Prediction of Traveltime and Longitudinal Dispersion in Rivers and Streams

By Harvey E. Jobson

U.S. GEOLOGICAL SURVEY
Water-Resources Investigations Report 96-4013

Reston, Virginia
1996

U.S. DEPARTMENT OF THE INTERIOR

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CONVERSION FACTORS

Multiply metric unit	By	To obtain inch-pound unit
kilogram (kg)	2.205	pound avoirdupois (lb avdp)
kilometer (km)	0.6214	mile (mi)
kilometer^2 (km^2)	0.3861	mile^2 (mi^2)
liter (L)	0.2642	gallon (gal)
meter (m)	3.281	foot (ft)
meter^3 per second (m^3/s)	35.31	foot^3 per second (ft^3/s)
milligrams (mg)	2.2046×10^{-6}	pound (lb)

Prediction of Traveltime and Longitudinal Dispersion in Rivers and Streams

by Harvey E. Jobson

Abstract

The possibility of a contaminant being accidentally or intentionally spilled upstream from a water supply is a constant concern to those diverting and using water from streams and rivers. Although many excellent models are available to estimate traveltime and dispersion, none can be used with confidence before calibration and verification to the particular river reach in question. Therefore, the availability of reliable input information is usually the weakest link in the chain of events needed to predict the rate of movement, dilution, and mixing of contaminants in rivers and streams.

Measured tracer-response curves produced from the injection of a known quantity of soluble tracer provide an efficient method of obtaining the necessary data. The purpose of this report is to use previously presented concepts along with extensive data collected on time of travel and dispersion to provide guidance to water-resources managers and planners in responding to spills. This is done by providing methods to estimate (1) the rate of movement of a contaminant through a river reach, (2) the rate of attenuation of the peak concentration of a conservative contaminant with time, and (3) the length of time required for the contaminant plume to pass a point in the river. Although the accuracy of the predictions can be greatly increased by performing time-of-travel studies on the river reach in question, the emphasis of this report is on providing methods for making estimates where few data are available.

Results from rivers of all sizes can be combined by defining the unit concentration as that concentration of a conservative pollutant that would result from injecting a unit of mass into a unit of flow. Unit-peak concentrations are compiled for more than 60 different rivers representing a wide range of sizes, slopes, and geomorphic types. Analyses of these data indicate that the unit-peak concentration is well correlated with the time required for a pollutant cloud to reach a specific point in the river. The variance among different rivers is, of course, larger than for a specific river reach. Other river characteristics that were compiled and included in the correlation included the drainage area, the reach slope, the mean annual discharge, and the discharge at the time of the measurement. The most significant other variable in the correlation was the ratio of the river discharge to mean annual discharge.

The prediction of the traveltime is more difficult than the prediction of unit-peak concentration; but the logarithm of stream velocity can be assumed to be linearly correlated with the logarithm of discharge. More than 980 subreaches for about 90 different rivers were analyzed and prediction equations were developed based on the drainage area, the reach slope, the mean annual discharge, and the discharge at the time of the measurement. The highest probable velocity, which will result in the highest concentration, is usually of concern after an accidental spill. Therefore, an envelope curve for which more than 99 percent of the velocities were smaller was developed to address this concern.

The time of arrival of the leading edge of the pollutant indicates when a problem will first exist and defines the overall shape of the tracer-response function. The traveltime of the leading edge is generally about 89 percent of the traveltime to the peak concentration.

The area under a tracer-response function (a known value when unit concentrations are used) can be closely approximated as the area under a triangle with a height of the peak concentration and a base extending from the leading edge to a point where the concentration has reduced to 10 percent of the peak. Knowing the time of the leading edge and the peak, the peak concentration, and the time when the response function has reduced to 10 percent of its peak value allows the complete response function to be sketched with fair accuracy.

Four example applications are included to illustrate how the prediction equations developed in this report can be used either to calibrate a mathematical model or to make predictions directly.

INTRODUCTION

The possibility of a contaminant being accidentally or intentionally spilled upstream from a water supply is a constant concern to those diverting and using water from streams and rivers. A method of rapidly estimating traveltime or dispersion is needed for pollution control or warning systems on streams where data are limited. As greater demands are placed on streams, the evaluation of significant forces of self-purification, such as deoxygenation-reaeration properties, becomes increasingly necessary. Therefore, the ability to simulate potential pollution buildup in streams, lakes, and estuaries becomes increasingly important.

Traveltime and mixing of water within a stream are basic streamflow characteristics that water-resources managers and planners should understand in order to predict the rate of movement and dilution of pollutants that may be introduced into streams. Mean velocities and mixing characteristics for a wide range of flows are basic data needed to address all of these concerns.

With the widespread availability of computers today, it is natural to think of numerical models as a means of answering these questions. Although many excellent models are available to make the types of calculations needed, none can be used with confidence before calibration and verification to the particular river reach in question. That is to say, all models must be provided with information from which flow velocities and mixing rates can be computed. In general there are no reliable methods of predicting dispersion coefficients (mixing rates) from commonly available hydraulic information. Stream velocities, typically predicted by use of a flow model, generally require very detailed channel geometry and flow resistance coefficients, which are seldom available. The availability of reliable input information is, therefore, almost always the weakest link in the chain of events needed to predict the rate of movement, dilution, and mixing of pollutants in rivers and streams.

Soluble tracers can be used to simulate the transport and dispersion of solutes in surface waters because they have virtually the same physical characteristics as water (Feurstein and Selleck, 1963; Smart and Laidlaw, 1977). This is the case in either a steady flowing river or in the unsteady oscillatory stage and flow of a tidal estuary. Measured tracer-response curves produced from the injection of a known quantity of soluble tracer provides an efficient method of obtaining the data necessary to calibrate and verify pollutant transport models. These data can also be used, in conjunction with the superposition principle, to simulate potential pollution buildup in streams, lakes, and estuaries without the need to use numerical models.

Extensive use of fluorescent dyes as water tracers to quantify the transport and dispersion in streams and rivers began in the United States in the early to mid-1960's. Kilpatrick (1993), using the concept of unit-peak concentration and the superposition principle, illustrated how these data, obtained in the time-of-travel studies, could be generalized to a wide range of flow conditions and even to other sites.

In this report, the concepts presented by Kilpatrick (1993), along with extensive data collected by the U.S. Geological Survey on time of travel and dispersion, are used to provide guidance to water-resources managers and planners in responding to spills. This will be done by providing methods to estimate (1) the rate of movement of a solute through a river reach, (2) the rate of attenuation of the peak concentration of a conservative solute with time, and (3) the length of time required for the solute plume to pass a point in the river. It will be shown how these estimates can be used alone to make the required predictions. In addition, they are precisely the data required to calibrate or verify pollutant transport models. The accuracy of these predictions will be greatly increased by performing time-of-travel studies on the river reach in question; but the emphasis of this report is on providing methods for making estimates in rivers where few data are available. Large fluctuations in the flow rates of the rivers during the downstream movement of a solute would cause significant differences between actual and predicted traveltimes. These cases can best be interpreted by use of numerical models. Traveltime and concentration attenuation of pollutants not dissolved in the water are beyond the scope of this report.

The report begins with a short discussion of the theory of movement and dispersion of dissolved pollutants and introduces the unit-peak concentration concept. A brief summary of the methods used to collect time-of-travel information is then given along with a summary of the data used in the report. Methods are recommended for estimating the rate of movement and attenuation of conservative pollutants based on an analysis of the data. The application of these results is then illustrated by use of three examples. The report concludes by introducing the superposition principle and illustrates its purpose by use of an example.

BACKGROUND AND TECHNIQUES

Theory of Transport and Dispersion for Instantaneous Sources

The response to the slug injection of a soluble tracer is assumed to imitate the characteristics of a soluble pollutant, so understanding of how tracers mix and disperse in a stream is essential to understanding their application in simulating pollution. Time-of-travel studies are often conducted to help understand these processes and to quantify traveltime and dispersion for a given reach of river. The general procedure for conducting a time-of-travel study is to instantaneously inject a known quantity of water-soluble tracer into a stream, usually at the center of flow, and to observe the variation in concentration of the tracer as it moves downstream. The general distribution of a tracer concentration resulting from a slug injection is shown in figure 1. The tracer-response curves in figure 1 are shown as a function of longitudinal distance and not as a function of time. Later in the report the response curves will generally be shown as a function of time.

The dispersion and mixing of a tracer in a receiving stream take place in all three dimensions of the channel (fig. 1). In this report, vertical and lateral diffusion will be referred to in a general way as mixing. The elongation of the tracer-response cloud longitudinally will be referred to as longitudinal dispersion. Vertical mixing is normally completed rather rapidly, within a distance of a few river depths. Lateral mixing is much slower but is usually complete within a few kilometers downstream. Longitudinal dispersion, having no boundaries, continues indefinitely. In other words, vertical mixing is likely to be complete at section I in figure 1, which is a very short distance downstream of the injection. At section II lateral mixing is still taking place rapidly, so mixing and dispersion are both significant processes between the injection and section III on figure 1. Downstream of section III the dominant mixing process is longitudinal dispersion, so the tracer concentration can generally be assumed to be uniform in the cross section.

For a midpoint injection, the tracer cloud moves faster than the mean stream velocity upstream of section III because the bulk of the tracer is in the high velocity part of the cross section. Preferably, all measurement cross sections for a time-of-travel study are at least as far downstream as the optimum distance (section III in fig. 1) so that longitudinal dispersion is the dominant process acting between measurement cross sections and so the tracer moves downstream at the mean stream velocity.

The conventional manner of displaying the response of a stream to a slug injection of tracer is to plot the variation of concentration with time (the tracer-response curve) as observed at two or more cross sections downstream of the injection, as illustrated on figure 2. The tracer-response curve, defined by the analysis of water samples taken at selected time intervals during the tracer-cloud passage is the basis for

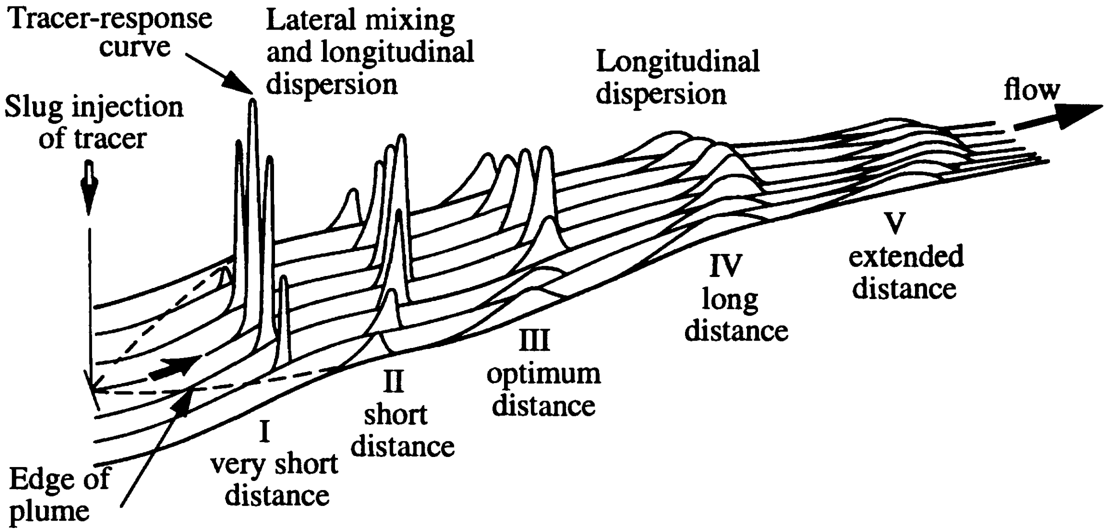

Figure 1. Lateral mixing and longitudinal dispersion patterns and changes in distribution of concentration downstream from a single, center, slug injection of tracer. (Modified from Kilpatrick, 1993, p. 2.)

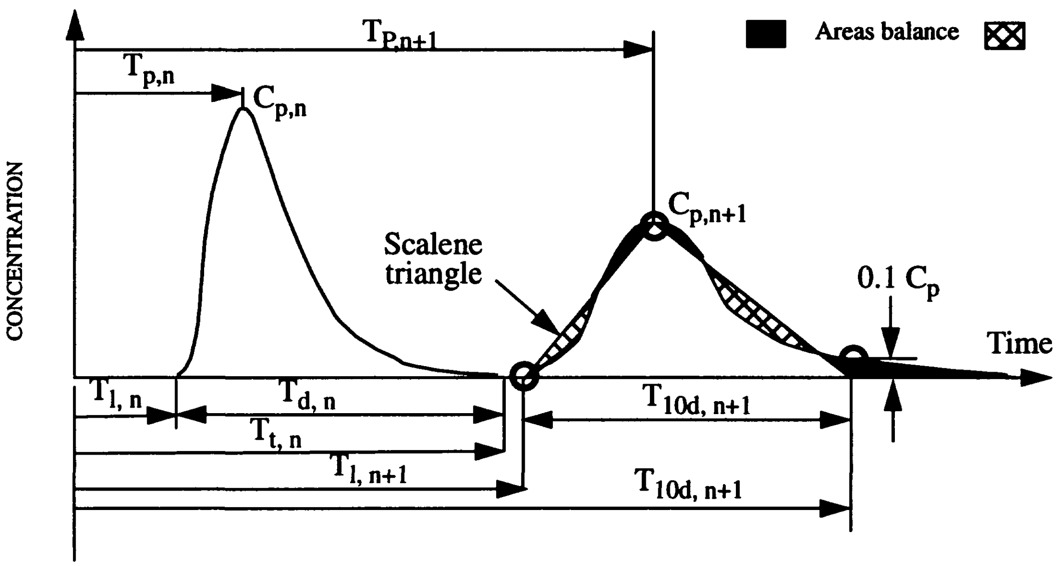

Figure 2. Definition sketch for tracer-response curves. Symbols are explained in text. (Modified from Kilpatrick and Wilson, 1989, p. 3.)

determining time-of-travel and dispersion characteristics of streams. A detailed explanation of the analysis and presentation of time-of-travel data are covered in the report by Kilpatrick and Wilson (1989).

The characteristics of the tracer-response curves shown in figure 2 are described in terms of elapsed time after an instantaneous tracer injection:

C_p, peak concentration of the tracer cloud;

T_l, elapsed time to the arrival of the leading edge of a tracer cloud at a sampling location;

T_p, elapsed time to the peak concentration of the tracer cloud;
T_t, elapsed time to the trailing edge of the tracer cloud;
T_d, duration of the tracer cloud (T_t-T_l);
T_{10d}, duration from leading edge until tracer concentration has reduced to within 10 percent of the peak concentration; and
n, number of sampling site downstream of injection.

The mass of tracer to pass a cross section, M_r, is computed as:

$$M_r = \int_{T_l}^{T_t}\int_{0}^{W} C_v \times q \times dwdt \tag{1}$$

where W is the total width of the river, C_v is the vertically averaged tracer concentration, and q is the unit discharge (discharge per unit width). Both C_v and q are given at time t and distance w from one bank. After mixing is complete in the cross section, the equation simplifies to:

$$M_r = \int_{T_l}^{T_t} C \times Q \times dt \tag{2}$$

where C is assumed to be uniform in the cross section and Q is the total discharge in the cross section at time t. If mixing is not complete, equation 2 can still be used as long as the concentration C is the discharge-weighted, cross-sectional-average concentration. If discharge is constant during the passage of a tracer cloud, it can also be factored out of the integral.

The shape and magnitude of the observed tracer-response curves shown in figures 1 and 2 are determined by four factors:

1. the quantity of tracer injected;
2. the degree to which the tracer is conservative;
3. the magnitude of the stream discharge; and
4. longitudinal dispersion.

All of these factors must be taken into consideration to predict the concentration of solutes from tracer-concentration data.

It is obvious that the magnitude of the tracer concentration in a stream is in direct proportion to the mass of tracer injected, M_i. Doubling the amount of injected tracer will double the observed concentrations, but the shape and duration of the tracer-response curve will remain constant. Thus, most investigators have normalized their data by dividing all observed tracer concentrations by the mass of tracer injected, M_i (Bailey and others, 1966; Martens and others, 1974).

It has also been found that various tracers are lost in transit due to adhesion on sediments and photochemical decay. Scott and others (1969) found fluorescent dyes to be absorbed on fine sediments such as clay. Rhodamine WT dye has been shown both in the field and laboratory to decay photochemically about 2 to 4 percent per day (Hetling and O'Connell, 1966; Tai and Rathbun, 1988). Kilpatrick (1993) noted decay rates tended to be higher in rivers, about 5 percent per day, compared to about 3 percent per day in estuaries.

To compare data and to have it simulate a conservative substance, it is desirable to eliminate the effects of tracer loss. If the stream discharge, Q, is measured at the same time and location as the tracer concentration, it is possible to evaluate the mass of tracer recovered, M_r, from equations 1 or 2. When the mass of the tracer injected, M_i, is known, the tracer recovery ratio R_r can be expressed as:

$$R_r = \frac{M_r}{M_i}. \tag{3}$$

A factor that inversely affects the magnitude of the tracer-response curves is the stream discharge. The diluting effect of tributary inflows, as well as that of natural ground-water accretion, differs from stream to stream and with location. To counter the variable diluting effects of differing discharges, it is desirable to adjust observed concentration data by multiplying by the stream discharge.

Observed concentrations can be adjusted for (1) the amount of tracer injected, (2) tracer loss, and (3) stream discharge (three of the four factors affecting the concentration) by use of what is called a "unit concentration." The unit concentration is defined as 1,000,000 times the concentration produced in a unit discharge due to the injection of a unit mass of conservative soluble substance. The unit concentration, C_u (units of inverse time), can be computed by the equation:

$$C_u = 1\times10^6 \times \frac{C}{R_r} \times \frac{Q}{M_i} = 1\times10^6 \times \frac{C}{M_r} \times Q. \tag{4}$$

The unit concentration can be visualized as the mass flux of solute (milligrams per liter times liters per second = milligrams per second) per unit of mass injected (milligrams). The 1,000,000 simply makes the numbers closer to unity. The discharge must be expressed in units that are consistent with the denominator of the concentration, and the injected mass must be in the same units as the numerator of the concentration. For example, if the concentration is expressed in milligrams per liter, the injected mass must be expressed in milligrams and the discharge must be expressed in liters per unit time. If the entire tracer cloud is sampled, the value of M_r can be computed and the mass of injected tracer need not be known.

Equation 4 can be used to convert any measured tracer-response curve to a unit-response (UR) curve. This UR curve can be used as the building block for simulating the concentrations to be expected from various pollutant loadings at different stream discharges. Normalizing the tracer-response curves, in effect, fits one unit of mass of tracer into one unit of flow. As such, when the flow is constant and mixing is complete, the area under UR curves is constant (1×10^6) for any cross section on a stream.

The Modeling Approach, Its Strengths and Weaknesses

A numerical model is one way to formally account for factors that influence the timing and shape of the tracer-response curves. Numerical models also tend to be complex and difficult to apply by someone without formal training. Although the use of numerical models is encouraged, it should be remembered that the accuracy of the model is critically dependent on the accuracy of the data used as input. Indeed, unless rather detailed and accurate field data are available, the modeling approach may add little to the reliability and accuracy of the predictions over what can be obtained by the much simpler and more straightforward approach outlined in this report.

All models solve three basic equations—the continuity of the mass of water, the conservation of momentum, and the conservation of the mass of the pollutant. Generally the first two equations are solved by use of a flow model to provide the water velocity, depth, and cross-sectional area as a function of time and position along the river. Three basic types of flow models are in common use. The simplest type, called the kinematic wave flow model, solves only the simplest form of the momentum equation by assuming the boundary friction force is always in balance with the weight component along the channel. Kinematic wave models generally provide satisfactory results for shallow flows over steep terrain, such as occurs in overland flow. The flow component in rainfall/runoff models often uses a kinematic wave approach to flow modeling. Kinematic wave models generally are not recommended for routing flows in rivers.

The most complex flow models, called the dynamic wave models, solve the complete form of the momentum equation. Examples are numerous including the BRANCH flow model of the Geological

Survey (Schaffranek and others, 1981), the DAMBRK model of the National Weather Service (Fread, 1977, 1984), and many others. These models work well for rivers with very flat slopes and in estuaries where flow reversals occur. They generally require at least two input boundary conditions (often the upstream discharge and the downstream stage) and detailed input information about the channel geometry and flow resistance. Dynamic wave models tend to become unstable as the river slope increases, particularly for rivers with shallow depths, slopes exceeding 0.5 m/km, or rivers with distinct riffles and pools.

Diffusive wave models ignore the inertia of the water and equate the sum of the pressure and friction forces to the weight component of the water. These models assume there is a unique relation between a steady-state flow and stage at each point in the river, so they generally do not require the specification of a downstream stage. They also generally operate satisfactorily with less detailed channel geometry information than required by the dynamic wave models and are much more stable and easy to use. Accuracy of diffusive wave models increase with increasing slope, and they cannot be used in situations where flow reversals occur. By using empirical geomorphological relations to represent channel geometry, the DAFLOW model (Jobson, 1989) has been shown to provide excellent accuracy using very limited data for slopes as small as 0.3 m/km. The DAFLOW model also allows wave speeds and transport speeds to be independently specified, which greatly facilitates the calibration of a transport model.

Transport models simulate four basic processes—advection, dilution, longitudinal mixing, and decay. Many excellent one-dimensional numerical models are available for simulating dissolved pollutant transport in rivers. The major models in use in the United States include the BLTM developed by the Geological Survey (Jobson, 1987), the WASP developed by the U.S. Environmental Protection Agency (Ambrose and others, 1987), and the CE-QUAL-RIV1 developed by the U.S. Army Corps of Engineers (Environmental Laboratory, 1990). All one-dimensional models solve the continuity of mass equation along the river thalweg, and so the differences between the models is generally less important than the quality of the data used to drive them.

Advection is simply the translation of the response function downstream with time. The water and the dissolved pollutant must move downstream at the cross-sectional mean water velocity that is supplied by the flow model. The accuracy of the timing, therefore, is dependent on the accuracy of the flow model, not the accuracy of the transport model. No matter which flow model is used, the channel geometry information will generally have to be adjusted (calibrated) to force the timing of the simulated and observed response functions in figure 2 to agree.

Dilution by tributary inflow is a simple process that all models simulate very well.

All models assume the spreading of the response function with time (fig. 2) is caused by a Fickian type of dispersion process. A Fickian process is one that assumes the flux of material along the channel is proportional to the concentration gradient. The proportionality constant is called the dispersion coefficient. Transport models can be grouped into two basic types called Eulerian models and Lagrangian models.

Eulerial models solve the continuity of mass equation at fixed locations along the channel, and Lagrangian models solve the continuity equation for a series of specific water parcels that move along the channel with the mean flow velocity. Eulerian models generally exhibit more numerical dispersion than Lagrangian models. In estuaries where reversing flow is predominant, numerical dispersion becomes much more troublesome. Paul Conrads (Geological Survey, personal commun., 1995) reported that while it was very difficult to calibrate an Eulerian model to simulate salinity throughout the Cooper River Estuary, the BLTM Lagrangian model was easy to calibrate and provided accurate simulations.

If Fickian dispersion correctly represented the total longitudinal mixing in rivers, the unit-peak concentration would decrease in proportion to the square root of time. Nordin and Sabol (1974) have reported that unit-peak concentration in natural rivers generally decreases more rapidly with time than predicted by the Fickian law. It is often assumed that other processes, presumably the movement of pollutant mass into and out of dead zone storage areas (Spreafico and van Mazijk, 1993), significantly contribute to the spreading of the response function in natural rivers. This process would tend to make the leading edge rise more steeply and the trailing edge fall more slowly than predicted by Fickian dispersion. Few models account for this process, so most models underpredict the tails on the concentration response function. Use of the empirical approach outlined herein, however, automatically accounts for all physical processes that contribute to the longitudinal spreading of the pollutant mass.

Transport models typically simulate a very limited number of chemical reactions. Prediction of the rates of chemical reactions is beyond the scope of this report.

Field Measurements

Time-of-travel studies may be conducted to improve the estimates of traveltimes and dispersion rates for specific river reaches and flow conditions.The Geological Survey has published a series of reports detailing the procedures to be used (Kilpatrick and Wilson, 1989; Kilpatrick and Cobb, 1985; Wilson and others, 1986), but the following will briefly outline the data collection needs to produce a full suite of traveltime and dispersion information. The following information should be obtained at each of two or more stream discharges that bracket the flows of interest.

1. Select the river reach and flow conditions of interest. Then establish two or more sampling cross sections where tracer concentration will be measured.
2. Attempt to conduct studies during times of reasonably steady flow.
3. Measure carefully the amount of tracer to be injected.
4. Retain a sample of the injected tracer for laboratory use in preparing standards.
5. Inject the tracer at a sufficient distance upstream so that lateral mixing is essentially complete by the first measurement section (section III on fig. 1). The distance required for essentially complete lateral mixing can be reduced by injecting the tracer at multiple points across the river if the amount of tracer injected at each point is proportional to the discharge in that subsection.
6. Measure for each sampling section the concentration at several points across the river during the passage of the entire tracer cloud or at least until a concentration of less than 10 percent of the peak concentration is reached. Measurement at several points across each sampling section allows one to better account for the entire mass of tracer recovered and to quantify the completeness of lateral dispersion.
7. Measure independently or evaluate stream discharges at every sampling cross section during the passage of the tracer cloud.

These data will provide information sufficient to allow nearly every kind of applicable analysis in the literature and provide the best practical information on predicting the effects of spills. It is often not practical to obtain the complete information as outlined above. Probably the most valuable information for improving forecasts is to measure the traveltimes of the peak concentrations at the center of the channel for various discharges. If only the peak traveltime is needed, the entire tracer cloud need not be sampled and it is not necessary to know the amount of tracer injected. It is important, however, that lateral mixing be nearly complete in the measurement reach and that the discharges be reasonably steady. Rather than measuring the discharge at each measurement cross section, the local discharge is sometimes assumed to be directly related to the flow measured at a remote index site.

The second most valuable information that can be gained from time-of-travel studies is the traveltimes for the leading edge of the tracer cloud. To obtain this information, sampling must begin before the arrival of the tracer and continue long enough to be sure the true peak concentration has passed.

If data are available for only one discharge, they can be extrapolated to other flows using equation 8 or other extrapolation techniques discussed later in the report.

Available Data

Starting in the 1960's, the Geological Survey conducted extensive time-of-travel studies to quantify the transport and dispersion in streams and rivers of the country. The results of some of these studies have been generalized by Godfrey and Frederick (1970), Boning (1974), Nordin and Sabol (1974), Eikenberry and Davis (1976), and Graf (1986). Some of the studies produced a full suite of time-of-travel and dispersion information, but many concentrated only on the traveltime of the tracer peak and did not obtain enough information to determine unit-peak concentration.

As many of the available data as time permitted were compiled for use in this report. All of the compiled data are listed in Appendix A. The appendix contains two tables and a list of references to the original studies. Table A-1 contains all the data for studies in which the unit-peak concentrations could be determined. Table A-2 contains all the data for studies in which the unit-peak concentrations could not be determined.

Appendix B contains a bibliography of other reports containing time-of-travel data that were not compiled because of time constraints.

ANALYSIS OF EXISTING DATA AND DEVELOPMENT OF PREDICTION EQUATIONS

Attenuation of Unit-Peak Concentration

The mixing processes have usually been interpreted by use of the Fickian theory of diffusion, and Fischer (1967) used this theory to define longitudinal dispersion coefficients for mixing in rivers. The peak concentration is a very important point on a tracer-response curve, and the variation in dispersion becomes most apparent if the unit-peak concentration is considered as a function of lapsed time since injection. According to Fischer's dispersion model, the peak concentration should attenuate with time as:

$$C_{up} \propto t^{-\beta} \tag{5}$$

in which C_{up} is the unit-peak concentration, t is time since injection, and β is a coefficient. The value of β should be approximately 1.5 for very short dispersion times (section I on fig. 1) and decrease to 0.5 for very long dispersion times (section V on fig. 1). Nordin and Sabol (1974) argue that a Fickian type equation cannot adequately describe longitudinal dispersion in rivers because the value of β never decreases to a value of 0.5. They conclude that a typical value of β is 0.7

After mixing in the cross section is complete, the decrease of the unit-peak concentration with time (as measured by β) is a measure of the longitudinal mixing efficiency. Larger values of β indicate more rapid longitudinal mixing. The presence of pools and riffles, bends, and other channel and reach characteristics will increase the rate of longitudinal mixing and almost always yield a value of β greater than the Fickian value of 0.5.

Unit-peak concentrations were compiled for 422 cross sections obtained from more than 60 different rivers in the United States. These data represent mixing conditions in rivers with a wide range of size, slope, and geomorphic type. For example, the slope in the study reach of the Mississippi River is 0.01 m/km and the mean annual discharge is about 11,000 m^3/s, whereas the study reach of Bear Creek has a slope of 36.0 m/km and a mean annual discharge of only about 1.3 m^3/s.

Figure 3 is a plot of the unit-peak concentrations (C_{up}) as a function of traveltime (T_p) of the peak concentration of all the data for which the mean annual flow was available. A tight correlation is shown by the data, indicating that a reasonable estimate of the unit-peak concentration can be determined from an expression of the form of equation 5. The regression equation based only on traveltime that best fit all of the data was:

$$C_{up} = 1025 \times T_p^{-0.887}. \tag{6}$$

This equation predicted the 422 available data points with a root mean square (RMS) error of 0.502 natural log units. The coefficient of variation was 0.112 and the coefficient of determination (R^2) value was 0.893. The standard error of estimate of the coefficient is 4.9 percent and the standard error of estimate for the exponent is 1.7 percent.

