

Estimating the Variation of Travel Time in Rivers by Use of Wave Speed and Hydraulic Characteristics

Water-Resources Investigations Report 00-4187

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By Harvey E. Jobson

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LIST OF SYMBOLS

A	cross-sectional area of flow
A_d	cross-sectional area downstream of wave
A_u	cross-sectional area upstream of wave
A_0	inactive flow area, or average cross-sectional area at zero flow
A_1	hydraulic-geometry coefficient for area
A_2	hydraulic-geometry exponent for area
C	celerity of moving wave
n	Manning's resistance coefficient
n_1	constant in relation between Manning's n and Q
n_2	exponent in relation between Manning's n and Q
P	wetted perimeter
Q	discharge
Q_{bf}	discharge of river flowing bank full
Q_d	discharge downstream of wave
Q_u	discharge upstream of wave
S	slope
W	top width of channel
W_{bf}	average top width of a river flowing bank full
W_1	hydraulic-geometry coefficient for width
W_2	hydraulic-geometry exponent for width
Δt	time increment
ΔX	distance moved by wave in time Δt

CONVERSION FACTORS

Multiply	By	To obtain
inch	25.4	millimeter (mm)
foot(ft)	0.3048	meter (m)
foot/second (ft/s)	0.3048	meters/second (m/s)
mile (mi)	1.609	kilometer (km)
square foot (ft ²)	0.09290	square meter (m ²)
square mile (mi ²)	2.590	square kilometer (km ²)
cubic foot (ft ³)	0.02832	cubic meter (m ³)
cubic foot per second (ft ³ /s)	0.02832	cubic meter per second (m ³ /s)

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Abstract

Predicting the effect of a pollutant spill on downstream water quality is primarily dependent on the speed of movement downstream, the rate of longitudinal mixing, and the chemical or physical reactions that affect the total mass of pollutant in the water column. Of these three processes, the speed of movement is probably the most important and most difficult to predict. A time-of-travel study often is conducted to quantify the mixing properties and speed of movement of a pollutant, but these results are expensive and applicable only for the flow conditions under which they were obtained. The purpose of this report is to provide guidance on extrapolating travel-time information obtained at one flow to a different flow in the same reach of river.

It will be shown that in many cases a time series of discharge data (such as provided by a U.S. Geological Survey stream-gaging station) at two or more points along the river can provide a reasonable basis for this extrapolation. Direct application of resistance equations, such as the Manning equation, provides very poor estimates of the variation of travel time with discharge.

For the 60 reaches on 12 rivers studied here, the accuracy of resistance equations could be greatly improved by assuming the total flow area is composed of two parts, an active and inactive area. An accuracy of about 10 percent, in extrapolating travel times from one within-bank flow to another within-bank flow, could be obtained by assuming the inactive area to be constant and predicting the active flow by use of a constant resistance coefficient. A Manning resistance coefficient (n) of 0.035 could be used for 50 of the 51 reaches with slopes greater than 0.0002 meter/meter. The predicted travel times were not very sensitive to the assumed values of bed slope or channel width. For slopes less than 0.0002, accurate predictions could be made by assuming the inactive flow area to be zero and the Manning coefficient to be a constant, but less than 0.035.

Introduction

Predicting the effect of a pollutant spill on the downstream water quality is a complex problem. In general, mathematical models are available to address the transport, dispersion, and chemical reactions that may occur, but the model accuracy is limited by the lack of reliable basic physical data, especially flow area and mixing coefficients.

Predicting the effect of a pollutant spill is primarily dependent on the ability to predict the speed of movement downstream, the rate of longitudinal mixing, and the chemical or physical reactions that affect the total mass of pollutant in the water column. It is often acceptable to assume the pollutant is conservative, to predict the worst-case scenario. The best way to obtain speed and mixing data is by conducting a time-of-travel study. These studies are expensive, however, and the results only apply to flow conditions that existed during the study. This report considers procedures for extrapolating time-of-travel results from one within-bank flow to a higher or lower within-bank flow.

A fundamental building block for predicting the effect of an accidental spill is predicting the speed of movement and attenuation of the unit-peak concentration resulting from an instantaneous spill. The unit-peak concentration is defined as the a million times the peak concentration of an instantaneous spill times the river discharge divided by the mass of pollutant to pass the cross section (Jobson, 1996, p. 6). If the mixing processes are interpreted by use of the Fickian theory of diffusion, which is the most common approach, the unit-peak concentration attenuates in proportion to the square root of travel time. Data

collected from nearly a hundred streams and rivers indicate that the unit-peak concentration tends to attenuate in proportion to travel time to the 0.89 power (Jobson, 1996).

The cross-sectional mean river velocity that controls the travel time is, therefore, a primary variable that needs to be predicted before any reasonable estimates can be made about pollutant concentrations. Unfortunately, the prediction of mean water velocity in long reaches of a river has proven difficult. Even the measurement of travel time through a river reach requires special techniques (Wilson, and others, 1986; Kilpatrick and Wilson, 1989). These measurements generally involve a time-of-travel study where a slug of dye is injected at one cross section and its arrival time is monitored at downstream sites. The U.S. Geological Survey (USGS) has conducted many time-of-travel studies throughout the United States, and Jobson (1996) has compiled references to many of these studies. Several velocity-prediction equations have been proposed, but the accuracy of these equations is poor (Boning, 1974; Eikenberry and Davis, 1976; Graf, 1986, Jobson, 1996).

A time-of-travel study gives an accurate measure of the reach-averaged water velocity at the river discharge that existed during the study. Because the water velocity varies with discharge, a means of extrapolating the velocity from one flow to another is needed. The purpose of this report is to apply principles developed in the fields of geomorphology and uniform progressive flow to improve the prediction of water velocity as a function of river discharge. Only the change in velocity with flow is addressed in this report, thus the velocity at some reference flow is still required.

A brief discussion of the concepts and principles to be used will be given first, followed by the application of these concepts, which will be illustrated using time-of-travel data collected in several rivers. It will be shown that an analysis of discharge records at each end of the reach in question can provide accurate estimates of the variation of water velocity with discharge. The USGS maintains discharge records at about 7,000 locations within the United States. If these data are not available for the river reach in question, then the extrapolation must be accomplished by use of a resistance type equation. It will be shown that the effective resistance coefficient is much more predictable when the flow area is considered to consist of an active and inactive part. The accuracy of the recommended procedures will be demonstrated and contrasted to predictions made using traditional applications of the resistance equation. One of the goals of this report is to demonstrate how this information can be used to improve estimates of the variation of travel time with discharge for dissolved constituents.

Working Concepts

Some physical concepts that are useful in predicting the variation of average stream velocity with river discharge are reviewed below.

Hydraulic Geometry

When rafting the Colorado River through the Grand Canyon, one is struck by the extreme variability of the flow, which includes rapids and pools, backwater eddies and central currents. When flying over the Grand Canyon, however, the river appears uniform and one-dimensional in nature. Geomorphology is a field of study that specializes in viewing the extreme variability of nature and estimating general trends. A geomorphologist tends to view a river from a distant perspective so that the general trends are apparent. An engineer or modeler, on the other hand, tends to view a river close up, to capture all of the detail. Sometimes a more distant perspective would be helpful, especially when attempting to predict the effect of pollutant spills.

Much geomorphic information suggest that, in an average sense, the top-width of a natural channel, W , can be approximated by an equation of the form:

$$W = W_1 \cdot Q^{W_2}, \quad (1)$$

in which W_1 and W_2 are constants called the hydraulic geometry coefficient and exponent for width, respectively, and Q is the river discharge. The value of the unitless exponent W_2 is generally found to range from 0.1 to 0.4 with a typical value of 0.26 (Leopold and Miller, 1956; Stall and Yang, 1970; Jobson, 1989). Hydraulic geometry exponents have been found to maintain relatively consistent values both at a site and between river reaches (Beven and Kirkby 1993).

Rivers and streams form their own channel by eroding the bed and the banks until equilibrium exists between the power of the flow to remove material and the supply and resistance of the material to be moved. Large flows have greater power to erode material, but these flows occur infrequently. Most of the time, a river occupies a channel that has the capacity to carry a much larger flow. The flow that is compatible with the size of the channel is called the channel-forming discharge. The channel-forming discharge is generally considered to be the flow that can just be carried within the banks of the river. In other words, the channel-forming discharge is the flow that occurs just before water spills onto the flood plane. Kilpatrick and Barnes (1964) and Wolman and Leopold (1957) suggest that the channel-forming discharge is the flow that has a recurrence interval of about 2 years. Generally speaking, geomorphic relations only apply for within-bank flow.

The discharge with a recurrence interval of 2 years can be estimated for any basin in the United States from regional regression equations (Jennings and others, 1994). Areas of drainage basins, which are the dominant variable in regression equations, have been tabulated by Seaber and others (1984) for all but the smallest basins within the United States.

Using data from the Missouri River Basin, Osterkamp (1980) found that channel geometry is somewhat influenced by the particle-size distribution of the streambed and bank material. He developed empirical equations for separate particle-size distributions that relate channel width to discharge. The average of these equations is:

$$W_{bf} = 7.5Q_{bf}^{0.56}, \quad (2)$$

in which W_{bf} is the average top width of the channel, in meters, when the river is flowing at bank-full discharge (about the 2-year flow), Q_{bf} in m^3/s . Equation 2 should not be considered as a substitute for field data, but it can provide a reasonable estimate of bank-full top width when field data are not available. For rivers with no width information available, the value of $W1$ in equation 1 can be estimated by assuming a value for $W2$, computing the bank-full width from equation 2, and solving equation 1 for $W1$ with $W = W_{bf}$, and $Q = Q_{bf}$.

Geomorphologists also suggest that, in an average sense, the cross-sectional area, A , of a natural channel can be approximated by an equation of the form:

$$A = A1 \cdot Q^{A2}, \quad (3)$$

in which A is the cross-sectional area of the river flow, $A1$ and $A2$ are constants called the hydraulic-geometry coefficient and exponent for area, respectively, and Q is the river discharge. Theoretically, the unitless $A2$ value can range from 0 to 1, but its value is usually between 0.5 and 0.8 with an average of 0.66 (Leopold and Maddock, 1953; Stall and Yang, 1970; Boning, 1974; Boyle and Spahr, 1985; Jobson, 1989). Its value tends to be consistent for different reaches of a given river. Equations 1, 2, and 3 are not homogeneous; thus, the units of $A1$ and $W1$ depend on the units selected for A , W and Q .

Equations 1 and 3 are developed for within-bank flow. As the flow covers the flood plain, the width and area increases more rapidly with discharge than for within-bank flow. The values of $W2$ and $A2$, therefore, usually increase as the flood plain is covered.

Flow resistance

Resistance in open channels is typically quantified by using an empirical equation of either the Manning or Chezy type. In the United States, the Manning form of the equation is most common. The Manning equation relates the discharge, Q , to measurable quantities as:

$$Q = \frac{1}{n} A \left(\frac{A}{P} \right)^{2/3} \sqrt{S}, \quad (4)$$

in which n is the Manning's resistance coefficient, P is wetted perimeter, and S is slope. The slope of the energy grade line should be used in the equation if non-uniform flow is being considered. The equation is often written in terms of the bed slope, in which case it should be applied for steady uniform flow only. The Manning's resistance coefficient is considered to be dimensionless; therefore, when the English system of units is used, the 1 in the numerator needs to be replaced with 1.49 to make the equation homogeneous.

In most natural channels, the width is much larger than the depth. Thus, for natural channels the wetted perimeter can be closely approximated by the top width, and the equation can be approximated as:

$$Q = \frac{1}{n} \frac{A^{5/3}}{W^{2/3}} \sqrt{S}. \quad (5)$$

Equation 5 will be used to represent the Manning equation throughout this report.

By solving equation 5 for the area in terms of the discharge:

$$A = (n^{0.6} S^{-0.3} W^{0.4}) Q^{0.6}, \quad (6)$$

it is seen that the Manning equation is consistent with the hydraulic-geometry equation 3. The value of A2 that is observed in the field is typically different from 0.6 because both flow resistance and width vary with discharge. If one assumes that the Manning resistance coefficient can be approximated from discharge by using a power form of the equation

$$n = n_1 Q^{n_2}, \quad (7)$$

in which n1 and n2 are constants, equations 1, 6, and 7 can be combined to yield:

$$A = (n_1^{0.6} S^{-0.3} W^{0.4}) Q^{(0.6+0.6n_2+0.4W^2)}. \quad (8)$$

For typical values of W2 and A2, 0.26 and 0.66 respectively, it is seen that n2 = - 0.073. A negative value of n2 implies that the Manning n decreases with increasing discharge. This is usually rationalized by stating that the relative roughness decreases as the water depth increases. It is seen from equation 8 that the hydraulic-geometry parameters for area include the combined effects of width, slope, and flow resistance. The hydraulic-geometry parameters tend to be consistent from reach to reach for a river, whereas the width, depth, and roughness tend to be quite variable. For this reason, the hydraulic-geometry parameters tend to be more easily predicted and provide a reasonable means of estimating reach-averaged areas (velocities) and widths in a modeling framework.

The Manning equation is typically applied to uniform flow. To apply it to non-uniform flow, the reach is subdivided into small subreaches, and some means of averaging the cross-sectional properties is incorporated. For example, the average area in a subreach is estimated as either the arithmetic or geometric mean of the areas at each end of the subreach. The hydraulic-geometry approach, on the other hand, considers data to be reach-averaged values.

Uniformly Progressive Flow and Moving Waves

Chow (1959) defines uniformly progressive flow as having a stable wave profile that does not change in shape as it moves down the channel. One common type of uniformly progressive flow, which approximates most flood waves in natural channels, is the monoclinal rising wave (Chow, 1959). Kleitz (1877) developed the mathematical principle of uniformly progressive waves, whereas Seddon (1900) and Wilkinson (1945) showed that the principle is applicable to natural rivers. Wilkinson found that the mid-point of the rise or fall in stages were best suited for determining the velocity of an observed wave.

