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Estimation of Watershed Lag Times and Times of Concentration for the Kansas City Area

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The regional lag-time equation was calibrated for the Kansas City area with observed lag times and channel and watershed characteristics for 30 gage sites in the Automated Local Evaluation in Real Time (ALERT) flash-flood warning system. Average lag times for the gaged watersheds were determined from archived rainfall and water-level records for significant runoff events. The new lag-time equation for the Kansas City area performs better than the 2001 KU-KDOT equation for urban lag time, which was developed from a study of 14 gaged watersheds in Johnson County (McEnroe & Zhao, 2001). The main advantage of the new lag-time equation is that it accounts directly for the higher velocities in the paved and enclosed segments of the flow path. The new equation's solid theoretical foundation is also an advantage.

Because time of concentration cannot be determined from gaging data, direct calibration of an equation for T_C is not possible. However, time of concentration and lag time are closely related, so T_C can be estimated from T_L. The Natural Resources Conservation System (NRCS, 2010) recommends the approximation $T_C = 5/3\ T_L$. The recommended time-of-concentration equation for the Kansas City area incorporates this approximation.

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Abstract

Lag time (T_L) and time of concentration (T_C) are two related measures of how quickly a stream responds to runoff-producing rainfall over its watershed. In this report, a general relationship for lag time is derived from the Manning equation for frictional resistance, the rational formula for bankfull discharge, and a generalized rainfall intensity-duration relationship. This relationship accounts for the length, average slope, and average Manning n value for the longest flow path and three watershed characteristics: average width, a rational runoff coefficient, and a reference rainfall intensity. A simpler form of this relationship is derived for regional calibration. The regional form of the lag-time equation accounts for the impacts of urbanization through two inputs: the fraction of the longest flow path that is paved or enclosed, and the fraction of the drainage area covered by impervious surfaces. The relationships indicate that lag time is strongly related to the length, slope, and roughness of the main channel and less strongly related to other watershed characteristics that affect the bankfull discharge in the channel.

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Because time of concentration cannot be determined from gaging data, direct calibration of an equation for T_C is not possible. However, time of concentration and lag time are closely related, so T_C can be estimated from T_L. The Natural Resources Conservation System (NRCS, 2010) recommends the approximation $T_C = 5/3\ T_L$. The recommended time-of-concentration equation for the Kansas City area incorporates this approximation.

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Chapter 1: Introduction

1.1 Lag Time and Time of Concentration

The peak discharge resulting from runoff-producing rainfall depends on how quickly the runoff reaches the watershed outlet. Hydrologic methods for calculation of peak discharge require some measure of the watershed's response time as an input. The two most common measures of hydrologic response time are time of concentration (T_C) and lag time (T_L). Lag time is needed for flood hydrograph calculations, and the time of concentration is needed to calculate peak flows by the rational method. The Natural Resources Conservation Service (NRCS) defines lag time as the time interval between the occurrence of a sudden burst of runoff-producing rainfall over the basin and the resulting peak at the basin outlet. Time of concentration (T_C) is defined as the time required for runoff to travel from the most remote point in the basin to the basin outlet (NRCS, 2010).

A watershed's lag time and time of concentration are closely related to the length, slope, and roughness of the longest flow path. Urban watersheds have much shorter lag times and times of concentration than rural watersheds due to the lower frictional resistance of the urban infrastructure (curb-and-gutter streets, storm sewers, etc.). The lag time of a gaged watershed can be determined from an analysis of precipitation and water-level records if the time intervals between data values are sufficiently short. However, time of concentration cannot be determined from gaging data or measured in the field. Time of concentration can be estimated with hydraulic calculations, but the estimate may be unreliable due to idealized representation of irregular field conditions and large uncertainties in Manning roughness coefficients and other inputs. Alternatively, T_c can be estimated from lag time.