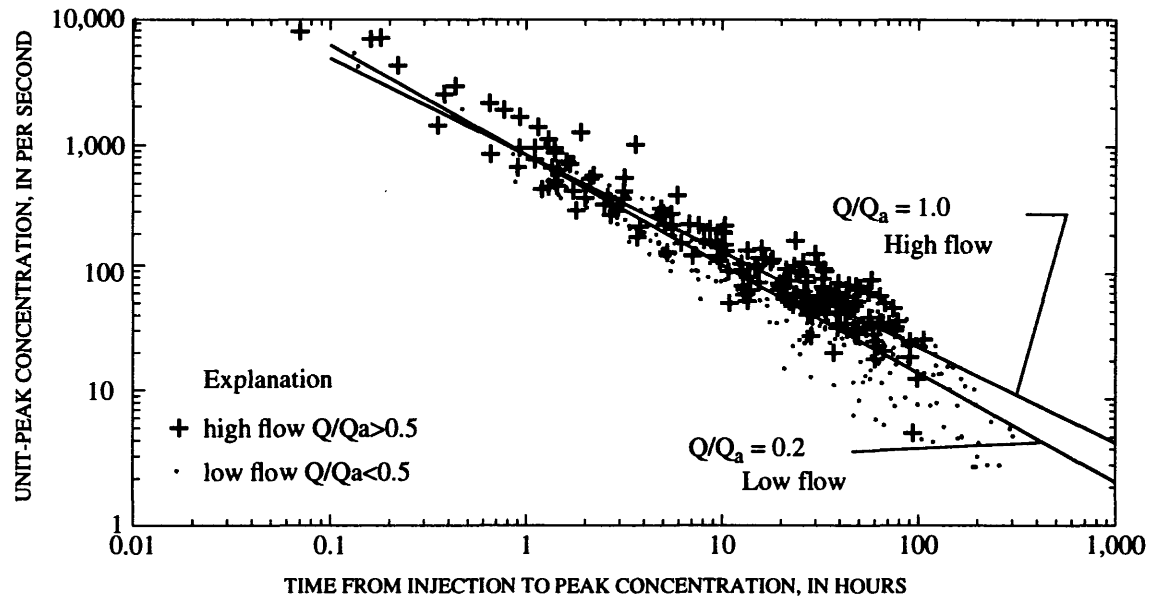

Figure 3. Unit concentrations as a function of traveltime with equation 7 plotted on the figure for two values of Q/Q_a.

Other river characteristics that were available to help define the relation included the drainage area (D_a), the reach slope (S), the mean annual river discharge (Q_a), and the discharge at the time of the measurement (Q). The most significant other variable in the correlation was the ratio of the river discharge to mean annual discharge giving a prediction equation:

$$C_{up} = 857 T_p^{-0.760\left(\frac{Q}{Q_a}\right)^{-0.079}} \tag{7}$$

in which Q is the river flow at the section at the time of the measurement and Q_a is the mean annual flow at the section. This equation predicted the 410 available data points with an RMS error of 0.426 natural log units. The coefficient of variation was 0.100 and the R^2 value was 0.910. The standard error of estimate of the coefficient is 4.3 percent, and the standard error of estimate for the exponent (0.760) is 1.6 percent.

The data in figure 3 are separated into two groups—one with values of relative discharge (Q/Q_a) greater than 0.5 (high flow) and one with a relative discharge less than 0.5 (low flow). The solid lines for high flow and low flow are plotted assuming constant values of relative discharge of 1.0 and 0.2, the approximate median value for each group of data.

Slope was not significant as an explanatory variable. Various regression models based on different combinations of discharge, mean annual discharge, and drainage area were tried. None of the equations produced a smaller RMS error or a larger R^2 value than equation 7.

Results for individual rivers generally define a much closer relation. For example, figure 4 presents measured concentrations of dye for the Shenandoah River as published by Taylor and others (1986). The points labeled as Q/Q_a=0.65 were actually taken at relative discharges ranging from 0.57 to 0.79 and the points labeled as Q/Q_a=0.27 actually ranged from 0.21 to 0.32. Notice that the data for the Shenandoah River show almost no correlation with relative discharge. Equations 6 and 7 are also plotted on the figure for reference. In this case the equations fit the data very closely.

Results for Wind/Bighorn Rivers and Copper Creek show a weak relation with relative discharge (figs. 5 and 6). Notice that the data for all of these rivers define a very good curve although the data for the Wind/ Bighorn Rivers are not especially well fit by either equation 6 or 7.

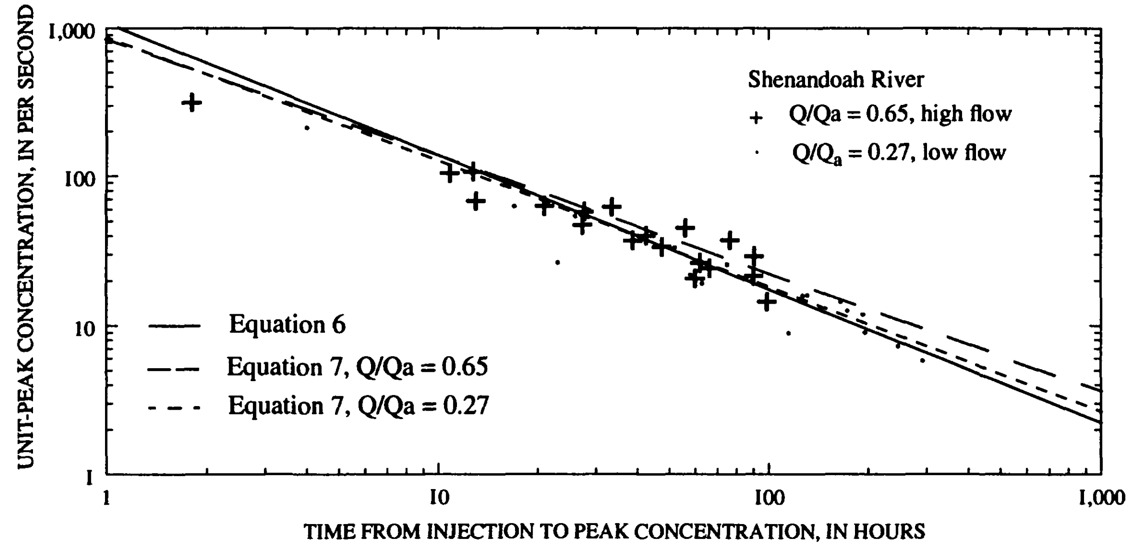

Figure 4. Unit-peak concentrations of dye for the Shenandoah River.

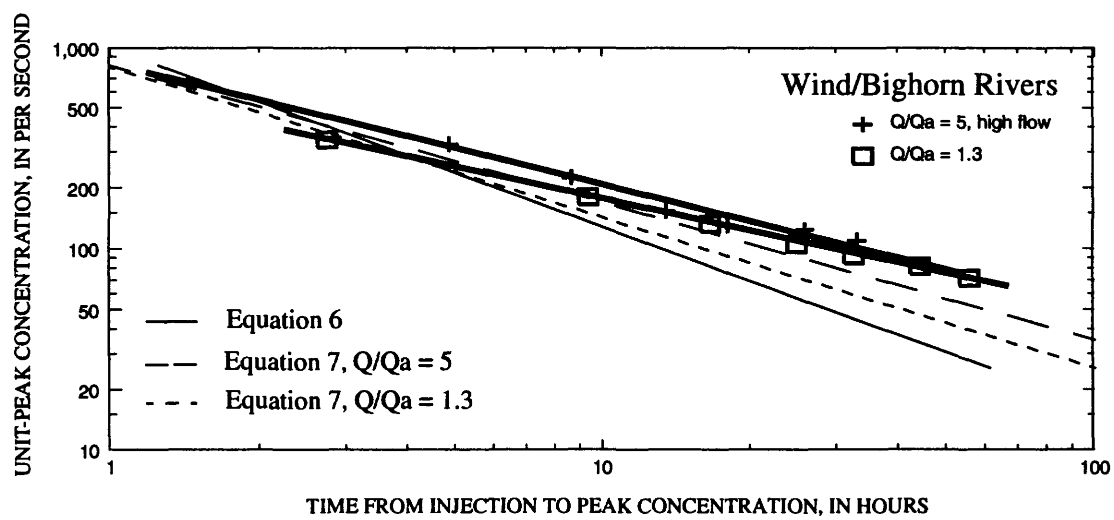

Figure 5. Unit-peak concentrations of dye for the Wind/Bighorn Rivers.

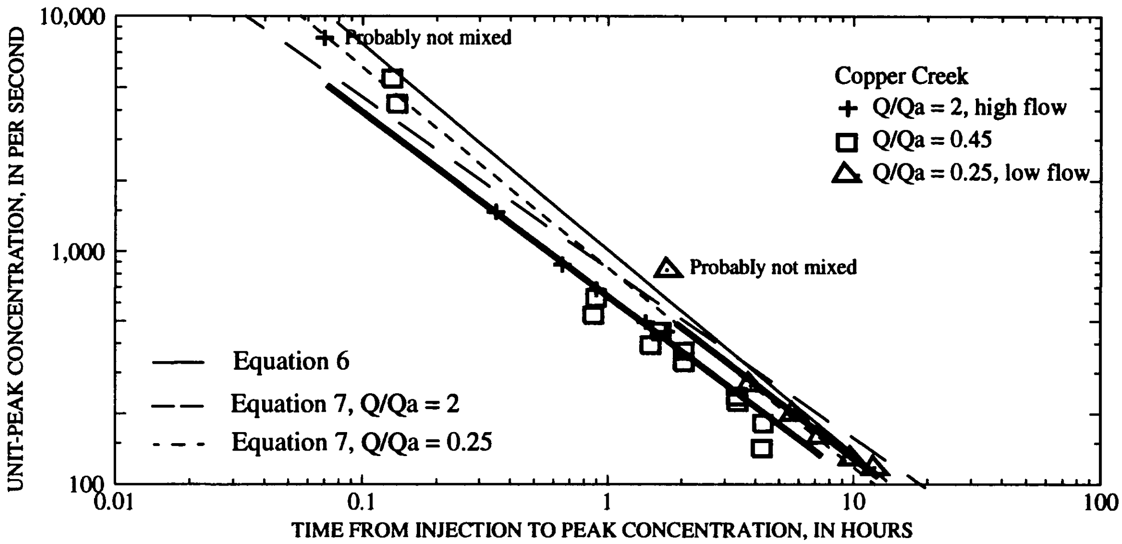

Figure 6. Unit-peak concentrations of dye for the Copper Creek.

The Sangamon River shows strong correlations with relative discharge (fig. 7). It should be noted, however, that one set of measurements was made at extremely low flow. At any rate, the scatter among points for a single river is typically much less than the scatter among all rivers (fig. 3) so there is significant value in collecting data for individual rivers to improve the ability to predict the variation of unit-peak concentration.

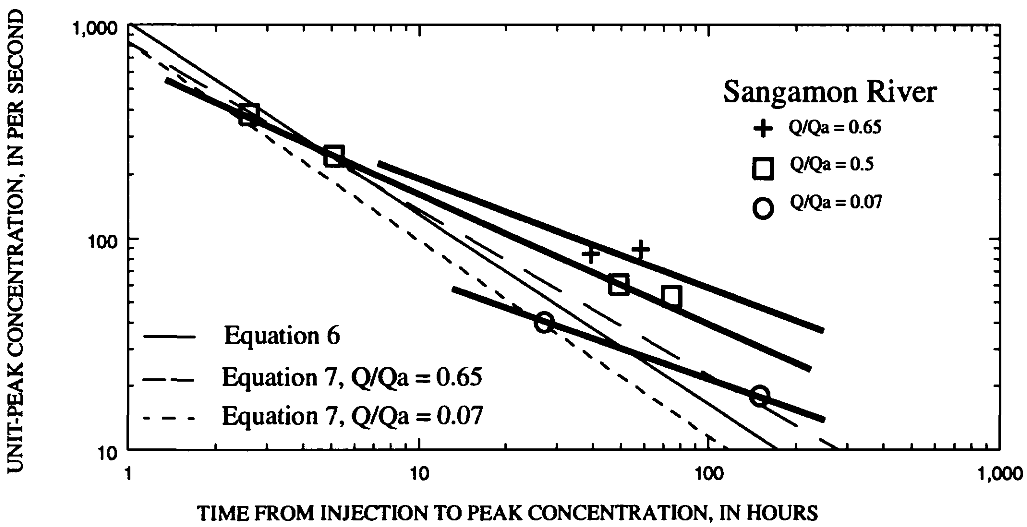

Figure 7. Unit-peak concentrations of dye for the Sangamon River.

A flow-duration curve is often used to provide a common base for comparison of streams of different sizes (Graf, 1986). A flow-duration curve for a site is developed by plotting the discharge as a function of the percentage of time the flow is exceeded. Several years of continuous discharge data are required but once the flow-duration curve is established for a site, flow-duration frequencies can be determined from the curve. Flows with low flow-duration frequencies are high discharges that occur during floods, whereas flows that occur with high flow-duration frequencies are low discharges that approach base-flow conditions.

Because the development of a flow-duration curve for a site requires data that are unlikely to be available where predictions are required, the relative discharge (discharge at measurement site/mean annual flow at measurement site, Q/Q_a) is used in this report to provide a common base for comparison of streams having different sizes. The mean annual flow (Q_a) can be easily estimated from drainage area and runoff relations for the region. An analysis of the data for the ten streams analyzed by Graf (1986) indicated that the relative discharge is equally as efficient as flow-duration frequency for predicting the unit-peak concentrations. Figure 8 is a plot of the relation between relative discharge and flow-duration frequency for Illinois streams as determined from the data of Graf (1986). As can be seen from the figure, the average flow in Illinois streams is one that is exceeded about 30 percent of the time.

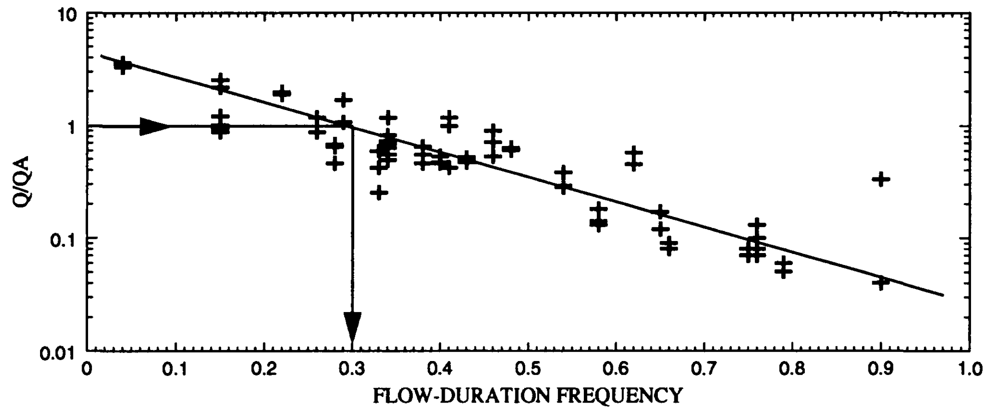

Figure 8. Relative discharge as a function of flow-duration frequency for Illinois streams and rivers.

The more efficient the mixing in a river, the steeper will be the relation between unit-peak concentration and traveltime. At high flow, river channels generally tend to be relatively uniform in shape, and they tend to increasingly exhibit a pool and riffle structure as the flow decreases. A pool and riffle structure offers great opportunities for tracer trapping; therefore, a pool and riffle structure tends to be efficient in mixing and attenuating the peak concentration. Equation 7 accounts for this process by decreasing the slope of UR curve for lower relative discharges.

Time of Travel of Peak Concentration

As shown in the preceding section, the time required for a tracer cloud to reach a specific point in a river is the dominant factor in determining the concentration that will occur. The traveltime itself is also of interest to local planners, who may be more interested in the minimum probable traveltime than the expected traveltime. The water velocity depends on many factors including the general morphology of the river and particularly the amount of ponding caused by dams or other manmade works. The prediction of the traveltime is, therefore, very important and it is often more difficult than the prediction of unit-peak concentration.

Stream velocity and, consequently, traveltime commonly vary with discharge. The relation of mean stream velocity, V, to discharge is generally assumed to take the form:

$$V = K \times Q^a \tag{8}$$

which is a straight line when the logarithm of discharge, Q, is plotted against the logarithm of velocity. For accurate estimates the constant, K, and exponent, a, must be defined for each river reach of interest, and two or more time-of-travel measurements are required to define the transport characteristics of the river reach. Geomorphic analyses by many investigators, however, suggest that the exponent in equation 8 typically has a value of about 0.34 (Jobson, 1989).

The velocity of the peak concentration and associated hydraulic data are compiled in Appendix A for more than 980 subreaches for about 90 different rivers in the United States representing a wide range of river sizes, slopes, and geomorphic types. Four variables were available in sufficient quantities for regression analysis. These included the drainage area (D_a), the reach slope (S), the mean annual river discharge (Q_a), and the discharge at the section at time of the measurement (Q). It was reasoned that these variables should be combined into the following dimensionless groups. The dimensionless peak velocity is defined as:

$$V'_p = \frac{V_p D_a}{Q}. \tag{9}$$

The dimensionless drainage area is defined as:

$$D'_a = \frac{D_a^{1.25} \times \sqrt{g}}{Q_a} \tag{10}$$

in which g is the acceleration of gravity. The dimensionless relative discharge is defined as:

$$Q'_a = \frac{Q}{Q_a}. \tag{11}$$

These equations are homogeneous, so any consistent system of units can be used in the dimensionless groups. The regression equations that follow, however, have a constant term that has specific units, meters per second. The most convenient set of units for use with the equations is, therefore, velocity in meters per second, discharge in cubic meters per second, drainage area in square meters, acceleration of gravity in m/s^2, and slope in meters per meter.

The most accurate prediction equation, based on 939 data points, for the peak velocity in meters per second was:

$$V_p = 0.094 + 0.0143 \times (D'_a)^{0.919} \times (Q'_a)^{-0.469} \times S^{0.159} \times \frac{Q}{D_a}. \tag{12}$$

The standard error of estimates of the constant and slope are 0.026 m/s and 0.0003, respectively. This prediction equation has an R^2 of 0.70 and an RMS error of 0.157 m/s. Figure 9 contains a plot of the observed velocities as a function of the variables on the right side of equation 12.

For responses to accidental spills, the highest probable velocity, which will result in the highest concentration, is usually a concern. On figure 9 an envelope line for which more than 99 percent of the observed velocities are smaller is also shown. The equation for this line, the maximum probable velocity, in meters per second (V_{mp}) is:

$$V_{mp} = 0.25 + 0.02 \times (D'_a)^{0.919} \times (Q'_a)^{-0.469} \times S^{0.159} \times \frac{Q}{D_a}. \tag{13}$$

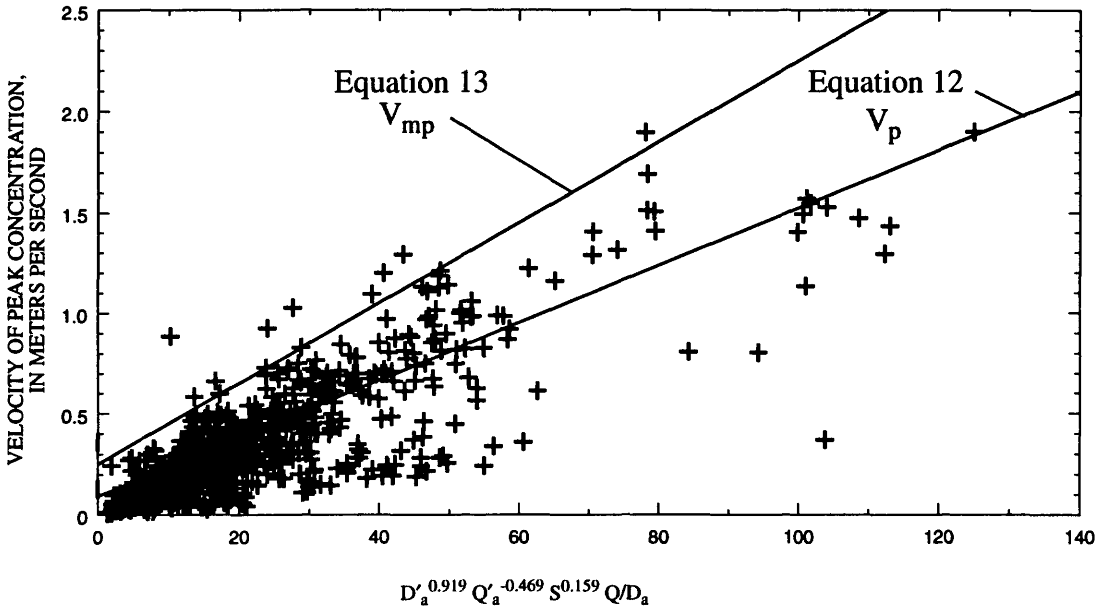

Figure 9. Plot of velocity of the peak concentration as a function of dimensionless drainage area, relative discharge, slope, local discharge, and drainage area.

The best equation for the velocity of the peak concentration, in meters per second, that did not include slope as a variable was:

$$V_p = 0.020 + 0.051 \times (D'_a)^{0.821} \times (Q'_a)^{-0.465} \times \frac{Q}{D_a}. \tag{14}$$

The standard error of estimates of the constant and slope are 0.009 m/s and 0.0013, respectively. The root-mean-square error of the prediction equation, based on 986 points, is 0.17 m/s with an R^2 of 0.62. Figure 10 presents a plot of the observed velocities as a function of the variables on the right side of equation 14. Also shown on the figure is a line for which 99 percent of the data points indicate a smaller velocity. The equation for this line, for the probable maximum velocity, in meters per second, is:

$$V_{mp} = 0.2 + 0.093 \times (D'_a)^{0.821} \times (Q'_a)^{-0.465} \times \frac{Q}{D_a}. \tag{15}$$

The best equation for the velocity of the peak concentration, in meters per second, using only drainage area was:

$$V_p = 0.152 + 8.1 \times (D''_a)^{0.595} \times \frac{Q}{D_a}. \tag{16}$$

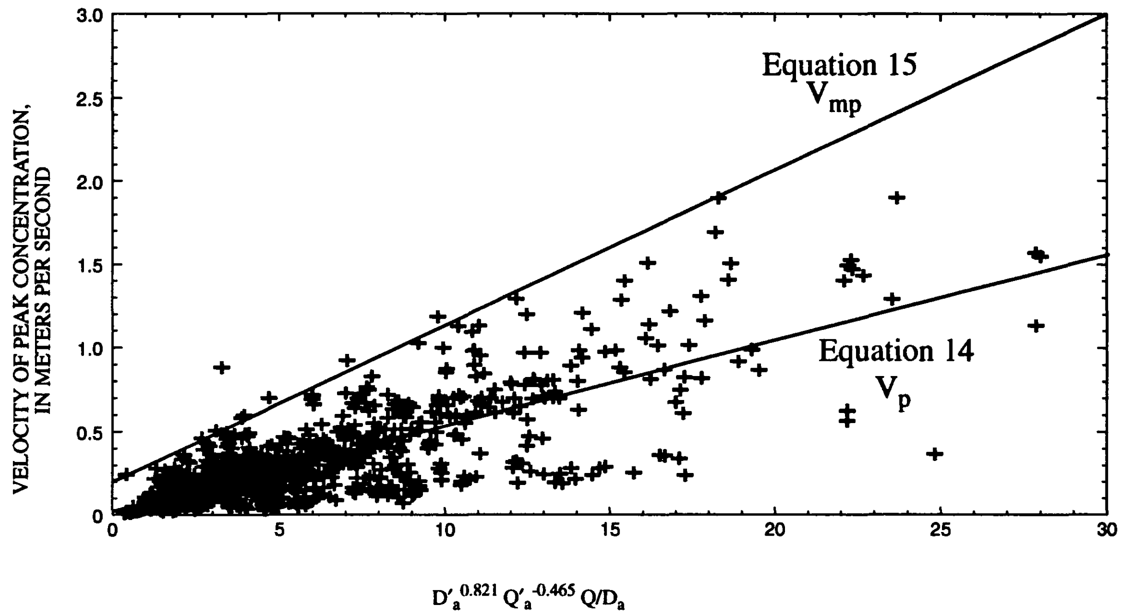

Figure 10. Plot of velocity of the peak concentration as a function of dimensionless drainage area, relative discharge, local discharge, and drainage area.

The term D''_a is defined by equation 10 except that the local discharge (Q) is used in place of the mean annual discharge (Q_a). The standard error of estimates, based on 986 points, of the constant and slope are 0.009 m/s and 0.28, respectively. The root-mean-square error of the prediction equation is 0.21 m/s with an R^2 of 0.46. Figure 11 presents a plot of the observed data as a function of the variables on the right side of equation 16. Also shown on the figure is a line for which 99 percent of the data points indicate a smaller velocity. The equation for this line is:

$$V_{mp} = 0.2 + 40.0 \times (D''_a)^{0.595} \times \frac{Q}{D_a} . \qquad (17)$$

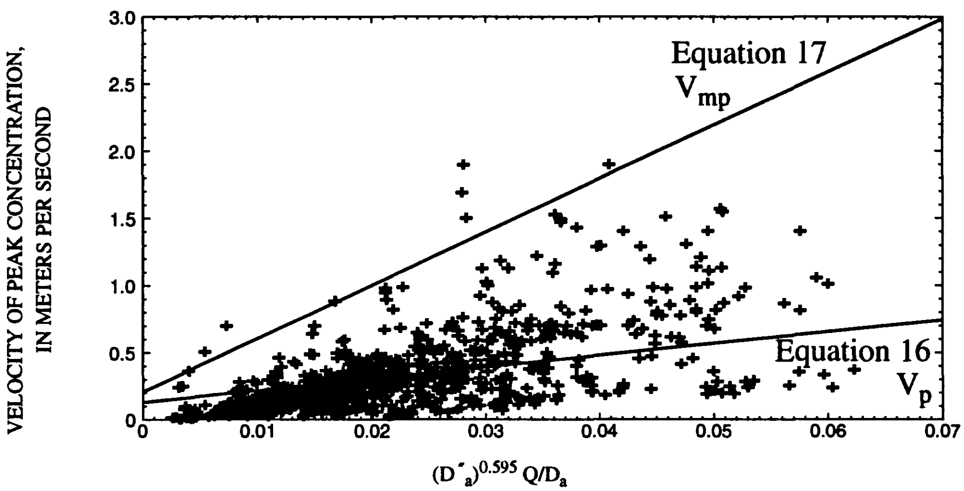

Figure 11. Plot of velocity of the peak concentration as a function of dimensionless drainage area, local discharge, and drainage area.

Time of Travel of Leading Edge

In addition to knowing when the peak concentration will arrive at a site, it is of great interest to know when the first pollutant will arrive. The time of arrival of the leading edge of the pollutant indicates when a local problem will first exist and defines the overall shape of the concentration response function.

Fewer data are available for the time-of-arrival of the leading edge (520 sites) than are available for the velocity of the peak concentration. Eight variables were available in sufficient quantities for regression analysis. These included the drainage area (D_a), the reach slope (S), the mean annual river discharge (Q_a), the discharge at the section at time of the measurement (Q), the velocity of the peak concentration (V_p), the width of the river, the depth of the river, and the time from the injection to the passage of the peak concentration (traveltime of the peak concentration, T_p). No significant correlation could be found between any of the variables and the time from injection to the arrival of the leading edge (T_l) except for the traveltime to the peak concentration. Figure 12 contains a plot of the traveltime of the leading edge as a function of the traveltime of the peak concentration. As can be seen from the figure, the correlation between these two variables is very good with an R^2 of 0.989, a coefficient of variation of 0.13, and a RMS error of 3.78 hours. These data indicate that the traveltime of the leading edge can be estimated from:

$$T_l = 0.890 \times T_p . \qquad (18)$$

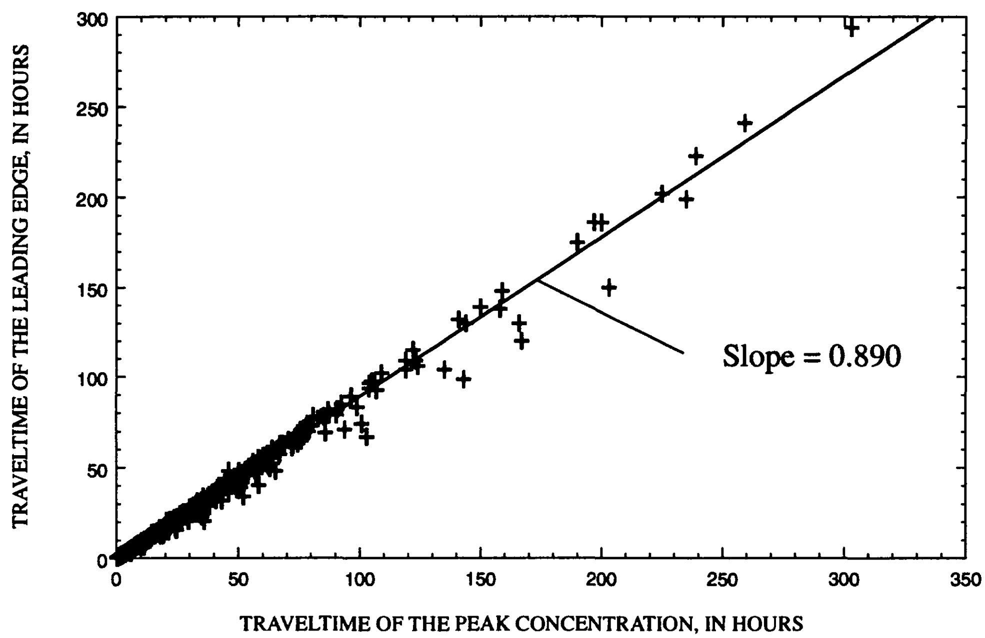

Figure 12. Plot of the time from injection to the first arrival of the leading edge of the tracer cloud as a function of the traveltime of the peak concentration.

Time of Passage of Pollutant

Methods have been developed for estimating the traveltime of the leading edge, T_l, the traveltime of the peak concentration, T_p, and the magnitude of the unit-peak concentration, C_{up}. This information defines two points on the tracer-response curve, shown as two of the large dots on figure 2. Kilpatrick and Taylor (1986) show that the area of a normal slug-produced tracer-response curve is very nearly equal to the area of a scalene triangle (three unequal sides) with a height equal to the peak concentration and the base extending from the leading edge to a point where the trailing edge concentration is equal to 0.1 times the peak concentration, T_{d10} (fig. 2). Because the area under the unit-response curve is 1×10^6, this information can be used to estimate a third point on the curve. The time of passage from the leading edge to a point where the concentration has been reduced to 10 percent of the peak concentration, T_{d10}, can be estimated from the equation:

$$T_{d10} = \frac{2 \times 10^6}{C_{up}}. \tag{19}$$

Furthermore, the area under the tail of the tracer-response curve should approximately balance the area between the falling limb portion of the tracer-response curve and the falling limb of the scalene triangle (fig. 2). This allows a complete tracer-response curve to be sketched in with reasonable accuracy based on the peak concentration and the times to the leading edge and peak.

Nonconservative Constituents

The unit concentration approach gives estimates of the solute concentration assuming no loss of mass during the transit from the injection to the point of observation (conservative transport). This will generally be a worst case estimate because losses normally occur with time. Losses may result from chemical transformations, photochemical decay, volatilization, trapping on sediments, or a number of other processes. Losses are often found to follow a first order decay law, which implies that the mass of material in the river decreases exponentially with time. One way to approximate this loss is to reduce the injected mass using the equation:

$$M_{ia} = M_i \times e^{-kT_p} \tag{20}$$

in which M_{ia} is the apparent mass of pollutant spilled after a time of T_p, M_i is the actual mass of pollutant spilled, and k is the decay coefficient with units of $time^{-1}$. The apparent mass of pollutant is then used in the unit concentration relation to determine the actual concentration from the unit concentration.