By considering the flow into and out of the control volume ABCD of figure 1, it is shown that:

$$Q_d \Delta t - Q_u \Delta t = (A_d - A_u) \Delta X,$$

or

$$C = \frac{\Delta X}{\Delta t} = \frac{Q_d - Q_u}{A_d - A_u} \Rightarrow \frac{\partial Q}{\partial A}, \quad (9)$$

in which C is the speed of the moving wave or celerity, ΔX is the distance moved by the wave in time Δt, Qu and Qd are discharge upstream and downstream of the wave, respectively, and Au and Ad are the cross-sectional areas upstream and downstream of the wave, respectively. Differentiating equation 3 and

inverting, it is seen that the wave speed is determined as:

$$C = \frac{Q^{(1-A^2)}}{A1 \cdot A2} \quad (10)$$

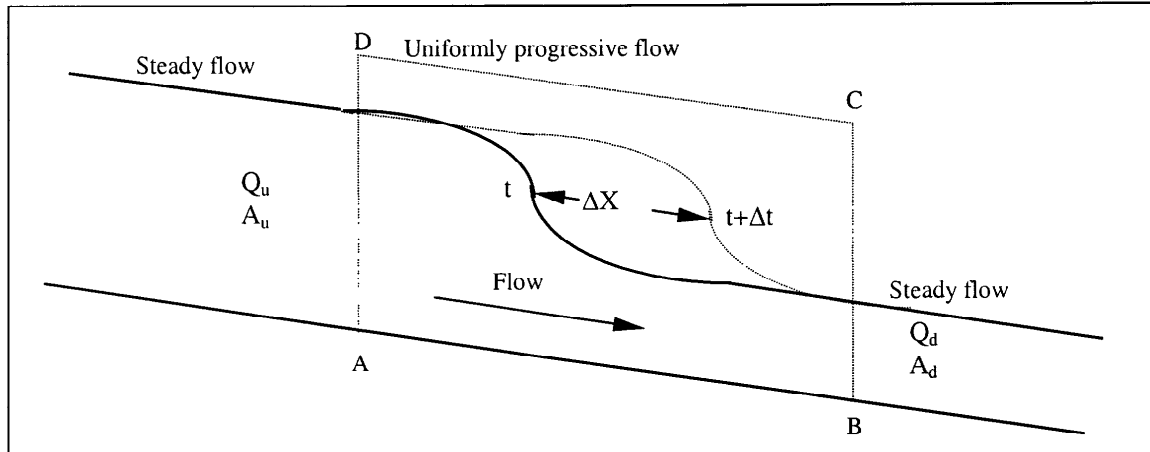


Figure 1. Schematic showing uniformly progressive flow and the relation of wave speed to change in flow area with discharge.

It can be seen from equation 9 that the speed of a flow wave is precisely controlled by the change in storage volume, or in the average area within a river reach. As shown in equation 8, the hydraulic-geometry parameters contain information about how the combined effects of flow resistance, slope, and width vary with discharge. Equation 10 illustrates that the speed of flow waves passing through a river reach contains that same information. The USGS continuously monitors the discharge at about 7,000 stream sites throughout the United States. The arrival time of flow waves can be determined from these records. If the amount of intervening flow is not too large, wave celerity between gages can be computed from the arrival times.

Inactive storage areas

Based on the information given above, the following steps could be followed to estimate the travel time of a dissolved constituent through a river reach between stream-gaging stations:

1. Measure the reach length, as well as the mean discharge and travel time of flow waves through the reach at two different discharges.
2. Calculate the wave celerity for each wave by dividing the reach length by the travel time of the wave. This yields two values of celerity, C , and discharge, Q .
3. By using equation 10, compute the values of $A1$ and $A2$ from the two values of celerity and discharge.
4. By using equation 3, compute the flow area for the discharge that occurs when the dissolved constituent is to be routed through the reach.
5. Divide the discharge by the flow area to determine the transport velocity and route the dissolved constituent using a model or the analytic solution to the convective diffusion equation.

When the above steps were applied to numerous time-of-travel studies, it was found that the predicted travel times were always substantially smaller than the measured values, indicating that equation 3 typically predicts areas that are smaller than observed in the field. It was finally concluded that equation 3 needed to be modified by adding a constant term:

$$A = A0 + A1 \cdot Q^{A2}, \quad (11)$$

in which $A0$ is the average inactive area or the cross-sectional area at zero flow. Similarly, the Manning

equation (Equations 6 or 8) should be modified to include the zero-flow cross-sectional area:

$$A = A_0 + (n^{0.6} \cdot S^{-0.3} \cdot W^{0.4}) \cdot Q^{0.6} \quad (12)$$

The last term in equations 11 and 12, the flow dependant term, is considered the active area, and the first term, A_0 , is considered to be the inactive area, or storage zone. Stall and Yang (1970) predicted travel times from hydraulic-geometry equations, like equation 3, and compared the results with time-of-travel studies conducted at various flow rates. Consistent with equations 11 and 12, they concluded that the accuracy of the approach decreased as the flow decreased (Stall and Yang, 1970, p. 38).

In natural channels the flow is seldom uniform in either width or depth. Rather, the flow tends to consist of pools that contain rather slow moving water separated by riffles, which are relatively high points in the bed. If the inflow to a natural channel were stopped and if water did not seep into the bed or evaporate, then the river typically would not drain completely. Instead, there would be periodic pools of stagnant water. These pools are easily recognized as an ephemeral stream dries up. Especially at low flow, the water level in a stream is controlled by flow over the riffles and thus, the overall "flow resistance" is insensitive to the depth of the pools; that is, the inactive area. Travel time of a dissolved constituent through the reach, however, is sensitive to the depth of the pools, because it influences the volume of water in the reach.

The equations 1, 11 and 12 are considered to be representative of area or width when averaged over a distance of several pool/riffle sequences. They may not be representative of the area or width at a specific cross section. Thus, the value of A_0 in equations 11 and 12 should be considered as the volume of water per unit length that would not drain if the flow were shut off. The second term on the right side of equations 11 and 12 is considered to represent the active area of flow, and this is the part of the flow that is influenced by flow resistance and that influences the wave celerity. Because the value of A_0 is a constant, the wave speed is independent of A_0 , and equation 10 remains valid.

It will be shown below that when the values of A_1 and A_2 are determined from wave speeds in a channel, and when A_0 is determined from the travel time at a specific discharge, equation 11 provides close predictions of the flow area as a function of discharge. Likewise, it will be shown that when the inactive area, A_0 , is assumed to be zero and the Manning coefficient is determined for a reach of river from equation 5 or 6 by using the transport velocity (area), slope and width, only poor estimates of the travel time at another flow are possible without major adjustments to the Manning coefficient. Improved estimates of transport velocity can be obtained by using equation 12 and a constant Manning coefficient.

Extrapolation of Transport Velocity Using Wave Speeds

Four examples are provided to illustrate how stream-gaging information can be used to extrapolate transport velocities to different flows and to demonstrate the accuracy that might be obtained. In each example the only hydraulic data to be used is a continuous record of river discharge at two points, a known distance apart. These data will be used to measure the variation of wave speed with discharge, and thus, to compute the values of A_1 and A_2 in equation 11. All examples are for stream reaches where at least two time-of-travel dye studies have been conducted. The low-flow dye study will then be used to "calibrate" the reach by determining the average transport velocity, and thus, the total flow area and value of A_0 in equation 11. The predictive ability of the equation will then be tested by comparing the predicted and measured travel times obtained at other flows.

New River, Hinton to Thurmond, West Virginia

The New River flows northward from its headwaters in North Carolina, through western Virginia, and into south-central West Virginia, where it joins the Gauley River to form the Kanawha River. The 60.51-km reach, between Hinton and Thurmond, West Virginia, figure 2, is within the New River Gorge National River and is widely used for white-water rafting. According to the USGS web site at <http://waterdata.usgs.gov/nwis-w/WV/>, the drainage area at Hinton (Station 03184500) is 16,203 km², and at Thurmond (Station 03185400) it is 20,544 km². The largest drainage into this reach of the New River Gorge is about 350 km² and drainages generally are less than 15 km² (Wiley, 1993).

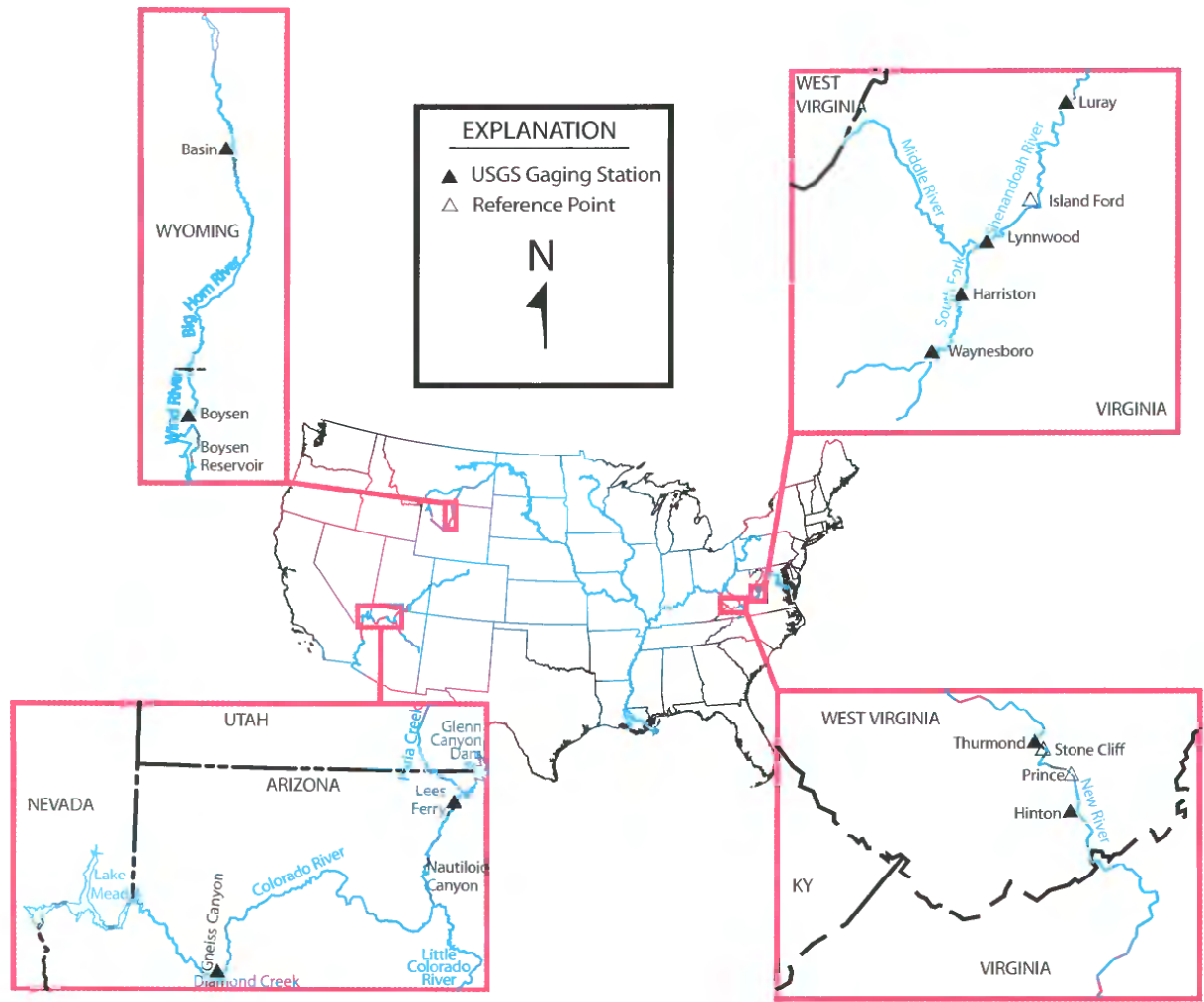


Figure 2. Gaging stations and reference points for rivers where wave celerity and travel time are computed.

The discharge at the two gaging stations starting on November 1, 1986, is shown in figure 3. The circular symbols define the time when a wave is assumed to have passed a site. The wave passage time is defined as follows: The discharge before and after the rise is estimated. Then, the time when the discharge is equal to the average of the beginning and ending flows is defined as the passage time. The wave celerity and average discharge for each of the seven waves is listed in table 1.

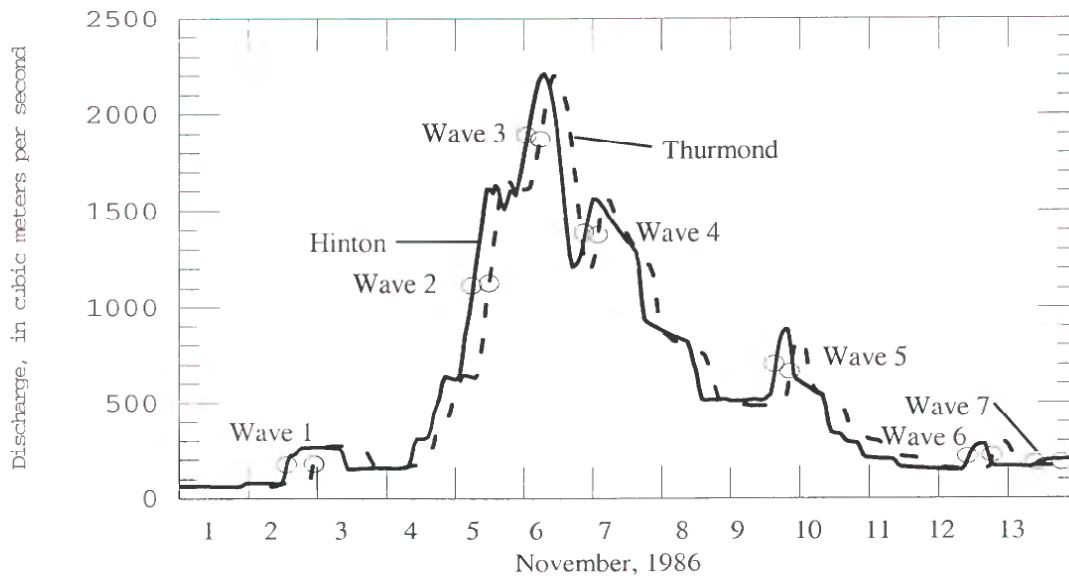


Figure 3. Discharge in the New River at Hinton and Thurmond, West Virginia. Wave passage times indicated by circles.