According to the NRCS, in an average natural watershed with an approximately uniform distribution of runoff, lag time and time of concentration are related by:

$$T_L = 0.6\ T_C$$

(NRCS, 2010) **Equation 1.1**

Although its origin is not well documented and its generality is uncertain, this relationship has been widely accepted in engineering practice for several decades. In this report, we assume Equation 1.1 is valid as a reasonable approximation and apply this relationship to estimate time of concentration from lag time.

1.2 Previous Work by Others

A recent paper by Gericke and Smithers (2014) provides a comprehensive review of methods for estimating lag time, time of concentration, and other time parameters. This paper catalogs 19 equations for T_C and 21 equations for T_L. Many of these equations are applicable only within a fairly narrow range of conditions. Most do not account for the effects of urban development, and many use non-standard definitions for T_L and T_C.

Methods for estimating T_L and T_C can be classified as hydraulic or empirical. Hydraulic methods include velocity methods and kinematic wave methods. Velocity methods use hydraulic relationships to calculate T_C for steady flow at system capacity and typically set T_L equal to $0.6 \cdot T_C$. Kinematic-wave methods generally define T_C as the time to equilibrium rather than the travel time at steady state. The time to equilibrium is the time needed for excess rainfall (rainfall minus losses) of a specified constant intensity to produce a constant peak discharge at the watershed outlet, starting from a dry surface.

Empirical methods relate lag time or time of concentration to watershed characteristics by fitting an equation of specified form to a set of data by regression analysis. Empirical equations generally provide estimates of the total lag time or time of concentration to the point of interest, accounting for both overland flow and channel flow. The best empirical equations incorporate some degree of physical reasoning. Empirical equations with forms derived from hydrologic or hydraulic principles are termed semi-analytical equations. Empirical formulas fitted to data from a particular geographic region might not work as well outside that region due to different geomorphological and climatological characteristics.

1.3 Local Practices

Most local governments in the Kansas City area require that storm drainage facilities be designed in accordance with the Section 5600 design guidance of the Kansas City Metro Chapter of the American Public Works Association (KC-APWA, 2011). In Section 5600, the time of concentration is considered the sum of “the overland flow time to the most upstream inlet or other point of entry to the system, Inlet Time, plus the time for flow in the system to travel to the point under consideration, Travel Time,” and lag time is approximated as $0.6 \cdot T_C$. The inlet time is computed by the Federal Aviation Administration (FAA) equation for sheet flow (FAA, 1970). The FAA equation generally overestimates inlet times because much of the flow to the inlet is typically shallow concentrated flow rather than sheet flow.

The Kansas Department of Transportation provides regression equations for estimating T_L and T_C for rural and urban watersheds in Kansas. The equations for rural watersheds relate T_L and T_C to the length and average slope of the main channel. The equations for urban and developing watersheds also account for the fraction of the drainage area that is impervious. The T_L equation for rural watersheds was derived from an analysis of USGS gaging data for 19 rural watersheds in Kansas with drainage areas from 2 to 14 mi^2 (McEnroe & Zhao, 1999). The T_L equation for urban and developing watersheds was derived from an analysis of ALERT-system data through 1999 for 14 sites in Johnson County with drainage areas from 170 acres to 28 mi^2 (McEnroe & Zhao, 2001). The T_C equations follow from the approximation $T_C = 5/3\ T_L$.

1.4 Overview of This Work

In this report, the average lag times of 30 gaged watersheds in the Kansas City area are determined from an analysis of ALERT rainfall and stage data for large rainfall events. Relevant physical characteristics of the gaged watersheds are determined from geospatial datasets and aerial images. A semi-analytical approach is used to formulate a general equation for urban lag time that accounts for five important watershed characteristics. The new equation is calibrated for the Kansas City area with the data from the gaged watersheds.