EXAMPLE APPLICATIONS

Three example applications for a slug injection will be given. The first example will assume that very few hydrologic data are available, and the second example will assume that time-of-travel measurements have been made at a relatively high and relatively low discharge. The third example will apply the method to a river for which some data are available that was not used in the development of the equations.

Example 1, Very Limited Data

Assume that a truck runs off the road and instantaneously spills 6,000 kg of a corrosive chemical into an ungaged stream. Estimate the most probable and the expected worst case effects of the spill on the water intake for a town that is located 15 km downstream. The worst case should occur for the shortest probable traveltime.

No data exist for the stream receiving the spill, but topographic maps show that the drainage area is 350 km^2 at the spill site and 430 km^2 at the intake for the town. A review of available data also indicates that a gaging station exists for a nearby stream with a drainage area of 452 km^2 and a mean-annual flow of 5.22 m^3/s. At the time of the spill the flow at the gaging station was 3.88 m^3/s. The hydrology and weather are assumed to be fairly uniform within the area so it will be assumed that the stream carrying the spill is flowing at about 3.88 (390/452) = 3.35 m^3/s, assuming the average drainage area for the reach is (350+430)/2 = 390 km^2. Likewise, the mean-annual flow of the ungaged stream is estimated to be about 5.22 (390/452) = 4.50 m^3/s.

The first step is to estimate traveltime of the peak concentration. Because the river slope is not available, equations 14 and 15 will be used to estimate the expected and fastest probable traveltimes in the stream. The dimensionless drainage area and discharge are computed first from equations 10 and 11:

$$D'_a = \frac{\left(390\times10^6\right)^{1.25}\times\sqrt{9.8}}{4.50} = 3.81\times10^{10}$$

$$Q'_a = \frac{3.35}{4.50} = 0.744\ .$$

Applying equation 14:

$$V_p = 0.020+0.0509(3.81\text{x}10^{10})^{0.821}(0.744)^{-0.465}(3.35/390\text{x}10^6) = 0.264 \text{ m/s}$$

while the maximum probable velocity from equation 15 is:

$$V_{mp} = 0.2+0.093(3.81\text{x}10^{10})^{0.821}(0.744)^{-0.465}(3.35/390\text{x}10^6) = 0.646 \text{ m/s}.$$

The most probable traveltime of the peak to the water intake is:

$$T_p = 15000/(0.264\text{x}3600) = 15.8 \text{ hours},$$

and the probable minimum traveltime of the peak is:

$$T_{pm} = 15000/(0.646\text{x}3600) = 6.4 \text{ hours}.$$

With the traveltimes known, the most probable unit-peak concentration at the town intake can be estimated from equation 7 as:

$$C_{up} = 857\times15.8^{-0.760\times0.744^{-0.079}} = 100 \text{ per second.}$$

Rearranging equation 4, to give the peak concentration:

$$C_p = \frac{C_{up} \cdot R_r \cdot M_i}{1x10^6 \cdot Q},$$

and using the injected mass, M_i, of $6x10^9$ mg, the flow rate at the intake, Q, of (3.88x(430/452)x1000) 3,690 L/s, and assuming the recovery ratio, R_r, to be 1.0, the most probable conservative-peak concentration can be computed as:

$C_p = 100x1.0x6x10^9/3690x10^6 = 162$ mg/L

occurring 15.8 hours after the injection.

At the highest probable velocity, the unit-peak concentration is 202 s^{-1} giving an estimated conservative-peak concentration of 328 mg/L occurring 6.4 hours after the spill.

When will the pollutant first arrive at the intake? As can be seen from equation 18, the time of arrival of the leading edge of the pollutant cloud should occur 0.89x15.8 = 14 hours after the accident. It is highly unlikely that the pollutant will arrive at the intake sooner than 0.89x6.4 = 5.7 hours after the spill.

How long will the intake be affected? As can be seen from equation 19, the most probable time required for the bulk of the dye cloud to pass the site (the concentration to be reduced to 10 percent of the peak value, 16 mg/L) is:

$T_{d10} = 2x10^6/(100x3600) = 5.6$

hours after the time of arrival, or 14+5.6 = 19.6 hours after the spill.

It is highly unlikely that the pollutant concentration will have reduced to less than 20 mg/L before;

$5.7+2x10^6/(202x3600) = 8.5$ hours after the spill.

All of the above computations were carried out assuming no loss of pollutant between the spill and the intake. Losses could occur by chemical reactions, volatilization, absorption on the streambed, or other processes. Equation 20 can be used to account for these losses.

Example 2, Traveltime Data Available

The second example assumes that 50 kg of a pollutant is spilled in the Apple River 25.9 km upstream of Elizabeth (10 km from the injection site) when the river discharge at the spill site is 2.4 m^3/s. Compute the probable impact, assuming no losses, of this spill on a water intake at Hanover, which is 41.1 km downstream of the spill.

Two time-of-travel studies have been completed on this reach of the Apple River and the data are contained in table A-1 of Appendix A as injection numbers 83 and 84. One of these studies was conducted at relatively low flow, when the river discharge was about 0.7 times the mean annual flow, and one was conducted at relatively high flow, when the flow rate was about 3.5 times the mean annual flow. The first step is to estimate the times of travel of the leading edge and peak of the pollutant cloud. The traveltimes of the peak concentrations as found in table A-1 are plotted in figure 13.

From table A-1 it is seen that the traveltime of the peak concentration to Elizabeth is 49.4 hours at a relative discharge of 0.68, while the traveltime to Whitton is 105.8 hours at a relative discharge of 0.62. Also it is seen that the distance from Elizabeth to Hanover is 16.1 km while the distance from Elizabeth to Whitton is 22.5 km, so Hanover is 72 percent of the way between Elizabeth and Whitton. By linear interpolation, it is easily seen that the traveltime from the injection site to Hanover would be about 49.4+(105.8-49.4)x0.72 = 89.8 hours and that the relative discharge at this point would have been about 0.68+(0.62-0.68)x0.72 = 0.64. Likewise, the traveltime from the town of Apple River to the spill site

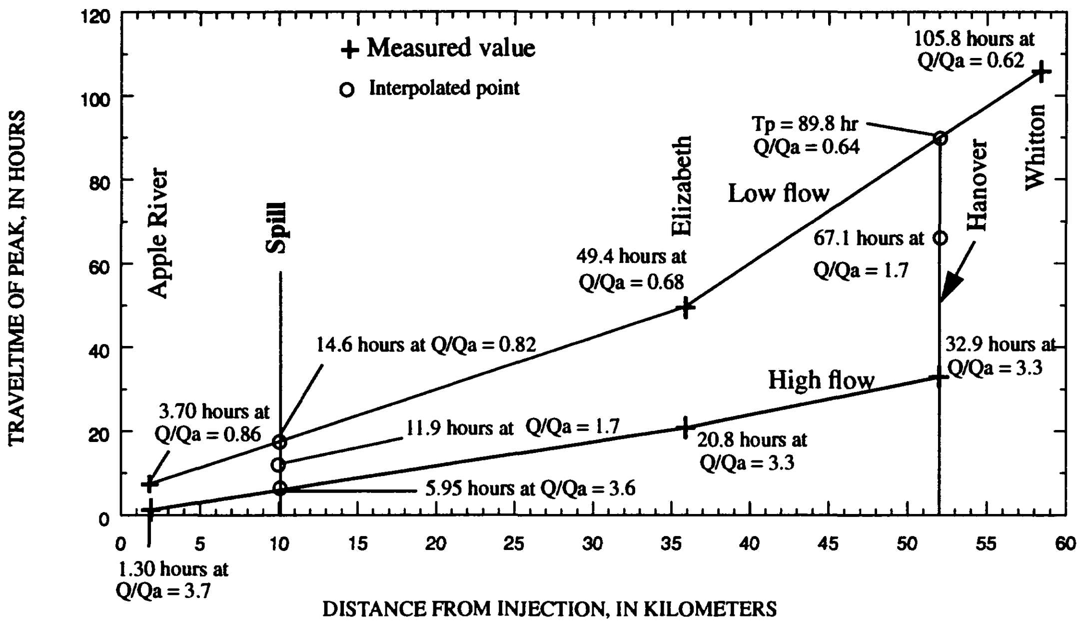

Figure 13. Traveltime distance relation for peak concentration in the Apple River.

would be 1.30+(20.80-1.30)x(10-1.9)/(35.9-1.9) = 5.95 hours at a relative discharge of 3.7+(3.3-3.7)x(10-1.9)/(35.9-1.9) = 3.6. In a similar manner, the traveltime from Apple River to the spill site would be 14.6 hours at a relative discharge of 0.82.

Assuming a mean annual flow at the spill site of 1.4 m^3/s, the relative discharge at the time of the spill is 2.4/1.4 = 1.7. Then by linear interpolation between the relative discharges, it is seen that the traveltime from Apple River to the spill site would be 5.95+(14.6-5.95)x(1.7-3.6)/(0.82-3.6) = 11.9 hours. Likewise the traveltime from Apple River to Hanover would be 67.1 hours. The traveltime from the spill site to Hanover should, therefore, be 67.1-11.9 = 55.2 hours.

With the relatively small amount of data contained in Appendix A for the Apple River, it is possible to estimate the timing of a spill on the river with much better accuracy than would have been possible by use of equations 12 to 17.

Figure 14 is a plot of the unit-peak concentrations measured on the Apple River during the two tests. As can be seen from the figure, the unit-peak concentration should be about 40 s^{-1} for a traveltime of 55 hours. Converting the spilled mass into milligrams ($5x10^7$ mg), the flow rate at Hanover (Q_{ave} = 5 m^3/s from table A-1) to liters per second (1.7x5.0x1000 = 8500), and assuming a recovery ratio of 1.0, the peak concentration at the intake can be estimated from equation 4 as:

$$C_p = 40x5x10^7x1.0/(1x10^6x8500) = 0.235 \text{ mg/L}.$$

The time required for the pollution cloud to pass the intake and the river concentration to be reduced to 10 percent of the peak value (0.024 mg/L) can be estimated by use of equation 19 as:

$$T_{t10} = 2x10^6/(40 \times 3600) = 13.9 \text{ hours}.$$

The times for the arrival of the leading edge of the tracer cloud, from table A-1, can also be plotted as in figure 14. The traveltime of the leading edge of the tracer cloud from the spill site to Hanover can then

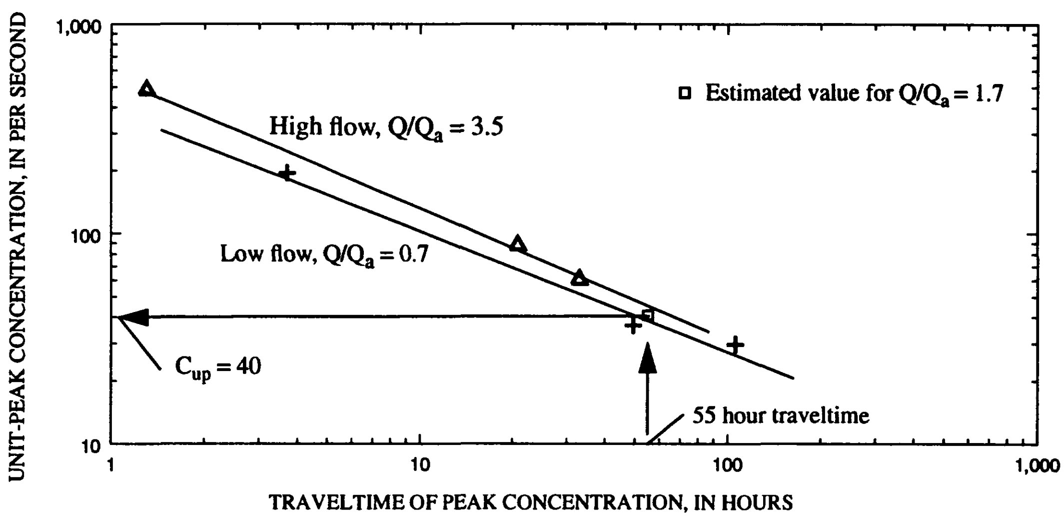

Figure 14. Unit-peak concentrations of dye for the Apple River.

be estimated using the same procedure as for the peak concentration, as 51.1 hours. After 51.1+13.9 = 65 hours the pollution cloud should have passed the intake and the concentration reduced to 0.024 mg/L.

In conclusion, the pollutant should first arrive at Hanover 51 hours after the spill. The peak concentration should pass the site 55 hours after the spill; and if there are no losses, it should arrive with a concentration of 0.24 mg/L. By 65 hours after the spill, the concentration should have fallen back to 0.024 mg/L. If there are losses or chemical reactions between the spill and the intake, the concentrations will be smaller and either equation 20 or a numerical model could be used for predictions.

Example 3, Application to the Rhine River

With a catchment area of 180,000 km^3, the Rhine River is a very important European river (Spreafico and van Mazijk, 1993, p. 19). Because of the high population density and heavy use, there is always the potential that the river will be accidently polluted. The International Rhine Commission has been set up to help reduce the danger of accidents and to help respond to them if they occur. The Commission developed, calibrated, and verified the Alarm model to be used in responding to accidental spills. As part of the calibration process, the response to a slug injection near river km 59 was measured at Eglisau (km 78.7) and Birsfelden (km 163.8) (Spreafico and van Mazijk, 1993, p. 95). In this example, the measured response curves will first be predicted based on the river discharge and drainage area. To illustrate the value of time-of-travel data, improved predictions of the unit-peak concentration, as well as the time of the leading edge, and time of passage of the cloud will then be made using the traveltime measured for the peak concentration.

The mean annual flow of the Rhine River is 0.0152 $m^3/s/km^2$ (Leeden and others, 1990, p. 181). Because the drainage area is approximately 16,000 and 48,000 km^2 at river km 59 and 163.8, respectively, the mean annual flow can be estimated as 240 m^3/s at the injection point and 730 m^3/s at Birsfelden.

The response function characteristics at Eglisau and Birsfelden are first estimated without the aid of traveltime information. Assuming the drainage area at Eglisau is the same as at the injection site, the dimensionless drainage area, for use in equation 14, is computed as:

$$D'_a = (16x10^9)^{1.25}x(9.81)^{0.5}/240 = 7.43x10^{10}.$$

During the test, the river flow was 490 m^3/s at the injection point (Spreafico and van Mazijk, 1993, p. 65) so the relative discharge is estimated as:

$$Q'_a = 490/240 = 2.04.$$

With these values the velocity can be predicted from equation 14 as:

$$V_p = 0.020+0.0509x(7.43x10^{10})^{0.821}x2.04^{-0.465}x(490/16x10^9) = 0.96 \text{ m/s},$$

so the traveltime to the peak concentration is estimated as:

$$T_p = (78.7-59)1000/(3600x0.96) = 5.7 \text{ hours}.$$

Applying equation 7, the unit-peak concentration can be estimated as:

$$C_{up} = 857 \cdot 5.7^{-0.760 \cdot 2.04^{-0.079}} = 245 s^{-1}.$$

The time of first arrival is estimated as 0.89x5.7 = 5.1 hours (equation 18). The time of passage of the pollutant can be determined from equation 19 to be 2.3 hours, so the time from the spill until the unit concentration has returned to within 24 s^{-1} is 7.4 hours.

The flow at Birsfelden during the test was 1,068 m^3/s (Spreafico and van Mazijk, 1993, p. 65) so the same procedure can be used to determine values a Birsfelden as:

$$D'_a = (48x10^9)^{1.25}x(9.81)^{0.5}/730 \qquad = 9.64x10^{10}$$

$$Q'_a = 1068/730 \qquad = 1.46$$

$$V_p = 0.020+0.0509x(9.64x10^{10})^{0.821}x1.46^{-0.465}x(1068/48x10^9) \qquad = 1.01 \text{ m/s}$$

$$T_p = (163.8-59)1000/(3600x1.01) \qquad = 28.8 \text{ hours}$$

$$C_{up} = 857 \cdot (28.8)^{-0.760 \cdot 1.46^{-0.079}} \qquad = 71.9 s^{-1}$$

$$T_l = 0.89x28.8 \qquad = 25.6 \text{ hours}$$

$$T_{10d} = 2x10^6/(71.9x3600) \qquad = 7.7 \text{ hours}$$

$$T_{10t} = 25.6+7.7 \qquad = 33.3 \text{ hours}$$

Figure 15 contains a plot of these computed values along with observed data from (Spreafico and van Mazijk, 1993, p. 95). As can be seen from the plot, the timing is not good. Prediction of the solute velocity is the least reliable component of the procedures outlined herein. A major reason for this is that most rivers and streams have been modified so that the storage volume has increased. Equation 16 contains data from rivers with varying degrees of manmade storage and no easy way of quantifying this storage was available. Boning (1974) has presented a traveltime prediction equation that includes the effect of storage volumes if these are available. It may be about as easy, and far more accurate, to measure traveltimes as to accurately quantify the storage volumes.

If the traveltime is available, the estimates can be much improved. For example, the traveltime of the peak concentration to Eglisau can be seen from figure 15 to be 6.5 hours, so time to the leading edge can be estimated as 0.89x6.5 = 5.8 hours (equation 18) and the unit-peak concentration can be estimated from equation 7 as 222 s^{-1}. The time for the concentration to be reduced to 10 percent of the peak can be estimated from the duration given by equation 19 (2.5 hours) plus the time to the leading edge (5.8 hours) to be 8.3 hours. Likewise, reading the traveltime of the peak to Birsfelden from figure 15 as 32.7 hours, the time to the leading edge can be estimated as 29.1 hours, the unit-peak concentration as 65.4 s^{-1}, and the time for the concentration to be reduced to 10 percent of the peak as 37.6 hours. The improved estimates

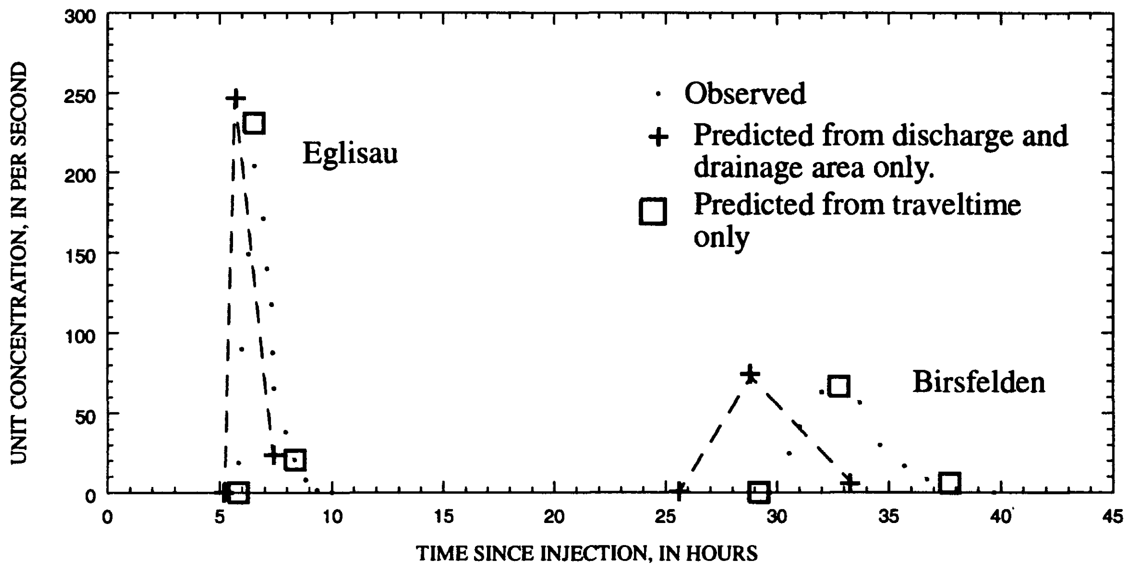

Figure 15. Prediction of unit response resulting from a dye injection on the Rhine River.

(based traveltime to the peak concentration) are also shown on figure 15 for comparison with the observed data and estimates made without the benefit of traveltime information. The entire response function can be predicted with a high degree of accuracy when only the traveltime of the peak concentration is accurately known.

EXTENSION TO CONTINUOUS SOURCES BY USE OF THE SUPERPOSITION PRINCIPLE

One of the most useful tools to hydrologists has been the unit-hydrograph method (Linsley and others, 1958) for predicting stream runoff from precipitation in a drainage basin. The unit-hydrograph theory assumes that the stream runoff response is linear and that unit hydrographs can be added to synthesize the response to different rainfalls.

Another application of the linear superposition approach is for the simulation of buildup of soluble-pollutant concentrations in streams and estuaries using tracer tests (Bailey and others, 1966; Yotsukura and Kilpatrick, 1973). By this method, the response to a slug injection of a soluble tracer is assumed to imitate the characteristic of a soluble pollutant, and as such, can be used to simulate it.

The superposition approach has the advantage of simplicity and accuracy when applied to steady flow or to the exact flow conditions for which the response function was measured. Its weaknesses are that it can only be used with flow conditions for which it was derived and chemical interactions cannot easily be considered. The strengths of the numerical modeling approach are that it can account for unsteady flow conditions, and chemical reactions can be easily simulated if the reaction rate constants are known.

Kilpatrick and Cobb (1985) showed that the response curve of a continuous, constant-rate injection of tracer could be simulated by adding tracer-response curves from a sequence of single slug injections on the same stream, location, and discharge. For example, assume a series of slug injections of tracer (simulating a constant injection), each of mass, M^m, is injected in the stream depicted in figure 16. A repetition of the same responses downstream at the different times shown would result.

There would be a buildup to a constant plateau concentration as shown in figure 16. If discharge remained constant and the injection were continued long enough, the same would occur at every distance downstream, so that plateaus of concentration would ultimately exist at every downstream section.

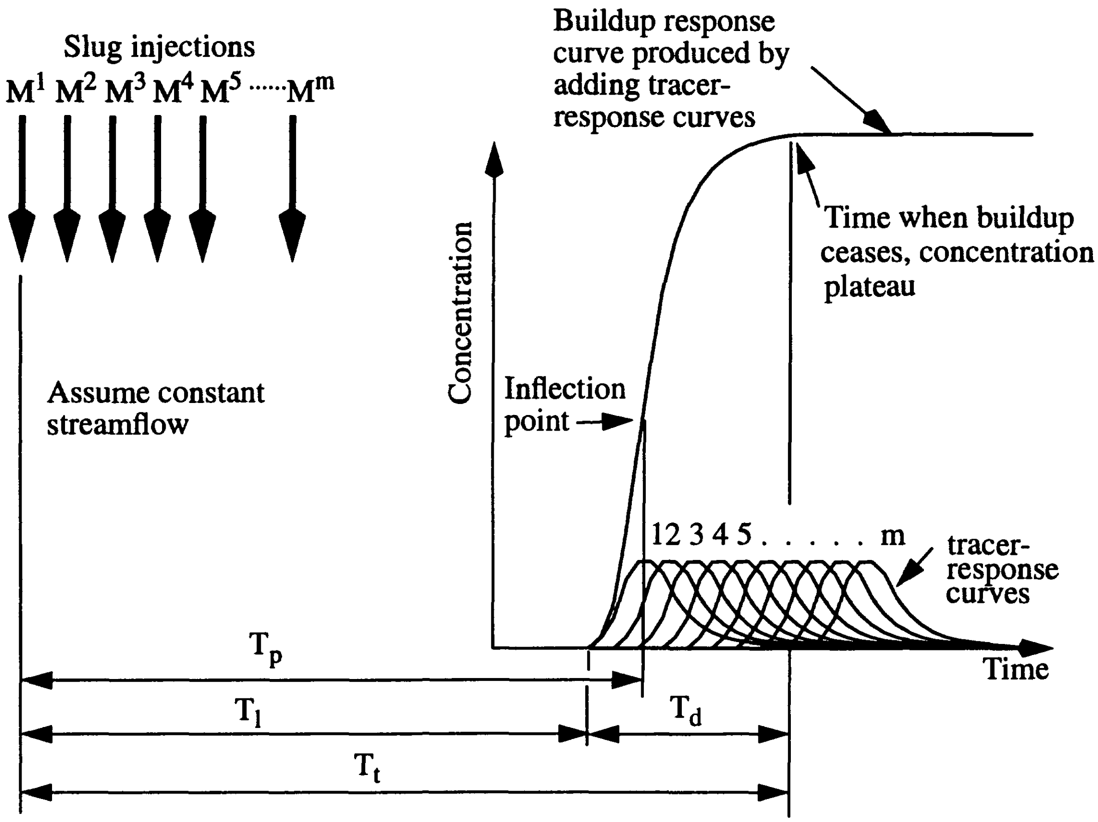

Figure 16. Superposition of tracer-response curves to simulate constant-injection buildup to a plateau at one location in a stream section.

As can be seen on figure 16, in order for a plateau to be reached at any particular location, a constant injection must be maintained for a length of time equal to the duration of the tracer-response curve, T_d. Similarly, the duration of the constant injection necessary to establish a plateau in the entire stream reach shown in figure 16 is dictated by the longest slug response duration at the most downstream location.

It becomes apparent that the tracer-response curve produced by a slug injection of tracer may be used as a building block with the superposition principle to simulate the buildup of a given pollutant in the stream. In fact, linearity permits the superposition of a variable loading of pollution to simulate the resulting response downstream. For convenience, it is practical to reduce all measured curves to UR curves using equation 4.

Example 4, Use of the Superposition Principle

Assume that a chemical plant is to be constructed on the Apple River at the location of the spill of example 2. Determine the maximum pollutant concentration at the Hanover intake for a chemical spill assuming a steady flow at the point of the spill of 1.2 m^3/s. Spills of 70, 300, 150, 140, and 80 kg occur at hours 0, 1, 7, 8, and 9, respectively.

The results of example 2 provided three points on the unit-response curve, the time of the leading edge (51.1 hours), the time (55.2 hours) and magnitude (40 s^{-1}) of the unit-peak concentration, and the time when the trailing edge has been reduced to 4 s^{-1} (65 hours). These three points are plotted on figure 17 along with a smooth curve drawn through the points. The ordinates of the smooth curve are given in table 1. Notice that the sum of the ordinates, 277.78 s^{-1}, times the number of seconds between ordinates, 3,600, equals 1×10^6.

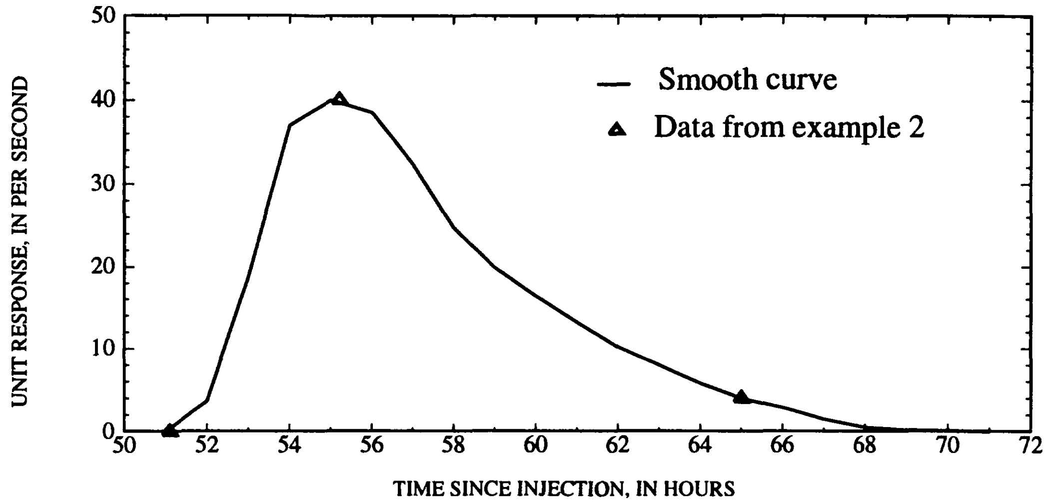

Figure 17. Unit-response function for concentration at Hanover on Apple River.

Table 1. Response function ordinates for Apple River at Hanover
[Unit concentrations given in per second]

Hour	Unit concentration	Hour	Unit concentration	Hour	Unit concentration
51	0.0	58	24.7	65	4.0
52	3.7	59	19.9	66	2.9
53	18.78	60	16.4	67	1.5
54	37.0	61	13.2	68	0.5
55	40.0	62	10.2	69	0.2
56	38.5	63	8.0	70	0.1
57	32.4	64	5.8	71	0.0
				Total	277.78

The unit values in table 1 must be converted to concentrations by use of equation 4. If the discharge at Apple River is 1.2 m^3/s, the flow at Hanover is likely to be 8.5 m^3/s as indicated in example 2. The computations for the response to each load are carried out in table 2. The Hanover concentration at hour 53, for example, is affected by only the first two loads as computed by:

$$C = (3.7 \times 300 + 18.78 \times 70) \times 1 \times 10^6 / (1 \times 10^6 \times 8.5 \times 1000) = 0.131 + 0.155 = 0.286 \text{ mg/L}$$

in which 3.7 and 18.78 are the first two ordinates of the unit concentration response function (C_u in equation 4) and 300 and 70 (times 10^6) are the loads, in milligrams, at hour 1 and 0, respectively (M_r in equation 4). The loads, responses to each load, and resulting concentration at Hanover are tabulated in table 2 and plotted on figure 18.

As can be seen from either figure 18 or table 2, the maximum concentration at Hanover results from the second spill, which has a smaller initial load but a longer duration. The advantage of the superposition principle is its simplicity and its disadvantage is that it assumes steady flow. If unsteady flow conditions are present or if complex chemical reactions are to be simulated, a numerical model would be needed.