The wave speeds in table 1 are plotted as a function of discharge on figure 4. The line on figure 4 represents a least squares fit of the points, which is defined by the equation:

$$C = 0.428 \cdot Q^{0.281}, \quad (13)$$

in which C is wave celerity in m/s, and Q is discharge in m^3/s . The arrows indicate discharges where time-of-travel dye studies were conducted.

From equations 10, 11, and 13, it is seen that the flow area of the New River should be represented by:

$$\Lambda = \Lambda_0 + 3.25 \cdot Q^{0.719}. \quad (14)$$

Table 1. Celerity and flow for waves in the New River between Hinton and Thurmond, West Virginia

Point	Time, in Hours	Flow, in cubic meters per second	Celerity, in Meters per second
Hinton	37.31	175.3	
Average		177.2	1.75
Thurmond	46.93	179.2	
Hinton	101.85	1115.9	
Average		1122.4	2.82
Thurmond	107.82	1128.8	
Hinton	120.79	1895.9	
Average		1884.4	3.43
Thurmond	125.68	1872.8	
Hinton	140.87	1390.0	
Average		1383.0	3.41
Thurmond	145.80	1376.1	
Hinton	207.19	702.4	
Average		682.1	3.03
Thurmond	212.74	661.8	
Hinton	274.01	216.5	
Average		219.7	1.98
Thurmond	282.50	222.9	
Hinton	297.47	182.4	
Average		184.2	1.85
Thurmond	306.57	185.9	

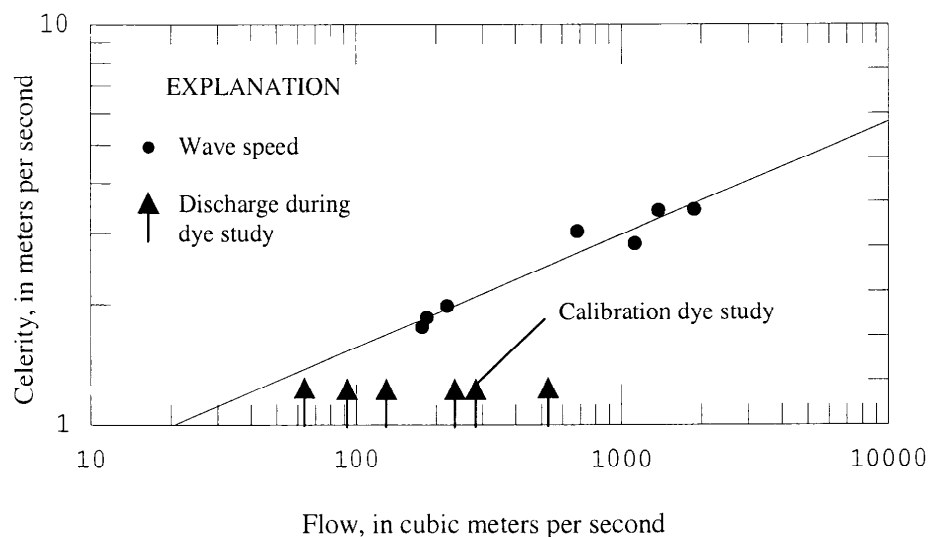


Figure 4. Wave celerity in the New River between Hinton and Thurmond, West Virginia, as a function of discharge.

Time-of-travel data obtained under steady-flow conditions in the New River are presented in table 2. According to Appel and Moles (1987), the distance is 21.7 km from Sandstone to Prince and 20.0 km from Prince to Stone Cliff. Sandstone is 5.2 km below the Hinton gage. Wiley and Appel (1989) did not specify the exact date of the two studies they reported, thus these studies are identified by the date of the report.

Table 2. Time-of-travel data for the New River between Sandstone and Stone Cliff, West Virginia (from Appel and Moles (1987) and Wiley and Appel (1989))
[m³/s, cubic meters per second]

Date of study	Sandstone Discharge m ³ /s	Travel Time Hours	Prince Discharge m ³ /s	Travel Time Hours	Stone Cliff Discharge m ³ /s
August 14, 1985	62.3	20.0	62.3		
October 24, 1985			90.6	13.0	90.6
1989 (report date)	127.4	11.5	127.4		
1989 (report date)	229.4	7.8	232.2		
May 15, 1986	263.4	7.0	280.3	6.4	297.3
November 8, 1985	518.2	4.8	526.7	4.0	538.0

The May 15, 1986 study was selected for computing the cross-sectional area at zero flow because it had the smallest discharge to span both subreaches. The total travel time between Sandstone and Stone Cliff is 13.4 hours; therefore, the average dye velocity is 0.864 m/s over this 41.7-km reach. Assuming an average discharge of 280.3 m³/s, which is the average of the values measured at the three sites, the volume of water displaced in 13.4 hours is 13.52 x10⁶ cubic meters. This represents an average cross-sectional area along the 41.7-km reach of 324.3 m² at this flow. Solving equation 14 yields A₀ = 137.3 m².

The total area for each of the other flow conditions was then computed from the observed travel time and distance as above, and the predicted area was computed from equation 14 with the value of A₀ = 137.3 m². The results are compiled in table 3, along with the predicted travel time, as well as the percent error in the predicted travel time. The total and predicted areas are plotted on figure 5. As shown on figure 5, the projection of the total areas to a zero discharge would indicate a non-zero value of area. Therefore, the data on figure 5 support the general form of equations 11 and 12. Any resistance equation with a constant resistance coefficient and a zero value of A₀ would pass through the origin and likely provide only a poor estimate of the areas on figure 5 except near the flow at which it was calibrated. As indicated by the arrows on figure 4, travel times were extrapolated for much lower discharges than were used for the calibration or were available for the wave-celerity measurements.

Table 3. Area and travel time for the New River, between Sandstone and Stone Cliff, West Virginia
[m³/s, cubic meters per second; m², square meters; N/A, not applicable]

Average Discharge m ³ /s	Effective Area m ²	Predicted Area m ²	Predicted Travel-time Hours	Measured Travel-time Hour	Error in Percent
280.3	324.3	324.3	13.4	13.4	N/A
62.3	206.7	200.7	19.42	20.0	-2.9
90.6	212.0	220.3	13.51	13.0	3.9
127.4	243.1	243.4	11.51	11.5	0.1
230.8	298.7	299.9	7.83	7.8	0.4
527.6	400.8	431.9	9.48	8.8	+7.8

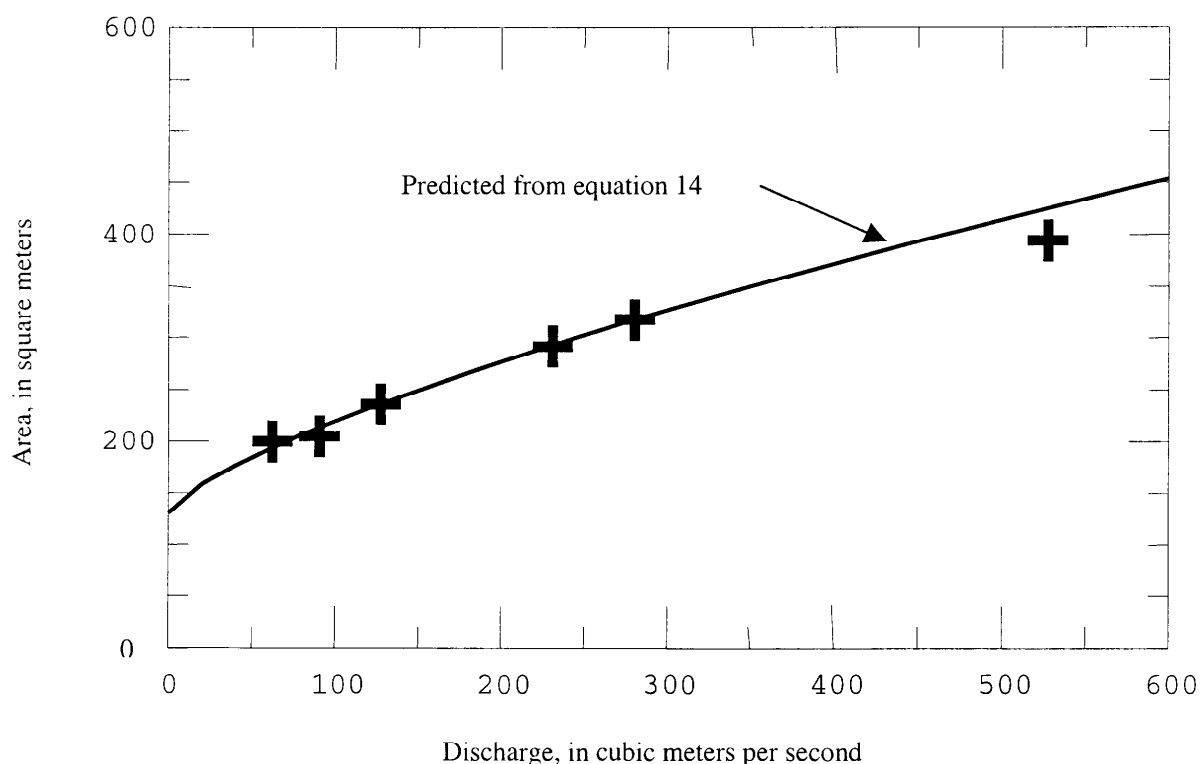


Figure 5. Variation of cross-sectional area with discharge for the New River, West Virginia. Symbols represent values computed from the observed travel time, and the line is computed from equation 14.

Wind/Bighorn River, Boysen Reservoir to Basin, Wyoming

The Wind/Bighorn River flows generally north from central Wyoming to Montana where it joins the Yellowstone River, figure 2. Stream-gaging stations are located on the Wind River just below Boysen Reservoir (Station 0625900) and on the Bighorn River at Basin, Wyoming (Station 06274300), about 70 km south of the Montana boarder and 165 km downstream from the Boysen gage. The name of the river changes from the Wind to Bighorn as it leaves the Wind River Canyon about 20 km downstream from Boysen Reservoir. The USGS web site at <http://waterdata.usgs.gov/nwis-w/WY> lists the drainage area of the Wind River below Boysen Reservoir as 19,945 km² and the gage datum as 1,404.70 m. Likewise the drainage area at Basin is 34,247 km², and the gage datum is 1,164.73 m.

A plot of discharge at the two gaging stations for two time periods when individual waves could be identified is shown in figure 6. The passage time of the four waves identified on figure 6, as well as the average discharge, and computed wave celerity for each wave are shown in table 4.

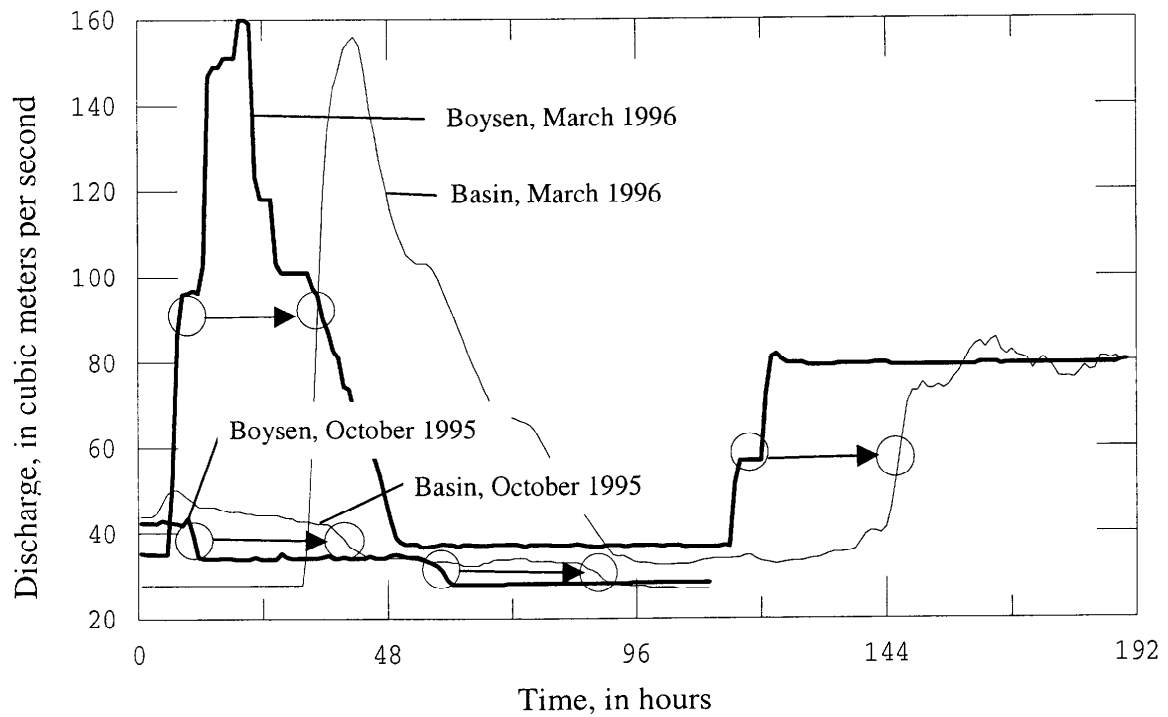


Figure 6. Discharge in Wind/Bighorn River below Boysen Reservoir and at Basin, Wyoming. Wave passage indicated by arrows.