Chapter 2: Lag Times and Physical Characteristics of Gaged Watersheds

2.1 Selection of Gaged Watersheds

Thirty ALERT gage sites in the Kansas City area were selected for study. Fifteen of these gages are located in Johnson County, Kansas; nine are in Clay County, Missouri; four are in Jackson County, Missouri; and two are in Platte County, Missouri. All of the selected sites have consistent and reliable rainfall depth and water-level records and are unaffected by upstream impoundments. On streams with multiple water-level gages, the gage site furthest upstream was selected (with one exception) so that the dataset would include as many small watersheds as possible. (The smaller the watershed, the better the gage rainfall approximates the watershed-average rainfall.) The selected watersheds span a wide range of sizes and land uses. The drainage areas range from 113 acres to 11.11 mi^2, and the percentages of impervious surface area range from 1% to 50%. Table 2.1 lists the selected gage sites and Figure 2.1 shows the locations of the gages and their watersheds.

2.2 Watershed Characteristics

The physical characteristics chosen to describe the selected watersheds are listed and defined in Table 2.2. In addition to drainage area, these characteristics include three measures of length, two measures of slope, an average width, a measure of imperviousness, and a measure of channel modification. The values of these characteristics for each site are listed in Table 2.1. All characteristics except channel development ratio were determined from digital geospatial data. The channel development ratios were determined from aerial photographs.

Johnson County's Automated Information Mapping System (AIMS) office provided data in ArcGIS format for the watersheds located in Kansas. These data consisted of aerial imagery, road centerlines, general land use, two-foot elevation contours, pavement edges, water bodies, stormwater drainage lines, building polygons, driveway centerlines, and a digital elevation model (DEM) with a cell size of 3 feet. All of the data were provided in the StatePlane coordinate system for Kansas North FIPS 1501 (US Feet).

Table 2.1: Selected Gage Sites and Watershed Characteristics (As Defined in Table 2.2)

ID	ALERT Site Name	State	Area (ac)	L (ft)	W (ft)	S (ft/ft)	S_{10-85} (ft/ft)	R_c	R_i
1140	143rd @ Indian Creek	KS	1,554	17,663	3,833	0.0053	0.0054	0.356	0.339
1400	Waterford (N. Br. Indian Cr.)	KS	3,415	21,542	6,906	0.0081	0.0076	0.429	0.427
1450	I-435 @ Quivira	KS	678	11,702	2,524	0.0135	0.0151	0.621	0.480
1650	Pflumm @ Tomahawk Creek	KS	1,010	10,188	4,318	0.0094	0.0086	0.313	0.255
1680	Wilshire Woods	KS	170	4,697	1,572	0.0178	0.0191	0.652	0.326
2090	191st St. @ E. Wolf Cr.	KS	3,864	26,707	6,303	0.0062	0.0052	0.004	0.039
2220	Lackman @ Wolf Cr.	KS	5,004	32,793	6,646	0.0039	0.0029	0.007	0.020
2540	96th & Brighton E. Fork Shoal Cr.	MO	2,156	22,376	4,197	0.0090	0.0064	0.027	0.098
2600	NE 112th Ter @ Rocky Branch Cr.	MO	182	5,367	1,475	0.0149	0.0157	0.581	0.234
2640	NE Vivion Rd @ Rock Creek	MO	412	7,342	2,446	0.0193	0.0188	0.560	0.307
2700	Hickman Mills Dr & I-470	MO	801	8,823	3,957	0.0160	0.0170	0.193	0.339
2720	Elm Rd @ White Oak Creek Trib.	MO	530	9,857	2,344	0.0134	0.0142	0.759	0.241
2730	E 83rd St @ White Oak Creek	MO	511	8,919	2,496	0.0153	0.0147	0.640	0.238
3020	69th @ Quail Cr. Tr. to Turkey Cr.	KS	660	10,523	2,732	0.0135	0.0135	0.621	0.311
3160	79th St @ Little Mill Creek	KS	2,806	16,796	7,276	0.0082	0.0106	0.282	0.372
3170	Woodland @ Clear Creek	KS	7,108	57,155	5,417	0.0049	0.0039	0.021	0.138
3250	119th @ Little Cedar Creek	KS	7,093	42,251	7,313	0.0055	0.0052	0.087	0.195
3310	143rd @ Kill Creek	KS	4,455	40,910	4,744	0.0040	0.0038	0.068	0.062
3350	151st @ Spoon Creek	KS	3,384	33,349	4,420	0.0045	0.0037	0.002	0.012
3660	Skyview @ 2nd Creek Trib.	MO	1,891	29,011	2,839	0.0055	0.0039	0.101	0.183
3690	Summit @ First Creek	MO	711	10,441	2,966	0.0066	0.0072	0.107	0.209
3720	Hwy 152 @ Upper Shoal Creek	MO	946	13,498	3,052	0.0097	0.0128	0.169	0.308
3840	NW Waukomis @ Old Maids Cr.	MO	1,038	21,514	2,101	0.0114	0.0091	0.141	0.296
3900	NW Vivion @ East Creek	MO	1,221	21,892	2,429	0.0118	0.0123	0.196	0.285
3940	N Jackson Dr @ Rock Creek	MO	1,769	23,384	3,296	0.0083	0.0060	0.144	0.318
3980	NE 79th @ E. Fork Little Shoal Cr.	MO	1,389	21,466	2,819	0.0111	0.0099	0.219	0.290
4080	Blue Ridge Cutoff @ Rnd. Gr. Cr.	MO	2,358	18,736	5,482	0.0127	0.0130	0.289	0.256
4150	NW 80th @ Walnut Creek	MO	113	5,445	908	0.0149	0.0131	0.417	0.496
5050	Lee Blvd @ Dykes Branch	KS	1,951	17,110	4,968	0.0117	0.0099	0.530	0.337
5700	Martway @ Rock Creek	KS	1,419	14,172	4,362	0.0123	0.0119	0.736	0.414