Table 2. Computation of resultant concentration at Hanover resulting from spills 45 kilometers upstream

Time since spill (hour)	Amount spilled (kilogram)	Time since spill (hour)	****** Response at Hanover ****** Concentration to result from indicated load Load 1	Load 2	Load 3	Load 4	Load 5	Total
0	70	51	0.0					0.0
1.0	300	52	0.030	0.0				0.030
2.0		53	0.155	0.131				0.286
3.0		54	0.305	0.663				0.968
4.0		55	0.329	1.306				1.635
5.0		56	0.317	1.412				1.729
6.0		57	0.267	1.359				1.626
7.0	150	58	0.203	1.144	0.0			1.347
8.0	140	59	0.164	0.872	0.065	0.0		1.101
9.0	80	60	0.135	0.702	0.331	0.061	0.0	1.229
		61	0.109	0.579	0.653	0.309	0.035	1.685
		62	0.084	0.466	0.706	0.609	0.177	2.042
		63	0.066	0.360	0.679	0.659	0.348	2.112
		64	0.048	0.282	0.572	0.634	0.376	1.912
		65	0.033	0.205	0.436	0.534	0.362	1.570
		66	0.024	0.141	0.351	0.407	0.305	1.228
		67	0.012	0.102	0.289	0.328	0.232	0.963
		68	0.004	0.053	0.233	0.270	0.187	0.747
		69	0.002	0.018	0.180	0.217	0.154	0.571
		70	0.001	0.007	0.141	0.168	0.124	0.441
		71	0.0	0.004	0.102	0.132	0.096	0.334
		72		0.0	0.071	0.096	0.075	0.242
		73			0.051	0.066	0.055	0.172
		74			0.026	0.048	0.038	0.112
		75			0.009	0.025	0.027	0.061
		76			0.004	0.008	0.014	0.026
		77			0.002	0.003	0.005	0.010
		78			0.0	0.002	0.002	0.004
		79				0.0	0.001	0.001
		80					0.0	0.0

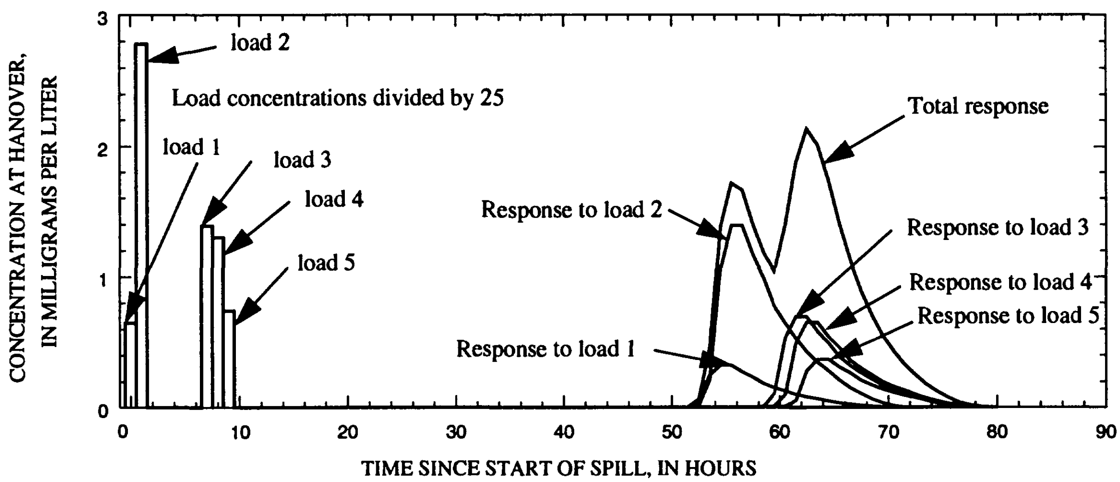

Figure 18. Example of using the superposition principle to determine response at Hanover to a chemical spill 42 kilometers upstream.

CONCLUSIONS

The possibility of a contaminant being accidentally or intentionally spilled in a river upstream of a water supply is an ever-present danger; and a method of rapidly estimating traveltime and dispersion in rivers is needed by all water-resources planners and managers. A numerical model is often considered to fulfill this need. Unfortunately, numerical models are not truly predictive because they must be calibrated with data from the river being modeled. Generally, mean stream velocities cannot be accurately predicted without very detailed cross-sectional geometry and flow resistance estimates. Time-of-travel studies typically provide more accurate traveltime estimates and are much cheaper to conduct than the detailed surveying necessary to obtain adequate channel-geometry data for flow models. Generally speaking, dispersion coefficients cannot be accurately predicted without dispersion studies on the river in question.

This report compiles information from a large number of time-of-travel and dispersion studies and presents empirical relations that appear to have general applicability. These relations are not recommended as a substitute for field studies but are believed to provide reasonable estimates in situations where adequate field data are not available. Empirical relations are given for the unit-peak concentrations, velocity of the peak concentration, velocity of the leading edge of a solute cloud, and the duration of the time of passage as measured from the leading edge to the point where the solute concentration has fallen to 10 percent of its peak value. It is shown how this information can be used to estimate the complete response function, which can then be used with the superposition principle to estimate the effect of multiple spills. The recommended methods are demonstrated by presenting four examples.

If the solute transport in the river is to be modeled, the model must be calibrated to provide the correct traveltimes and rates of attenuation of the peak concentration. The relations presented in this report can be used to calibrate a solute-transport model for use on a river that has little field data.

The relation for unit-peak concentration is the best defined of all the relations needed to predict the transport and dispersion of pollutants. Field data show that the peak concentration tends to decrease more rapidly with time than predicted by Fickian dispersion. Because almost all numerical models are based on the Fickian relation, model dispersion coefficients must be assumed to increase with time for the model results to duplicate observed data.

The relation for predicting mean stream velocity (traveltime) is the least accurately defined of all relations presented in this report. Traveltime information is, therefore, the most valuable information that can be collected to improve the ability to predict the transport and dispersion in a river. These data should be collected at two or more flows, preferably a low flow and a high flow.

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APPENDIXES

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APPENDIX A. BASIC DATA

The following tables contain data compiled for use in this report. Table A-1 contains the data from studies where the complete tracer-response curve was measured at each site. Data from the entire curve allows the recovery ratio to be determined and the unit-peak concentration to be computed. The data in table A-2 are from studies where the complete dye curve was not sampled and the emphasis of the study was on only the traveltime.

Some data, particularly slope, drainage area, and mean annual flow, are not available in the listed references. When slope was missing from the referenced report, it was estimated from topographic maps. Generally, a map scale of 1/25,000 was used for short reaches in the small streams and a map scale of 1/100,000 was used for long reaches of large rivers.

When the drainage area and mean annual flow were missing from the referenced reports, they were generally determined from data contained in annual reports published by the Geological Survey (Water Resources Data STATE Water Year XXX). These reports give daily mean discharge, river mile, and drainage area at each gage operated by the Geological Survey. Generally one or more gaging stations were located on the river within a study reach. Drainage areas at specific cross sections were estimated by assuming that the logarithm of the drainage area varied linearly with the river mile between points of known drainage area. In some cases, where the study reach extended downstream of an available Geological Survey gage, the drainage area of the entire basin (at river mile 0.0) was determined from the State Hydrologic Unit Maps (Seaber and others, 1984). In a very few cases the drainage area was measured from a topographic map.

The mean annual discharge at specific cross sections was computed by assuming that the discharge was proportional to drainage area for the reach between available gages. In the case of the Mississippi River below Baton Rouge, La., the mean flow was assumed to be two-thirds of the mean flow at Vicksburg, Miss., because the U.S. Army Corps of Engineers diverts one-third of the flow in the Mississippi River to the Atchafalaya River above Baton Rouge.

The column headings in the tables are brief in order to save space. The following is a more complete description of the data contained in tables:

River:	The name of the stream or river where the data were collected. In table A-2 the begin and end points are briefly defined.
Inj No:	A different number is given for each dye injection.
Km:	For table A-1 it is the distance, in kilometers, of the sampling cross section downstream of the dye injection. For table A-2 it is the distance, in kilometers, between the sampling cross sections that define the reach within which the velocity is determined.
Q:	The discharge, in cubic meters per second, at the sampling cross section during the passage of the dye cloud.
Tl:	The time, in hours, from the injection until the dye first reached the sampling cross section.
Tp:	The time, in hours, from the injection until the peak concentration was observed at the sampling cross section.
Tt:	The time, in hours, from the injection until the dye concentration at the sampling cross section was reduced to 0.1 times the peak. For injection numbers 52 to 59 it is the time until the dye concentration at the sampling cross section was reduced to 0.05 times the peak.
Qave:	The mean annual flow, in cubic meters per second, at the sampling cross section.
Da Ar:	The drainage area, in square kilometers, of the river at the sampling section.
Slope:	The slope of the river, in meter per meter, in the subreach upstream of the sampling section.
Depth:	The water depth, in meters, at the sampling cross section.
Width:	The width of the water surface, in meters, at the sampling cross section.
Cup:	The unit-peak dye concentration at the sampling cross section as defined by equation 4, with units of per second.
Inj Mass:	The mass of dye injected, in grams.
R ratio:	The recovery ratio for the measurement cross section as determined by equation 3.
Ref:	Reference number as given in the references to Appendix A.
Vp:	The velocity of the dye, in meters per second, as determined by the distance between the measurement cross sections divided by the difference in the times to the peak dye concentrations.

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Table A-1. Compiled data for studies publishing the complete dy curve

River	Inj No	Km	Q	Ti	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Antietam Creek	1	2.6	1.2	4.51	3.20	4.51	2.6	240	0.00270	0.3	12.8	264.6	464	0.871	24
Antietam Creek	1	9.6	1.2	10.56	13.00	16.39	2.7	255	0.00110	0.5	11.0	99.0	464	0.755	24
Antietam Creek	1	21.5	1.6	36.49	42.50	52.90	4.6	429	0.00110	0.3	12.8	39.9	464	0.433	24
Antietam Creek	1	29.6	1.8	57.25	67.00	82.40	4.9	466	0.00080			26.8	464	0.318	24
Antietam Creek	2	2.6	5.1	1.01	1.35	1.80	2.6	240	0.00270			682.1	2474	1.057	24
Antietam Creek	2	9.6	5.2	4.61	5.50	6.48	2.7	255	0.00110			296.1	2474	1.074	24
Antietam Creek	2	21.5	7.4	14.06	15.90	17.79	4.6	429	0.00110	0.5	28.0	157.4	2474	0.926	24
Antietam Creek	2	29.6	7.8	21.07	23.40	26.15	4.9	466	0.00080		0.0	116.0	2474	0.664	24
Antietam Creek	2	42.2	8.5	30.09	33.20	37.50	5.4	509	0.00150	0.7	23.5	76.2	2474	0.610	24
Antietam Creek	2	49.2	10.2	34.11	38.00	42.58	6.4	604	0.00170			71.6	2474	0.490	24
Antietam Creek	2	59.2	12.2	40.07	43.30	48.94	7.7	727	0.00100	1.0	24.1	66.1	2474	0.467	24
Antietam Creek	2	66.8	13.0	43.25	47.40	53.28	8.0	757	0.00100			60.1	2474	0.442	24
Antietam Creek	3	2.6	2.4	2.02	2.70	4.03	2.6	240	0.00270			290.0	928	1.017	24
Antietam Creek	3	9.6	2.4	7.85	9.50	11.73	2.7	255	0.00110			138.1	928	0.928	24
Antietam Creek	3	21.5	3.4	23.86	27.40	32.47	4.6	429	0.00110	0.7	11.9	65.8	928	0.628	24
Antietam Creek	3	29.6	4.0	36.60	40.10	46.87	4.9	466	0.00080			56.1	928	0.501	24
Antietam Creek	4	12.6	4.5	11.84	13.80	17.92	5.4	509	0.00150	0.5	25.0	93.2	1546	0.692	24
Antietam Creek	4	19.6	5.3	18.35	21.20	26.34	6.4	604	0.00170			68.5	1546	0.661	24
Antietam Creek	4	29.6	6.4	25.54	29.00	35.86	7.7	727	0.00100	0.5	21.0	54.4	1546	0.626	24
Antietam Creek	4	37.1	6.5	30.61	34.20	41.55	8.0	757	0.00100			58.4	1546	0.560	24
Monocacy River	5	10.3	5.4	11.02	12.50	16.41	19.7	1586	0.00060			93.1	715	0.916	21
Monocacy River	5	18.3	5.7	20.12	23.00	30.86	20.5	1650	0.00060			54.0	715	1.058	21
Monocacy River	5	26.8	6.4	28.24	33.50	42.91	22.6	1819	0.00050	0.6	48.8	37.3	715	1.142	21
Monocacy River	5	34.3	7.6	37.10	43.50	54.20	26.2	2115	0.00030			31.2	715	1.215	21
Monocacy River	6	7.5	8.1	7.07	8.50	13.38	26.2	2115	0.00030			94.1	1546	1.231	21
Monocacy River	6	18.8	8.4	25.14	29.00	38.72	27.6	2221	0.00030			39.7	1546	1.004	21
Monocacy River	6	27.6	9.3	35.08	41.00	53.53	30.4	2454	0.00060			30.3	1546	0.972	21
Monocacy River	6	33.8	9.5	42.16	48.00	62.42	31.2	2517	0.00030	0.7	93.0	28.0	1546	0.930	21
Monocacy River	7	10.3	14.3	6.23	7.10	9.88	19.7	1586	0.00060	0.8	37.2	138.8	1906	1.146	21
Monocacy River	7	18.3	15.1	11.71	13.60	17.39	20.5	1650	0.00060	1.0	50.0	94.3	1906	1.122	21
Monocacy River	7	26.8	15.9	17.01	19.60	24.21	22.6	1819	0.00050			74.9	1906	1.034	21
Monocacy River	7	34.3	18.5	22.05	25.80	31.27	26.2	2115	0.00030	0.6	51.2	58.9	1906	1.252	21
Monocacy River	8	7.3	20.4	4.01	5.25	7.31	26.2	2115	0.00030	0.6	51.2	147.0	3711	1.567	21
Monocacy River	8	18.8	20.4	13.27	15.50	19.40	27.6	2221	0.00030	1.2	59.4	85.5	3711	1.225	21

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Monocacy River	8	27.8	22.1	18.88	22.00	27.81	30.4	2454	0.00060	0.8	48.2	60.0	3711	1.056	21
Monocacy River	8	33.8	22.1	24.24	27.50	33.16	31.2	2517	0.00030	1.1	97.5	58.8	3711	0.964	21
Monocacy River	9	7.5	3.1	13.82	17.50	26.79	26.2	2115	0.00030	0.3	46.6	41.0	1855	0.797	21
Monocacy River	9	18.8	3.2	52.14	52.50	96.30	27.6	2221	0.00030	0.6	33.5	13.6	1855	0.571	21
Monocacy River	9	27.6	3.5	69.06	86.00	125.36	30.4	2454	0.00060	0.3	51.2	10.3	1855	0.479	21
Monocacy River	9	33.8	3.5	83.22	99.00	153.00	31.2	2517	0.00030	0.5	31.4	9.0	1855	0.440	21
Conococheague Creek	10	4.4	6.8	2.41	3.10	3.62	16.5	1279		0.0	0.0	441.4	1546	1.544	24
Conococheague Creek	10	8.7	6.9	5.81	6.60	8.38	17.2	1330	0.00050	0.5	35.1	208.8	1546	1.407	24
Conococheague Creek	10	13.4	6.9	9.61	10.20	13.11	17.5	1356	0.00070	0.3	46.0	170.0	1546	1.072	24
Conococheague Creek	10	19.9	6.9	14.42	15.20	19.33	17.9	1381	0.00070	0.4	65.5	123.3	1546	0.881	24
Conococheague Creek	10	26.0	6.9	19.63	21.10	26.70	17.9	1381	0.00060	0.7	43.3	76.0	1546	0.796	24
Conococheague Creek	10	33.9	7.1	29.94	33.80	42.73	18.9	1458	0.00060	0.7	41.1	45.1	1546	0.566	24
Conococheague Creek	11	4.4	2.6	4.62	5.40	8.38	16.5	1279	0.00070	0.4	24.1	138.6	1429	1.119	24
Conococheague Creek	11	8.7	2.8	11.26	13.90	19.69	17.2	1330	0.00050	0.4	35.7	69.1	1429	0.992	24
Conococheague Creek	11	13.4	2.9	18.88	24.80	38.00	17.5	1356	0.00070	0.5	61.0	29.6	1429	1.031	24
Conococheague Creek	11	19.9	2.9	31.51	37.70	51.35	17.9	1381	0.00070	0.4	42.7	30.1	1429	0.928	24
Conococheague Creek	12	4.4	29.4	1.25	1.40	1.71	16.5	1279	0.00070	0.8	59.4	894.6	1855	1.490	24
Conococheague Creek	12	8.7	29.4	2.81	3.15	4.12	17.2	1330	0.00050			445.8	1855	0.906	24
Conococheague Creek	12	13.4	29.7	4.60	5.00	6.71	17.5	1356	0.00070			301.4	1855	0.942	24
Conococheague Creek	12	19.9	30.0	7.00	7.60	9.32	17.9	1381	0.00070			240.2	1855	0.862	24
Conococheague Creek	12	26.0	30.3	9.41	10.40	12.81	17.9	1381	0.00060	1.1	43.0	150.0	1855	0.821	24
Conococheague Creek	12	33.9	30.6	13.32	14.80	18.22	18.9	1458	0.00060			110.5	1855	0.776	24
Chattahoochee River	13	16.0	136.2	4.02	4.83	5.94	62.0	2848	0.00050	0.0	0.0	286.2	7813	0.698	16
Chattahoochee River	13	27.9	179.5	7.02	8.08	9.57	65.1	3029	0.00028	2.9	65.8	183.3	7813	1.054	16
Chattahoochee River	13	36.5	174.7	9.09	10.33	12.47	66.9	3107	0.00061			168.7	7813	0.966	16
Chattahoochee River	13	49.6	169.9	13.03	14.80	17.78	62.4	3262	0.00031	2.3	77.7	117.0	7813	1.072	16
Chattahoochee River	13	57.1	107.6	17.70	19.83	25.67	65.9	3443	0.00002			81.0	7813	0.742	16
Chattahoochee River	13	60.5	107.6	19.03	21.50	28.35	65.9	3443	0.00044			63.9	7813	0.836	16
Chattahoochee River	13	76.6	107.6	24.75	28.82	39.28	72.6	3495	0.00099	2.0	75.6	47.3	7813	0.929	16
Chattahoochee River	14	10.5	140.2	2.20	2.60	3.49	82.8	4479	0.00020			402.9	6218	0.883	16
Chattahoochee River	14	30.1	141.0	8.15	9.60	11.54	103.8	5333	0.00025	2.3	74.1	159.8	6218	0.998	16
Chattahoochee River	14	65.4	139.6	18.57	22.16	25.86	113.5	6291	0.00029	2.4	84.7	88.7	6218	0.977	16
Chattahoochee River	14	104.6	139.6	33.40	38.33	46.00	122.5	6938	0.00045	2.5	100.0	49.4	6218	0.981	16
Salt Creek	15	9.3	2.5	6.62	7.75	9.53	7.2	1993	0.00051	0.2	24.4	173.8	910	0.821	16

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Ti	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Salt Creek	15	15.3	2.6	10.68	12.50	15.54	8.0	2226	0.00048	0.2	39.0	114.5	910	0.836	16
Salt Creek	15	31.3	3.0	22.62	25.00	31.13	10.6	2925	0.00050	0.3	29.0	70.6	910	0.654	16
Salt Creek	15	44.6	3.0	34.04	37.50	46.46	13.4	3702	0.00043	0.3	29.0	47.1	910	0.573	16
Salt Creek	15	51.8	4.1	43.09	48.00	63.59	15.2	4194	0.00023	0.7	35.1	33.8	910	0.508	16
Difficult Run	16	0.6	1.0	0.81	1.20	2.07	1.7	150				463.5	24	1.027	16
Difficult Run	16	2.2	1.0	3.01	3.80	5.13	1.7	150	0.00195	0.4	11.6	234.1	24	0.972	16
Difficult Run	16	3.2	1.1	5.41	6.20	8.24	1.7	150	0.00058	0.2	17.4	174.3	24	1.065	16
Bear Creek	17	1.1	10.2	0.18	0.22	0.30	1.3	330	0.01080			4347.7	101	0.958	16
Bear Creek	17	6.0	10.4	1.70	1.88	2.10	1.4	329	0.01770			1305.5	101	0.690	16
Bear Creek	17	10.9	10.5	3.38	3.60	3.89	1.5	425	0.03670	0.9	13.7	1040.7	101	0.649	16
Ltl Piney Creek	18	0.6	1.4	0.61	0.85	1.47	4.4	518	0.00130	0.2	15.8	873.2	48	0.972	16
Ltl Piney Creek	18	3.4	1.4	3.75	5.10	7.70	4.4	518				138.1	48	0.948	16
Ltl Piney Creek	18	5.2	1.6	6.03	7.40	10.80	4.4	518				106.1	48	0.988	16
Ltl Piney Creek	18	7.3	1.6	9.21	12.00	17.15	4.4	518				75.1	48	1.000	16
Bayou Anacoca	19	11.4	2.0	17.05	20.00	23.88	13.6	951	0.00047	0.3	15.2	81.1	953	0.896	16
Bayou Anacoca	19	23.2	2.6	37.73	43.00	50.80	14.9	1045	0.00057			42.8	953	0.815	16
Bayou Anacoca	19	29.8	2.7	47.57	52.70	61.31	15.7	1102	0.00057	0.6	19.8	39.6	953	0.713	16
Bayou Anacoca	19	38.0	2.7	62.99	68.70	78.41	16.8	1177	0.00057			36.8	953	0.607	16
Comite River	20	6.8	0.8	20.50	23.70	27.63	4.0	219	0.00068	0.5	6.1	76.1	1668	1.228	16
Comite River	20	26.9	0.9	80.16	87.20	98.19	6.0	328	0.00079	0.3	11.6	28.9	1668	0.603	16
Comite River	20	48.0	1.0	102.11	109.00	125.02	9.3	500	0.00076	0.2	19.8	26.1	1668	0.436	16
Comite River	20	60.4	1.1	114.50	122.00	142.90	12.1	642	0.00040	0.2	11.6	22.8	1668	0.413	16
Comite River	20	78.9	1.0	132.15	141.00	168.80	17.8	931	0.00023	0.3	13.1	18.1	1668	0.239	16
Bayou Bartholomew	21	3.2	4.1	4.54	5.75	8.46	43.5	2937	0.00011	1.2	22.9	146.8	2817	0.811	16
Bayou Bartholomew	21	25.7	4.8	52.19	61.50	71.00	47.0	3173	0.00007			28.9	2817	0.842	16
Bayou Bartholomew	21	59.5	6.5	108.54	123.00	139.78	52.8	3564	0.00006	0.7	29.3	18.5	2817	0.844	16
Bayou Bartholomew	21	117.5	8.1	199.30	234.50	281.45	64.4	4349	0.00008	2.1	37.5	6.6	2817	1.404	16
Amite River	22	10.1	5.7	11.10	13.90	20.46	18.5	1022	0.00059	0.5	21.3	64.9	4288	0.816	16
Amite River	22	38.8	7.4	46.61	57.25	80.51	25.4	1405	0.00053			17.0	4288	0.514	16
Amite River	22	94.1	9.9	92.49	107.30	144.68	46.9	2602	0.00051			16.1	4288	0.341	16
Amite River	22	148.1	8.9	105.52	124.25	152.55	68.3	3797	0.00035	0.5	46.0	9.3	4288	0.417	16
Tickfau River	23	6.4	2.0	8.81	11.20	13.65	5.3	342	0.00160	0.4	12.2	102.4	1191	0.829	16
Tickfau River	23	22.5	2.2	47.54	54.20	62.35	7.5	461	0.00095	0.8	19.2	36.9	1191	0.764	16
Tickfau River	23	38.6	1.9	78.78	87.00	99.38	10.5	621	0.00095	0.5	13.4	28.6	1191	0.560	16

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	inj No	Km	Q	Ti	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	inj Mass	R ratio	Ref
Tickfau River	23	49.9	2.9	97.40	105.00	117.91	13.9	806	0.00020	1.0	41.5	27.3	1191	0.781	16
Tangipahoa River	24	8.2	5.8	8.00	9.75	12.56	4.5	226	0.00076	1.1	23.2	127.3	4764	1.185	16
Tangipahoa River	·24	18.0	9.8	18.60	21.30	26.10	5.8	292	0.00071	0.8	31.7	109.3	4764	1.170	16
Tangipahoa River	24	41.5	11.9	31.97	35.00	42.75	10.7	540	0.00066	0.8	32.9	55.7	4764	1.045	16
Tangipahoa River	24	55.4	12.4	40.79	45.00	53.00	15.3	775	0.00057	0.8	29.9	50.8	4764	0.945	16
Tangipahoa River	24	71.0	14.4	51.06	56.80	65.74	23.0	1165	0.00038	1.3	43.0	41.2	4764	1.047	16
Tangipahoa River	24	82.1	17.8	60.30	66.00	76.14	30.8	1557	0.00027			40.6	4764	0.891	16
Tangipahoa River	24	94.0	18.7	71.28	77.17	86.49	34.7	1755	0.00019			39.6	4764	0.869	16
Tangipahoa River	25	8.2	3.5	11.45	13.75	18.88	4.5	226	0.00076	0.5	17.1	74.4	3811	1.023	16
Tangipahoa River	25	18.0	4.6	25.36	30.00	36.50	5.8	292	0.00071			51.5	3811	0.973	16
Tangipahoa River	25	41.5	6.9	41.08	47.00	56.26	10.7	540	0.00069			36.9	3811	0.802	16
Tangipahoa River	25	55.4	8.1	52.95	59.00	69.56	15.3	775	0.00066	0.4	29.9	32.7	3811	0.860	16
Tangipahoa River	25	71.0	8.6	66.15	73.50	83.38	23.0	1165	0.00057	0.5	40.2	33.2	3811	0.741	16
Tangipahoa River	25	82.1	9.0	76.92	83.00	93.61	30.8	1557	0.00038	0.8	31.7	30.8	3811	0.660	16
Tangipahoa River	25	94.0	10.8	88.99	96.50	106.72	34.7	1755	0.00027			29.9	3811	0.696	16
Red River	26	5.7	235.3	2.25	2.50	4.09	739.2	167422	0.00009	4.8	156.7	290.1	23440	0.741	16
Red River	26	75.6	245.2	28.90	33.50	39.19	815.3	172761	0.00009			56.6	23440	0.740	16
Red River	26	132.8	249.5	53.88	60.00	68.75	848.0	175697	0.00007			38.1	23440	0.695	16
Red River	26	193.1	249.5	84.86	92.50	104.75	868.2	179886	0.00007	1.6	253.6	27.2	23440	0.587	16
Red River	27	8.0	139.6	2.92	3.35	5.89	530.1	149561	0.00011	1.3	209.4	177.7	13673	1.203	16
Red River	27	54.7	139.6	27.50	32.00	40.86	665.2	161251	0.00015			40.8	13673	0.805	16
Red River	27	103.0	170.8	46.96	52.00	62.78	712.1	165331	0.00013			37.6	13673	0.854	16
Red River	27	159.3	184.9	79.03	87.50	98.52	739.1	167414	0.00009	4.0	161.5	26.3	13673	0.772	16
Red River	28	8.0	187.5	4.11	4.75	7.63	530.1	149561	0.00011			172.1	18199	1.342	16
Red River	28	54.7	187.5	28.91	33.50	41.27	665.2	161251	0.00015			45.4	18199	1.040	16
Red River	28	103.0	235.0	48.98	54.25	63.29	712.1	165331	0.00013			37.6	18199	0.924	16
Red River	28	159.3	251.7	78.00	85.25	99.12	739.1	167414	0.00009	3.7	152.4	27.7	18199	0.847	16
Red River	29	12.1	107.6	6.28	7.00	9.87	739.1	167414	0.00009	2.5	132.6	181.5	18199	0.868	16
Red River	29	82.1	113.3	40.05	45.00	51.95	815.3	172761	0.00009			45.2	18199	0.758	16
Red River	29	139.2	135.9	72.79	79.50	90.30	848.0	175697	0.00007			31.3	18199	0.621	16
Red River	29	199.6	166.5	114.57	122.00	134.11	868.2	179886	0.00007	1.0	268.2	26.6	18199	0.616	16
Sabine River	30	7.9	127.4	2.09	2.50	3.54	166.1	18809	0.00018			350.6	4621	0.821	16
Sabine River	30	17.2	127.4	5.59	6.80	8.23	164.3	19080	0.00017	2.2	90.5	244.6	4621	0.744	16
Sabine River	30	39.9	118.9	24.98	27.20	30.76	168.1	19724	0.00015	0.0	0.0	86.7	4621	0.391	16

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Sabine River	30	79.8	110.7	34.47	38.00	42.82	186.7	20864	0.00014	1.6	116.4	67.0	4621	0.402	16
Sabine River	31	17.2	337.0	4.84	5.58	7.09	164.3	19080	0.00018	2.3	152.4	241.8	22773	0.848	16
Sabine River	31	79.8	359.6	24.74	26.75	30.48	186.7	20864	0.00015	2.3	154.8	97.0	22773	0.687	16
Sabine River	31	135.3	399.3	44.01	47.42	53.32	223.7	22513	0.00014	2.3	161.5	55.6	22773	0.501	16
Sabine River	31	165.4	419.1	54.99	59.83	81.32	246.9	23451	0.00010	2.3	164.6	29.0	22773	0.584	16
Sabine River	31	209.7	433.3	71.03	94.00	171.32	282.7	24901				5.4	22773	0.995	16
Sabine River	32	22.2	0.7	54.88	61.00	70.90	14.8	2251	0.00026	0.5	5.9	6.9	2270	0.658	15
Sabine River	32	54.2	1.3	148.37	159.00	172.44	21.4	2801	0.00025	0.8	21.6	4.9	2270	0.354	15
Sabine River	32	68.9	0.5	186.17	197.00	229.90	24.9	3096	0.00023	0.4	6.7	2.9	2270	0.177	15
Sabine River	32	85.9	1.2	241.12	259.00	281.08	29.4	3479	0.00021	1.2	17.4	3.0	2270	0.175	15
Sabine River	33	17.1	1.0	58.60	67.00	75.31	29.4	3479	0.00021	1.2	17.4	6.2	9080	0.636	15
Sabine River	33	37.2	1.4	108.55	119.00	132.87	32.4	4007	0.00018	0.5	12.2	4.6	9080	0.585	15
Sabine River	33	72.6	2.5	185.67	200.00	221.53	38.3	5142	0.00018	0.3	13.4	3.5	9080	0.373	15
Sabine River	33	121.0	6.9	294.10	303.00	316.17	49.1	7234	0.00016	1.0	16.5	5.1	9080	0.158	15
Sabine River	34	21.1	6.2	40.35	47.00	55.49	50.2	7353	0.00012	1.4	32.0	7.4	9080	0.772	15
Sabine River	34	55.4	7.1	93.56	104.00	116.77	52.0	7563	0.00013	0.9	21.3	4.8	9080	0.644	15
Sabine River	34	93.5	6.2	175.23	190.00	215.95	60.8	8575	0.00011	1.5	46.0	2.9	9080	0.473	15
Sabine River	34	113.5	9.5	202.17	225.00	247.00	67.0	9298	0.00017	1.0	36.6	3.0	9080	0.574	15
Mississippi River	35	35.4	0.0	10.55	12.23	16.74	10987	2912484	0.00001	12.6	807.7	0.0	662840	1.000	20
Mississippi River	35	100.6	0.0	36.83	41.75	50.11	10987	2912484	0.00001	17.3	670.6	0.0	662840	0.268	20
Mississippi River	35	147.7	6796.1	55.15	62.75	73.15	10987	2912484	0.00001	17.7	731.5	38.7	662840	0.471	20
Mississippi River	35	204.7	6796.1	81.86	90.75	107.23	10987	2912484	0.00001	24.8	731.5	26.8	662840	0.505	20
Mississippi River	36	54.7	2633.5	12.53	14.25	16.93	5376	1818949	0.00012	5.3	468.2	132.5	90800	1.134	16
Mississippi River	36	96.6	2633.5	21.77	23.75	29.24	5554	1830119	0.00012			73.9	90800	1.024	16
Mississippi River	36	118.0	2775.1	28.78	33.50	44.23	5646	1835859	0.00012	4.9	533.4	35.8	90800	1.118	16
Mississippi River	36	294.5	2973.3	70.56	75.75	89.80	5624	1854833	0.00012	4.1	707.7	35.1	90800	0.635	16
Mississippi River	37	54.7	6824.4	9.51	10.25	11.79	5376	1818949	0.00012	9.2	534.0	238.4	108960	0.972	16
Mississippi River	37	96.6	6824.4	16.77	17.75	22.60	5554	1830119	0.00012			123.3	108960	0.843	16
Mississippi River	37	118.0	6824.4	19.83	23.00	31.12	5646	1835859	0.00012	8.9	537.4	56.3	108960	1.113	16
Mississippi River	37	294.5	6824.4	50.59	54.25	67.65	5624	1854833	0.00012	7.3	690.1	36.2	108960	0.589	16
Wind/Bighorn River	38	9.2	54.9	2.15	2.75	3.74	40.3	20165	0.00289	1.3	55.5	345.6	3904	1.071	14
Wind/Bighorn River	38	32.7	54.9	8.27	9.38	11.11	40.5	20763	0.00200			179.3	3904	0.932	14
Wind/Bighorn River	38	50.4	56.6	15.04	16.63	18.99	41.2	22460	0.00100	0.8	77.7	131.5	3904	0.911	14
Wind/Bighorn River	38	75.3	56.6	22.84	25.00	27.79	42.3	25089	0.00123			104.6	3904	0.840	14