Table 4. Celerity and flow for waves in the Wind/Bighorn River between Boysen and Basin, Wyoming

Point	Time Hours	Flow cubic meters/second	Celerity meters/second
Boysen	10.76	38.14	1.58
Average		38.14	
Basin	39.77	38.14	
Boysen	58.52	31.08	1.50
Average		30.63	
Basin	88.94	30.17	
Boysen	9.50	90.63	1.84
Average		91.19	
Basin	34.33	91.74	
Boysen	117.50	58.28	1.61
Average		57.73	
Basin	145.92	57.17	

Wave celerity as a function of discharge, along with the line that represents a least-squares fit to the data are shown in figure 7. The equation of the fitted line is:

$$C = 0.828 \cdot Q^{0.173} \quad (15)$$

From a comparison of equations 9, 11, and 15, it is seen that the area in the Wind/Bighorn River can be represented by:

$$A = A_0 + 1.46 \cdot Q^{0.827} \quad (16)$$

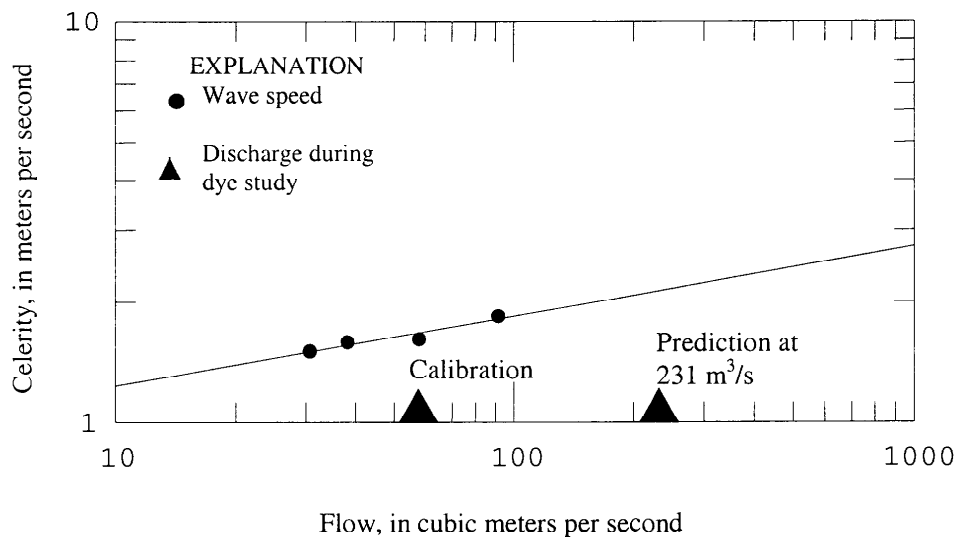


Figure 7. Wave celerity in Wind/Bighorn River between Boysen and Basin, Wyoming, as a function of discharge.

Time-of-travel data for the Wind/Bighorn River were collected on March 21-24 and June 29-30, 1971 (Lowham and Wilson, 1971), and Nordin and Sabol (1974) published the complete concentration as a function time data for the study. Dye was injected near the Boysen gage, and concentrations were measured at seven points downstream, the first being 9.2 km below the injection site and the last being at Greybull, Wyoming, which is 181.4 km downstream from the injection site and about 13 km downstream from the Basin gage. The average of the discharges measured at the injection point and seven data collection sites during the first and second injection were 57.5 and 231.3 m³/s, respectively (Lowham and Wilson, 1971, p. 6). The travel time of the peak concentration was 53.25 and 31.65 hours, respectively, for the low- and high-flow studies (Lowham and Wilson, 1971, p. 6). The mean velocities, during low and high flow, were 0.898 and 1.51 m/s, respectively, and the average areas were 64.0 and 153 m², respectively. By solving equation 16 with a flow of 57.5 m³/s and a total area of 64.0 m², the low-flow time-of-travel indicates a value of $A_0 = 22.4 \text{ m}^2$.

Solving equation 16 with $A_0 = 22.4 \text{ m}^2$ for a discharge of 231.3 m³/s yields a predicted area of 154 m² and a velocity of 1.50 m/s. This predicted velocity is 0.7 percent lower than the value measured by the high-flow time-of-travel study. Thus, the predicted travel time for the high-flow study would be 31.89 hours, which is 0.24 hours (0.8 percent) longer than the observed value. An accurate prediction was obtained even though the discharge during the predicted dye study was more than 2.5 times the highest flow where the wave celerity was observed (see figure 7), and the wave speeds were measured more than 25 years after the time-of-travel study was conducted.

South Fork Shenandoah River, Waynesboro to Luray, Virginia

The South Fork Shenandoah River provides an example application where flow records are more difficult to interpret. The Shenandoah River flows generally northeast near the western boundary of Virginia and joins the Potomac River at Harpers Ferry, West Virginia, figure 2. Travel-time data were collected from Waynesboro, Virginia to Harpers Ferry, West Virginia (Taylor and others, 1986), but only two reaches will be considered here. Many low-head dams are located along the South Fork Shenandoah River, although in the most cases these dams do not store sufficient water to substantially impede the movement of dye. Only the dam located about 8 km downstream from the Luray gage (Station 01629500), retards the flow noticeably, particularly during low flow (Taylor and others, 1986, p. 5). The dam probably had little or no effect on the flow upstream from the Luray gage. Table 5 lists the data for sites considered here.

Table 5. Time-of-travel data for the South Fork Shenandoah River between Waynesboro and Luray, Virginia, (Taylor and others, 1986)

[km, kilometer; m/m, meter fall per meter of distance; m ³ /s, cubic meters per second]						
Site	Distance below injection (km)	Slope (m/m)	Low Flow		High Flow	
			Discharge (m ³ /s)	Peak-to-peak Travel time (Hours)	Discharge (m ³ /s)	Peak-to-peak Travel time (Hours)
Waynesboro	2.25	0.0017	1.57	71	4.04	37.0
Harriston	32.99					
Lynnwood	49.99					
Island Ford	59.64					
Luray	118.23	.0013	9.56	108	22.26	55.7

The first reach extends from Waynesboro (Station 01626000) to Harriston (Station 01627500) where the drainage area increases from 329 to 549 km². Discharge hydrographs at Waynesboro and Harriston under low-flow conditions starting November 20, 1994 are shown in figure 8. Although it is generally recommended that the wave celerity be computed from the time lag of the midpoint of the rising hydrograph (Wilkinson, 1945), it is seen that the hydrograph at Harriston often contains a small rise before the main peak occurs. This is apparently the result of local runoff between the gages. It also is noted that there is substantial inflow between the two gages, which makes it difficult to identify the passage of specific waves. The inflow along the reach significantly decreases the apparent travel time of the midpoint of the rise and causes unreasonably large apparent celerity, especially for the rise occurring on December 10 (figure 8). The peaks that were judged acceptable for computing the wave celerity are indicated by circles on the figure. The wave labeled 2, which is based on the midpoint of the rise, was considered acceptable because the rise at Harriston had receded before the midpoint of the main wave arrived.

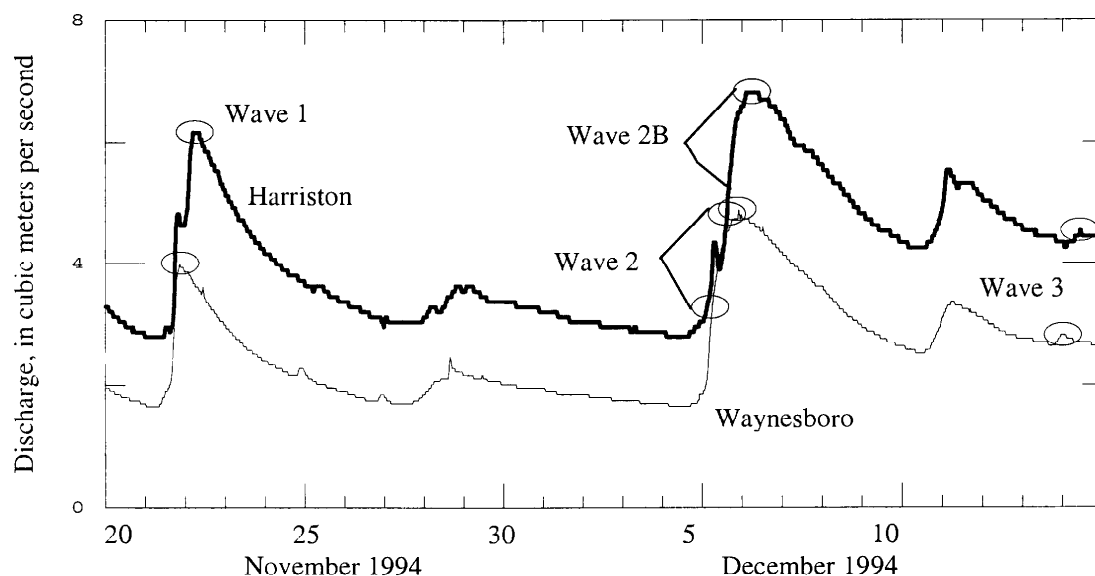


Figure 8. Low-flow discharge in the South Fork Shenandoah River at Waynesboro and Harriston, Virginia.

Discharge hydrographs at Waynesboro and Harriston under high-flow conditions starting June 22, 1995 are shown in figure 9. Again, the peaks were used to define the passage time of the waves, and the largest wave, which occurred on June 28, was not used because of the multiple peaks at Harriston, which occurred at 1-hour intervals.

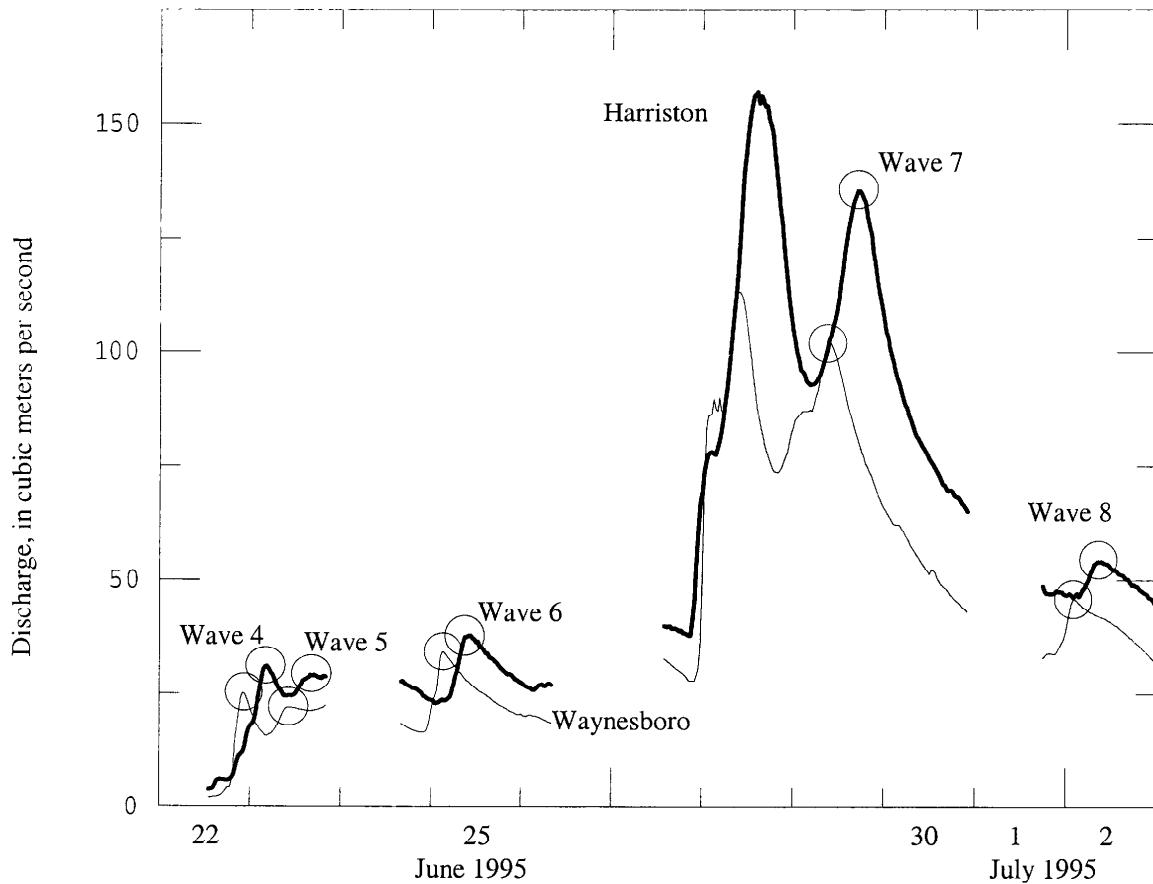


Figure 9. High-flow discharge in the South Fork Shenandoah River at Waynesboro and Harriston, Virginia.

Wave celerity as a function of discharge for the nine waves shown in figures 8 and 9 is shown in figure 10. A least squares fit of the points, which have considerable scatter, indicates that the wave celerity can be approximated using the equation:

$$C = 0.825 Q^{0.108} . \quad (17)$$

As before, the hydraulic geometry for the South Fork Shenandoah River between Waynesboro and Harriston becomes:

$$A = A_0 + 1.36 Q^{0.892} . \quad (18)$$

By use of the low-flow data in table 5, it was determined that the average velocity of the dye in this reach was 0.120 m/s at an average flow of 1.57 m³/s. This flow is much lower than the lowest discharge for which a wave speed is known (see the calibration arrow in figure 10). The average cross-sectional area, however, was 13.05 m², and the value of A₀ in equation 18 is 11.02 m². By extrapolating to a high flow of 4.04 m³/s, the area predicted by equation 18 is 15.75 m², giving a predicted travel time of 33.3 hours. The predicted value is 3.7 hours less than the measured value and represents an error of 10.0 percent. This is a larger error than observed for either of the other examples. The large inflow between the two gages is probably the major cause of the error. The inflow complicates the measurement of the wave speed and probably contributes to the scatter on figure 10 and errors in equation 17. Furthermore, the value of A₀ represents about 85 percent of the total area at low flow. This also may influence the accuracy, in addition to the extrapolation that was mostly outside the range of flows where the wave celerity was observed.