Figure 2.1: Locations of Selected Gage Sites and Watersheds

The City of Kansas City, Missouri, provided GIS data for most of the watersheds in Missouri. These data included road centerlines, stormwater drainage lines, inlets, impervious surfaces polygons, 2-foot elevation contours, water bodies, outlets, and manhole and outfall point locations. Kansas City also provided DEMs with 1-meter cell size for Clay and Platte Counties, and LiDAR points of bare-earth terrain for Jackson County, which we converted to raster data with the same resolution as the other DEMs. The cities of Raytown and Gladstone provided stormwater drainage maps. The coordinate system for all Missouri data was StatePlane Missouri West FIPS 2403 (US Feet). Information on impervious surfaces was not provided for areas in

Missouri outside the City of Kansas City. In these areas, we estimated imperviousness from a 30-m National Land Cover Database (NLCD) 2011 Percent Developed Imperviousness layer.

Table 2.2: Watershed Characteristics Considered in this Study

Symbol	Definition
A	drainage area (acres) at the watershed outlet
L	length of longest flow path (ft), measured from a point on the drainage divide to the watershed outlet
S	average slope of longest flow path (ft/ft), defined as the total fall in elevation divided by the flow-path length
$S_{10\text{-}85}$	average slope of longest flow path (ft/ft), computed between two points located at 10% and 85% of the path length (measured from the watershed outlet)
W	average width of the watershed (ft), defined as A/L
R_c	channel development ratio (dimensionless), defined as the fraction of the longest flow path with low frictional resistance (e.g., pavements, gutters, enclosed conduits, and channels with paved bottoms)
R_i	impervious area ratio, defined as the fraction of the drainage area covered by impervious surfaces

Geospatial data were processed in ESRI® ArcMap version 10.1 and the corresponding version of Arc Hydro Data Model and Tools. First, any vector data containing information on the flow paths for streams (whether natural or from the stormwater drainage network) upstream of the gages were used to recondition the DEMs and "burn" channels onto them. This was done to force runoff to flow through enclosed conduits rather than follow the downward gradient of the ground surface. To delineate the watersheds, we used the following tools from Arc Hydro: "Fill sinks" to eliminate depression in the DEMs, "Flow Direction," "Flow Accumulation," "Stream Definition" with the default inputs (threshold depending on size of DEM), "Stream Segmentation," "Catchment Grid Delineation," "Catchment Polygon Processing," "Drainage Line Processing," "Adjoint Catchment Processing," and "Point Delineation." We used the "Longest Flow Path" and "Flow Path Parameters from 2D Line" tools to delineate the longest flow path and calculate its slope. Other watershed characteristics were computed with basic ArcToolbox functions.