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Wind/Bighorn River	38	99.9	56.6	30.06	32.63	35.62	43.4	27986	0.00063	1.4	44.2	91.7	3904	0.808	14
Wind/Bighorn River	38	141.9	56.6	41.66	44.50	48.65	43.7	28602	0.00097			81.4	3904	0.773	14
Wind/Bighorn River	38	181.4	68.8	53.08	56.00	60.48	56.7	38315	0.00070			71.0	3904	0.752	14
Wind/Bighorn River	39	9.2	233.0	1.14	1.44	1.98	40.3	20165	0.00289	2.4	60.4	659.6	3700	1.120	14
Wind/Bighorn River	39	32.7	235.0	4.20	4.88	5.92	40.5	20763	0.00200			326.7	3700	1.122	14
Wind/Bighorn River	39	50.4	235.0	7.73	8.67	10.29	41.2	22460	0.00100	2.4	85.3	223.5	3700	0.827	14
Wind/Bighorn River	39	75.3	220.9	12.08	13.50	15.58	42.3	25089	0.00123			152.4	3700	0.795	14
Wind/Bighorn River	39	99.9	215.2	16.55	18.08	20.57	43.4	27986	0.00063			129.6	3700	0.682	14
Wind/Bighorn River	39	141.9	218.0	24.19	26.00	28.18	43.7	28602	0.00097			123.2	3700	0.569	14
Wind/Bighorn River	39	181.4	254.9	31.35	33.17	36.47	56.7	38315	0.00070			109.0	3700	0.518	14
Copper Creek	40	0.2	1.6	0.11	0.14	0.27	4.0	274	0.00116	0.6	14.3	4262	67000	1.042	12
Copper Creek	40	1.0	1.6	0.65	0.88	1.70	4.0	274	0.00144	0.4	18.0	530	67000	0.963	12
Copper Creek	40	1.7	1.6	1.08	1.50	2.41	4.0	274	0.00144	0.5	15.5	393	67000	0.836	12
Copper Creek	40	2.4	1.6	1.55	2.04	3.28	4.0	274	0.00144	0.5	14.6	332	67000	0.886	12
Copper Creek	40	3.3	1.6	2.57	3.36	4.90	4.0	274	0.00129	0.4	17.7	238	67000	0.908	12
Copper Creek	40	4.1	1.6	3.09	4.26	7.40	4.0	274	0.00129	0.6	17.7	143	67000	1.276	12
Clinch River	41	0.7	10.8	0.37	0.47	0.67	44.5	2922	0.00048	1.0	45.7	1965	744000	1.050	12
Clinch River	41	1.6	9.9	1.09	1.25	1.66	44.7	2930	0.00063	0.6	47.2	1051	744000	1.106	12
Clinch River	41	2.5	9.5	1.67	1.80	2.59	44.8	2939	0.00063	0.6	41.1	628	744000	1.060	12
Clinch River	41	3.6	10.5	2.38	2.65	3.64	45.0	2949	0.00048	1.0	50.6	497	744000	1.196	12
Clinch River	41	4.7	10.6	3.49	3.84	5.28	45.1	2960	0.00044	1.1	53.6	318	744000	1.110	12
Clinch River	41	5.9	9.8	5.02	5.70	8.52	45.3	2972	0.00036	1.9	47.5	195	744000	1.076	12
Copper Creek	42	0.7	1.0	1.56	1.74	2.23	4.0	274	0.00463	0.4	9.4	829	64000	0.895	12
Copper Creek	42	2.2	1.0	3.23	3.84	5.28	4.0	274	0.00449	0.4	20.1	264	64000	0.769	12
Copper Creek	42	3.3	1.0	4.87	5.75	7.51	4.0	274	0.00397	0.3	20.1	199	64000	0.593	12
Copper Creek	42	4.8	0.9	6.25	7.37	9.68	4.0	274	0.00336	0.4	18.6	158	64000	0.641	12
Copper Creek	42	6.5	1.1	7.71	9.75	12.37	4.0	274	0.00303	0.4	17.7	130	64000	0.737	12
Copper Creek	42	8.4	1.1	10.22	12.18	15.67	4.0	274	0.00291	0.4	16.8	118	64000	0.524	12
Powell River	43	1.0	4.1	1.23	1.46	2.85	16.9	943	0.00044	1.1	31.4	418	260000	0.861	12
Powell River	43	1.7	4.6	2.51	2.92	4.99	16.9	947	0.00044	0.5	30.5	246	260000	1.163	12
Powell River	43	3.3	4.2	4.13	4.88	7.43	17.1	956	0.00036	0.8	36.0	140	260000	0.804	12
Powell River	43	4.8	3.9	6.05	7.60	11.73	17.3	965	0.00029	1.1	37.2	90	260000	0.933	12
Powell River	43	6.2	4.0	7.87	9.91	13.95	17.4	974	0.00033	0.7	37.2	87	260000	0.709	12
Clinch River	44	0.9	5.8	0.75	0.93	1.45	37.0	2442	0.00063	0.5	38.4	950	377000	0.767	12

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Clinch River	44	1.9	6.1	1.74	2.00	3.21	37.2	2459	0.00036	0.6	43.0	435	377000	0.725	12
Clinch River	44	3.3	5.9	3.16	3.65	6.04	37.7	2488	0.00044	0.5	26.8	222	377000	0.784	12
Clinch River	44	4.5	5.7	4.14	4.85	7.88	38.0	2510	0.00040	0.6	25.6	144	377000	0.681	12
Clinch River	44	5.6	5.1	6.14	7.54	10.97	38.4	2532	0.00033	0.8	33.5	104	377000	0.573	12
Clinch River	44	6.6	5.9	7.74	9.34	13.33	38.7	2553	0.00033	0.5	47.5	87	377000	0.622	12
Copper Creek	45	0.2	7.1	0.05	0.07	0.14	4.0	274	0.00096	1.0	14.0	8084	450000	1.129	12
Copper Creek	45	1.0	7.8	0.28	0.35	0.66	4.0	274	0.00137	0.7	19.2	1476	450000	1.211	12
Copper Creek	45	1.7	7.9	0.50	0.65	1.09	4.0	274	0.00130	0.8	17.4	876	450000	1.163	12
Copper Creek	45	2.4	7.6	0.68	0.90	1.45	4.0	274	0.00137	0.8	17.7	686	450000	0.985	12
Copper Creek	45	3.3	8.1	1.04	1.43	2.13	4.0	274	0.00130	0.8	19.8	494	450000	0.930	12
Copper Creek	45	4.1	8.7	1.30	1.73	2.42	4.0	274	0.00130	0.9	21.3	450	450000	0.897	12
Clinch River	46	0.7	104.8	0.13	0.18	0.22	44.5	2922	0.00090	1.9	62.2	7183	3770000	1.052	12
Clinch River	46	1.6	105.9	0.33	0.38	0.55	44.7	2930	0.00063	1.6	64.0	2583	3770000	1.714	12
Clinch River	46	2.5	104.5	0.58	0.64	0.83	44.8	2939	0.00053	2.0	56.4	2183	3770000	2.068	12
Clinch River	46	3.6	110.2	0.83	0.92	1.19	45.0	2949	0.00044	2.3	59.4	1705	3770000	1.571	12
Clinch River	46	4.7	104.2	1.18	1.29	1.78	45.1	2960	0.00040	2.2	62.2	1137	3770000	1.260	12
Clinch River	46	5.9	101.9	1.50	1.67	2.50	45.3	2972	0.00040	2.8	52.1	730	3770000	1.586	12
Coachella Canal	47	0.3	25.8	0.10	0.11	0.13			0.00012	1.6	24.4	16613	1580000	0.862	12
Coachella Canal	47	0.9	25.4	0.31	0.35	0.39			0.00012	1.6	23.8	7102	1580000	1.181	12
Coachella Canal	47	1.8	25.1	0.65	0.71	0.79			0.00011	1.4	24.4	4007	1580000	1.286	12
Coachella Canal	47	2.7	25.5	0.97	1.05	1.17			0.00009	1.5	25.0	3013	1580000	1.097	12
Coachella Canal	47	5.5	25.6	1.99	2.16	2.46			0.00011	1.7	21.9	1378	1580000	0.845	12
Coachella Canal	48	0.3	26.9	0.08	0.11	0.13			0.00012	1.6	24.4	10871	1210000	1.504	12
Coachella Canal	48	0.9	26.9	0.30	0.33	0.38			0.00012	1.6	23.8	6762	1210000	0.936	12
Coachella Canal	48	1.8	26.9	0.94	1.05	1.20			0.00011	1.4	24.4	2643	1210000	1.147	12
Coachella Canal	48	2.7	26.9	1.42	1.54	1.76			0.00009	1.5	25.0	1892	1210000	0.983	12
Coachella Canal	48	4.0	26.9	1.98	2.14	2.47				1.5	27.1	1276	1210000	0.769	12
Coachella Canal	48	5.5	26.9	6.95	7.45	8.16			0.00011	1.7	21.9	446	1210000	0.916	12
Clinch River	49	0.7	85.8	0.13	0.16	0.22	44.5	2922	0.00101	1.7	61.0	7049	2450000	1.983	12
Clinch River	49	1.6	79.9	0.36	0.43	0.57	44.7	2930	0.00078	1.6	50.3	2979	2450000	2.038	12
Clinch River	49	2.5	89.2	0.66	0.76	0.98	44.8	2939	0.00058	2.0	48.8	1950	2450000	2.188	12
Clinch River	49	3.6	86.9	1.03	1.14	1.46	45.0	2949	0.00044	2.3	55.8	1416	2450000	2.122	12
Clinch River	49	4.7	83.8	1.23	1.39	1.97	45.1	2960	0.00044	2.2	53.3	970	2450000	1.989	12
Clinch River	49	5.9	85.2	1.91	2.11	3.24	45.3	2972	0.00040	2.7	50.6	559	2450000	1.904	12

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Copper Creek	50	0.2	1.8	0.13	0.17		4.0	274	0.00116	0.7	13.4	5451	112000	1.000	12
Copper Creek	50	1.0	1.8	0.70	0.90		4.0	274	0.00144	0.4	15.5	630	112000	1.000	12
Copper Creek	50	1.7	1.8	1.16	1.52		4.0	274	0.00137	0.4	17.4	450	112000	1.000	12
Copper Creek	50	2.4	1.9	1.58	2.07		4.0	274	0.00144	0.5	14.9	371	112000	1.000	12
Copper Creek	50	3.3	1.9	2.61	3.38		4.0	274	0.00130	0.5	18.3	225	112000	1.000	12
Copper Creek	50	4.1	1.9	3.50	4.30		4.0	274	0.00130	0.5	16.5	182	112000	1.000	12
Missouri River	51	65.7	883.5	10.54	12.78	16.62	848.0	821845	0.00020	3.8	185.9	80	54480	0.824	12
Missouri River	51	134.4	976.9	21.69	25.45	30.30	880.8	829646	0.00019	3.0	182.9	55	54480	0.817	12
Missouri River	51	186.7	948.6	28.79	34.03	39.94	905.9	835633	0.00021	3.4	175.9	48	54480	0.763	12
Missouri River	51	227.4	962.8	34.58	40.00	47.79	913.5	837449	0.00020	2.9	178.3	40	54480	0.720	12
Antietam Creek	52	12.6	2.0	19.80	25.00	43.80	5.4	509	0.00150			22.7	773	0.000	24
Antietam Creek	52	19.6	2.9	31.50	37.20	65.00	6.4	604	0.00170			12.9	773	0.000	24
Antietam Creek	52	29.6	3.1	43.60	50.00	82.00	7.7	727	0.00100			9.7	773	0.000	24
Antietam Creek	53	10.1	3.2	10.10	12.80	17.60	7.7	727	0.00100			71.8	619	0.000	24
Antietam Creek	53	17.5	3.2	17.70	21.30	28.00	8.0	757	0.00100			42.3	619	0.000	24
Conococheague Creek	54	6.1	3.0	12.20	17.00	20.00	17.9	1381	0.00060			99.1	825	0.000	24
Conococheague Creek	54	14.0	3.0	35.00	42.00	52.50	18.9	1458	0.00060			27.0	825	0.000	24
Monocacy River	55	2.8	1.1	26.50	35.50	57.00	5.0	388				28.2	619	0.002	21
Monocacy River	55	7.6	1.2	39.00	51.00	81.00	5.7	448				19.8	619	0.002	21
Monocacy River	56	9.3	2.5	21.80	27.20	45.00	8.9	699				34.4	928	0.001	21
Monocacy River	56	22.8	4.0	48.20	57.00	82.00	16.5	1333				14.5	928	0.000	21
Monocacy River	56	31.5	4.7	62.20	72.50	98.00	19.7	1586	0.0006			10.1	928	0.000	21
Monocacy River	57	2.8	2.7	8.60	10.90	25.00	5.0	388				58.5	309	0.002	21
Monocacy River	57	7.6	2.6	14.80	19.00	33.00	5.7	448				40.4	309	0.001	21
Monocacy River	58	9.3	5.5	10.40	13.30	19.00	8.9	699				68.9	2165	0.000	21
Monocacy River	58	22.8	11.5	24.00	28.50	39.00	16.5	1333				31.8	2165	0.000	21
Monocacy River	58	33.1	12.6	32.00	37.20	50.00	19.7	1586	0.0006			23.3	2165	0.000	21
Monocacy River	59	10.3	2.0	22.00	27.00	41.00	19.7	1586	0.0006			29.1	1237	0.000	21
Monocacy River	59	18.3	2.0	45.50	54.00	80.00	20.5	1650	0.0006			15.3	1237	0.000	21
Monocacy River	59	26.8	2.3	64.50	76.00	115.00	22.6	1819	0.0005			8.8	1237	0.000	21
Souris River	60	2.7	0.4	32.00	46.75	73.60	3.7	24773	0.0001	0.9	18.3	13.6	222	0.850	25
Souris River	60	4.0	0.4	48.00	65.25	92.40	3.7	24859	0.0001	0.9	18.3	13.4	222	0.800	25
Souris River	61	1.8	0.3	17.00	23.25	33.40	4.4	27335	0.000174	1.0	18.6	37.0	168	0.780	25
Souris River	61	3.2	0.3	41.50	49.75	64.10	4.4	27442	0.000174	1.0	18.6	29.7	168	0.750	25

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Souris River	63	2.1	0.6	15.50	17.00	19.20	4.7	27973	0.000174	0.7	18.6	55.5	168	0.850	25
Souris River	63	14.6	0.9	51.50	59.75	70.40	4.8	28134	0.000174	0.7	15.8	25.9	168	0.790	25
Souris River	64	1.4	0.4	3.50	5.75	8.40	5.2	28825	0.000174	0.7	15.8	145.0	173	0.900	25
Souris River	64	6.4	0.7	19.00	23.33	28.00	5.3	28891	0.000144	0.4	12.2	72.7	173	0.790	25
Souris River	64	14.2	0.8	40.00	44.50	50.40	5.3	28994	0.000144	0.4	12.2	51.4	173	0.750	25
Souris River	65	1.6	1.3	3.00	4.75	7.20	5.8	30845	0.00083	0.8	20.7	177.3	182	0.890	25
Souris River	65	10.5	1.3	29.50	34.75	42.80	5.8	31040	0.00083	0.8	20.7	42.6	182	0.860	25
New River	66	2.6	0.1	10.59	13.50	19.17	0.2	10	0.00005			60.5	400	1.261	4
New River	66	13.5	0.1	61.06	72.00	102.35	0.7	39	0.00005			16.1	400	0.773	4
Flint River	67	4.2	1.5	4.76	5.33	7.64	17.4	2438	0.00052			225.7	1800	0.551	5
Flint River	67	18.3	3.9	21.41	26.38	35.00	18.1	2547	0.00052			27.8	1800	1.517	5
Flint River	67	35.9	5.2	43.54	49.40	58.30	19.1	2710	0.00064			31.4	1800	1.088	5
Flint River	67	64.5	6.3	70.09	77.08	90.54	21.5	3076	0.00076			18.8	1800	1.138	5
Flint River	68	0.5	2.2	1.23	1.47	2.47	16.7	2347	0.00092			623.5	360	0.681	5
Flint River	68	1.7	2.3	4.43	5.48	10.60	16.8	2355	0.00092			105.0	360	0.791	5
Flint River	68	3.5	2.4	13.48	20.83	48.17	16.9	2367	0.00092			17.5	360	1.033	5
Flint River	68	4.4	2.4	22.74	28.38	57.33	16.9	2373	0.00092			15.0	360	0.911	5
Hoosic, Route 346	69	3.1	6.5	2.51	2.90	4.21	11.8	540	0.001817	0.6	19.5	312.0	3800	0.597	6
Hoosic, Route 11	69	8.4	6.3	5.92	7.20	9.10	15.0	685	0.001817	0.7	29.0	104.9	3800	0.396	6
Hoosic, Route 7	69	13.6	6.7	9.31	10.90	13.65	18.4	841	0.001817	0.4	28.0	51.8	3800	0.763	6
Hoosic, Route22	69	19.5	6.9	13.54	15.57	18.74	21.3	971	0.001817			61.5	3800	0.325	6
Shenandoah, Waynesboro	70	2.3	1.0	3.00	4.00	6.00	4.0	329	0.001705			212.4	2400	1.070	22
Shenandoah, Waynesboro	71	2.3	3.1	1.30	1.80	3.10	4.0	329	0.001705			315.3	4540	0.990	22
Shenandoah, Hopeman	70	10.8	1.6	21.00	26.00	34.00	5.7	384	0.001705			54.0	2400	0.820	22
Shenandoah, Hopeman	71	10.8	3.7	10.00	13.00	18.50	5.7	384	0.001705			68.2	4540	0.880	22
Shenandoah,Crimora	70	22.7	1.8	44.00	52.00	66.00	7.1	477	0.001705			33.2	2400	0.760	22
Shenandoah,Crimora	71	22.7	4.5	22.20	27.30	34.20	5.7	477	0.001705			47.1	4540	0.890	22
Shenandoah,Harriston	70	33.0	1.9	64.00	75.00	91.00	7.2	549	0.001705			25.7	2400	0.670	22
Shenandoah,Harriston	71	33.0	4.8	32.50	38.80	47.50	7.2	549	0.001705			37.0	4540	0.830	22
Shenandoah,Island Ford	70	60.0	7.7	116.00	131.00	158.00	30.0	2975	0.001705			16.0	2400	0.500	22
Shenandoah,Island Ford	71	60.0	19.3	58.00	66.30	80.50	30.0	2975	0.001705			24.3	4540	0.680	22
Shenandoah,Shenandoah	70	81.8	9.3	156.00	173.00	210.00	33.4	3311	0.001136			12.6	2400	0.460	22
Shenandoah,Shenandoah	71	81.8	20.7	79.50	90.30	106.00	33.4	3311	0.001136			21.6	4540	0.590	22
Shenandoah,Shenandoah	72	11.3	24.1	8.80	10.80	14.00	33.4	3311	0.001136			104.4	6810	1.080	22

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Shenandoah,Grove Hill	70	94.5	10.6	174.00	193.00	236.00	33.7	3340	0.001136			11.9	2400	0.520	22
Shenandoah,Grove Hill	73	12.7	9.5	14.00	17.00	24.00	33.7	3340	0.001136			62.7	7000	1.120	22
Shenandoah,Grove Hill	72	24.0	23.2	17.50	21.00	14.00	33.7	3340	0.001136			62.9	6810	0.910	22
Shenandoah,US Hwy 211	73	36.9	10.7	50.00	63.00	87.00	37.2	3564	0.001136			19.2	7000	0.910	22
Shenandoah,US Hwy 211	72	36.9	24.1	37.00	42.50	51.20	37.2	3564	0.001136			40.1	6810	0.740	22
Shenandoah, Bixler Bridge	73	48.1	11.2	84.00	115.00	163.00	38.5	3620	0.001136			8.9	7000	0.950	22
Shenandoah, Bixler Bridge	72	59.4	23.2	53.00	60.00	76.50	38.5	3620	0.001136			20.8	6810	0.770	22
Shenandoah, Bentonville	73	90.1	12.8	162.00	196.00	248.00	43.4	40.81	0.001136			9.0	7000	0.980	22
Shenandoah, Bentonville	72	101.4	24.6	85.00	98.50	123.50	43.4	4081	0.001136			14.5	6810	0.750	22
Shenandoah, Bentonville	74	42.0	28.9	30.00	33.50	39.00	45.2	4081	0.001136			62.3	13620	0.960	22
Shenandoah, Front Royal	73	114.9	13.0	208.00	246.00	305.00	45.2	4251	0.001136			7.3	7000	1.020	22
Shenandoah, Front Royal	74	66.8	29.4	51.00	56.00	64.00	45.2	4251	0.001136			45.0	13620	0.760	22
Shenandoah, Morgan Ford	73	131.3	18.1	256.00	292.00	378.00	71.2	7175	0.001136			5.8	7000	0.720	22
Shenandoah, Morgan Ford	75	7.9	16.7	13.00	23.00	111.00	71.2	7175	0.001136			26.5	3000	1.010	22
Shenandoah, Morgan Ford	74	83.2	43.0	71.50	76.50	86.00	71.2	7175	0.001136			37.3	13620	0.770	22
Shenandoah, HWWY 17&50	75	25.4	15.4	36.00	58.00	143.00	71.8	7237	0.000568			22.0	3000	0.890	22
Shenandoah, HWWY 17&50	74	100.7	43.0	84.00	90.50	102.00	71.8	7237	0.000568			29.1	13620	0.770	22
Shenandoah, HWWY 17&50	76	17.5	48.7	11.20	12.70	16.50	71.8	7237	0.000568			106.7	9080	0.940	22
Shenandoah, HWY 7	75	48.8	15.9	64.00	89.00	176.00	75.5	7612	0.000568			19.8	3000	0.830	22
Shenandoah, HWY 7	76	40.9	48.7	25.30	27.70	34.30	75.5	7612	0.000568			57.7	9080	0.920	22
Shenandoah, HWY 9	75	70.8	16.9	99.00	127.00	219.00	77.3	7799	0.000568			15.9	3000	0.780	22
Shenandoah, HWY 9	76	62.9	48.1	40.70	47.50	56.30	77.3	7799	0.000568			33.7	9080	0.880	22
Shenandoah, Harpers Ferry	75	83.0	16.1	131.00	165.00	262.00	78.6	7924	0.002178			14.5	3000	0.630	22
Shenandoah, Harpers Ferry	76	75.2	49.0	54.70	62.00	74.50	78.6	7924	0.002178			26.3	9080	0.840	22
Potomac, Cumberland	77	3.5	6.9	4.00	4.80	6.30	36.2	2265	0.000663			335.4	4960	0.930	23
Potomac, North Branch	77	17.5	7.2	25.00	31.50	43.00	36.2	2265	0.000663			38.3	4960	0.760	23
Potomac, Oldtown	77	31.5	7.9	41.00	50.50	67.00	36.2	2265	0.000663			27.6	4960	0.810	23
Potomac, Paw Paw	77	48.8	13.0	63.00	74.50	93.50	93.4	8049	0.000663			22.6	4960	0.790	23
Potomac, Doe Gully	77	80.6	12.2	104.00	119.00	148.00	103.1	9268	0.000511			15.6	4960	0.740	23
Potomac, Hancock	77	109.8	14.9	138.00	158.00	195.00	117.3	10544	0.00036			8.9	4960	1.150	23
Potomac, Hancock	78	6.6	16.1	7.50	9.00	12.00	117.3	10544	0.00036			166.2	3040	1.150	23
Potomac, Fort Frederick	78	25.1	17.3	30.00	33.50	44.50	126.6	11408	0.00036			50.0	3040	0.910	23
Potomac, Dam # 5	78	40.7	15.6	104.00	135.00	218.00	135.00	12192	0.00036			6.0	3040	1.030	23
Potomac, Williamsport	78	51.3	17.6	130.00	166.00	259.00	141.0	12756	0.00036			5.1	3040	1.020	23

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Potomac, Williamsport	79	9.7	19.5	21.50	24.00	29.50	141.0	12756	0.00036			95.5	6060	1.080	23
Potomac, Dam #4	79	34.6	19.5	120.00	167.00	261.00	156.3	14184	0.00036			5.0	6060	0.840	23
Potomac, Shepherdstown	79	53.4	20.1	150.00	203.00	304.00	169.0	15368	0.00036			4.5	6060	0.820	23
Potomac, Shepherdstown	80	7.2	31.7	12.50	15.40	21.00	169.0	15368	0.00036			99.3	10000	0.990	23
Potomac, Dam #3	80	24.1	32.3	59.00	66.50	89.00	177.0	16095	0.000455			23.6	10000	1.000	23
Potomac, Brunswick	80	35.9	40.8	70.00	79.0	103.00	253.0	23965	0.000568			21.5	10000	1.140	23
Potomac, Point of Rocks	80	46.0	40.2	79.00	90.50	114.50	265.2	24985	0.000663			19.7	10000	1.100	23
Potomac, Point of Rocks	81	8.7	47.6	8.00	9.50	11.00	265.2	24985	0.000663			172.5	13520	1.020	23
Potomac, Whites Ferry	81	28.6	51.0	29.00	31.50	35.50	286.7	27183	0.000663			110.9	13520	1.020	23
Potomac, Seneca	81	49.9	50.1	63.00	67.00	76.00	299.3	28478	0.000663			61.1	13520	0.850	23
Potomac, Little Falls Dam	81	76.9	51.0	130.0	144.00	184.00	313.5	29927	0.000663			11.9	13520	0.600	23
N Platte, Mystery Bridge	82	8.7	26.6	3.00	3.17	9.00	40.6	32361	0.00088			572.6	1802	0.980	2
N Platte, Cole Cr Rd Br	82	32.3	26.6	11.10	12.60	20.00	42.1	33655	0.00088			120.4	1802	0.920	2
N Platte, Old Hwy Bridge	82	58.6	26.6	21.60	24.00	35.00	43.5	35048	0.00088			69.7	1802	0.890	2
Apple R, near Apple River	83	1.9	2.6	1.00	1.30	3.40	0.7	93	0.00151	0.5	9.1	485.2	2400	0.760	13
Apple R, Elizabeth	83	35.9	12.4	18.50	20.80	35.60	3.8	487	0.00151	0.5	9.1	87.9	2400	0.660	13
Apple R, Hanover	83	52.0	16.7	30.40	32.90	50.20	5.0	639	0.00151	0.5	9.1	60.5	2400	0.840	13
Apple R, near Apple River	84	1.9	0.6	2.70	3.70	7.80	0.7	93	0.00151	0.5	9.1	193.8	450	1.050	13
Apple R, Elizabeth	84	35.9	2.6	43.80	49.40	72.80	3.8	487	0.00151	0.5	9.1	36.5	450	0.640	13
Apple R, near Whitton	84	58.4	3.1	97.20	105.80	129.50	5.0	645	0.00151	0.5	9.1	29.7	450	0.570	13
Cedar C, near Galesburg	85	4.7	1.5	5.30	5.90	7.80	0.6	86	0.00078	0.6	7.6	418.1	400	1.050	13
Cedar C, near Monmouth	85	30.9	3.1	28.10	30.00	45.10	1.4	199	0.00078	0.6	7.6	143.2	400	0.900	13
Cedar C, near Bald Bluff	85	61.8	3.7	48.10	50.30	72.80	3.0	427	0.00078	0.6	7.6	75.9	400	0.740	13
Cedar C, near Galesburg	86	4.7	0.5	8.80	10.20	13.60	0.6	86	0.00078	0.6	7.6	207.7	400	0.580	13
Cedar C, near Monmouth	86	30.9	1.0	46.80	50.60	67.80	1.4	199	0.00078	0.6	7.6	77.4	400	0.550	13
Cedar C, Little York	86	52.9	1.3	72.60	78.30	107.40	2.4	337	0.00078	0.6	7.6	41.8	400	0.420	13
Cedar C, near Galesburg	87	4.7	0.3	8.80	10.10	14.00	0.6	86	0.00078	0.6	7.6	211.8	150	0.500	13
Cedar C, near Monmouth	87	30.9	0.6	57.60	62.10	93.20	1.4	199	0.00078	0.6	7.6	50.3	150	0.610	13
Elkhorn C, near Haldane	88	1.6	0.4	1.80	2.20	3.50	0.4	54	0.00070	0.3	5.5	596.6	800	0.650	13
Elkhorn C, near Penrose	88	46.8	2.8	45.00	48.40	61.70	2.8	378	0.00070	0.3	5.5	79.8	800	0.530	13
Elkhorn C, near Emerson	88	64.9	3.6	60.60	63.90	80.30	4.1	557	0.00070	0.3	5.5	66.4	800	0.460	13
Elkhorn C, near Haldane	89	1.6	0.2	2.80	3.30	5.10	0.4	54	0.00070	0.3	5.5	406.7	200	0.860	13
Elkhorn C, near Milledgeville	89	20.9	0.7	28.00	31.00	37.90	1.3	179	0.00070	0.3	5.5	121.1	200	0.420	13
Elkhorn C, near Penrose	89	46.8	1.5	63.10	67.50	88.20	2.8	378	0.00070	0.3	5.5	58.2	200	0.470	13