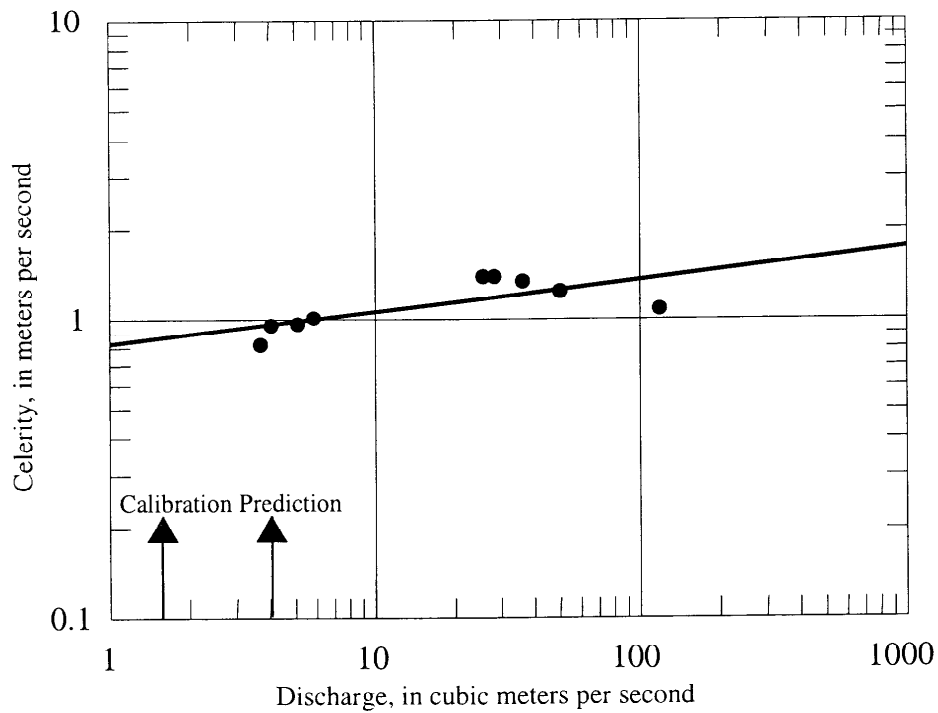


Figure 10. Wave celerity in South Fork Shenandoah River between Waynesboro and Harriston, Virginia, as a function of discharge.

A major tributary, the North River, joins the South Fork Shenandoah in the reach between Harriston and Lynnwood (Station 01628500), thus this reach was not used. The drainage area at Lynnwood, which is 2,808 km², is more than five times that at Harriston. Luray (Station 01629500) is the next stream-gaging station below Lynnwood and has a drainage area of 3,566 km². Dye concentrations were not available at Lynnwood but were measured at Island Ford, which is about 10 km downstream. Discharge hydrographs for selected storms at Lynnwood and Luray under low-flow conditions are shown in figure 11. The time of passage of the selected waves are indicated by circles. Hydrographs for high-flow conditions are shown in figure 12.

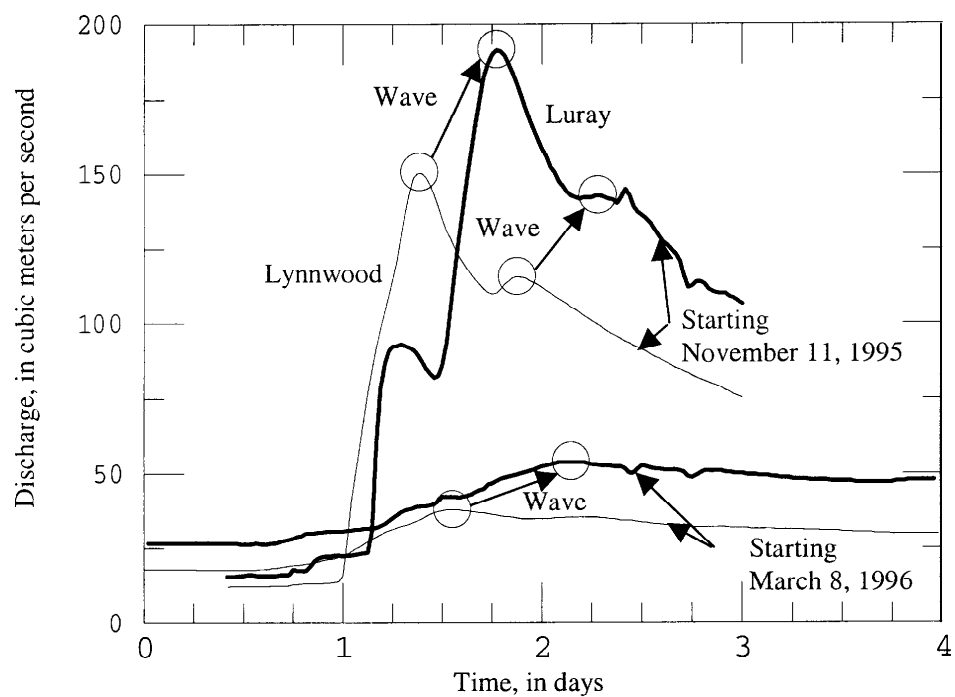


Figure 11. Low-flow discharge in the South Fork Shenandoah River at Lynnwood and Luray, Virginia.

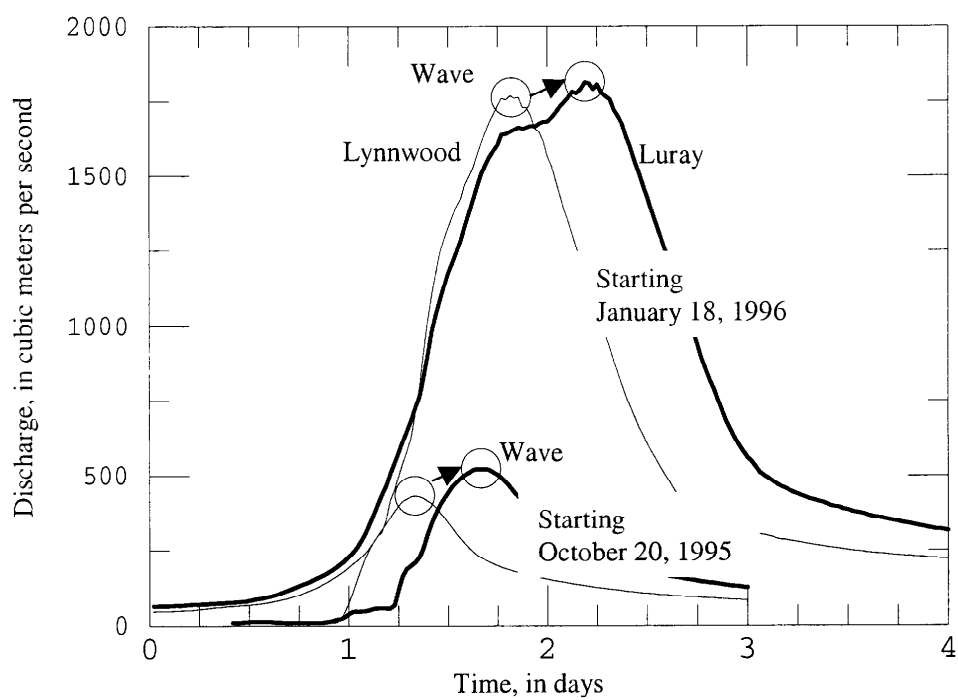


Figure 12. High-flow discharge in the South Fork Shenandoah River at Lynnwood and Luray, Virginia.

Wave celerity as a function of discharge along with a line fitted to the data is shown in figure 13. The equation of the fitted line is:

$$C = 0.990 Q^{0.122}. \quad (20)$$

As before, the hydraulic-geometry equation for the South Fork Shenandoah River between Lynnwood and Luray becomes:

$$A = A_0 + 1.15 Q^{0.878}. \quad (21)$$

By using the low-flow data in table 5, at an average flow of $9.56 \text{ m}^3/\text{s}$, the average velocity of the dye between Island Ford and Luray was calculated to be 0.151 m/s , thus the average cross-sectional area was calculated to be 63.3 m^2 , and the value of A_0 is 55.0 m^2 , which is 87 percent of the total area. By extrapolating to a high flow of $22.26 \text{ m}^3/\text{s}$, the area predicted by equation 21 is 72.5 m^2 , giving a predicted travel time of 53.0 hours. This predicted value is 2.7 hours less than the measured value (table 5) and represents an error of 4.8 percent. Although the scatter on figure 13 is large, the amount of inflow between the gages is not large except, perhaps, for flows starting on November 11, 1995 (figure 11). The predicted dye velocity is reasonably accurate, even though the extrapolation takes place for much lower flows than were used to measure the wave celerity.

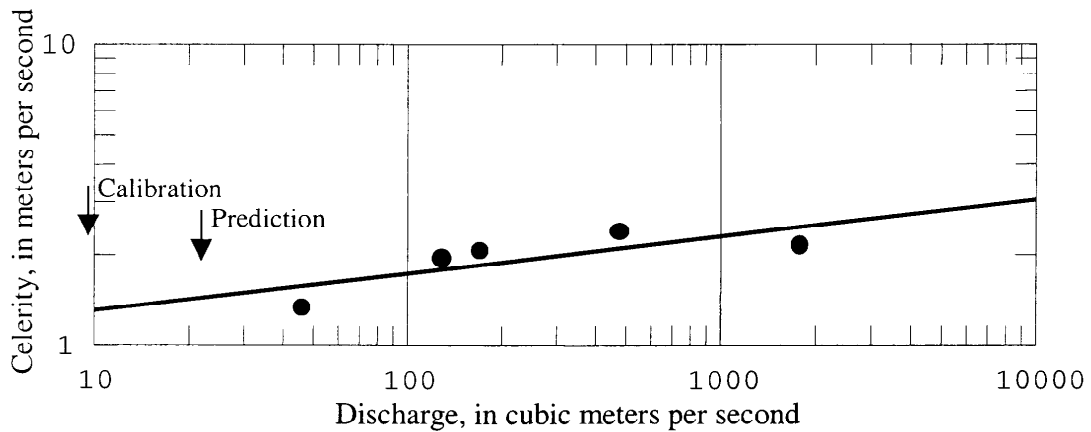


Figure 13. Wave celerity in South Fork Shenandoah River between Lynnwood and Luray, Virginia, as a function of discharge.

Colorado River, Lees Ferry to Diamond Creek, Arizona

Flow in the Colorado River below Glenn Canyon Dam (see figure 2) is almost totally controlled by dam releases, and the releases are greatly influenced by demands for power. The USGS provides real-time data for gaging stations on the Colorado River at <http://az.water.usgs.gov/>. Discharges at Lees Ferry (Station 09380000) and above Diamond Creek (Station 09404200) that were obtained from the internet on October 19, 1999 are shown in figure 14. The times-of-occurrence as well as the magnitude of the minimum and maximum flows were estimated for six pulses at Lees Ferry and Diamond Creek. As shown in figure 14, it takes about 2 days for a flow peak to traverse the 362.1-km reach of river. Averaging the six estimates of peak and minimum flows indicates that the average wave speed is 2.55 m/s at $411 \text{ m}^3/\text{s}$ and 2.67 m/s at $592 \text{ m}^3/\text{s}$. Solving for A_1 and A_2 from equation 10, yields:

$$A = A_0 + 0.958 \cdot Q^{0.874}. \quad (22)$$

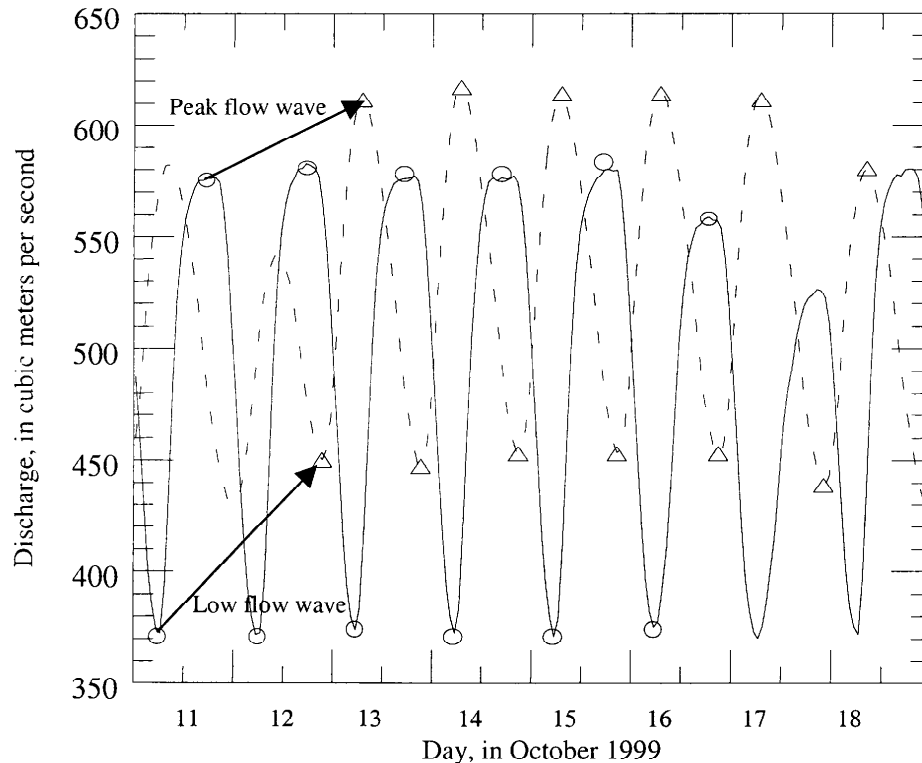


Figure 14. Discharge in the Colorado River at Lees Ferry, solid curve, and above Diamond Creek, Arizona, dashed curve.

At a steady flow of $432 \text{ m}^3/\text{s}$ the total travel time of dye from the Nautiloid Canyon section (57.7 km below Lees Ferry) to Gneiss Canyon (380.5 km below Lees Ferry) is 89.0 hours (Graf, 1995, p. 273). Thus the average flow area is calculated as the discharge divided by velocity to be 428.8 m^2 . Solving the equation 22 yields $A_0 = 236.1 \text{ m}^2$.

The data for a high flow time-of-travel study is available at the USGS web site at <http://az.water.usgs.gov/flood2.html>. These results indicate that at a steady flow of $1,317 \text{ m}^3/\text{s}$ the travel time from the Nautiloid Canyon section (57.8 km below Lees Ferry) to Diamond Creek (362 km below Lees Ferry) is 46.0 hours. Thus, the high-flow average area is 717 m^2 . Solving the equation 22 with $Q = 1,317 \text{ m}^3/\text{s}$, yields an area of 746 m^2 and a velocity of 1.77 m/s . The predicted travel time is 47.9 hours, which is 1.9 hours (4.1 percent) greater than the observed value. The predicted travel time is at a flow more than 2.2 times larger than the largest flow for which the wave speed is known and over three times the flow where the low-flow dye travel time is known.