2.3 Selection of Rainfall-Runoff Events

The City of Overland Park provided the archived rainfall and water-level records for the selected gages. Rainfall records from the ALERT tipping-bucket gages show the exact times corresponding to 1-mm increments of rainfall. Water-level records show the exact times of occurrence of discrete water levels at 0.05-ft intervals.

Rainfall-runoff events were discarded if the rise in stage could not be explained by the recorded rainfall. Events with long durations and low intensities were dropped because reliable lag-time estimates could not be obtained for these events. Short, intense events with a single peak stage were preferred. Figure 2.2 shows the rainfall and water-level record for one such event. A total of 220 individual events at the 30 gages sites were selected for analysis.

2.4 Determination of Lag Times for Selected Events

Lag times for individual events were determined by calibration of a hydrologic simulation model for the event. The simulations were performed with the HEC-HMS Hydrologic Modeling System, Version 3.5, of the U.S. Army Corps of Engineers (Scharffenberg & Fleming, 2010). Each event model was calibrated by adjusting the lag time so that the simulated peak discharge occurred at the same time as the observed peak stage. Figure 2.3 shows a plot of the simulation results for a successfully calibrated event model. This plot displays the observed rainfall hyetograph (inverted, in blue) and the simulated excess-rainfall hyetograph (inverted, in red), the observed stage record (yellow), and the simulated discharge hydrograph (blue).

In the HEC-HMS event models, the watershed was treated as a single basin. The rainfall recorded at the gage was applied uniformly over the entire watershed. The rainfall records were input as cumulative depths at one-minute intervals. Incremental runoff depths were computed by the NRCS curve-number method. In accordance with the KC-APWA (2011) Section 5600 guidance, the curve number for the pervious areas was set to 74, which is representative of grass-covered ground in good condition with soils in NRCS hydrologic group C (soils with moderately high runoff potential when thoroughly wet). The initial abstraction was set to one-fifth of the maximum potential retention, the NRCS-recommended default setting. HEC-HMS assumes that all rainfall on impervious surfaces becomes direct runoff with no losses. Direct runoff was transformed into discharge at the watershed outlet by the NRCS unit-hydrograph method. Base flow was set to zero in all cases.

2.5 Average Lag Times for Watersheds

The median lag time from the set of events for each watershed was calculated and considered to be the representative lag time for the watershed. The median was chosen as the representative lag time because it is a measure of central tendency that is not greatly influenced by outliers. Table 2.3 lists the median lag times for the 30 watersheds. The Appendix provides information on the individual events at each site. The event lag times showed considerable variation at some gage sites. The differences in lag times are largely attributable to differences in rainfall spatial patterns, storm movement across the watershed, and antecedent moisture conditions. Event lag times showed greater consistency on the smaller watersheds because the rainfall recorded at the gage was more representative of the average rainfall over the watershed.

