with representative cross sections for each. Reach lengths are measured directly from topographic maps. Roughness coefficients (Manning's n) must be estimated for main channels as well as overbank areas. If information is available to estimate an approximate energy grade line slope (friction slope, S_f), that slope should be used instead of the bed slope. If no information is available to estimate the slope of the energy grade line, the channel bed slope should be used.

(4) Advantages and limitations. The Muskingum-Cunge routing technique is considered to be a nonlinear coefficient method that accounts for hydrograph diffusion based on physical channel properties and the inflowing hydrograph. The advantages of this method over other hydrologic techniques are: the parameters of the model are physically based, and therefore this method will make for a good ungauged routing technique; several studies have shown that the method compares very well with the full unsteady flow equations over a wide range of flow conditions (Ponce 1983 and Brunner 1989); and the solution is independent of the user-specified computation interval. The major limitations of the Muskingum-Cunge technique are that the method can not account for backwater effects, and the method begins to diverge from the complete unsteady flow solution when very rapidly rising hydrographs (i.e., less than 2 hr) are routed through flat channel sections (i.e., channel slopes less than 1 ft/mile). For hydrographs with longer rise times (T_r), the method can be used for channel reaches with slopes less than 1 ft/mile.

9-4. Applicability of Routing Techniques

a. Selecting the appropriate routing method. With such a wide range of hydraulic and hydrologic routing techniques, selecting the appropriate routing method for each specific problem is not clearly defined. However, certain thought processes and some general guidelines can be used to narrow the choices, and ultimately the selection of an appropriate method can be made.

b. Hydrologic routing method. Typically, in rainfall-runoff analyses, hydrologic routing procedures are utilized on a reach-by-reach basis from upstream to downstream. In general, the main goal of the rainfall-runoff study is to calculate discharge hydrographs at several locations in the watershed. In the absence of significant backwater effects, the hydrologic routing models offer the advantages of simplicity, ease of use, and computational efficiency. Also, the accuracy of hydrologic methods in calculating discharge hydrographs is normally well within the range of acceptable values. It should be remembered, however, that insignificant backwater effects alone do not always justify the use of a hydrologic method. There are many other factors that must be considered when deciding if a hydrologic model will be appropriate, or if it is necessary to use a more detailed hydraulic model.

c. Hydraulic routing method. The full unsteady flow equations have the capability to simulate the widest range of flow situations and channel characteristics. Hydraulic models, in general, are more physically based since they only have one parameter (the roughness coefficient) to estimate or calibrate. Roughness coefficients can be estimated with some degree of accuracy from inspection of the waterway, which makes the hydraulic methods more applicable to ungauged situations.

d. Evaluating the routing method. There are several factors that should be considered when evaluating which routing method is the most appropriate for a given situation. The following is a list of the major factors that should be considered in this selection process:

(1) Backwater effects. Backwater effects can be produced by tidal fluctuations, significant tributary inflows, dams, bridges, culverts, and channel constrictions. A floodwave that is subjected to the influences of backwater will be attenuated and delayed in time. Of the hydrologic methods discussed previously, only the modified puls method is capable of incorporating the effects of backwater into the solution. This is accomplished by calculating a storage-discharge relationship that has the effects of backwater included in the relationship. Storage-discharge relationships can be determined from steady flow-water surface profile calculations, observed water surface profiles, normal depth calculations, and observed inflow and outflow hydrographs. All of these techniques, except the normal depth calculations, are capable of including the effects of backwater into the storage-discharge relationship. Of the hydraulic methods discussed in this chapter, only the kinematic wave technique is not capable of accounting for the influences of backwater on the floodwave. This is due to the fact that the kinematic wave equations are based on uniform flow assumptions and a normal depth downstream boundary condition.