If the inflow between gages is not too large, discharge hydrographs can be used to determine representative wave-celerity information for a river reach between the gages. The celerity information can subsequently be used to infer the change of travel time with flow with reasonable accuracy. For the examples given, travel time could be predicted to within 10 percent as the flow ranged from 0.22 to 4.0 times the flow rate where travel-time information was known. In general, it was found that the inactive flow area was a significant portion of the total flow area, thus extrapolation of travel times using the Manning equation directly will likely require the resistance coefficient to vary significantly with flow. The next section of this report demonstrates that the modified form of the Manning equation (equation 12) can be used with reasonable accuracy to extrapolate travel-time information to different flows while keeping the resistance coefficient constant.

Extrapolation of Transport Velocity Using a Resistance Equation

The previous section has shown that continuous discharge records can be used to predict the variation of travel time with flow. Although the USGS maintains about 7,000 continuous stream-flow gaging stations throughout the country, for many river reaches this information is not available or is not suitable for the analysis described above. For example, although continuous discharge records are available on the Mississippi River at St. Louis, Missouri and Thebes, Illinois, about 220 km downstream, no consistent relation between wave speed and discharge could be established. Because gages often do not exist at desired locations and because inflow or other factors can limit the ability of flow records to infer wave speeds, it generally is necessary to predict the variation in travel time with discharge from other basic hydrologic information.

Use of a resistance equation, such as the Manning equation, requires additional data, including channel width, slope, and a resistance coefficient. Fortunately, the results are not very sensitive to channel width and slope and, if the modified form of the equation (eq. 12) is used as shown below, the Manning's n varies in a predictable manner. Channel slope can be obtained from a topographic map, and, channel width can generally be inferred from the drainage area (see the Appendix for an example). Drainage areas are available at many points along most streams in the United States (Seaber and others, 1984) and at other points, the drainage area can be determined by use of a topographic map.

To demonstrate the ability of the modified Manning approach to predict the variation of travel time with discharge, river reaches for which two or more time-of-travel studies are available are analyzed below. Table A-1 in the appendix contains the basic data necessary to calibrate 60 river reaches on 12 rivers. Therefore, the Manning equation can be used to estimate travel time as a function of discharge.

The following steps were used to calibrate each river reach in table A-1 to an existing time-of-travel study:

1. The data, including slope, width, discharge, travel time, reach length, and Manning's n were assembled.
2. The hydraulic-geometry parameters for width were determined, so that widths could be estimated for other flows. The hydraulic-geometry exponent (W_2) was assumed to be 0.26, except for the Colorado River through the Grand Canyon. The hydraulic-geometry coefficient (W_1) was computed using equation 1 and the known width at a specified discharge.
3. The Manning's n was assumed to be equal to 0.035.
4. The travel-time information was converted to transport velocity.
5. The total flow area was computed by dividing the discharge by the transport velocity.
6. The active flow cross-sectional area was computed by use of equation 6.
7. The inactive flow area (AO) was computed as the difference between the total and active flow area by use of equation 12.
8. If the value of AO was negative, then its value was set to zero, and a Manning's n was computed by use of equation 6.

The first reach of Antietam Creek and the first reach of the Mississippi River will be used to illustrate these steps.

Referring to the data in table A-1 for Antietam Creek, it is seen that at a reach average flow of $(1.16 + 1.20)/2 = 1.18 \text{ m}^3/\text{s}$, the reach average width is $(12.8 + 11.0)/2 = 11.9 \text{ m}$. Solving equation 1 for W_1 yields:

$$W_1 = 11.9 / (1.18^{0.26}) = 11.4.$$

A time of $13.0 - 3.2 = 9.8$ hours was required for the dye peak to traverse the $9.6 - 2.6 = 7.0$ kilometers from site 1 to 2, thus the average transport velocity is $7,000 \text{ meters} / (9.8 \times 3,600 \text{ s}) = 0.198 \text{ m/s}$. The total flow area in the reach is subsequently computed as the discharge/velocity or:

$$\text{Total area} = 1.18 \text{ m}^3/\text{s} / 0.198 \text{ m/s} = 5.95 \text{ m}^2.$$

By using the average slope (0.0019), the active flow area is computed from equation 6:

$$\text{Active area} = 0.035^{0.6} \times 11.9^{0.4} \times 1.18^{0.6} / 0.0019^{0.3} = 2.61 \text{ m}^2.$$

The difference between the total area (5.95 m^2) and active area (2.61 m^2) is the inactive area, $AO = 3.34 \text{ m}^2$. In this case, the value of AO is positive and the reach is calibrated.

Applying the above analysis yielded negative values of AO for 15 percent of the river reaches in table A-1. The first reach of the Mississippi River, which has a flat slope, was one such reach. The data in table

A-1 for the Mississippi River indicates that at a flow of 2,633 m³/s, the reach average width was (468 + 501)/2 = 484.5 m. Solving for W1 from equation 1 yields:

$$W1 = 484.5 / 2,633^{0.26} = 62.5.$$

A time of 23.75 – 14.10 = 9.65 hours was required for the dye peak to traverse the 96.5 – 54.7 = 41.8 km from site 1 to 2. The average velocity, therefore, was 41,800 m/ (9.65 x 3,600 s) = 1.203 m/s, and the total flow area, which was computed as the discharge/velocity, was:

$$\text{Total area} = 2633 \text{ m}^3/\text{s} / 1.203 \text{ m/s} = 2188 \text{ m}^2.$$

By using the slope (0.000118), the active flow area was computed from equation 6:

$$\text{Active area} = 0.035^{0.6} \times 484.5^{0.4} \times 2633^{0.6} / 0.000118^{0.3} = 2699 \text{ m}^2.$$

Because the total area, 2188 m², is less than the active area, 2699 m², the inactive flow area (AO) is negative, which is physically unrealistic. Therefore, its value was assumed to be zero, and the value of Manning's n was computed from the total flow area using equation 6:

$$n = \{2188 \times 0.000118^{0.3} / (484.5^{0.4} \times 2633^{0.6})\}^{1/0.6} = 0.0246.$$

Only nine reaches in table A-1 have values of Manning's n smaller than 0.035, which indicates that the inactive flow area (AO) has been assumed to be zero. Eight of the nine small values occur where the slope is less than 0.00015 m/m, and only two of the 12 reaches with slopes of less than 0.0002 have AO values greater than zero.

In all cases, except for Conococheague Creek and the New River, the dye study conducted at the lowest flow is used to calibrate the reach. The inactive flow area will be the largest portion of the total area at low flow; thus, the calculation of AO should be most accurate. On the New River, the lowest flow with data available in all three reaches was used for calibration. The time-of-travel data for the extremely low flow in Conococheague Creek (the flow is exceeded 90 percent of the time [Taylor and Solley, 1971]) were suspected of being in error; therefore, the intermediate flow (exceeded 60 percent of the time) was used for calibration.

Once a river reach is calibrated, as shown in table A-1, the travel time at any other within-bank flow can be computed by assuming that the inactive flow area and Manning's n do not vary with discharge. The procedure is as follows:

1. For the discharge where the travel time is desired, compute the river width by assuming W2 = 0.26 and using the value of W1 shown in table A-1.
2. Compute the flow area by use of equation 12, which is based on the slope, width, Manning's n, and the inactive area shown in table A-1.
3. Divide the discharge by the flow area to yield the flow velocity.
4. Convert the flow velocity to travel time, based on reach length.

Time-of-travel data obtained at different discharges for the river reaches tabulated in table A-1 are summarized in table A-2. To illustrate the prediction procedure, the travel time of the first reach of Antietam Creek at high flow is computed. The average high flow shown in table A-2 for the first reach is (5.10 + 5.24)/2 = 5.17 m³/s. Assuming that W2 = 0.26 and W1 = 11.4 (from table A-1), the width at high flow is computed from equation 1:

$$\text{Width} = 11.4 \times 5.17^{0.26} = 17.5 \text{ m}.$$

The flow area can then be computed from equation 12:

$$\text{Flow area} = 3.34 + \{0.035^{0.6} \times 17.5^{0.4} \times 5.17^{0.6} / 0.0019^{0.3}\} = 10.72 \text{ m}^2.$$

The velocity is subsequently computed as the discharge divided by the area:

$$\text{Velocity} = 5.17 \text{ m}^3/\text{s} / 10.72 \text{ m}^2 = 0.482 \text{ m/s}.$$

The travel time for the 7.0-km reach would be 7,000 m / 0.482 m/s = 14,523 seconds or 4.03 hours, which is less than half of the travel time at low flow. This value compares closely with 4.15 hours, which was observed in the field. The flow in Antietam Creek exceeds 5.17 m³/s only 12 percent of the time, whereas it exceeds the 1.18 m³/s 75 percent of the time, (Taylor and Solley, 1971). The intermediate flow of Antietam Creek is exceeded 40 percent of the time.

The above procedure was used to estimate the travel time for all reaches and flows tabulated in table A-2, and the results are plotted on figure 15 as a function of the observed travel times.

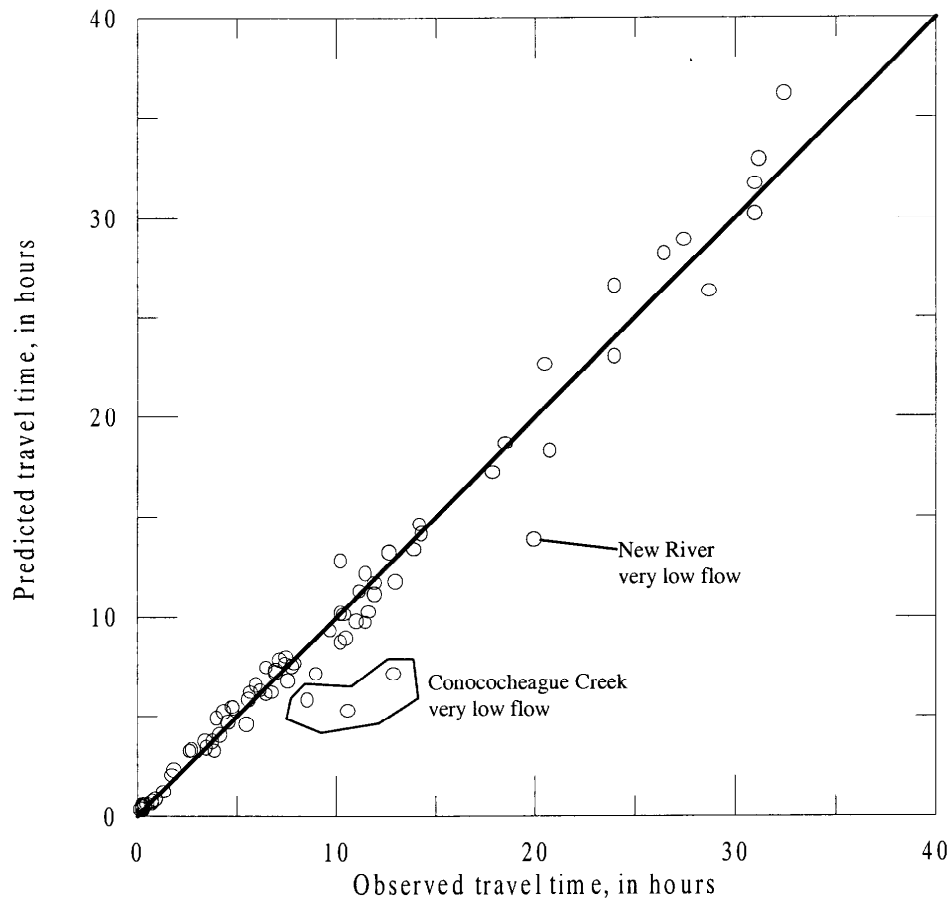


Figure 15. Predicted and observed travel time through each reach listed in table A-2.

Three of the outliers shown in figure 15 represent Conococheague Creek at very low flow ($2.8 \text{ m}^3/\text{s}$); the reach was calibrated at $6.9 \text{ m}^3/\text{s}$. The other outlier represents the upstream reach in the New River at a flow of $62 \text{ m}^3/\text{s}$. The New River reach was calibrated at a flow of $271 \text{ m}^3/\text{s}$. These outliers probably indicate that it is not advisable to extrapolate from high to low flows. It should be noted that the travel time for low flow in the New River was predicted closely by using the wave speed, as shown in table 3.

Computing the travel time for the entire reach of each injection reduces the scatter from random errors. The predicted and observed total travel time of each dye injection listed in table A-2 are shown in figure 16. Although several of the errors for individual reaches were larger than 10 percent, only the low flows in Conococheague Creek had an error in the total travel time that exceeded 10 per cent.

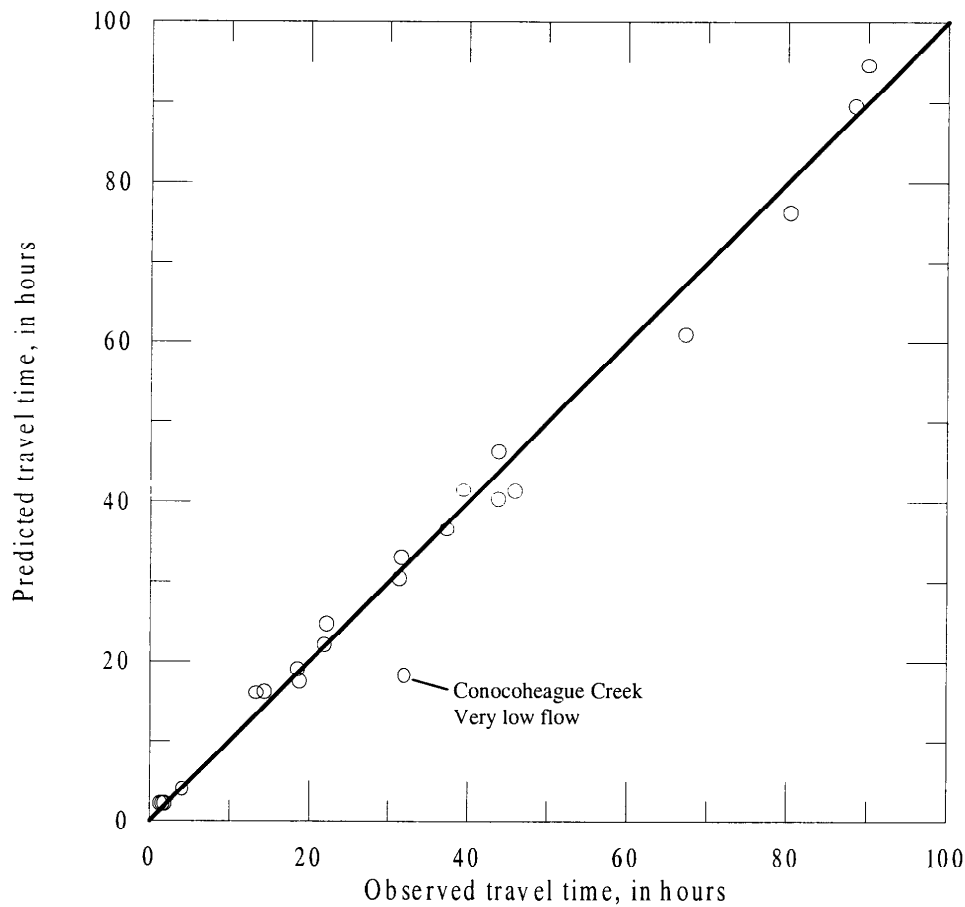


Figure 16. Predicted and observed total travel time of each time-of-travel study listed in table A-2.