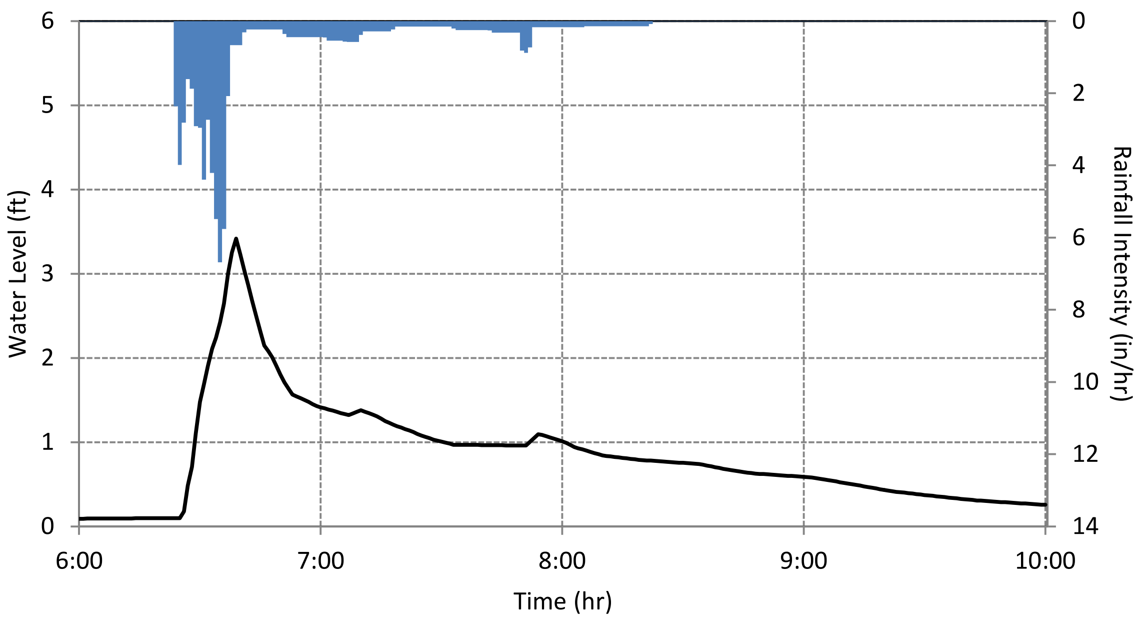

Figure 2.2: Sample Hyetograph and Hydrograph for Event on 8/26/2005 at Site 1680

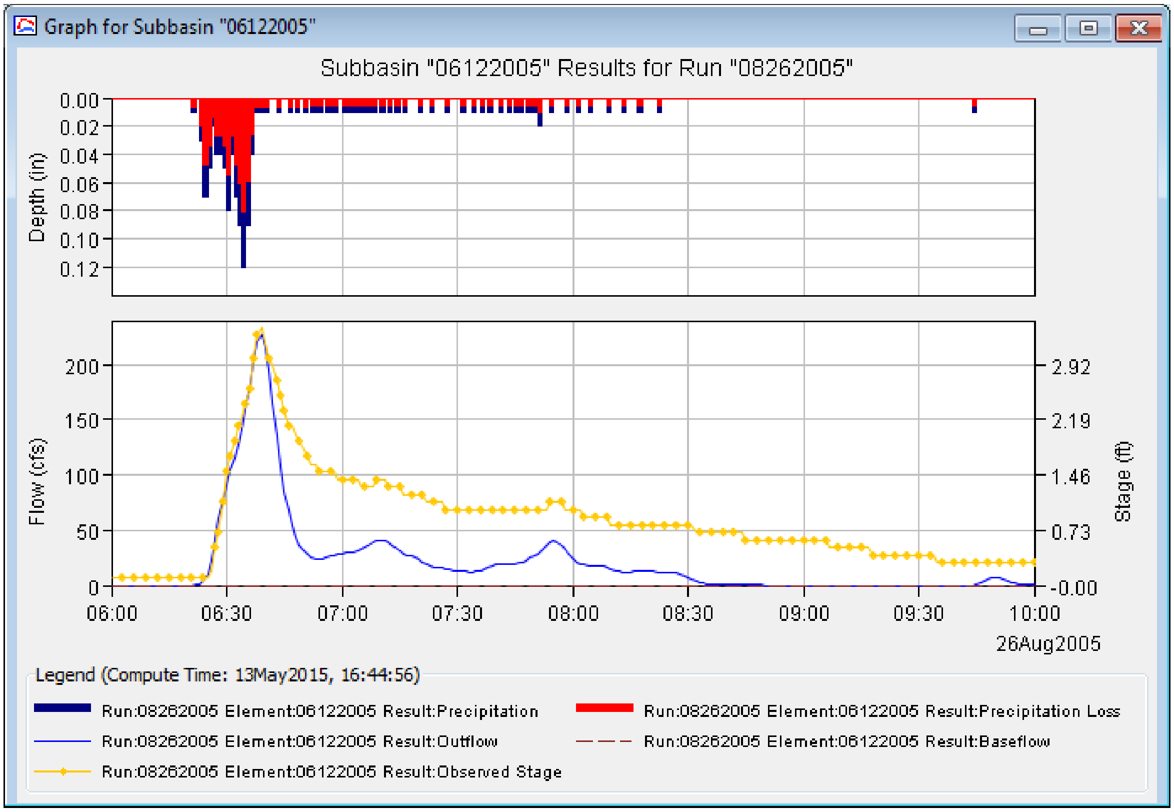

Figure 2.3: Results from Calibrated Simulation Model for Event on 8/26/2005 at Site 1680

Table 2.3: Median Lag Times for the Selected Watersheds

Site ID	Name	Lag time (minutes)
1140	143rd @ Indian Creek	32.5
1400	Waterford (N. Br. Indian Cr.)	41
1450	I-435 @ Quivira	18
1650	Pflumm @ Tomahawk Creek	26
1680	Wilshire Woods	6
2090	191st St. @ E. Wolf Cr.	73
2220	Lackman @ Wolf Cr.	152
2540	96th & Brighton East Fork Shoal Creek	57
2600	NE 112th Ter @ Rocky Branch Creek	15
2640	NE Vivion Rd @ Rock Creek	11
2700	Hickman Mills Dr & I-470	18
2720	Elm Rd @ White Oak Creek Trib	7
2730	E 83rd St @ White Oak Creek	10
3020	69th @ Quail Crk Trib to Turkey	13
3160	79th St @ Little Mill Creek	38
3170	Woodland @ Clear Creek	98
3250	119th @ Little Cedar Creek	72
3310	143rd @ Kill Creek	118.5
3350	151st @ Spoon Creek	139
3660	Skyview @ 2nd Creek Trib	103
3690	Summit @ First Creek	39
3720	Hwy 152 @ Upper Shoal Creek	43
3840	NW Waukomis @ Old Maids Creek	55.5
3900	NW Vivion @ East Creek	37
3940	N Jackson Dr @ Rock Creek	33
3980	NE 79th @ East Fork Little Shoal Creek	56.5
4080	Blue Ridge Cutoff @ Round Grove Creek	31
4150	NW 80th @ Walnut Creek	17
5050	Lee Blvd @ Dykes Branch	28
5700	Martway @ Rock Creek	12

Chapter 3: New Semi-Analytical Relationships for Lag Time