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Elkhorn C, near Emerson	89	64.9	1.9	81.40	87.00	113.20	4.1	557	0.00070	0.3	5.5	43.3	200	0.430	13
Embarras R,Greenup	90	2.4	27.5	0.80	0.92	1.90	23.4	2646	0.00029	1.2	35.1	977.5	1600	0.850	13
Embarras R, Ste Marie	90	67.3	27.8	30.70	33.60	44.60	31.9	3925	0.00029	1.2	35.1	104.5	1600	0.830	13
Embarras R, Greenup	91	2.4	8.9	1.20	1.60	3.80	23.4	2646	0.00029	1.2	35.1	478.7	1000	1.120	13
Embarras R, Rose Hill	91	26.4	8.7	19.40	21.60	34.00	30.4	3435	0.00029	1.2	35.1	97.0	1000	0.920	13
Embarras R, Newton	91	48.8	9.4	37.40	40.90	53.00	31.9	3604	0.00029	1.2	35.1	64.6	1000	0.830	13
Embarras R, Greenup	92	2.4	1.2	3.80	5.00	10.30	23.4	2646	0.00029	1.2	35.1	158.4	500	0.960	13
Embarras R, Rose Hill	92	26.4	1.5	51.60	58.20	77.40	30.4	3435	0.00029	1.2	35.1	50.8	500	0.640	13
Embarras R, Falmouth	92	37.3	1.9	73.20	80.0	106.10	31.3	3534	0.00029	1.2	35.1	31.7	500	0.700	13
Embarras R, Newton	92	48.8	1.9	96.30	104.40	127.10	34.7	3604	0.00029	1.2	35.1	29.3	500	0.570	13
Kaskaskia R, Bondville	93	3.2	0.1	3.30	3.90	7.60	0.3	32	0.00028	0.1	2.7	394.7	300	0.460	13
Kaskaskia R, Grange	93	26.7	1.0	24.30	26.20	52.00	1.5	174	0.00028	0.1	2.7	72.9	300	0.420	13
Kaskaskia R, Ficklin	93	42.6	1.7	38.80	41.30	70.70	3.0	324	0.00028	0.1	2.7	55.1	300	0.440	13
Kaskaskia R, Bondville	94	3.2	0.1	3.80	4.40	6.80	0.3	32	0.00028	0.1	2.7	392.7	300	0.900	13
Kaskaskia R, Pesotum	94	32.5	3.1	28.40	31.10	51.10	2.6	282	0.00028	0.1	2.7	67.4	300	1.140	13
Kaskaskia R, Chesterville	94	60.5	8.6	53.20	61.60	97.10	8.7	932	0.00028	0.1	2.7	26.4	300	0.710	13
Kaskaskia R, Bondville	95	1.6	0.0	6.10	10.40	25.70	0.3	32	0.00028	0.1	2.7	89.0	250	1.710	13
Kaskaskia R, Hayes	95	36.7	1.0	53.70	60.60	100.70	2.9	311	0.00028	0.1	2.7	23.7	250	0.490	13
Mackinaw R, Mackinaw	96	1.9	16.2	0.90	1.10	2.30	15.2	2213	0.00061	0.5	33.5	794.0	2400	1.120	13
Mackinaw R, Green Valley	96	49.9	32.6	22.20	23.70	27.80	19.4	2827	0.00061	0.5	33.5	180.9	2400	0.690	13
Mackinaw R, Mackinaw	97	1.9	11.1	1.00	1.40	2.90	15.2	2213	0.00061	0.5	33.5	538.4	1800	0.810	13
Mackinaw R, Green Valley	97	49.9	10.7	28.80	32.70	40.90	19.4	2827	0.00061	0.5	33.5	79.2	1800	0.500	13
Mackinaw R, Mackinaw	98	1.9	1.0	5.40	7.80	25.80	15.2	2213	0.00061	0.5	33.5	62.8	500	0.800	13
Mackinaw R, Tremont	98	10.3	1.3	24.40	29.00	56.20	16.2	2358	0.00061	0.5	33.5	34.2	500	0.590	13
MF Vermilion R, Armstrong	99	4.8	8.7	2.60	3.10	4.70	7.4	722	0.00077	0.9	15.2	390.6	1000	0.970	13
MF Vermilion R, Oakwood	99	50.7	5.6	29.60	33.90	49.30	11.5	1118	0.00077	0.9	15.2	65.2	1000	0.650	13
MF Vermilion R, Armstrong	100	4.8	1.3	6.90	8.70	18.70	7.4	722	0.00077	0.9	15.2	120.7	300	0.790	13
MF Vermilion R, Oakwood	100	50.7	1.4	66.80	103.00	167.00	11.5	1118	0.00077	0.9	15.2	15.3	300	0.500	13
Sangamon R, Mahomet	101	3.1	3.5	2.20	2.60	4.60	7.5	937	0.00027	0.6	30.5	380.0	500	0.800	13
Sangamon R, Monticello	101	40.9	7.8	36.40	39.30	45.90	11.4	1424	0.00027	0.6	30.5	84.5	500	0.590	13
Sangamon R, Cisco	101	60.5	8.3	55.10	58.20	63.30	13.0	1618	0.00027	0.6	30.5	88.9	500	0.280	13
Sangamon R, near Mahomet	102	5.1	3.6	3.90	5.10	8.20	7.6	942	0.00027	0.6	30.5	243.3	1600	0.570	13
Sangamon R, Monticello	102	40.9	5.7	44.40	49.00	67.40	11.4	1424	0.00027	0.6	30.5	60.3	1600	0.460	13
Sangamon R, Cisco	102	60.5	6.7	68.80	74.40	88.10	13.0	1618	0.00027	0.6	30.5	53.1	1600	0.400	13

Table A-1. Compiled data for studies publishing the complete dy curve—Continued

River	Inj No	Km	Q	Tl	Tp	Tt	Qave	Da Ar	Slope	Depth	Width	Cup	Inj Mass	R ratio	Ref
Sangamon R, near Mahonet	103	5.1	0.5	21.60	27.10	44.70	7.6	942	0.00027	0.6	30.5	40.1	400	1.810	13
Sangamon R, Allerton Prk	103	47.6	0.9	139.00	150.00	193.90	11.9	1483	0.00027	0.6	30.5	17.9	400	0.410	13
Shoal Cr, Old Ripley	104	2.7	5.6	1.50	1.60	2.70	9.4	1201	0.00019	0.8	18.3	764.0	1600	0.840	13
Shoal Cr, Jamestown	104	29.8	5.9	24.20	26.60	32.20	14.2	1812	0.00019	0.8	18.3	122.1	1600	0.830	13
Shoal Cr, near Breese	104	57.0	3.8	59.10	63.80	80.10	14.9	1903	0.00019	0.8	18.3	46.7	1600	0.660	13
Shoal Cr, Old Ripley	105	2.7	1.2	2.20	2.70	4.40	9.4	1201	0.00019	0.8	18.3	427.1	600	0.790	13
Shoal Cr, Jamestown	105	29.8	2.5	40.80	44.30	51.60	14.2	1812	0.00019	0.8	18.3	85.2	600	0.600	13
Shoal Cr, Frogtown	105	48.8	2.1	73.10	79.00	101.70	14.7	1882	0.00019	0.8	18.3	40.3	600	0.640	13
Shoal Cr, Old Ripley	106	2.7	0.7	3.40	4.60	8.50	9.4	1201	0.00019	0.8	18.3	202.4	400	0.840	13
Shoal Cr, Jamestown	106	29.8	1.3	67.60	73.90	103.00	14.2	1812	0.00019	0.8	18.3	35.7	400	0.710	13
Vermilion R, near Leonore	107	3.5	43.6	0.90	1.10	2.10	23.2	3239	0.00108	1.1	36.6	978.4	1200	0.910	13
Vermilion R, Lowell	107	14.3	46.7	4.50	5.10	7.60	23.7	3309	0.00108	1.1	36.6	243.9	1200	0.870	13
Vermilion R, near Leonore	108	3.5	14.0	1.70	2.00	5.20	23.2	3239	0.00108	1.1	36.6	394.5	600	1.060	13
Vermilion R, Ogesby	108	26.9	15.5	17.70	19.70	28.70	24.7	3441	0.00108	1.1	36.6	84.0	600	0.970	13
Vermilion R, near Leonore	109	3.5	2.9	4.80	5.60	14.40	23.2	3239	0.00108	1.1	36.6	124.9	550	0.760	13
Vermilion R, Ogesby	109	26.9	2.4	51.10	62.30	100.70	24.7	3441	0.00108	1.1	36.6	20.6	550	0.490	13

Table A–2. Data from studies yielding traveltime only

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Buffalo R, Checktowaga–STP W Seneca	0.5	1.0	0.16	0.00005	5.7	368	10
Buffalo R, Checktowaga STP–W Seneca	0.5	0.3	0.10	0.00005	10.8	690	10
Buffalo R, Checktowaga STP–W Seneca	0.5	2.7	0.30	0.00005	10.8	690	10
Buffalo R, W Seneca –S. Ogden	1.0	1.0	0.06	0.00005	10.8	690	10
Buffalo R, W Seneca –S. Ogden	1.0	0.3	0.04	0.00005	10.8	690	10
Buffalo R, W Seneca –S. Ogden	1.0	2.7	0.11	0.00005	10.8	690	10
Cattaraugus C, Gowanda Vill–Gowanda STP	3.7	3.7	0.37	0.00199	21.0	1129	10
Cattaraugus C, Gowanda Vill–Gowanda STP	3.7	3.3	0.30	0.00199	21.0	1129	10
Cattaraugus C, Gowanda Vill–Gowanda STP	3.7	13.4	0.69	0.00199	21.0	1129	10
Cattaraugus C, Gowanda STP – Versailles	9.2	3.3	0.33	0.00329	21.0	1547	10
Cattaraugus C, Gowanda STP – Versailles	9.2	2.4	0.19	0.00329	21.0	1547	10
Cattaraugus C, Gowanda STP – Versailles	9.2	13.3	0.71	0.00329	21.0	1547	10
Cayuga C, Lancaster STP – Broadway	0.3	0.2	0.06	0.00066	3.8	250	10
Cayuga C, Lancaster STP – Broadway	0.3	0.1	0.05	0.00066	3.8	250	10
Cayuga C, Lancaster STP – Broadway	0.3	0.8	0.12	0.00066	3.8	250	10
Cayuga C, Broadway –Transit Road	3.0	0.2	0.06	0.00066	3.8	250	10
Cayuga C, Broadway –Transit Road	3.0	0.1	0.04	0.00066	3.8	250	10
Cayuga C, Broadway –Transit Road	3.0	0.8	0.17	0.00066	3.8	250	10
Cayuga C, Transit Road – Borden road	1.5	0.2	0.05	0.00066	3.8	250	10
Cayuga C, Transit Road – Borden road	1.5	0.1	0.03	0.00066	3.8	250	10
Cayuga C, Transit Road – Borden road	1.5	0.8	0.14	0.00066	3.8	250	10
Ellicott C (Alden), Amhrst STP – Snd Rdge	2.7	0.5	0.10	0.00096	1.8	102	10
Ellicott C (Alden), Amhrst STP – Snd Rdge	2.7	0.0	0.02	0.00096	1.8	102	10
Ellicott C (Alden), Amhrst STP – Snd Rdge	2.7	0.2	0.05	0.00096	1.8	102	10
Ellicott C (Amherst) Amhrst STP –Mple Rd	1.0	0.7	0.17	0.00050	3.8	218	10
Ellicott C (Amherst) Amhrst STP –Mple Rd	1.0	0.2	0.10	0.00050	3.8	218	10
Ellicott C (Amherst) Amhrst STP –Mple Rd	1.0	0.7	0.18	0.00050	3.8	218	10
Ellicott C (Amherst) Amhrst STP –Mple Rd	1.0	0.3	0.11	0.00050	3.8	218	10
Ellicott C (Amherst) Amhrst STP –Millersport	3.3	0.7	0.12	0.00126	4.2	240	10
Ellicott C (Amherst) Mple Rd –Millersport	3.3	0.2	0.08	0.00126	4.2	240	10
Ellicott C (Amherst) Mple Rd –Millersport	3.3	0.7	0.14	0.00126	4.2	240	10
Ellicott C (Amherst) Mple Rd –Millersport	3.3	0.3	0.06	0.00126	4.2	240	10
Ellicott C (Amherst) Millersport–Sweet Hm	4.7	0.7	0.08	0.00126	4.8	275	10
Ellicott C (Amherst) Millersport–Sweet Hm	4.7	0.2	0.04	0.00126	4.8	275	10
Ellicott C (Amherst) Millersport–Sweet Hm	4.7	0.3	0.05	0.00126	4.8	275	10
Ellicott C (Pen) Pen STP–Waldem Ave	1.6	0.0	0.01	0.00165	1.9	107	10
Ellicott C (Pen) Pen STP–Waldem Ave	1.6	0.0	0.01	0.00165	1.9	107	10
Ellicott C (Pen) Pen STP–Waldem Ave	1.6	0.3	0.07	0.00165	1.9	107	10
Ellicott C (Pen) Waldem Ave–Erie C Home	0.5	0.0	00.01	0.00138	1.9	109	10
Ellicott C (Pen) Waldem Ave–Erie C Home	0.5	0.0	0.01	0.00138	1.9	109	10
Ellicott C (Pen) Waldem Ave–Erie C Home	0.5	0.3	0.04	0.00138	1.9	109	10
Ellicott C (Pen) Erie C Home–Zoeller Rd	1.8	0.0	0.01	0.00138	2.0	115	10
Ellicott C (Pen) Erie C Home–Zoeller Rd	1.8	0.0	0.01	0.00138	2.0	115	10
Ellicott C (Pen) Erie C Home–Zoeller Rd	1.8	0.3	0.05	0.00138	2.0	115	10
Murder C , Alrpm STP–Simpson's grove	7.0	0.3	0.11	0.00221	2.8	157	10
Murder C , Alrpm STP–Simpson's grove	7.0	0.1	0.07	0.00221	2.8	157	10
Murder C , Alrpm STP–Simpson's grove	7.0	0.6	0.19	0.00221	2.8	157	10
Tonawanda C, Prospect St–Stroh Rd	2.4	0.8	0.14	0.00292	3.9	238	10

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Tonawanda C, Prospect St–Stroh Rd	2.4	1.5	0.22	0.00292	3.9	238	10
Tonawanda C, Prospect St–Stroh Rd	2.4	0.2	0.05	0.00292	3.9	238	10
Tonawanda Cr, Stroh Rd–Railroad Ave	4.0	0.8	0.14	0.00148	3.9	238	10
Tonawanda Cr, Stroh Rd–Railroad Ave	4.0	1.5	0.23	0.00148	3.9	238	10
Tonawanda Cr, Stroh Rd–Railroad Ave	4.0	0.2	0.05	0.00148	3.9	238	10
Tonawanda Cr, S. Lyon–Main st	1.6	0.3	0.11	0.00050	6.0	443	10
Tonawanda Cr, S. Lyon–Main st	1.6	0.3	0.10	0.00050	6.0	443	10
Tonawanda Cr, S. Lyon–Main st	1.6	2.7	0.25	0.00050	6.0	443	10
WB Cazenovia C, Glenwood–Colden	4.9	3.2	0.27	0.00765	1.5	82	10
WB Cazenovia C, Glenwood–Colden	4.9	2.0	0.26	0.00765	1.5	82	10
WB Cazenovia C, Glenwood–Colden	4.9	0.6	0.11	0.00765	1.5	82	10
WB Cazenovia C, Colden–West Falls	7.5	3.2	0.35	0.00521	2.1	110	10
WB Cazenovia C, Colden–West Falls	7.5	2.0	0.27	0.00521	2.1	110	10
WB Cazenovia C, Colden–West Falls	7.5	0.6	0.12	0.00521	2.1	110	10
WB Cazenovia C, West Falls–Griffin Mills	3.7	3.2	0.20	0.00436	2.4	128	10
WB Cazenovia C, West Falls–Griffin Mills	3.7	2.2	0.14	0.00436	2.4	128	10
WB Cazenovia C, Griffin Mills–Mouth	6.1	3.2	0.34	0.00348	3.1	163	10
WB Cazenovia C, Griffin Mills–Mouth	6.1	2.4	0.31	0.00348	3.1	163	10
WB Cazenovia C, West Falls–Grover Road	7.2	0.5	0.07	0.00365	2.8	147	10
WB Cazenovia C, Grover Rd–Holmwood Rd	1.6	0.5	0.18	0.00298	3.0	157	10
WB Cazenovia C, Holmwood Rd–Mouth	0.7	0.5	0.10	0.00331	3.1	163	10
EB Cazenovia C, Holland–S Wales	9.8	2.8	0.30	0.00542	1.5	78	10
EB Cazenovia C, Holland–N Canada St	2.3	0.3	0.11	0.00765	1.1	58	10
EB Cazenovia C, N Canada St–1 Br,Hwy 16	3.0	0.3	0.08	0.00435	1.2	66	10
EB Cazenovia C, 1 Br Hwy 16–2 Br, Hwy 16	1.8	0.3	0.08	0.00439	1.3	71	10
EB Cazenovia C, 2 Br, Hwy 16–S Wales	2.4	0.3	0.11	0.00368	1.5	78	10
EB Cazenovia C, S Wales–Emery Rd	1.6	0.3	0.10	0.00432	1.6	84	10
EB Cazenovia C, Emery Rd–Sweet Rd	5.6	0.5	0.11	0.00231	2.0	104	10
EB Cazenovia C, Sweet Rd–Ab Lapham Rd	3.1	2.4	0.14	0.00065	2.2	118	10
EB Cazenovia C, E Aurora Dam–Mill Rd	1.5	0.5	0.05	0.00414	2.6	136	10
EB Cazenovia C, Mill Rd–Mouth	2.7	0.5	0.13	0.00414	2.9	151	10
EB Cazenovia C, Holland–1 Br, Hwy 16	5.4	2.0	0.24	0.00552	1.2	66	10
EB Cazenovia C, 1 Br, Hwy 16–South Wales	4.2	2.2	0.24	0.00406	1.5	78	10
EB Cazenovia C, South Wales–Sweet Rd	7.2	2.2	0.30	0.00186	2.0	104	10
EB Cazenovia C, Ab Lapham Rd–Center St.	1.9	2.4	0.04	0.00481	2.4	127	10
EB Cazenovia C, Center St.–Dam	0.3	2.4	0.06	0.00414	2.4	128	10
EB Cazenovia C, Dam–Mouth	4.2	2.4	0.25	0.00414	2.9	151	10
Cazenovia C, Confluence–Northrup Rd	11.5	3.2	0.40	0.00269	6.3	332	10
Cazenovia C, Confluence–Northrup Rd	11.5	0.3	0.07	0.00269	6.3	332	10
Cazenovia C, Northrup Rd–Leydecker Rd	4.4	3.2	0.49	0.00364	6.4	339	10
Cazenovia C, Northrup Rd–Leydecker Rd	4.4	0.3	0.08	0.00364	6.4	339	10
Cazenovia C, Northrup Rd–Leydecker Rd	4.4	2.4	0.43	0.00364	6.4	339	10
Cazenovia C, Leydecker Rd–Ebenezer	6.3	3.2	0.48	0.00159	6.6	350	10
Cazenovia C, Leydecker Rd–Ebenezer	6.3	0.3	0.11	0.00159	6.6	350	10
Cazenovia C, Leydecker Rd–Ebenezer	6.3	2.4	0.41	0.00159	6.6	350	10
Cazenovia C, Ebenezer–Cezenovia park	4.5	3.2	0.22	0.00083	6.7	357	10
Cazenovia C, Leydecker Rd–Ebenezer	4.5	0.3	0.05	0.00083	6.7	357	10
Cazenovia C, Leydecker Rd–Ebenezer	4.5	2.4	0.24	0.00083	6.7	357	10
Cazenovia C, Confluence–Willardshire Rd	4.8	2.4	0.40	0.00202	6.1	322	10

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Cazenovia C, Willardshire Rd–Northrup Rd	6.7	2.4	0.37	0.00269	6.3	332	10
Fall Ck., McLean–Malloryville	4.7	0.2	0.07	0.00345	2.3	141	9
Fall Ck., McLean–Malloryville	4.7	0.8	0.13	0.00345	2.3	141	9
Fall Ck., McLean–Malloryville	4.7	2.7	0.26	0.00345	1.7	105	9
Fall Ck., Malloryville–N George Rd	4.5	0.2	0.07	0.00163	2.3	141	9
Fall Ck., Malloryville–N George Rd	4.5	0.8	0.14	0.00163	2.3	141	9
Fall Ck., Malloryville–N George Rd	4.5	2.7	0.29	0.00163	2.0	127	9
Fall Ck., N George Rd–Freeville	2.4	0.2	0.05	0.00125	2.3	141	9
Fall Ck., N George Rd–Freeville	2.4	0.8	0.10	0.00125	2.3	141	9
Fall Ck., N George Rd–Freeville	2.4	2.7	0.20	0.00125	2.3	141	9
Fall Ck., Freeville–Virgil crk	3.5	0.2	0.02	0.00053	2.3	145	9
Fall Ck., Freeville–Virgil crk	3.5	0.8	0.05	0.00053	2.3	145	9
Fall Ck., Freeville–Virgil crk	3.5	2.7	0.32	0.00053	2.3	145	9
Fall Ck., Virgil Creek–Etna	3.5	0.2	0.09	0.00184	4.6	288	9
Fall Ck., Virgil Creek–Etna	3.5	0.8	0.14	0.00184	4.6	288	9
Fall Ck., Virgil Creek–Etna	3.5	2.8	0.24	0.00184	4.6	288	9
Fall Ck., Etna–Pinckney Rd	2.5	0.2	0.11	0.00231	4.8	299	9
Fall Ck., Etna–Pinckney Rd	2.5	0.8	0.20	0.00231	4.8	299	9
Fall Ck., Etna–Pinckney Rd	2.5	2.8	0.38	0.00231	4.8	299	9
Fall Ck., Pinckney Rd–Monkey Run Rd	3.6	0.2	0.13	0.00316	5.0	312	9
Fall Ck., Pinckney Rd–Monkey Run Rd	3.6	0.8	0.25	0.00316	5.0	312	9
Fall Ck., Pinckney Rd–Monkey Run Rd	3.6	2.8	0.38	0.00316	5.0	312	9
Fall Ck., Monkey Run Rd–Freese Rd	3.0	0.2	0.12	0.00557	5.1	320	9
Fall Ck., Monkey Run Rd–Freese Rd	3.0	0.8	0.19	0.00557	5.1	320	9
Fall Ck., Monkey Run Rd–Freese Rd	2.9	2.8	0.41	0.00575	5.1	320	9
Fall Ck.,Freese Rd–Ithaca Gage	3.4	0.2	0.08	0.00840	5.2	326	9
Fall Ck.,Freese Rd–Ithaca Gage	3.4	0.8	0.15	0.00840	5.2	326	9
Fall Ck.,Freese Rd–Ithaca Gage	3.4	2.8	0.30	0.00840	5.2	326	9
Virgil Crk, Dryden(Hwy38)–Spring House Rd	1.9	0.1	0.08	0.00318	1.1	71	9
Virgil Crk, Dryden(Hwy38)–Spring House Rd	1.9	0.3	0.08	0.00318	1.1	71	9
Virgil Crk, Dryden(Hwy38)–Spring House Rd	1.9	0.7	0.15	0.00318	1.1	71	9
Virgil Crk, Spring House Rd–S George Rd	2.0	0.1	0.28	0.00150	1.4	87	9
Virgil Crk, Spring House Rd–S George Rd	2.0	0.3	0.05	0.00150	1.4	87	9
Virgil Crk, Spring House Rd–S George Rd	2.0	0.7	0.08	0.00150	1.4	87	9
Virgil Crk, S George Rd–Johnson St	3.1	0.1	0.22	0.00194	1.5	96	9
Virgil Crk, S George Rd–Johnson St	3.1	0.3	0.08	0.00194	1.5	96	9
Virgil Crk, S George Rd–Johnson St	3.1	0.7	0.15	0.00194	1.5	96	9
Virgil Crk, Johnson St–Mouth	1.3	0.1	0.11	0.00194	1.6	102	9
Virgil Crk, Johnson St–Mouth	1.3	0.3	0.03	0.00194	1.6	102	9
Virgil Crk, Johnson St–Mouth	1.3	0.7	0.06	0.00194	1.6	102	9
Hudson, Fort Edward–Thomson	16.7	127.4	0.20	0.00015	146.4	7296	17
Hudson, Fort Edward–Thomson	16.7	76.2	0.16	0.00015	146.4	7296	17
Hudson, Fort Edward–Thomson	16.7	71.6	0.16	0.00015	146.4	7296	17
Hudson, Thomson–Clarks Mills	1.8	134.5	0.24	0.00138	155.9	7770	17
Hudson, Thomson–Clarks Mills	1.8	76.2	0.14	0.00138	155.9	7770	17
Hudson, Thomson–Clarks Mills	1.8	81.6	0.15	0.00138	155.9	7770	17
Hudson, Clarks Mills–Schuylerville	1.8	61.2	0.31	0.00293	176.7	8806	17
Hudson, Clarks Mills–Schuylerville	1.8	94.6	0.28	0.00293	176.7	8806	17
Hudson, Clarks Mills–Schuylerville	1.8	93.4	0.35	0.00293	176.7	8806	17

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Hudson, Schuylerville–Stillwater	20.8	152.9	0.21	0.00018	184.2	9772	17
Hudson, Schuylerville–Stillwater	20.8	109.6	0.18	0.00018	184.2	9772	17
Hudson, Schuylerville–Stillwater	20.8	86.4	0.15	0.00018	184.2	9772	17
Hudson, Stillwater–W Vir Pupl	3.7	112.7	0.26	0.00004	196.3	10360	17
Hudson, Stillwater–W Vir Pupl	3.7	134.5	0.28	0.00004	196.3	10360	17
Hudson, Stillwater–W Vir Pupl	3.7	82.1	0.19	0.00004	196.3	10360	17
Hudson, W Vir Pupl–Mechanicville	0.8	112.7	0.38	0.00871	213.2	11251	17
Hudson, W Vir Pupl–Mechanicville	0.8	134.5	0.45	0.00871	213.2	11251	17
Hudson, W Vir Pupl–Mechanicville	0.8	82.1	0.22	0.00871	213.2	11251	17
Hudson, Mechanicville–Loc No 2	3.7	109.0	0.30	0.00058	213.2	11251	17
Hudson, Mechanicville–Loc No 2	3.7	133.1	0.40	0.00058	213.2	11251	17
Hudson, Mechanicville–Loc No 2	3.7	82.1	0.29	0.00058	213.2	11251	17
Hudson, Loc No 2–Lock No 1	5.8	109.9	0.14	0.00073	229.4	11942	17
Hudson, Loc No 2–Lock No 1	5.8	150.1	0.21	0.00073	229.4	11942	17
Hudson, Loc No 1–Waterford	4.3	109.9	0.16	0.00073	229.4	11942	17
Hudson, Loc No 1–Waterford	4.3	150.1	0.25	0.00073	229.4	11942	17
Hudson, Waterford–112 st. Troy	1.9	107.6	0.11	0.00073	229.4	11942	17
Hudson, Waterford–112 st. Troy	1.9	150.1	0.21	0.00073	229.4	11942	17
Hudson, Waterford–112 st. Troy	1.9	150.1	0.21	0.00073	229.4	11942	17
Hudson, Waterford–112 st. Troy	1.9	248.6	0.46	0.00073	229.4	11942	17
Hudson, 112 st. Troy–Troy Lock	2.3	137.9	0.11	0.00073	388.5	20953	17
Hudson, 112 st. Troy–Troy Lock	2.3	231.1	0.18	0.00073	388.5	20953	17
Hudson, 112 st. Troy–Troy Lock	2.3	175.6	0.15	0.00073	388.5	20953	17
Hudson, 112 st. Troy–Troy Lock	2.3	548.2	0.36	0.00073	388.5	20953	17
Mohawk, Gate 6 Rome–Oriskany	8.9	7.4	0.15	0.00099	10.6	389	17
Mohawk, Gate 6 Rome–Oriskany	8.9	6.3	0.15	0.00099	10.6	389	17
Mohawk, Gate 6 Rome–Oriskany	8.9	34.0	0.19	0.00099	10.6	389	17
Mohawk, Oriskany–Whitesboro	7.7	10.5	0.30	0.00099	18.7	751	17
Mohawk, Oriskany–Whitesboro	7.7	11.2	0.34	0.00099	18.7	751	17
Mohawk, Oriskany–Whitesboro	7.7	39.1	0.61	0.00099	18.7	751	17
Mohawk, Whitesbor–N S Arterial, Utica	7.6	12.4	0.18	0.00099	34.0	1432	17
Mohawk, Whitesbor–N S Arterial, Utica	7.6	13.9	0.22	0.00099	34.0	1432	17
Mohawk, Leland Ave Utica–Lealand Ave Dam	3.5	19.7	0.15	0.00024	36.4	1541	17
Mohawk, Leland Ave Utica–Lealand Ave Dam	3.5	12.7	0.10	0.00024	36.4	1541	17
Mohawk, Lealand Ave Dam–Dyke Rd Schuyler	8.4	20.0	0.35	0.00024	42.9	1833	17
Mohawk, Lealand Ave Dam–Dyke Rd Schuyler	8.4	17.0	0.27	0.00024	42.9	1833	17
Mohawk, Lealand Ave Dam–Dyke Rd Schuyler	8.4	17.0	0.27	0.00024	42.9	1833	17
Mohawk, Dyke Rd Schuyler–Frankford	10.6	21.5	0.33	0.00024	53.0	2284	17
Mohawk, Dyke Rd Schuyler–Frankford	10.6	15.9	0.21	0.00024	53.0	2284	17
Mohawk, Frankford–Harkimer	7.7	24.9	0.15	0.00024	61.9	2680	17
Mohawk, Frankford–Harkimer	7.7	21.0	0.07	0.00024	61.9	2680	17
Mohawk, Harkimer–Lock 18	7.4	38.5	0.34	0.00082	71.8	3124	17
Mohawk, Harkimer–Lock 18	7.4	57.2	0.63	0.00082	71.8	3124	17
Mohawk, Harkimer–Lock 18	7.4	27.8	0.27	0.00082	71.8	3124	17
Mohawk, Lock 18–Hansen Isl, Lit Falls	5.1	32.3	0.10	0.00040	79.7	3476	17
Mohawk, Lock 18–Hansen Isl, Lit Falls	5.1	69.4	0.20	0.00040	79.7	3476	17
Mohawk, Hansen Isl–Lit Falls–Lock 17	1.8	37.1	0.23	0.00040	80.5	3531	17
Mohawk, Hansen Isl–Lit Falls–Lock 17	1.8	88.9	0.45	0.00040	80.5	3531	17
Mohawk, Lock 17–Fivemile Dam	6.1	39.4	0.09	0.00040	83.5	3731	17