(2) Floodplains. When the flood hydrograph reaches a magnitude that is greater than the channels carrying capacity, water flows out into the overbank areas. Depending on the characteristics of the overbanks, the flow can be slowed greatly, and often ponding of water can occur. The effects of the floodplains on the floodwave can be very significant. The factors that are important in evaluating to what extent the floodplain will impact the hydrograph are the width of the floodplain, the slope of the floodplain in the lateral direction, and the resistance to flow due to vegetation in the floodplain. To analyze the transition from main channel to overbank flows, the modeling technique must account for varying conveyance between the main channel and the overbank areas. For 1-D flow models, this is normally accomplished by calculating the hydraulic properties of the main channel and the overbank areas separately, then combining them to formulate a composite set of hydraulic relationships. This can be accomplished in all of the routing methods discussed previously except for the Muskingum method. The Muskingum method is a linear routing technique that uses coefficients to account for hydrograph timing and diffusion. These coefficients are usually held constant during the routing of a given floodwave. While these coefficients can be calibrated to match the peak flow and timing of a specific flood magnitude, they can not be used to model a range of floods that may remain in bank or go out of bank. When modeling floods through extremely flat and wide floodplains, the assumption of 1-D flow in itself may be inadequate. For this flow condition, velocities in the lateral direction (across the floodplain) may be just as predominant as those in the longitudinal direction (down the channel). When this occurs, a two-dimensional (2-D) flow model would give a more accurate representation of the physical processes. This subject is beyond the scope of this chapter. For more information on this topic, the reader is referred to EM 1110-2-1416.

(3) Channel slope and hydrograph characteristics. The slope of the channel will not only affect the velocity of the floodwave, but it can also affect the amount of attenuation that will occur during the routing process. Steep channel slopes accelerate the floodwave, while mild channel slopes are prone to slower velocities and greater amounts of hydrograph attenuation. Of all the routing methods presented in this chapter, only the complete unsteady flow equations are capable of routing floodwaves through channels that range from steep to extremely flat slopes. As the channel slopes become flatter, many of the methods begin to break down. For the simplified hydraulic methods, the terms in the momentum equation that were excluded become more important in magnitude as the channel slope is decreased. Because of this, the range of applicable channel slopes decreases with the number of terms excluded from the momentum equation. As a rule of thumb, the kinematic wave equations should only be applied to relatively steep channels (10 ft/mile or greater). Since the diffusion wave approximation includes the pressure differential term in the momentum equation, it is applicable to a wider range of slopes than the kinematic wave equations. The diffusion wave technique can be used to route slow rising floodwaves through extremely flat slopes. However, rapidly rising floodwaves should be limited to mild to steep channel slopes (approximately 1 ft/mile or greater). This limitation is due to the fact that the acceleration terms in the momentum equation increase in magnitude as the time of rise of the inflowing hydrograph is decreased. Since the diffusion wave method does not include these acceleration terms, routing rapidly rising hydrographs through flat channel slopes can result in errors in the amount of diffusion that will occur. While "rules of thumb" for channel slopes can be established, it should be realized that it is the combination of channel slope and the time of rise of the inflow hydrograph together that will determine if a method is applicable or not.

(a) Ponce and Yevjevich (1978) established a numerical criteria for the applicability of hydraulic routing techniques. According to Ponce, the error due to the use of the kinematic wave model (error in hydrograph peak accumulated after an elapsed time equal to the hydrograph duration) is within 5 percent, provided the following inequality is satisfied:

$$\frac{TS_o u_o}{d_o} \geq 171 \tag{9-45}$$

where

T = hydrograph duration, in seconds

S_o = friction slope or bed slope

u_o = reference mean velocity

d_o = reference flow depth

When applying Equation 9-45 to check the validity of using the kinematic wave model, the reference values should correspond as closely as possible to the average flow conditions of the hydrograph to be routed.

(b) The error due to the use of the diffusion wave model is within 5 percent, provided the following inequality is satisfied:

$$TS_o \left(\frac{g}{d_o} \right)^{1/2} \geq 30 \tag{9-46}$$

where g = acceleration of gravity. For instance, assume S_o = 0.001, u_o = 3 ft/s, and d_o = 10 ft. The kinematic wave model will apply for hydrographs of duration larger than 6.59 days. Likewise, the diffusion wave model will apply for hydrographs of duration larger than 0.19 days.

(c) Of the hydrologic methods, the Muskingum-Cunge method is applicable to the widest range of channel slopes and inflowing hydrographs. This is due to the fact that the Muskingum-Cunge technique is an approximation of the diffusion wave equations, and therefore can be applied to channel slopes of a similar range in magnitude. The other hydrologic techniques use an approximate relationship in place of the momentum equation. Experience has shown that these techniques should not be applied to channels with slopes less than 2 ft/mi. However, if there is gauged data available, some of the parameters of the hydrologic methods can be calibrated to produce the desired attenuation effects that occur in very flat streams.