In this chapter, we develop some general semi-analytical relationships for watershed lag time. These relationships account for the physical characteristics of the main channel and watershed and local rainfall characteristics.

3.1 General Relationships

The approximation that lag time is a fixed fraction of the time of concentration, defined as the total time of flow along the longest flow path, is widely accepted in engineering practice (e.g., NRCS, 2010). It follows that lag time can be considered directly proportional to the length of the longest flow path and inversely proportional to a representative velocity on this flow path:

$$T_L \propto \frac{L}{V}$$ **Equation 3.1**

In which:

T_L = lag time

L = length of the longest flow path

V = representative velocity on the longest flow path

The objective is to relate lag time to the length, slope, and roughness of the longest flow path and watershed characteristics closely tied to bankfull discharge. This objective is accomplished by the following steps:

1. The Manning equation is used to express the velocity in terms of representative values of bankfull hydraulic radius, slope, and Manning resistance factor for the longest flow path.
2. Approximate geomorphic relationships are used to express the hydraulic radius in terms of a representative cross-sectional area.
3. The continuity equation for steady flow is used to express the cross-sectional area in terms of the bankfull discharge and velocity.
4. Approximate flood-frequency and rainfall-frequency relationships are used to express the bankfull discharge in terms of watershed and rainfall characteristics.