To illustrate the sensitivity of the results to the assumed values of width and slope, the travel times in a large and small river were predicted for various assumed values of slope, width, and hydraulic-geometry exponent for width. The Mississippi River at high flow is considered first, and the results are tabulated in table 6. The minimum and maximum values represent the extremes among the reaches. Extremely small slopes or large widths require a very small Manning's n to keep the inactive area (A_0) from being negative. Reducing the width by 50 percent produced what is probably an unrealistic width-to-depth ratio. As can be seen from table 6, the predicted travel time is not sensitive to the assumed values of width and slope. If the value of the hydraulic-geometry exponent for width is reduced from 0.26 to 0.1, the width-to-depth ratio of the Mississippi River decreases as the discharge increases, which seems unrealistic. Nevertheless, the predicted travel times varied by only 2 or 3 percent for any of the adjustments except for the assumed value of W_2 , which varied by about 5 percent.

Table 6. Effect of width and slope on the predicted travel time of the Mississippi River.

Condition	Predicted Travel Time (hour)	Travel Time Error (percent)	Minimum Width/depth Ratio	Maximum Width/depth Ratio	Minimum Manning n	Maximum Manning n
Published data	46.3	5.2	60	195	0.022	0.035
Double slope	45.9	4.4	60	195	0.0312	0.035
Half of slope	46.9	6.5	60	195	0.0156	0.035
Double low-flow width	47.1	6.9	240	780	0.0139	0.035
Half of low-flow width	45.6	3.5	15	49	0.035	0.035
W2 = 0.1	43.8	- 0.4	50	200	0.022	0.035
W2 = 0.4	48.6	10.4	60	225	0.022	0.035

The effect of river size is illustrated by presenting the same analysis for the Antietam Creek in table 7. Even though reducing the widths to half of the published values gives an extremely small width-to-depth ratio of 3, the error in the predicted travel time is less than 5 percent.

Table 7. Effect of width and slope on the predicted travel time of the Antietam Creek, Maryland.

Condition	Predicted Travel Time (hour)	Travel Time Error (percent)	Minimum Width/depth Ratio	Maximum Width/depth Ratio	Minimum Manning n	Maximum Manning n
Published data	36.5	- 1.5	11	24	0.035	0.035
Double slope	35.4	- 4.5	11	24	0.035	0.035
Half of slope	37.9	2.2	11	24	0.035	0.035
Double low-flow width	38.1	2.3	28	97	0.035	0.035
Half of low-flow width	35.4	- 4.2	3	12	0.035	0.035
W2 = 0.1	35.8	- 3.3	11	24	0.035	0.035
W2 = 0.4	37.1	0.2	11	30	0.035	0.035

The data in tables 6 and 7 indicate that the prediction procedure is not sensitive to errors in the width, slope or the assumed value of W2.

When the Manning equation is applied directly for rivers with slopes greater than 0.0002 m/m, errors as large as 50 percent were common, and typical values were about 30 percent. The procedure for calibrating a river reach by applying the Manning equation directly would be as follows:

1. Assemble the data, including slope, width, discharge, travel time and reach length.
2. Determine the hydraulic-geometry parameters for width.
3. Convert the travel-time information to mean transport velocity.
4. Compute the flow area by dividing the discharge by the transport velocity.
5. Calculate Manning's n, by use of equation 5.

Once the Manning's n is calculated for the reach, it could be used to predict flow areas at other discharges, and the transport velocity could be computed as the discharge divided by the area.

To illustrate the direct use of the Manning equation, the travel time for the high flow in the first reach of Antietam Creek will be estimated. The first two steps have been completed above for low flow and the results are shown in table A-1, giving $S = 0.0019$, $Q = 1.18 \text{ m}^3/\text{s}$, travel time = 9.8 hours, reach length = 7,000 m, and $W1 = 11.4$. As shown above, the transport velocity is $7,000/(9.8 \times 3,600) = 0.198 \text{ m/s}$, and the flow area is $1.18/0.198 = 5.95 \text{ m}^2$. Solving equation 5 for Manning's n:

$$\text{Manning's } n = 5.95^{1.67} \times 0.0019^{0.5} / (1.18 \times 11.9^{0.67}) = 0.138$$

At this point the reach is calibrated; however, this is a large value of Manning's n that may indicate that the equation is not realistic. With no other knowledge of the river, there is little basis for adjusting the Manning's n . With the reach calibrated, the next step is to predict the travel time at high flow. This process is identical with what was done above except the value of AO is zero, and Manning's n is 0.138. Thus at the high flow of $5.17 \text{ m}^3/\text{s}$, the width can be estimated as before as 17.5 m, and equation 6 can be used to predict the flow area:

$$\text{Flow area} = 0.138^{0.6} \times 17.5^{0.4} \times 5.17^{0.6} / 0.0019^{0.3} = 16.8 \text{ m}^2 .$$

Computing the velocity as the discharge divided by the area gives $5.17/16.8 = 0.308 \text{ m/s}$, and the travel time through the 7-km reach becomes $7,000/0.308 = 22,700 \text{ sec}$ or 6.3 hours. As shown in table A-2, the observed travel time at this flow was 4.15 hours, giving an error of 52 percent.

Summary and Conclusions

Time-of-travel data are available for many rivers in the United States at a single flow, but few river reaches have data available at more than a single discharge. This report provides a procedure to extrapolate the time-of-travel information from one within-bank flow to another within-bank flow.

The report makes two points. First, that it is useful to consider the cross sectional area of a stream as being composed of two parts - an active area, whose size varies with discharge, and an inactive area of constant size. Second, the variation in size of the active area with discharge can be predicted either from an analysis of wave speeds determined from streamflow-gaging records or from estimates of channel slope and width by using a constant value of the Manning resistance coefficient.

The travel time of a dissolved constituent is directly proportional to the volume of water stored in the river. Natural rivers usually contain a significant volume of water stored in inactive storage or pools, which would not drain if the inflow were stopped. Dividing the inactive storage volume by the reach length yields the average inactive area (AO), which is sometimes called the dead zone. The inactive area significantly affects the time-of-travel of pollutants in the river but has little effect on the flow resistance or the speed of flow waves. Physically measuring the speed of a dissolved constituent, usually dye, is the easiest way to quantify the total volume of water stored in the river at a given flow. With the total volume known, the active volume can be computed from either an analysis of the speed of flow waves or by use of a resistance equation, such as the Manning equation. The inactive volume can then be computed as the difference between the total and active volume. The variation of the active area (volume/reach length) with discharge can be predicted using either a resistance equation or a geomorphic relation that is based on an analysis of wave speeds. Once the active and inactive areas are known, the travel time of pollutants can be estimated by adding the constant inactive area to the active flow area.

An analysis of the time series of discharge obtained at two or more points along the river may yield the relation of wave speed to discharge. From this relation, the theory of uniformly progressive flow can be used to develop the relation between the active flow area and discharge.

Geomorphic as well as Manning type resistance equations relate the size of the active flow area to discharge. It is shown that the variable components of the Manning equation (slope, width, and flow resistance) combine to produce the geomorphic form of the area equation. Although the slope, width, and flow resistance are each highly variable from point to point in a river, they can be combined by use of a resistance equation to yield a parameter that appears to have less variability and, thus, is more predictable. By accounting for the inactive area separately, a resistance equation, such as the Manning equation, can be applied meaningfully to long reaches of natural channels, even with limited data. When predicting the active area for bed slopes greater than 0.0002, it is usually acceptable to assume Manning's n is equal to 0.035.

Considering the reaches evaluated by using both the wave speeds and the modified form of the Manning equation, the wave-speed predictions were more accurate than the predictions made by use of the modified Manning equation. Typical prediction errors that result when using the wave speed approach were about 3 percent, whereas typical values of the errors when using the modified Manning approach were a little less than 10 per cent. Typical errors when using the direct form of the Manning equation were about 30 percent.

The hydraulic-geometry equations generally do not apply when the water covers the flood plain; therefore, the procedures developed in this report should not be used for out-of-bank (flood) flows.

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APPENDIX

APPENDIX. Basic Data and Predicted Times of Travel

The data used to determine the inactive flow area for each river reach used in the report is given in table A-1, and the data used to check the predicted travel times at different flows for the river reaches contained in table A-1 is given in table A-2. The hydraulic-geometry exponent (W2) is dimensionless, but the hydraulic geometry coefficient (W1) has units. The value of W1 in table A-1 gives the width in meters when the discharge is given in m³/s. Some assumptions had to be made to supplement the data available in the references. The following is a summary of assumptions made in this analysis.

Width was the most common variable to be missing. For the data in table A-1, widths were sometimes estimated on the basis of consistent hydraulic geometry coefficients (W1), or consistent width-depth ratios. Reaches for which this estimation procedure was used are indicated by a symbol ^a.

Because of the confined conditions of the Colorado River through the Grand Canyon, the variation of width with discharge is assumed to vary less than for typical rivers. A value of W2 = 0.1 was assumed, and the measured widths of Graf (1995, p. 269) obtained at 680 m³/s were used to compute W1. The widths at other discharges were then computed from equation 1. These values are indicated by a symbol ^b.

Wiley (1993, p. 2) estimated the average width of the New River for each reach at a flow of 57 m³/s. Assuming W2 = 0.26, the value of W1 was determined from equation 1 using the flows and estimated widths. The widths in table A-1 are then computed from equation 1 for the indicated discharge and flagged by use of a symbol ^c.

Widths and slopes were not available for the South Fork Shenandoah, but stream gages exist at Waynesboro, Harriston, Lynnwood, and Luray, and the gage datum and drainage area are available from the USGS web page. The slope for the reach between gages was estimated as the difference in gage datum divided by the distance between gages. These numbers are flagged in table A-1 by use of a symbol ^d.

The values of W1 for the South Fork Shenandoah River were estimated from the channel forming discharge and equation 2. The average drainage area for the reach was approximated as the average of the drainage area at each end of the reach. For example, the average drainage area between Waynesboro (site 1) and Harriston (site 4) is (329 + 549)/2 = 439 km², or 169 mi². Following Jennings and others (1994, p. 162), the flow with a 2-year return period (Q2) is given for the Shenandoah Valley by the equation:

$$Q2 = 25.2 A^{0.83} S^{0.26},$$

in which Q2 is the peak discharge in cubic feet per second, A is the drainage area in mi², and S is slope in feet per mile. The slope of 0.00165 m/m is equal to 8.71 ft per mile. Solving the above equation yields:

$$Q2 = 25.2 (169)^{0.83} (8.71)^{0.26} = 3126 \text{ cfs, or } 88.5 \text{ m}^3/\text{s}.$$

Solving equation 2 for the bank full width yields:

$$W_{bf} = 7.5 (88.5)^{0.56} = 92.3 \text{ m}.$$

Assuming a value of W2 = 0.26, equation 1 can then be solved for the hydraulic geometry coefficient:

$$W1 = 92.3 / 88.5^{0.26} = 28.8,$$

and the width at the flow of (1.02 + 1.59)/2 = 1.30 m³/s is also computed from equation 1 as:

$$W = 28.8 (1.30)^{0.26} = 30.8 \text{ m}.$$

Widths computed in this way are flagged using a ^e.

One slope on Concocheague Creek was simply estimated; it is indicated by an ^f.