(4) Flow networks. In a dendritic stream system, if the tributary flows or the main channel flows do not cause significant backwater at the confluence of the two streams, any of the hydraulic or hydrologic routing methods can be applied. If significant backwater does occur at the confluence of two streams, then the hydraulic methods that can account for backwater (full unsteady flow and diffusion wave) should be applied. For full networks, where the flow divides and possibly changes direction during the event, only the full unsteady flow equations and the diffusion wave equations can be applied.

(5) Subcritical and supercritical flow. During a flood event, a stream may experience transitions between subcritical and supercritical flow regimes. If the supercritical flow reaches are long, or if it is important to calculate an accurate stage within the supercritical reach, the transitions between subcritical and supercritical flow should be treated as internal boundary conditions and the supercritical flow reach as a separate routing section. This is normally accomplished with hydraulic routing methods that have specific routines to handle supercritical flow. In general, none of the hydrologic methods have knowledge about the flow regime (supercritical or subcritical), since hydrologic methods are only concerned with flows and not stages. If the supercritical flow reaches are short, they will not have a noticeable impact on the discharge hydrograph. Therefore, when it is only important to calculate the discharge hydrograph, and not stages, hydrologic routing methods can be used for reaches with small sections of supercritical flow.

(6) Observed data. In general, if observed data are not available, the routing methods that are more physically based are preferred and will be easier to apply. When gauged data are available, all of the methods should be calibrated to match observed flows and/or stages as best as possible. The hydraulic methods, as well as the Muskingum-Cunge technique, are considered physically based in the sense that they only have one parameter (roughness coefficient) that must be estimated or calibrated. The other hydrologic methods may have more than one parameter to be estimated or calibrated. Many of these parameters, such as the Muskingum X and the number of subreaches (NSTPS), are not related directly to physical aspects of the channel and inflowing hydrograph. Because of this, these methods are generally not used in ungauged situations. The final choice of a routing model is also influenced by other factors, such as the required accuracy, the type and availability of data, the type of information desired (flow hydrographs, stages, velocities, etc.), and the familiarity and experience of the user with a given method. The modeler must take all of these factors

into consideration when selecting an appropriate routing technique for a specific problem. Table 9-3 contains a list of some of the factors discussed previously, along with some guidance as to which routing methods are appropriate and which are not. This table should be used as guidance in selecting an appropriate method for routing discharge hydrographs. By no means is this table all inclusive.

Table 9-3
Selecting the Appropriate Channel Routing Technique

Factors to consider in the selection of a routing technique.	Methods that are appropriate for this specific factor.	Methods that are not appropriate for this factor.
1. No observed hydrograph data available for calibration.	* Full Dynamic Wave * Diffusion Wave * Kinematic Wave * Muskingum-Cunge	* Modified Puls * Muskingum * Working R&D
2. Significant backwater that will influence discharge hydrograph.	* Full Dynamic Wave * Diffusion Wave * Modified Puls * Working R&D	* Kinematic Wave * Muskingum * Muskingum-Cunge
3. Flood wave will go out of bank into the flood plains.	* All hydraulic and hydrologic methods that calculate hydraulic properties of main channel separate from overbanks.	* Muskingum
4. Channel slope > 10 ft/mile and $\frac{TS_o u_o}{d_o} \geq 171$	* All methods presented	* None
5. Channel slopes from 10 to 2 ft/mile and $\frac{TS_o u_o}{d_o} < 171$	* Full Dynamic Wave * Diffusion Wave * Muskingum-Cunge * Modified Puls * Muskingum * Working R&D	* Kinematic Wave
6. Channel slope < 2 ft/mile and $TS_o \left(\frac{g}{d_o}\right)^{1/2} \geq 30$	* Full Dynamic Wave * Diffusion Wave * Muskingum-Cunge	* Kinematic Wave * Modified Puls * Muskingum * Working R&D
7. Channel slope < 2 ft/mile and $TS_o \left(\frac{g}{d_o}\right)^{1/2} < 30$	* Full Dynamic Wave	* All others