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Mohawk, Lock 17–Fivemile Dam	6.1	98.0	0.23	0.00040	83.5	3731	17
Mohawk, Fivemile Dam–Lock 16	7.1	52.1	0.37	0.00040	87.1	3975	17
Mohawk, Fivemile Dam–Lock 16	7.1	56.6	0.41	0.00040	87.1	3975	17
Mohawk, Fivemile Dam–Lock 16	7.1	132.0	0.69	0.00040	87.1	3975	17
Mohawk, Lock 16–Lock 15	10.6	34.0	0.09	0.00040	92.9	4373	17
Mohawk, Lock 16–Lock 15	10.6	154.3	0.25	0.00040	92.9	4373	17
Mohawk, Lock 15–Lock 14	5.6	34.0	0.08	0.00040	96.3	4599	17
Mohawk, Lock 15–Lock 14	5.6	154.3	0.20	0.00040	96.3	4599	17
Mohawk, Lock 14–Lock 13	12.6	34.0	0.05	0.00040	104.4	5148	17
Mohawk, Lock 14–Lock 13	12.6	154.3	0.24	0.00040	104.4	5148	17
Mohawk, Lock 13–Fonda	7.4	22.7	0.04	0.00040	109.6	5501	17
Mohawk, Lock 13–Fonda	7.4	226.5	0.26	0.00040	109.6	5501	17
Mohawk, Fonda–Lock 12	8.2	17.9	0.05	0.00040	114.1	5807	17
Mohawk, Fonda–Lock 12	8.2	198.8	0.24	0.00040	114.1	5807	17
Mohawk, Lock 12–Lock 11	6.9	27.6	0.05	0.00040	118.1	6078	17
Mohawk, Lock 12–Lock 11	6.9	283.2	0.24	0.00040	118.1	6078	17
Mohawk, Lock 11–Lock 10	6.9	27.6	0.04	0.00049	122.3	6362	17
Mohawk, Lock 11–Lock 10	6.9	283.2	0.34	0.00049	122.3	6362	17
Mohawk, Lock 10–Lock 9	9.7	27.4	0.04	0.00049	128.4	6781	17
Mohawk, Lock 10–Lock 9	9.7	224.6	0.29	0.00049	128.4	6781	17
Mohawk, Lock 9–Lock 8	7.9	27.4	0.04	0.00049	133.8	7143	17
Mohawk, Lock 9–Lock 8	7.9	224.6	0.28	0.00049	133.8	7143	17
Mohawk, Lock 8–Hwy 50 Schenectady	6.6	31.7	0.09	0.00049	138.5	7460	17
Mohawk, Lock 8–Hwy 50 Schenectady	6.6	205.3	0.28	0.00049	138.5	7460	17
Mohawk, Hwy 50 Schenectady–Lock 7	10.9	26.6	0.04	0.00049	146.7	8019	17
Mohawk, Hwy 50 Schenectady–Lock 7	10.9	205.3	0.19	0.00049	146.7	8019	17
Mohawk, Lock 7–Crescent Dam	16.4	28.3	0.03	0.00049	160.2	8936	17
Mohawk, Lock 7–Crescent Dam	16.4	228.2	0.22	0.00049	160.2	8936	17
Susquehanna, Bingham–Vestal	13.8	12.7	0.13	0.00044	176.0	10207	8
Susquehanna, Bingham–Vestal	13.8	58.0	0.40	0.00044	176.0	10207	8
Susquehanna, Bingham–Vestal	13.8	27.2	0.23	0.00044	176.0	10207	8
Susquehanna, Vestal–Owego	23.2	12.7	0.07	0.00009	190.5	11049	8
Susquehanna, Vestal–Owego	23.2	58.0	0.23	0.00009	190.5	11049	8
Susquehanna, Vestal–Owego	23.2	27.2	0.11	0.00009	190.5	11049	8
Susquehanna, Owego–Smithboro	16.4	12.2	0.12	0.00048	211.0	12238	8
Susquehanna, Owego–Smithboro	16.4	58.0	0.33	0.00048	211.0	12238	8
Susquehanna, Owego–Smithboro	16.4	26.8	0.28	0.00048	211.0	12238	8
Susquehanna, Smithboro–State line	10.0	14.9	0.16	0.00049	213.2	12362	8
Susquehanna, Smithboro–State line	10.0	75.6	0.92	0.00049	213.2	12362	8
Susquehanna, Smithboro–State line	10.0	36.2	0.28	0.00049	213.2	12362	8
Susquehanna, Smithboro–Athens, PA	16.4	14.9	0.11	0.00048	216.6	12564	8
Susquehanna, Smithboro–Athens, PA	16.4	75.6	0.73	0.00048	216.6	12564	8
Susquehanna, Smithboro–Athens, PA	16.4	36.2	0.28	0.00048	216.6	12564	8
Canaserage Cr,Dansville Hwy 245–Hwy 36	1.6	2.4	0.49	0.00078	4.7	396	7
Canaserage Cr,Dansville Hwy 245–Hwy 36	1.6	0.9	0.32	0.00078	4.7	396	7
Canaserage Cr,Dansville Hwy 245–Hwy 36	1.6	0.4	0.20	0.00078	4.7	396	7
Canaserage Cr,Dansville Hwy 236–White Br	3.1	2.4	0.58	0.00078	5.0	426	7
Canaserage Cr,Dansville Hwy 236–White Br	3.1	0.9	0.33	0.00078	5.0	426	7
Canaserage Cr,Dansville Hwy 236–White Br	3.1	0.4	0.22	0.00078	5.0	426	7

Table A-2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	De Ar	Ref
Canaserage Cr,White Br–Everman Rd	1.8	2.6	0.28	0.00307	5.3	444	7
Canaserage Cr,White Br–Everman Rd	1.8	0.9	0.15	0.00307	5.3	444	7
Canaserage Cr,White Br–Everman Rd	1.8	0.4	0.09	0.00307	5.3	444	7
Canaserage Cr, Everman Rd–W Sparta Station	1.6	2.6	0.60	0.00307	5.5	461	7
Canaserage Cr, Everman Rd–W Sparta Station	1.6	0.9	0.22	0.00307	5.5	461	7
Canaserage Cr, Everman Rd–W Sparta Station	1.6	0.5	0.20	0.00307	5.5	461	7
Canaserage Cr, W Sparta Sta–No Bridge Rd	2.8	2.7	0.36	0.00307	5.8	491	7
Canaserage Cr, W Sparta Sta–No Bridge Rd	2.8	0.8	0.20	0.00307	5.8	491	7
Canaserage Cr, W Sparta Sta–No Bridge Rd	2.8	0.5	0.13	0.00307	5.8	491	7
Canaserage Cr, No Bridge Rd–Gvld Hwy 258	2.4	2.7	0.29	0.00307	6.2	520	7
Canaserage Cr, No Bridge Rd–Gvld Hwy 258	2.4	0.8	0.15	0.00307	6.2	520	7
Canaserage Cr, No Bridge Rd–Gvld Hwy 258	2.4	0.5	0.08	0.00307	6.2	520	7
Canaserage Cr, Gvld Hwy 258–Gvld RR Br	1.8	2.7	0.22	0.00307	6.4	541	7
Canaserage Cr, Gvld Hwy 258–Gvld RR Br	1.8	0.8	0.12	0.00307	6.4	541	7
Canaserage Cr, Gvld Hwy 258–Gvld RR Br	1.8	0.5	0.07	0.00307	6.4	541	7
Honeoye Cr, Honeloy Falls– LV RR Br	0.8	2.4	0.24	0.00271	3.5	510	7
Honeoye Cr, Honeloy Falls– LV RR Br	0.8	0.7	0.11	0.00271	3.5	510	7
Honeoye Cr, Honeloy Falls– LV RR Br	0.8	0.1	0.02	0.00271	3.5	510	7
Honeoye Cr, LV RR Br–Sebley Rd	1.8	2.4	0.31	0.00271	3.6	518	7
Honeoye Cr, LV RR Br–Sebley Rd	1.8	0.7	0.11	0.00271	3.6	518	7
Honeoye Cr, LV RR Br–Sebley Rd	1.8	0.1	0.02	0.00271	3.6	518	7
Oatka Cr, Allen St Warsaw–Hwy 20	0.6	0.5	0.21	0.00172	1.7	109	7
Oatka Cr, Allen St Warsaw–Hwy 20	0.6	0.2	0.11	0.00172	1.7	109	7
Oatka Cr, Allen St Warsaw–Hwy 20	0.6	0.1	0.08	0.00172	1.7	109	7
Oatka Cr, Hwy 20–Abv ditch	0.8	0.5	0.24	0.00175	1.7	113	7
Oatka Cr, Hwy 20–Abv ditch	0.8	0.2	0.11	0.00175	1.7	113	7
Oatka Cr, Hwy 20–Abv ditch	0.8	0.1	0.07	0.00175	1.7	113	7
Oatka Cr, Abv ditch–Village line Rd	1.3	0.5	0.09	0.00175	1.8	120	7
Oatka Cr, Abv ditch–Village line Rd	1.3	0.2	0.05	0.00175	1.8	120	7
Oatka Cr, Abv ditch–Village line Rd	1.3	0.1	0.03	0.00175	1.8	120	7
Oatka Cr,Village line Rd–Hwy 19	2.7	0.6	0.08	0.00042	2.1	137	7
Oatka Cr,Village line Rd–Hwy 19	2.7	0.2	0.03	0.00042	2.1	137	7
Oatka Cr,Village line Rd–Hwy 19	2.7	0.1	0.02	0.00042	2.1	137	7
Oatka Cr, Hwy 19–School Rd Wyoming	9.7	0.6	0.08	0.00042	3.2	214	7
Oatka Cr, Hwy 19–School Rd Wyoming	9.7	0.1	0.03	0.00042	3.2	214	7
Oatka Cr, Hwy 19–School Rd Wyoming	9.7	0.1	0.02	0.00042	3.2	214	7
Genesee R, Rochester, GOC STP–Erie Can	3.5	59.5	0.26		79.2	6390	7
Genesee R, Rochester, GOC STP–Erie Can	3.5	4.5	0.04		79.2	6390	7
Genesee R, Rochester, GOC STP–Erie Can	3.5	12.3	0.05		79.2	6390	7
Genesee R, Rchstr, Kodad Pk STP–Stutson Br	5.7	119.6	0.26		79.2	6390	7
Genesee R, Rchstr, Kodad Pk STP–Seneca Pk	2.1	17.8	0.07		79.2	6390	7
Genesee R, Rchstr, Seneca Pk–blw Rattlesnk pt	2.7	16.7	0.11		79.2	6390	7
Genesee R, Rchstr, blw Rattlesnk pt–Stutson Br	1.0	18.4	0.07		79.2	6390	7
Genesee R, Rchstr, Kodad Pk STP– blw Seneca Pk	2.4	26.1	0.07		79.2	6390	7
Genesee R, Rchstr, blw Seneca Pk–Rattlesnake pt	1.4	26.1	0.04		79.2	6390	7
Genesee R, Rchstr, Rattlesnake pt–Stutson Br	1.9	26.1	0.06		79.2	6390	7
Catharine Cr, Millport–Croton Rd	3.7	0.3	0.24	0.00944	2.2	220	19
Catharine Cr, Croton Rd–S Genesee Rd	4.3	0.3	0.23	0.00738	2.2	220	19
Catharine Cr, S Genesee Rd–St Hwy 224	2.6	0.4	0.29	0.00340	2.2	220	19

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Catharine Cr, St Hwy 224–Wakins Glen Hwy 414	4.7	0.4	0.04	0.00125	2.2	220	19
Keuka Inlet, SWP Taylor Wine Co–Dead–end Rd	0.8	0.0	0.24	0.00549	0.4	44	19
Keuka Inlet, Dead–end Rd–St Hwy 54A	2.1	0.2	0.03	0.00335	0.7	65	19
Keuka Lk Outlet,Penn Yan–Keuka Mills	1.0	1.8	0.32	0.01078	4.8	471	19
Keuka Lk Outlet,Penn Yan–Keuka Mills	1.0	1.4	0.25	0.01078	4.8	471	19
Keuka Lk Outlet,Penn Yan–Keuka Mills	1.0	0.4	0.13	0.01078	4.8	471	19
Keuka Lk Outlet, Keuka Mills–Mays Mills	4.5	1.8	0.49	0.01078	5.1	503	19
Keuka Lk Outlet, Keuka Mills–Mays Mills	4.5	1.4	0.41	0.01078	5.1	503	19
Keuka Lk Outlet, Keuka Mills–Mays Mills	4.5	0.4	0.21	0.01078	5.1	503	19
Keuka Lk Outlet, Mays Mills–Dresden	4.6	1.8	0.49	0.00441	5.5	536	19
Keuka Lk Outlet, Mays Mills–Dresden	4.6	1.4	0.43	0.00441	5.5	536	19
Keuka Lk Outlet, Mays Mills–Dresden	4.6	0.4	0.21	0.00441	5.5	536	19
Fall Cr, Hwy366–Etna	3.4	1.9	0.22	0.00184	4.6	287	19
Fish Cr, Holcomb STP–Victor/Hoccomb Rd	1.1	0.1	0.16	0.00912	0.2	16	19
Fish Cr, Victor/Hoccomb Rd–site 566	1.1	0.1	0.13	0.00906	0.2	16	19
Fish Cr, site 566–Pond Rd	0.8	0.1	0.10	0.00906	0.2	16	19
Fish Cr, Pond Rd–Rice Rd 568	1.3	0.1	0.12	0.00906	0.2	16	19
Fish Cr, Pond Rd–Brace Rd 571	3.4	0.1	0.08	0.00543	0.2	16	19
Fish Cr, Rice Rd–Brace Rd	2.1	0.1	0.07	0.00338	0.2	16	19
Fish Cr, Rice Rd–Boughton Hill Rd 572	5.1	0.1	0.08	0.00479	0.2	16	19
Fish Cr, Brace Rd–Boughton Hill Rd	3.1	0.1	0.10	0.00575	0.2	16	19
Fish Cr, Boughton Hill Rd–St Hwy 96	2.1	0.1	0.11	0.01375	0.2	16	19
Ganargua Cr, Mud Cr.@Rte 96–Gillis Rd	5.3	0.7	0.09	0.00209	3.1	329	19
Ganargua Cr, Gillis Rd–Wilson	5.1	0.8	0.15	0.00209	3.1	329	19
Ganargua Cr, Wilson–Macedon	5.3	1.4	0.15	0.00209	3.1	329	19
Ganargua Cr, Macedon–Yellow Mills rd	4.8	1.4	0.20	0.00159	3.1	329	19
Ganargua Cr, Wilson–Yellow Mills Rd	10.1	1.4	0.11	0.00194	3.1	329	19
Ganargua Cr, Macedon–St Hwy 31	4.2	0.3	0.06	0.00159	3.1	329	19
Ganargua Cr, Hogback Rd–St Hwy 88	12.6	3.5	0.15	0.00031	3.3	353	19
Ganargua Cr, Hogback Rd–Town Line Rd	6.1	16.0	0.31	0.00031	3.3	345	19
Ganargua Cr, Town Line Rd–St Hwy 88	6.4	16.0	0.31	0.00031	3.3	353	19
Ganargua Cr, Town Line Rd–St Hwy 88	6.4	3.5	0.18	0.00031	3.3	353	19
Ganargua Cr, Town Line Rd–St Hwy 88	6.4	4.3	0.21	0.00031	3.3	353	19
Ganargua Cr, St Hwy 88–Norsen Rd	5.1	16.0	0.32	0.00031	3.4	359	19
Ganargua Cr, St Hwy 88–Norsen Rd	5.1	4.3	0.16	0.00031	3.4	359	19
Ganargua Cr, Norsen Rd–Lyons Newark Rd	8.2	16.0	0.28	0.00031	3.5	370	19
Naples Cr, St Rte 245–Parish Flat Rd	2.7	1.8	0.54	0.00099	0.8	80	19
Naples Cr, St Rte 245–Parish Flat Rd	2.7	0.3	0.19	0.00099	0.8	80	19
Naples Cr, Parish Flat Rd–West R nr Mouth	3.2	1.4	0.04	0.00099	1.9	199	19
Cananddaigua O,US 20–Shortsville	12.4	1.7	0.18	0.00221	6.5	689	19
Cananddaigua O,US 20–Shortsville	12.4	0.8	0.09	0.00221	6.5	689	19
Cananddaigua O,Shortsville–Clifton Springs	10.9	2.6	0.25	0.00251	7.6	796	19
Cananddaigua O,Shortsville–Clifton Springs	10.9	0.9	0.14	0.00251	7.6	796	19
Cananddaigua O, Clifton Springs–Flint Cr	9.5	4.0	0.35	0.00193	8.6	901	19
Cananddaigua O, Clifton Springs–Flint Cr	9.5	0.9	0.14	0.00193	8.6	901	19
Cananddaigua O, Flint Cr–Gifford Rd	9.8	8.5	0.41	0.00109	10.4	1092	19
Cananddaigua O, Flint Cr–Gifford Rd	9.8	1.6	0.12	0.00109	10.4	1092	19
Cananddaigua O, Gifford Rd–Lyons	10.6	8.8	0.21	0.00100	12.8	1344	19
Cananddaigua O, Gifford Rd–Lyons	10.6	1.6	0.06	0.00100	12.8	1344	19

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Clyde R/Erie Canal, Lock 27–Creager	4.8	25.5	0.15	0.00000	16.7	1761	19
Clyde R/Erie Canal, Creager–St Hwy 414	10.3	25.5	0.12	0.00000	17.7	1863	19
Clyde R/Erie Canal, St Hwy 414–Lock 26	3.7	25.5	0.11	0.00000	18.1	1901	19
Clyde R/Erie Canal, Lock 26–St Hwy 89	10.3	24.1	0.10	0.00000	19.1	2012	19
Clyde R/Erie Canal, St Hwy 89–Seneca Canal	3.1	22.7	0.13	0.00000	19.4	2046	19
Owasco Inlet, St Hwy 222–St Hwy 38	3.9	0.3	0.10	0.00635	1.8	117	19
Owasco Inlet, St Hwy 38–St Hwy 38/RR	4.0	0.5	ERR	0.00587	1.8	117	19
Owasco Inlet, St Hwy 38/RR–Dead End Rd	5.5	0.7	0.12	0.00382	2.7	174	19
Owasco Inlet, St Hwy 90 at Locke–Dead End Rd	2.1	0.7	0.18	0.00361	2.7	174	19
Owasco Inlet, Dead End Rd–Long Hill Rd	4.8	0.8	0.15	0.00282	3.8	248	19
Owasco Inlet, St Hwy 90 at Locke–Long Hill Rd	6.9	0.8	0.16	0.00306	3.8	248	19
Owasco Inlet, Long Hill Rd–St Hwy 38 at Moravia	1.0	1.1	0.17	0.00158	4.1	267	19
Owasco Outlet, Canoga St/Auburn–Troopsville	5.1	1.0	0.19	0.00477	8.2	534	19
Owasco Outlet, Canoga St/Auburn–Troopsville	5.1	2.0	0.30	0.00477	8.2	534	19
Owasco Outlet, Troopsville–Hayden Rd	5.6	1.0	0.19	0.00357	9.5	621	19
Owasco Outlet, Troopsville–Hayden Rd	5.6	2.0	0.29	0.00357	9.5	621	19
Owasco Outlet, Hayden Rd–St Rte 32 Port Byron	3.1	1.0	0.20	0.00166	10.3	675	19
Owasco Outlet, Hayden Rd–SH 32 Port Byron	3.1	2.0	0.29	0.00166	10.3	675	19
Owasco Outlet, SH 32/Byron–SH 38	3.4	1.0	0.10	0.00166	11.3	739	19
Owasco Outlet, SH 32/Byron–SH 38	3.4	2.0	0.16	0.00166	11.3	739	19
Owasco Outlet, St Rte 38–Bridge at mouth	2.6	1.0	0.11	0.00166	12.1	793	19
Owasco Outlet, St Rte 38–Bridge at mouth	2.6	2.0	0.19	0.00166	12.1	793	19
Skaneateles Cr, Skan/Eliz St–Willow Glen	1.6	0.2	0.06	0.00637	3.0	196	19
Skaneateles Cr, Skan/Eliz St–Mottville	3.2	0.3	0.09	0.00637	3.0	196	19
Skaneateles Cr, Skan/Eliz St–Mottville	3.2	0.3	0.11	0.00637	3.0	196	19
Skaneateles Cr, Skan/Eliz St–Mottville	3.2	0.3	0.15	0.00637	3.0	196	19
Skaneateles Cr, Mottville–Long Bridge	1.1	0.3	0.17	0.00924	3.0	196	19
Skaneateles Cr, Mottville–Jordon Rd Sk Fk	3.1	0.3	0.16	0.00924	3.0	196	19
Skaneateles Cr, Mottville–Jordon Rd Sk Fk	3.1	0.4	0.17	0.00924	3.0	196	19
Skaneateles Cr, Mottville–Hamlton Rd	9.2	0.4	0.14	0.00805	3.0	196	19
Skaneateles Cr, Jordon Rd Sk Fk–Depot St	0.5	0.6	0.34	0.02037	3.0	196	19
Skaneateles Cr, Jordon Rd Sk Fk–Irish Rd	1.4	0.6	0.18	0.02037	3.0	196	19
Skaneateles Cr, Jordon Rd Sk–Chatfield Rd	3.9	0.9	0.23	0.00963	3.0	196	19
Skaneateles Cr, Jordon Rd Sk–Hamilton Rd	6.1	0.9	0.19	0.00745	3.0	196	19
Skaneateles Cr, Chatfield–Hamilton Rd	2.3	1.4	0.31	0.00513	3.0	196	19
Skaneateles Cr, Chatfield–SH 31C	6.6	1.0	0.29	0.00513	3.0	196	19
Skaneateles Cr, Hamilton Rd–SH 32 Jordon	7.1	1.0	0.33	0.00513	3.0	196	19
Skaneateles Cr, Hamilton Rd–SH 32 Jordon	7.1	0.5	0.16	0.00513	3.0	196	19
Skaneateles Cr, SH 31C–SH 32 Jordon	2.7	1.4	0.46	0.00513	3.0	196	19
Skaneateles Cr, SH 31C–Mechanic St	1.6	1.4	0.50	0.00513	3.0	196	19
Ninemile Cr, SH 175 Marcellus–SH 174 Martisco	4.2	5.8	0.66	0.01549	1.9	127	19
Ninemile Cr, SH 175 Marcellus–SH 174 Martisco	4.2	2.3	0.34	0.01549	1.9	127	19
Ninemile Cr, SH 174 Martisco– SH 5 Camillus	5.5	6.5	0.39	0.00235	3.3	218	19
Ninemile Cr, SH 174 Martisco– SH 5 Camillus	5.5	2.8	0.26	0.00235	3.3	218	19
Ninemile Cr, SH 5 Camillus–A Rd blw Amboy	6.9	2.8	0.24	0.00105	4.5	267	19
Ninemile Cr, SH 5 Camillus–A Rd blw Amboy	6.9	7.1	0.36	0.00105	4.5	267	19
Ninemile Cr, A Rd blw Amboy–I–690 Lakeland	3.7	10.8	0.40	0.00105	5.5	298	19
Seneca R,SH96A–Lock 4	7.7	23.4	0.27	0.00000	21.6	1823	19
Seneca R, Lock 4–Lock 2/3	6.9	23.4	0.23	0.00000	21.9	1845	19

Table A-2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Seneca R, Lock 2/3–Lock 1	6.4	28.3	0.31	0.00000	22.1	1865	19
Seneca R, Lock 1–Jct Clyde R	6.0	28.3	0.21	0.00000	70.4	5934	19
Seneca R, Jct Clyde R–Penn Central RR	7.7	57.8	0.22	0.00000	95.0	8004	19
Seneca R, Penn Central RR–SH 38	5.3	64.3	0.30	0.00000	95.2	8020	19
Seneca R, SH 38–SH 34	8.0	23.2	0.06	0.00000	95.5	8045	19
Seneca R, Penn Central RR–SH 34	13.4	65.1	0.41	0.00000	95.5	8045	19
Seneca R, SH 34–River Road	6.9	66.5	0.23	0.00000	95.7	8066	19
Seneca R, SH 34–River Road	6.9	24.6	0.10	0.00000	95.7	8066	19
Seneca R, River Road–Jones Point	3.7	68.0	0.07	0.00000	95.9	8078	19
Seneca R, Jones Point–SH 48, Lock 24	16.1	68.8	0.15	0.00000	96.4	8127	19
Seneca R, Jones Point–SH 48, Lock 24	16.1	32.3	0.07	0.00000	96.4	8127	19
Seneca R, SH 48, Lock 24–SH 370	8.9	75.6	0.17	0.00000	96.8	8155	19
Seneca R, SH 48, Lock 24–SH 370	8.9	39.6	0.11	0.00000	96.8	8155	19
Seneca R, SH 370–SH 31	7.7	75.6	0.20	0.00000	105.8	8917	19
Seneca R, SH 370–SH 31	7.7	39.6	0.12	0.00000	105.8	8917	19
Seneca R, SH 31–Three Rivers	3.2	75.6	0.21	0.00000	105.9	8927	19
Seneca R, SH 31–Three Rivers	3.2	39.6	0.16	0.00000	105.9	8927	19
Scondoa Cr, SH 234 Vernon–Williams St	4.5	0.2	0.13	0.00547	2.4	153	19
Scondoa Cr, Williams St–Sholtz Rd	3.1	0.2	0.07	0.00547	2.9	184	19
Oneida Cr, Scon Cr, Sholtz Rd–Sconondoa St	3.4	0.7	0.07	0.00189	3.6	227	19
Oneida Cr, Sconondoa St–SH 46 Durhamville	4.2	0.8	0.13	0.00092	4.7	293	19
Oneida Cr, SH 46 Durhamville–Shallow Rd	7.4	0.8	0.07	0.00080	5.8	364	19
Oneida Cr, Shallow Rd–SH 31	2.9	0.8	0.13	0.00080	6.3	397	19
Oneida Cr, SH 31–SH 13	3.7	0.8	0.03	0.00080	7.1	443	19
Cowaselon Cr, SH 13/Canastota–Oniontown	6.9	0.8	0.26	0.00093	1.9	119	19
Cowaselon Cr, SH 13/Canastota–Oniontown	6.9	1.5	0.30	0.00093	1.9	119	19
Cowaselon Cr, Oniontown–End Ditch Rd	2.6	1.1	0.13	0.00108	3.3	210	19
Limestone Cr, Fayetteville Dam–Kirkville Rd	9.8	1.1	0.09	0.00097	3.5	221	19
Limestone Cr, Kirkville Rd–Manlius Rd	3.7	1.6	0.08	0.00037	3.9	245	19
Chittenango Cr, Tuscarora Rd–Bolivar Rd	5.5	0.6	0.04	0.00138	5.5	347	19
Chittenango Cr, Bolivar Rd–Hoag Rd	4.2	0.6	0.06	0.00073	5.8	360	19
Chittenango Cr, Hoag Rd–Kirkville Rd	3.5	0.7	0.05	0.00043	5.9	372	19
Chittenango Cr, Kirkville Rd–N Manlius	5.0	0.7	0.06	0.00037	6.2	389	19
Chittenango Cr, N Manlius–Peck Rd	1.9	2.3	0.21	0.00060	6.3	396	19
Chittenango Cr, Peck Rd–Oxbow Rd	6.6	2.4	0.11	0.00021	10.5	660	19
Chittenango Cr, Oxbow Rd–Bridgeport	6.1	2.5	0.05	0.00021	13.0	812	19
Chittenango Cr, Bridgeport–Mouth	4.5	4.0	0.05	0.00009	15.1	945	19
Oneida R,Brewerton–Caughdenoy	6.9	41.6	0.13	0.00000	72.0	3579	19
Oneida R,Caughdenoy–Erie at Oak Orchard	8.7	41.6	0.16	0.00001	73.7	3662	19
Oneida R, Oak Orchard–Horseshoe I us	2.9	42.2	0.14	0.00001	74.2	3690	19
Oneida R, Oak Orchard–Horseshoe I us	2.9	5.7	0.04	0.00001	74.2	3690	19
Oneida R, Horseshoe I us–Horseshoe I ds	7.6	10.5	0.19	0.00001	75.7	3763	19
Oneida R, Horseshoe I us–Horseshoe I ds	7.6	1.4	0.05	0.00001	75.7	3763	19
Oneida R, Horseshoe I ds–Erie C at SH 57	1.9	42.5	0.13	0.00001	76.1	3782	19
Oneida R, Horseshoe I ds–Erie C at SH 57	1.9	5.7	0.03	0.00001	76.1	3784	19
Oswego R, 3 R point–Lock 1 at Phoenix	3.5	118.9	0.18	0.00000	257.4	12795	19
Oswego R, Lock 1 at Phoenix–Lock 3	16.4	54.1	0.10	0.00000	262.3	13040	19
Oswego R, Armstrong Cork Co.–Lock 5	6.4	55.8	0.14	0.00000	265.6	13201	19
Oswego R, Lock 5–Lock 6	5.5	55.8	0.06	0.00000	265.7	13209	19