Table A-1. Hydraulic data used to compute the inactive area of river reaches

[km, kilometer; m³/s, cubic meter per second; m, meter; m/100m, meter per 1000 meter; hr, hour; m², square meter]

Site	Distance km	Flow m ³ /s	Width m	Slope m/1,000m	Time to Peak hr	Manning n unitless	Width Coeff W1	Inactive Area m ²
Antietam Creek				Source of data - Nordin and Sabol, 1974				
1	2.6	1.16	12.8	2.7	3.2			
						0.035	11.4	3.34
2	9.6	1.20	11.0	1.1	13.0			
						0.035	10.8	9.47
3	21.5	1.64	12.8	1.1	43.0			
						0.035	14.2	13.67
4	29.6	1.78	19.8 ^a	0.8	67.0			
Clinch River				Nordin and Sabol, 1974				
1	0.69	10.79	45.7	0.484	0.467			
						0.035	24.5	9.32
2	1.58	9.85	44.2	0.625	1.250			
						0.035	23.7	0.02
3	2.49	9.46	41.1	0.625	1.800			
						0.035	25.2	4.65
4	3.59	10.51	50.6	0.484	2.650			
						0.035	28.2	15.18
5	4.67	10.59	53.6	0.441	3.840			
						0.035	27.7	28.75
6	5.89	9.77	47.5	0.361	5.700			
Colorado River				Graf, 1995				
1	57.7	425		1.41	18.5			
						0.035	17.6 ^b	333
2	98.3	425		1.26	33.1			
						0.035	22.3 ^b	171
3	123.2	430		2.74	39.3			
						0.035	17.6 ^b	273
4	189.3	433		1.95	58.8			
						0.035	12.1 ^b	227
5	267.9	436		1.51	79.0			
						0.035	15.5 ^b	196
6	343.6	436		1.34	98.4			
						0.035	16.3 ^b	213
7	380.5	436		1.61	108.3			
Conococheague Creek				Taylor and Solley, 1971				
1	4.4	6.82	31.0 ^a	0.7 ^f	3.0			
						0.035	20.0	52.8
2	8.7	6.85	35.1	0.5	6.6			
						0.035	24.6	21.4
3	13.4	6.88	46.0	0.7	10.3			
						0.035	33.7	13.8
4	19.9	6.94	65.5	0.7	15.5			
						0.035	32.9	49.5
5	26.0	6.94	43.3	0.6	21.3			
						0.035	25.4	239.1
6	33.9	7.08	41.1	0.6	33.8			

Table A-1. Hydraulic data used to compute the inactive area of river reaches (continued)

Site	Distance km	Flow m ³ /s	Width m	Slope m/1,000m	Time to Peak hr	Manning n unitless	Width Coeff W1	Inactive Area m ²
Copper Creek					Nordin and Sabol, 1974			
1	0.19	1.64	14.3	1.16	0.138			
						0.035	14.2	1.29
2	1.01	1.64	18.0	1.44	0.875			
						0.035	14.8	1.21
3	1.72	1.59	15.5	1.44	1.500			
						0.035	13.4	0.85
4	2.40	1.59	14.6	1.44	2.040			
						0.035	14.3	4.06
5	3.35	1.59	17.7	1.295	3.360			
						0.035	15.6	2.50
6	4.14	1.64	17.7	1.295	4.260			
Mississippi River					Nordin and Sabol, 1974			
1	54.7	2633	468	0.118	14.10			
						0.0246	62.5	0
2	96.5	2633	501 ^a	0.118	23.75			
						0.035	66.3	1669
3	117.9	2775	533	0.118	33.60			
						0.0220	87.9	0
4	294.4	2973	860	0.118	76.12			
Monocacy River, Reach 1					Nordin and Sabol, 1974			
1	10.3	5.38	29.0 ^a	0.6	12.5			
						0.035	21.6	11.9
2	18.3	5.66	38.4 ^a	0.6	23.0			
						0.035	27.3	10.1
3	26.8	6.37	48.8	0.5	33.5			
						0.035	26.9	13.2
4	34.3	7.65	40.5 ^a	0.3	43.5			
Monocacy River, Reach 2					Nordin and Sabol, 1974			
1	7.5	3.06	46.6	0.3	17.5			
						0.035	29.8	33.0
2	18.8	3.20	33.5	0.3	64.0			
						0.035	31.0	17.6
3	27.6	3.45	51.2	0.6	86.0			
						0.035	29.9	13.7
4	33.8	3.51	31.4	0.3	99.0			
New River					Appel and Moles, 1987			
1	85.8	263						
			321	1.61	7.00	0.035	74.7 ^c	48
2	64.0	280						
			256	1.61	6.40	0.035	58.7 ^c	80
3	44.1	297						
			211	3.13	8.20	0.035	48.0 ^c	166
4	19.8	297						

Table A-1. Hydraulic data used to compute the inactive area of river reaches (continued)

Site	Distance km	Flow m ³ /s	Width m	Slope m/1,000m	Time to Peak hr	Manning n unitless	Width Coeff W1	Inactive Area m ²
Red River, Reach A					Nordin and Sabol, 1974			
1	8.0	140	209	0.112	3.35			
						0.0337	56.0	0
2	54.7	140	195 ^a	0.151	32.00			
						0.0203	50.5	0
3	103.0	171	180 ^a	0.129	52.00			
						0.035	44.4	44
4	159.3	185	162	0.092	87.5			
Red River, Reach B					Nordin and Sabol, 1974			
1	12.1	108	133	0.092	7.00			
						0.0232	45.5	0
2	82.1	113	177 ^a	0.089	45.00			
						0.0240	56.7	0
3	139.2	136	221 ^a	0.072	79.50			
						0.0279	66.3	0
4	199.5	167	268	0.066	121.5			
South Fork Shenandoah River					Taylor and others, 1986			
1	287.2	1.02			0.00			
			30.8 ^e	1.65 ^d		0.035	28.8	7.9
2	278.7	1.59			22.0			
			32.9 ^e	1.65 ^d		0.035	28.8	8.1
3	266.8	1.76			48.0			
			33.7 ^e	1.65 ^d		0.035	28.8	9.4
4	256.5	1.93			71.0			
			61.5 ^e	2.05 ^d		0.035	40.9	24.5
5	229.4	7.70			127.0			
			80.7 ^e	1.30 ^d		0.035	46.3	38.5
6	207.7	9.26			169.0			
			84.1 ^e	1.30 ^d		0.035	46.3	33.4
7	195.0	10.65			189.0			
Tangipahoa River					Nordin and Sabol, 1974			
1	8.2	3.45	17.1	0.756	13.87			
						0.035	15.0	14.5
2	18.0	4.64	26.2 ^a	0.71	29.87			
						0.035	17.4	2.3
3	41.5	6.94	28.7 ^a	0.687	46.87			
						0.035	17.3	7.7
4	55.3	8.10	29.9	0.663	58.87			
						0.035	20.2	8.9
5	71.0	8.61	40.2	0.569	73.40			
						0.035	20.4	5.7
6	82.1	9.03	31.7	0.379	82.87			
						0.035	19.1	16.2
7	94.0	10.85	37.8 ^a	0.265	96.40			

Table A-1. Hydraulic data used to compute the inactive area of river reaches (continued)

Site	Distance km	Flow m ³ /s	Width m	Slope m/1,000m	Time to Peak hr	Manning N unitless	Width Coeff W1	Inactive Area m ²
Wind/Bighorn River					Nordin and Sabol, 1974			
1	9.2	54.9	55	3.30	2.75			
						0.035	21.5	12.1
2	32.7	54.9	66 ^a	2.70	9.38			
						0.035	25.3	27.7
3	50.4	56.6	78 ^a	1.00	16.63			
						0.035	24.3	3.3
4	75.3	56.6	61	1.00	25.00			
						0.035	18.4	8.6
5	99.9	56.6	44	1.50	32.63			
						0.035	18.4	4.3
6	141.9	56.6	61 ^a	1.20	44.50			
						0.0339	21.0	0
7	181.3	68.8	62 ^a	0.70	56.00			

- a Estimated value based on a consistent width depth ratio or value of W1.
- b Computed from measured widths at 680 m³/s (Graf, 1995, p. 269), assuming W2 = 0.1.
- c Estimated from Wiley (1993).
- d Estimated from the difference in gage datum.
- e Estimated from the channel-forming discharge.
- f Slope is estimated.

Table A-2. Computed and observed times of travel for verification computations
[km, kilometer; m³/s, cubic meter per second; hr, hour]

Site	Distance km	Flow m³/s	Time to Peak hr	Travel Time, in hr	
				Observed	Computed
Antietam Creek			Nordin and Sabol, 1974		
1	2.6	5.10	1.35		
				4.15	4.03
2	9.6	5.24	5.50		
				10.40	10.11
3	21.5	7.36	15.90		
				7.50	7.93
4	29.6	7.79	23.40		
1	2.6	2.41	2.7		
				6.8	6.16
2	9.6	2.44	9.5		
				17.9	17.19
3	21.5	3.40	27.4		
				12.7	13.16
4	29.6	3.96	40.1		
Clinch River			Nordin and Sabol, 1974		
1	0.69	104.8	0.175		
				.208	0.304
2	1.58	105.9	0.383		
				.259	0.271
3	2.49	104.5	0.642		
				.274	0.364
4	3.59	110.2	0.916		
				.374	0.424
5	4.67	104.2	1.290		
				.380	0.549
6	5.89	101.9	1.670		
1	0.69	85.8	0.160		
				0.273	0.330
2	1.58	79.9	0.433		
				0.329	0.289
3	2.49	89.2	0.762		
				0.378	0.388
4	3.59	86.9	1.140		
				0.250	0.462
5	4.67	83.8	1.390		
				0.720	0.597
6	5.89	85.2	2.110		

Table A-2. Computed and observed times of travel for verification runs (continued)

Site	Distance km	Flow m³/s	Time to Peak hr	Travel Time, in hr	
				Observed	Computed
Colorado River			Graf, 1995		
1	57.8	1303	9.75		
				7.58	6.74
2	98.3	1291	17.33		
				3.92	3.25
3	123.4	1291	21.25		
				10.25	10.14
4	196.3	1291	31.50		
				10.50	8.92
5	267.9	1297	42.00		
				3.50	3.43
6	294.4	1354	45.50		
				10.25	8.69
7	362.0	1410	55.75		
Conococheague Creek			Taylor and Solley, 1971		
1	4.4	29.4	1.35		
				1.80	1.98
2	8.7	29.4	3.15		
				1.85	2.24
3	13.4	29.7	5.00		
				2.65	3.23
4	19.9	30.0	7.65		
				2.75	3.28
5	26.0	30.3	10.40		
				4.40	5.20
6	33.9	30.6	14.80		
1	4.4	2.58	5.4		
				8.6	5.77
2	8.7	2.83	14.0		
				10.6	5.22
3	13.4	2.89	24.6		
				12.9	7.10
4	19.9	2.89	37.5		
Copper Creek			Nordin and Sabol, 1974		
1	0.19	7.05	0.070		
				0.275	0.396
2	1.01	7.82	0.345		
				0.300	0.329
3	1.72	7.93	0.645		
				0.250	0.295
4	2.40	7.62	0.895		
				0.530	0.538
5	3.35	8.10	1.425		
				0.305	0.409
6	4.14	8.72	1.730		

Table A-2. Computed and observed times of travel for verification runs (continued)

Site	Distance	Flow	Time to Peak hr	Travel Time, in hr	
				Observed	Computed
	km	m ³ /s			
Copper Creek			Nordin and Sabol, 1974		
1	0.19	1.78	0.132		
				0.768	0.709
2	1.01	1.78	0.900		
				0.750	0.597
3	1.72	1.78	1.650		
				0.390	0.507
4	2.40	1.90	2.040		
				1.34	1.17
5	3.35	1.93	3.380		
				0.920	0.817
6	4.14	1.93	4.300		
Mississippi River			Nordin and Sabol, 1974		
1	54.7	6824	10.25		
				7.00	7.27
2	96.5	6824	17.25		
				5.75	6.15
3	117.9	6824	23.00		
				31.25	32.88
4	294.4	6824	54.25		
Monocacy River, Reach 1			Nordin and Sabol, 1974		
1	10.3	14.3	7.10		
				6.50	6.06
2	18.3	15.1	13.60		
				6.00	6.49
3	26.8	15.9	19.60		
				6.20	6.26
4	34.3	18.5	25.80		
Monocacy River, Reach 2			Nordin and Sabol, 1974		
1	7.5	8.10	8.5		
				20.5	22.62
2	18.8	8.35	29.0		
				12.0	11.66
3	27.6	9.34	41.0		
				7.0	7.15
4	33.8	9.49	48.0		
1	7.5	20.39	5.25		
				10.25	12.74
2	18.8	20.39	15.50		
				6.50	7.41
3	27.6	22.09	22.00		
				5.50	4.56
4	33.8	22.09	27.50		

Table A-2. Computed and observed times of travel for verification runs (continued)

Site	Distance km	Flow m³/s	Time to Peak hr	Travel Time, in hr	
				Observed	Computed
New River			Appel and Moles (1987)		
2	64.0	91			
				13.00	11.76
3	44.1	91			
				18.50	18.66
4	19.8	91			
1	85.8	518			
				4.80	5.44
2	64.0	527			
				4.00	4.89
3	44.1	538			
				5.60	5.80
4	19.8	538			
1	85.8	62			
				20	13.80
2	64.0	62			
3	44.1	62			
				24.0	26.54
4	19.8	62			
1	85.8	127			
				11.50	9.70
2	64.0	127			
3	44.1	337			
				7.45	7.62
4	19.8	334			
1	85.8	229			
				7.80	7.48
2	64.0	232			
3	44.1	125			
				14.20	14.60
4	19.8	127			
Red River Reach A			Nordin and Sabol, 1974		
1	8.0	187	4.75		
				28.75	26.26
2	54.7	187	33.50		
				20.75	18.25
3	103.0	235	54.25		
				31.00	31.65
4	159.3	252	85.25		
Red River Reach B			Nordin and Sabol, 1974		
1	5.7	235	2.50		
				31.00	30.12
2	75.6	245	33.50		
				26.50	28.15
3	132.7	249	60.00		
				32.50	36.21
4	193.1	249	92.50		

Table A-2. Computed and observed times of travel for verification runs (continued)

Site	Distance	Flow	Time to Peak hr	Travel Time, in hr	
	km	m ³ /s		Observed	Computed
South Fork Shenandoah River			Taylor and others, 1986		
1	287.2	3.11	0.00		
				11.2	11.27
2	278.7	3.68	11.20		
				14.3	14.15
3	266.8	4.53	25.50		
				11.5	12.11
4	256.5	4.81	37.00		
				27.5	28.85
5	229.4	19.3	64.50		
				24.0	22.96
6	207.7	20.7	88.50		
Tangipahoa River			Nordin and Sabol, 1974		
1	8.2	5.78	9.83		
				11.67	10.21
2	18.0	9.80	21.50		
				13.92	13.34
3	41.5	11.9	35.42		
				9.75	9.29
4	55.3	12.4	45.17		
				12.00	11.07
5	71.0	14.4	57.17		
				9.00	7.10
6	82.1	17.8	66.17		
				11.00	9.73
7	94.0	18.7	77.17		
Wind/Bighorn River			Nordin and Sabol, 1974		
1	9.2	233	1.44		
				3.44	3.72
2	32.7	235	4.88		
				3.79	3.72
3	50.4	235	8.67		
				4.83	5.37
4	75.3	224	13.50		
				4.58	4.68
5	99.9	215	18.08		
				7.92	7.62
6	141.9	218	26.00		
				7.17	7.77
7	181.3	255	33.17		

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