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Potomac,Cumberland–North Branch	14.0	7.2	0.15	0.00066	36.2	2266	23
Potomac, N Branch–Oldtown	14.0	7.9	0.20	0.00066	36.2	2266	23
Potomac, Oldtown– Paw Paw	17.2	13.0	0.20	0.00066	93.4	8052	23
Potomac, Paw Paw–Doe Gully	31.9	12.2	0.20	0.00051	103.1	9273	23
Potomac, Paw Paw–Handcock	29.1	14.9	0.21	0.00036	117.3	10549	23
Potomac, Handcock–Fort Frederick	6.6	17.3	0.07	0.00036	126.5	7874	23
Potomac, Ft Federick–Dam # 5	15.6	15.6	0.04	0.00036	135.0	7874	23
Potomac, Dam #5–Williamsport	10.6	17.6	0.10	0.00036	141.0	7874	23
Potomac, Williamsport–Dam #4	24.9	19.5	0.05	0.00036	156.3	14191	23
Potomac, Dam #4–Shepherdstown	18.8	20.1	0.15	0.00036	169.0	15374	23
Potomac,Shepardstown–Dam #3	16.9	32.3	0.09	0.00045	177.0	16102	23
Potomac, Dam #3–Brunswick	11.7	40.8	0.26	0.00057	253.0	23976	23
Potomac, Brunswick–Point of Rocks	10.1	40.2	0.24	0.00066	265.2	24996	23
Potomac, Point of Rocks–Whites Ferry	20.0	51.0	0.25	0.00066	286.7	27195	23
Potomac, Whites Ferry–Seneca	21.2	50.1	0.17	0.00066	299.3	28490	23
Potomac, Seneca–Little Falls Dam	27.0	51.0	0.10	0.00066	313.5	29940	23
Shenandoah, Waynesboro–Hopeman	8.5	1.6	0.11	0.00170	5.7	384	22
Shenandoah, Waynesboro–Hopeman	8.5	3.7	0.21	0.00170	5.7	384	22
Shenandoah, Hopeman–Crimora	11.9	1.8	0.13	0.00170	7.1	478	22
Shenandoah, Hopeman–Crimora	11.9	4.5	0.23	0.00170	5.7	478	22
Shenandoah, Crimora–Harriston	10.3	1.9	0.12	0.00170	7.2	549	22
Shenandoah, Crimora–Harriston	10.3	4.8	0.25	0.00170	7.2	549	22
Shenandoah, Harriston–Island Ford	27.0	7.7	0.13	0.00170	30.0	2976	22
Shenandoah, Harriston–Island Ford	27.0	19.3	0.27	0.00170	30.0	2976	22
Shenandoah, Island Ford–Shenandoah	21.7	9.3	0.14	0.00114	33.4	3313	22
Shenandoah, Island Ford–Shenandoah	21.7	20.7	0.25	0.00114	33.4	3313	22
Shenandoah, Shenandoah–Grove Hill	12.7	10.6	0.18	0.00114	33.7	3341	22
Shenandoah, Shenandoah–Grove Hill	12.7	23.2	0.35	0.00114	33.7	3341	22
Shenandoah, Grove Hill–US Hwy 211	24.1	10.7	0.15	0.00114	37.2	3566	22
Shenandoah, Grove Hill–US Hwy 211	24.1	24.1	0.31	0.00114	37.2	3566	22
Shenandoah, US Hwy 211–Bixler Bridge	11.3	11.2	0.06	0.00114	38.5	3622	22
Shenandoah, US Hwy 211–Bixler Bridge	11.3	23.2	0.18	0.00114	38.5	3622	22
Shenandoah, Bixler Bridge–Bentonville	42.0	12.8	0.14	0.00114	43.4	4083	22
Shenandoah, Bixler Bridge–Bentonville	42.0	28.9	0.67	0.00114	43.4	4083	22
Shenandoah, Bentonville–Front Royal	24.8	13.0	0.14	0.00114	45.2	4253	22
Shenandoah, Bentonville–Front Royal	24.8	29.4	0.31	0.00114	45.2	4253	22
Shenandoah, Front Royal–Morgan Ford	16.4	18.1	0.10	0.00114	71.2	7179	22
Shenandoah, Front Royal–Morgan Ford	16.4	43.0	0.22	0.00114	71.2	7179	22
Shenandoah, Morgon Ford–HWWY 17&50	17.5	15.4	0.14	0.00057	71.8	7242	22
Shenandoah, Morgon Ford–HWWY 17&50	17.5	43.0	0.35	0.00057	71.8	7242	22
Shenandoah, Hwwy 17/50–HWY 7	23.3	15.9	0.21	0.00057	75.5	7615	22
Shenandoah, Hwwy 17/50–HWY 7	23.3	48.7	0.43	0.00057	75.5	7615	22
Shenandoah, HWY 7–HWY 9	22.0	16.9	0.16	0.00057	77.3	7804	22
Shenandoah, HWY 7–HWY 9	22.0	48.1	0.31	0.00057	77.3	7804	22
Shenandoah, HWY 9–Harpers Ferry	12.2	16.1	0.09	0.00218	78.6	7928	22
Shenandoah, HWY 9–Harpers Ferry	12.2	49.0	0.23	0.00218	78.6	7928	22
Passaic R, Bernardsville–Chatham	32.0	3.6	0.17	0.00052	4.5	259	1
Passaic R, Bernardsville–Chatham	32.0	0.4	0.04	0.00052	4.5	259	1
Passaic R, Chatham–Florham Park	6.8	1.6	0.19	0.00050	5.9	340	1

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Passaic R, Chatham–Florham Park	6.8	0.2	0.04	0.00050	5.9	340	1
Passaic R, Florham Park–Hanover	7.9	1.8	0.11	0.00031	5.7	332	1
Passaic R, Florham Park–Hanover	7.9	0.1	0.04	0.00031	5.7	332	1
Passaic R, Hanover–Pine Brook	9.7	4.6	0.14	0.00006	14.9	904	1
Passaic R, Hanover–Pine Brook	9.7	0.4	0.04	0.00006	14.9	904	1
Passaic R, Pine Brook–Clinton	5.5	4.7	0.16	0.00011	15.3	927	1
Passaic R, Pine Brook–Clinton	5.5	0.4	0.10	0.00011	15.3	927	1
Passaic R, Clinton–Two Bridges	14.5	4.8	0.09	0.00004	15.5	935	1
Passaic R, Clinton–Two Bridges	14.5	0.4	0.07	0.00004	15.5	935	1
Passaic R, Two Bridges–Little Falls	5.5	11.3	0.08	0.00017	32.2	1974	1
Passaic R, Two Bridges–Little Falls	5.5	0.8	0.03	0.00017	32.2	1974	1
Rockaway R, Dover–Boonton	23.8	2.8	0.18	0.00307	5.7	300	1
Rockaway R, Dover–Boonton	23.8	11.4	0.47	0.00307	5.7	300	1
Whippany R, Whippany–Pine Brook	9.2	0.6	0.11	0.00199	3.2	177	1
Whippany R, Whippany–Pine Brook	9.2	1.9	0.18	0.00199	3.2	177	1
Whippany R, Whippany–Pine Brook	9.2	3.6	0.24	0.00199	3.2	177	1
PomptonR, Pompton Planes–Two Bridges	10.9	1.4	0.06	0.00056	13.5	984	1
PomptonR, Pompton Planes–Two Bridges	10.9	2.8	0.12	0.00056	13.5	984	1
N Platte, Mystery Br–Cole Cr Rd Br	23.7	26.6	0.70	0.00088	42.1	33670	2
N Platte, Cole Cr Rd Br–Old Hwy Bridge	26.2	26.6	0.64	0.00088	43.5	35063	2
Yampa River, site 3–site 7	11.4	67.4	1.51	0.00408	15.2	1856	3
Yampa River, site 7–site 10	7.1	129.1	1.40	0.00258	16.9	2109	3
Yampa River, site 15–site 16	15.4	126.6	1.41	0.00158	27.7	3704	3
Yampa River, site 16–site 17	35.9	130.5	1.31	0.00127	33.4	4533	3
Yampa River, site 3–site 4	1.9	2.2	0.46	0.00442	13.2	1564	3
Yampa River, site 4–site 5	4.2	2.4	0.39	0.00364	14.0	1687	3
Yampa River, site 5–site 6	2.4	2.0	0.15	0.00581	14.5	1762	3
Yampa River, site 8–site 9	2.9	4.3	0.34	0.00253	16.5	2043	3
Yampa River, site 9–site 11	4.7	4.8	0.33	0.00202	17.6	2216	3
Yampa River, site 13–site 14	6.8	181.5	0.37	0.00149	20.7	2661	3
Little Snake R, site 3–site 6	17.5	45.9	1.40	0.00261	9.5	1399	3
Little Snake R, site 6–site 8	23.7	58.6	1.29	0.00219	14.6	2559	3
Little Snake R, site 9–site 13	53.6	47.9	1.13	0.00097	28.9	5822	3
Little Snake R, site 2–site 3	4.2	16.9	1.19	0.00510	7.4	917	3
Little Snake R, site 3–site 4	8.5	20.1	1.13	0.00322	8.2	1110	3
Little Snake R, site 4–site 5	7.9	20.1	0.88	0.00155	9.1	1326	3
Little Snake R, site 5–site 6	2.4	20.1	1.00	0.00568	9.5	1399	3
St Marys River, Pleasant Mills–Scheiman Bridge	23.2	0.6	0.05	13.5			11
St Marys River, Pleasant Mills–Scheiman Bridge	23.2	3.4	0.16	13.5			11
St Marys River, Pleasant Mills–Scheiman Bridge	23.2	17.6	0.38	13.5			11
St Marys River, Pleasant Mills–Scheiman Bridge	23.2	22.9	0.43	13.5			11
St Marys River, Pleasant Mills–Scheiman Bridge	23.2	34.5	0.54	13.5			11
White River, I 70–USGS gage at Brookville	81.4	3.8	0.23	15.0	1347	0.00129	11
White River, I 70–USGS gage at Brookville	81.4	7.5	0.35	15.0	1347	0.00129	11
White River, I 70–USGS gage at Brookville	81.4	15.0	0.50	15.0	1347	0.00129	11
White River, I 70–USGS gage at Brookville	81.4	30.1	0.72	15.0	1347	0.00129	11
East Fork White River, Dam–Beelor Road	9.3	0.8	0.17	3.3	313	0.00195	11
East Fork White River, Dam–Beelor Road	9.3	1.6	0.28	3.3	313	0.00195	11
East Fork White River, Dam–Beelor Road	9.3	3.3	0.46	3.3	313	0.00195	11

Table A-2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
East Fork White River, Dam–Beelor Road	9.3	6.5	0.76	3.3	313	0.00195	11
Eest Fork White River, Beelor Rd–SR 44	20.0	1.5	0.17	6.1	518	0.00116	11
Eest Fork White River, Beelor Rd–SR 44	20.0	3.1	0.26	6.1	518	0.00116	11
Eest Fork White River, Beelor Rd–SR 44	20.0	6.1	0.40	6.1	518	0.00116	11
Eest Fork White River, Beelor Rd–SR 44	20.0	12.3	0.62	6.1	518	0.00116	11
Whitewater River, USGS gage at Brookville–Harrison, C	34.4	8.8	0.34	35.3	3170	0.00091	11
Whitewater River, USGS gage at Brookville–Harrison, C	34.4	17.7	0.48	35.3	3170	0.00091	11
Whitewater River, USGS gage at Brookville–Harrison, C	34.4	35.3	0.68	35.3	3170	0.00091	11
Whitewater River, USGS gage at Brookville–Harrison, C	34.4	70.6	0.97	35.3	3170	0.00091	11
Walbash River, New Corydon–Linn Grove	34.8	1.3	0.11	5.1	679	0.00019	11
Walbash River, New Corydon–Linn Grove	34.8	2.6	0.17	5.1	679	0.00019	11
Walbash River, New Corydon–Linn Grove	34.8	5.1	0.24	5.1	679	0.00019	11
Walbash River, New Corydon–Linn Grove	34.8	10.3	0.35	5.1	679	0.00019	11
Walbash River, Linn Grove–Bluffton	18.2	2.3	0.14	9.0	1173	0.00025	11
Walbash River, Linn Grove–Bluffton	18.2	4.5	0.21	9.0	1173	0.00025	11
Walbash River, Linn Grove–Bluffton	18.2	9.0	0.29	9.0	1173	0.00025	11
Walbash River, Linn Grove–Bluffton	18.2	18.1	0.40	9.0	1173	0.00025	11
Walbash River, Buffton–Walbash	75.3	4.1	0.21	16.3	1867	0.00065	11
Walbash River, Buffton–Walbash	75.3	8.1	0.32	16.3	1867	0.00065	11
Walbash River, Buffton–Walbash	75.3	16.3	0.42	16.3	1867	0.00065	11
Walbash River, Buffton–Walbash	75.3	32.5	0.58	16.3	1867	0.00065	11
Eel River, Whitley Co Rd 260 W–Miami Co Rd 500 E	62.4	2.4	0.18	9.7	1080	0.0004	11
Eel River, Whitley Co Rd 260 W–Miami Co Rd 500 E	62.4	4.9	0.27	9.7	1080	0.0004	11
Eel River, Whitley Co Rd 260 W–Miami Co Rd 500 E	62.4	9.7	0.42	9.7	1080	0.0004	11
Eel River, Whitley Co Rd 260 W–Miami Co Rd 500 E	62.4	19.4	0.62	9.7	1080	0.0004	11
Eel River, Miami Co Rd 500 E–US 24 & 35	50.4	5.0	0.19	20.1	2044	0.0006	11
Eel River, Miami Co Rd 500 E–US 24 & 35	50.4	10.1	0.29	20.1	2044	0.0006	11
Eel River, Miami Co Rd 500 E–US 24 & 35	50.4	20.1	0.45	20.1	2044	0.0006	11
Eel River, Miami Co Rd 500 E–US 24 & 35	50.4	40.3	0.70	20.1	2044	0.0006	11
Tippecanoe River, Fox Farm Rd–Marshall Co Rd 18 E	30.3	0.7	0.11	2.7	293	0.00034	11
Tippecanoe River, Fox Farm Rd–Marshall Co Rd 18 E	30.3	1.4	0.17	2.7	293	0.00034	11
Tippecanoe River, Fox Farm Rd–Marshall Co Rd 18 E	30.3	2.7	0.27	2.7	293	0.00034	11
Tippecanoe River, Fox Farm Rd–Marshall Co Rd 18 E	30.3	5.5	0.43	2.7	293	0.00034	11
Tippecanoe River, Marshall Co Rd 18 E– SR 16 & 39	138.2	5.5	0.27	22.1	2217	0.00028	11
Tippecanoe River, Marshall Co Rd 18 E– SR 16 & 39	138.2	11.0	0.36	22.1	2217	0.00028	11
Tippecanoe River, Marshall Co Rd 18 E– SR 16 & 39	138.2	22.1	0.48	22.1	2217	0.00028	11
Tippecanoe River, Marshall Co Rd 18 E– SR 16 & 39	138.2	44.1	0.64	22.1	2217	0.00028	11
Wildcat Creek, Kokomo–Burlington	35.9	1.6	0.20	6.3	627	0.00066	11
Wildcat Creek, Kokomo–Burlington	35.9	3.1	0.25	6.3	627	0.00066	11
Wildcat Creek, Kokomo–Burlington	35.9	6.3	0.34	6.3	627	0.00066	11
Wildcat Creek, Kokomo–Burlington	35.9	12.6	0.45	6.3	627	0.00066	11
Wildcat Creek, Burlington–Wolf Rd	45.2	2.4	0.17	9.4	1026	0.00072	11
Wildcat Creek, Burlington–Wolf Rd	45.2	4.7	0.25	9.4	1026	0.00072	11
Wildcat Creek, Burlington–Wolf Rd	45.2	9.4	0.33	9.4	1026	0.00072	11
Wildcat Creek, Burlington–Wolf Rd	45.2	18.9	0.43	9.4	1026	0.00072	11
SF Wildcat Cr, Tippecanoe Co Rd 5 S–Carey Camp	17.7	0.0	0.25	0.2	629	0.00119	11
SF Wildcat Cr, Tippecanoe Co Rd 5 S–Carey Camp	17.7	0.1	0.36	0.2	629	0.00119	11
SF Wildcat Cr, Tippecanoe Co Rd 5 S–Carey Camp	17.7	0.2	0.51	0.2	629	0.00119	11
SF Wildcat Cr, Tippecanoe Co Rd 5 S–Carey Camp	17.7	0.4	0.70	0.2	629	0.00119	11

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Wildcar Creek, Wolf Rd–USGS gage near Lafayette	21.6	4.9	0.22	19.8	2056	0.00092	11
Wildcar Creek, Wolf Rd–USGS gage near Lafayette	21.6	9.9	0.32	19.8	2056	0.00092	11
Wildcar Creek, Wolf Rd–USGS gage near Lafayette	21.6	19.8	0.47	19.8	2056	0.00092	11
Wildcar Creek, Wolf Rd–USGS gage near Lafayette	21.6	39.6	0.66	19.8	2056	0.00092	11
Sugar Creek, US 231–Davis Bridge	18.8	3.0	0.29	12.1	1318	0.00131	11
Sugar Creek, US 231–Davis Bridge	18.8	6.0	0.42	12.1	1318	0.00131	11
Sugar Creek, US 231–Davis Bridge	18.8	12.1	0.59	12.1	1318	0.00131	11
Sugar Creek, US 231–Davis Bridge	18.8	24.1	0.85	12.1	1318	0.00131	11
Sugar Creek, Davis Bridge–Laffayette Road	43.1	4.4	0.30	17.7	1735	0.00071	11
Sugar Creek, Davis Bridge–Laffayette Road	43.1	8.8	0.47	17.7	1735	0.00071	11
Sugar Creek, Davis Bridge–Laffayette Road	43.1	17.7	0.72	17.7	1735	0.00071	11
Sugar Creek, Davis Bridge–Laffayette Road	43.1	35.3	1.10	17.7	1735	0.00071	11
White River, US 27–Anderson	89.3	1.4	0.11	5.7	624	0.00075	11
White River, US 27–Anderson	89.3	2.9	0.17	5.7	624	0.00075	11
White River, US 27–Anderson	89.3	5.7	0.26	5.7	624	0.00075	11
White River, US 27–Anderson	89.3	11.5	0.40	5.7	624	0.00075	11
White River, Anderson–Noblesville	48.1	2.6	0.21	10.4	1052	0.00058	11
White River, Anderson–Noblesville	48.1	5.2	0.26	10.4	1052	0.00058	11
White River, Anderson–Noblesville	48.1	10.4	0.38	10.4	1052	0.00058	11
White River, Anderson–Noblesville	48.1	20.8	0.55	10.4	1052	0.00058	11
White River, Noblesville–Nora	25.1	5.6	0.18	22.5	2222	0.00032	11
White River, Noblesville–Nora	25.1	11.2	0.29	22.5	2222	0.00032	11
White River, Noblesville–Nora	25.1	22.5	0.42	22.5	2222	0.00032	11
White River, Noblesville–Nora	25.1	44.9	0.70	22.5	2222	0.00032	11
White River, Nora–Indianapolis	27.8	7.5	0.07	29.8	3157	0.00058	11
White River, Nora–Indianapolis	27.8	14.9	0.11	29.8	3157	0.00058	11
White River, Nora–Indianapolis	27.8	29.8	0.17	29.8	3157	0.00058	11
White River, Nora–Indianapolis	27.8	59.6	0.32	29.8	3157	0.00058	11
Big Blue River, SR3–SR9	72.9	1.3	0.16	5.4	477	0.00091	11
Big Blue River, SR3–SR9	72.9	2.7	0.25	5.4	477	0.00091	11
Big Blue River, SR3–SR9	72.9	5.4	0.40	5.4	477	0.00091	11
Big Blue River, SR3–SR9	72.9	10.7	0.63	5.4	477	0.00091	11
Big Blue River, SR9–Edinburg	40.2	3.2	0.16	12.7	1090	0.00084	11
Big Blue River, SR9–Edinburg	40.2	6.3	0.28	12.7	1090	0.00084	11
Big Blue River, SR9–Edinburg	40.2	12.7	0.38	12.7	1090	0.00084	11
Big Blue River, SR9–Edinburg	40.2	25.4	0.66	12.7	1090	0.00084	11
Driftwood River, Edinburg–Columbus	23.2	7.8	0.23	31.1	2745	0.00047	11
Driftwood River, Edinburg–Columbus	23.2	15.6	0.32	31.1	2745	0.00047	11
Driftwood River, Edinburg–Columbus	23.2	31.1	0.45	31.1	2745	0.00047	11
Driftwood River, Edinburg–Columbus	23.2	62.3	0.70	31.1	2745	0.00047	11
St Joseph River, Newville–Cedarville Reservoir	34.8	3.5	0.15	13.8	1593	0.00025	11
St Joseph River, Newville–Cedarville Reservoir	34.8	6.9	0.24	13.8	1593	0.00025	11
St Joseph River, Newville–Cedarville Reservoir	34.8	13.8	0.37	13.8	1593	0.00025	11
St Joseph River, Newville–Cedarville Reservoir	34.8	27.7	0.59	13.8	1593	0.00025	11
Cedar Cr, Dekalb Rd 24–Cedar Chapel Bridge	23.5	0.5	0.12	1.9	226	0.00095	11
Cedar Cr, Dekalb Rd 24–Cedar Chapel Bridge	23.5	1.0	0.20	1.9	226	0.00095	11
Cedar Cr, Dekalb Rd 24–Cedar Chapel Bridge	23.5	1.9	0.34	1.9	226	0.00095	11
Cedar Cr, Dekalb Rd 24–Cedar Chapel Bridge	23.5	3.8	0.52	1.9	226	0.00095	11
Cedar Cr, Cedar Chapel Bridge–SR1	21.1	1.6	0.18	6.5	699	0.00073	11

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
Cedar Cr, Cedar Chapel Bridge–SR1	21.1	3.2	0.27	6.5	699	0.00073	11
Cedar Cr, Cedar Chapel Bridge–SR1	21.1	6.5	0.38	6.5	699	0.00073	11
Cedar Cr, Cedar Chapel Bridge–SR1	21.1	13.0	0.59	6.5	699	0.00073	11
St Marys River,NY,C hi,&St.L RR–Scheiman Bridge	29.3	3.4	0.15	13.5	1608	0.0002	11
St Marys River,NY,C hi,&St.L RR–Scheiman Bridge	29.3	6.8	0.21	13.5	1608	0.0002	11
St Marys River,NY,C hi,&St.L RR–Scheiman Bridge	29.3	13.5	0.30	13.5	1608	0.0002	11
St Marys River,NY,C hi,&St.L RR–Scheiman Bridge	29.3	27.1	0.48	13.5	1608	0.0002	11
St Marys River, Scheiman Bridge–Spy run	34.9	3.9	0.16	15.6	1974	0.00017	11
St Marys River, Scheiman Bridge–Spy run	34.9	7.8	0.22	15.6	1974	0.00017	11
St Marys River, Scheiman Bridge–Spy run	34.9	15.6	0.34	15.6	1974	0.00017	11
St Marys River, Scheiman Bridge–Spy run	34.9	31.1	0.55	15.6	1974	0.00017	11
Walbash River, Walbash–Peru	26.9	10.3	0.30	41.1	4579	0.00026	11
Walbash River, Walbash–Peru	26.9	20.5	0.44	41.1	4579	0.00026	11
Walbash River, Walbash–Peru	26.9	41.1	0.67	41.1	4579	0.00026	11
Walbash River, Walbash–Peru	26.9	82.1	0.97	41.1	4579	0.00026	11
Walbash River,Peru–Logansport	27.0	16.2	0.29	64.8	6957	0.00036	11
Walbash River,Peru–Logansport	27.0	32.4	0.50	64.8	6957	0.00036	11
Walbash River,Peru–Logansport	27.0	64.8	0.66	64.8	6957	0.00036	11
Walbash River,Peru–Logansport	27.0	129.6	0.94	64.8	6957	0.00036	11
Walbash River,Logansport–Delphi	36.9	22.7	0.35	91.0	9788	0.00044	11
Walbash River,Logansport–Delphi	36.9	45.5	0.49	91.0	9788	0.00044	11
Walbash River,Logansport–Delphi	36.9	91.0	0.70	91.0	9788	0.00044	11
Walbash River,Logansport–Delphi	36.9	182.0	0.98	91.0	9788	0.00044	11
Walbash River, Delphi–Lafayette	30.4	23.5	0.42	94.1	10546	0.00013	11
Walbash River, Delphi–Lafayette	30.4	47.1	0.54	94.1	10546	0.00013	11
Walbash River, Delphi–Lafayette	30.4	94.1	0.71	94.1	10546	0.00013	11
Walbash River, Delphi–Lafayette	30.4	188.3	0.88	94.1	10546	0.00013	11
Walbash River, Lafayette–Covington	65.7	44.2	0.43	176.8	18822	0.00014	11
Walbash River, Lafayette–Covington	65.7	88.4	0.58	176.8	18822	0.00014	11
Walbash River, Lafayette–Covington	65.7	176.8	0.79	176.8	18822	0.00014	11
Walbash River, Lafayette–Covington	65.7	353.5	1.01	176.8	18822	0.00014	11
Walbash River, Covington–Montezuma	50.1	49.7	0.41	198.7	21259	0.00011	11
Walbash River, Covington–Montezuma	50.1	99.3	0.51	198.7	21259	0.00011	11
Walbash River, Covington–Montezuma	50.1	198.7	0.65	198.7	21259	0.00011	11
Walbash River, Covington–Montezuma	50.1	397.3	0.82	198.7	21259	0.00011	11
Walbash River, Montezuma–Terre Haute	41.2	65.8	0.39	263.0	28796	0.00017	11
Walbash River, Montezuma–Terre Haute	41.2	131.5	0.52	263.0	28796	0.00017	11
Walbash River, Montezuma–Terre Haute	41.2	263.0	0.70	263.0	28796	0.00017	11
Walbash River, Montezuma–Terre Haute	41.2	526.0	0.92	263.0	28796	0.00017	11
Walbash River, Terre Haute–Riverton	81.9	72.4	0.36	289.7	31766	0.00012	11
Walbash River, Terre Haute–Riverton	81.9	144.8	0.51	289.7	31766	0.00012	11
Walbash River, Terre Haute–Riverton	81.9	289.7	0.70	289.7	31766	0.00012	11
Walbash River, Terre Haute–Riverton	81.9	579.4	0.99	289.7	31766	0.00012	11
White River, USGS gage at Indianapolis–Waverly	33.8	9.5	0.29	38.0	4235	0.00043	11
White River, USGS gage at Indianapolis–Waverly	33.8	19.0	0.42	38.0	4235	0.00043	11
White River, USGS gage at Indianapolis–Waverly	33.8	38.0	0.59	38.0	4235	0.00043	11
White River, USGS gage at Indianapolis–Waverly	33.8	76.0	0.81	38.0	4235	0.00043	11
White River, Waverly–Paragon	49.1	15.9	0.20	63.4	6330	0.0004	11
White River, Waverly–Paragon	49.1	31.7	0.30	63.4	6330	0.0004	11

Table A–2. Data from studies yielding traveltime only—Continued

River, begin–end	Km	Q	Vp	Slope	Qave	Da Ar	Ref
White River, Waverly–Paragon	49.1	63.4	0.42	63.4	6330	0.0004	11
White River, Waverly–Paragon	49.1	126.8	0.63	63.4	6330	0.0004	11
White River, Paragon–Worthington	74.8	21.0	0.40	83.8	7739	0.00026	11
White River, Paragon–Worthington	74.8	41.9	0.52	83.8	7739	0.00026	11
White River, Paragon–Worthington	74.8	83.8	0.70	83.8	7739	0.00026	11
White River, Paragon–Worthington	74.8	167.7	0.97	83.8	7739	0.00026	11
White River, Worthington–Petersburg	142.7	31.8	0.46	127.0	12090	0.0002	11
White River, Worthington–Petersburg	142.7	63.5	0.62	127.0	12090	0.0002	11
White River, Worthington–Petersburg	142.7	127.0	0.84	127.0	12090	0.0002	11
White River, Worthington–Petersburg	142.7	254.0	1.14	127.0	12090	0.0002	11
East Fork White River, Columbus–Azalia	19.2	12.5	0.31	50.0	4421	0.00045	11
East Fork White River, Columbus–Azalia	19.2	25.0	0.44	50.0	4421	0.00045	11
East Fork White River, Columbus–Azalia	19.2	50.0	0.55	50.0	4421	0.00045	11
East Fork White River, Columbus–Azalia	19.2	100.0	0.74	50.0	4421	0.00045	11
East Fork White River, Azealia–Williams Bridge	164.3	25.0	0.29	99.9	10000	0.0002	11
East Fork White River, Azealia–Williams Bridge	164.3	50.0	0.42	99.9	10000	0.0002	11
East Fork White River, Azealia–Williams Bridge	164.3	99.9	0.60	99.9	10000	0.0002	11
East Fork White River, Azealia–Williams Bridge	164.3	199.9	0.85	99.9	10000	0.0002	11
East Fork White River, Williams Bridge–SR 57	123.1	37.3	0.31	149.3	12761	0.00014	11
East Fork White River, Williams Bridge–SR 57	123.1	74.7	0.44	149.3	12761	0.00014	11
East Fork White River, Williams Bridge–SR 57	123.1	149.3	0.61	149.3	12761	0.00014	11
East Fork White River, Williams Bridge–SR 57	123.1	298.6	0.87	149.3	12761	0.00014	11
White River, Petersburg–Hazelton	44.4	79.2	0.45	316.9	28814	0.00015	11
White River, Petersburg–Hazelton	44.4	158.4	0.56	316.9	28814	0.00015	11
White River, Petersburg–Hazelton	44.4	316.9	0.70	316.9	28814	0.00015	11
White River, Petersburg–Hazelton	44.4	633.7	0.87	316.9	28814	0.00015	11
Maumee River, Columbia Ave (Ft Wayne)–state line	44.1	10.5	0.23	41.9	5095	0.00026	11
Maumee River, Columbia Ave (Ft Wayne)–state line	44.1	20.9	0.35	41.9	5095	0.00026	11
Maumee River, Columbia Ave (Ft Wayne)–state line	44.1	41.9	0.48	41.9	5095	0.00026	11
Maumee River, Columbia Ave (Ft Wayne)–state line	44.1	83.7	0.71	41.9	5095	0.00026	11

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APPENDIX B. OTHER DATA AVAILABLE

Limitations on time prevented the analysis of all available data. The following is a list of references to other time-of-travel studies conducted in the United States that were not included in this analysis.

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APPENDIX C. SYMBOLS

a	exponent in the relation of mean stream velocity to discharge
β	exponent on the unit-peak concentration vs. time relation
C	average, discharge weighted, cross-sectional tracer concentration
C_p	peak concentration of the tracer cloud
C_u	unit concentration (units of inverse time)
C_v	vertically averaged tracer concentration
C_{up}	unit-peak concentration
CV	coefficient of variation
D_a	drainage area of the river at the point of measurement
D'_a	dimensionless drainage area based on mean annual discharge
D''_a	dimensionless drainage area based on local discharge
g	acceleration of gravity
K	constant in the relation of mean stream velocity to discharge
k	decay coefficient describing loss of pollutant with time
M_i	mass of tracer injected
M_{ia}	apparent mass of tracer injected that accounts for losses
M_r	mass of tracer to pass a cross section
M^m	mass of tracer injected at time m
n	number of sampling site downstream of injection
N	number of points upon which the regression is based
Q	total discharge at the cross section at time t
Q_a	mean annual flow at the section
Q'_a	dimensionless mean discharge
q	unit discharge
RMS	the root-mean-square error of the regression equation
R^2	the r squared value
R_r	tracer recovery ratio
S	reach slope
T_d	duration of the tracer cloud (T_t-T_l)
T_l	elapsed time to the arrival of the leading edge of a tracer cloud at a sampling location
T_p	elapsed time to the peak concentration of the tracer cloud
T_{pm}	minimum probable elapsed time to the peak concentration of the tracer cloud
T_t	elapsed time to the trailing edge of the tracer cloud
T_{10d}	duration from leading edge until tracer concentration has reduced to within 10 percent of the peak concentration
T_{10t}	elapsed time until the trailing edge of the tracer cloud has been reduced to 10 percent of the peak concentration
t	time since injection
V	mean stream velocity
V_p	velocity of the peak concentration
V'_p	dimensionless peak velocity
V_{mp}	probable maximum velocity
W	total width of the river