The Manning resistance equation relates the representative velocity to representative values of the channel's bankfull hydraulic radius, slope, and Manning resistance factor, n. The average slope of the longest flow path is selected as the representative slope, and the average n value on the longest flow path is selected as the representative n value:

$$V \propto \frac{R^{2/3} S^{1/2}}{\bar{n}} \qquad \textbf{Equation 3.2}$$

In which:

R = representative hydraulic radius

S = average slope of longest flow path

$\bar{n}$ = average Manning n value on longest flow path

Approximate geomorphic relationships can be used to express the hydraulic radius in terms of the cross-sectional area of the bankfull channel. Cross-sectional area and hydraulic radius both increase in the downstream direction as discharge increases. If the shape of the bankfull channel remained constant in the downstream direction, cross-sectional area would increase in proportion to the square of hydraulic radius. However, it is well known that a channel's width-to-depth ratio tends to increase in the downstream direction as discharge increases. The relative rates of increase in width and depth with discharge depend to some extent on geologic and climatic conditions. For the purposes of this study, average relationships will suffice. Leopold and Maddock (1953) showed that on average, for rivers in the Midwestern US, the width, W_b, and average depth, $\bar{y}$, of the bankfull cross-section vary with mean annual discharge, $\bar{Q}$, as follows:

$$W_b \propto \bar{Q}^{0.5} \qquad \textbf{Equation 3.3}$$

$$\bar{y} \propto \bar{Q}^{0.4} \qquad \textbf{Equation 3.4}$$

We accept Equations 3.3 and 3.4 as adequate approximations for our purposes and proceed accordingly. Average depth, $\bar{y}$, is defined as A_b/W_b, where A_b is the cross-sectional area of the bankfull channel. Because average depth and hydraulic radius are nearly equal in natural channels, the relationship between average depth and discharge should also apply to hydraulic radius as an adequate approximation:

$$R \propto \overline{Q}^{0.4} \qquad \textbf{Equation 3.5}$$

Because A_b equals $W_b \cdot \overline{y}$, it follows from Equations 3.3 and 3.4 that:

$$A_b \propto \overline{Q}^{0.9} \qquad \textbf{Equation 3.6}$$

Solving both Equations 3.5 and 3.6 for $\overline{Q}$ and equating the results leads to the desired relationship between A_b and R:

$$R \propto {A_b}^{4/9} \qquad \textbf{Equation 3.7}$$

Substituting ${A_b}^{4/9}$ for R in Equation 3.3 leads to:

$$V \propto \frac{{A_b}^{8/27} S^{1/2}}{\overline{n}} \qquad \textbf{Equation 3.8}$$

Making use of the continuity relationship for steady flow, Q/V can be substituted for A_b in Equation 3.8 and the resulting relationship can be solved for V to obtain:

$$V \propto \frac{Q^{8/35}\, S^{27/70}}{\overline{n}^{27/35}} \qquad \textbf{Equation 3.9}$$

This relationship for V can be inserted into Equation 3.1 to obtain:

$$T_L \propto \frac{L\, \overline{n}^{27/35}}{Q^{8/35}\, S^{27/70}} \qquad \textbf{Equation 3.10}$$

It is advantageous to select the bankfull or capacity discharge at the watershed outlet, Q_o, as the representative discharge. The rational formula relates this discharge to the drainage area, an appropriate rainfall intensity, and a coefficient that accounts for other relevant factors such as land use, soils, and climate:

$$Q_o = C\ i\ A \qquad \textbf{Equation 3.11}$$

In which:

C = rational runoff coefficient

i = rainfall intensity for appropriate duration and same annual exceedance probability as Q_o

A = watershed area

Young, McEnroe, and Rome (2009) and Young and McEnroe (2014) have shown that the rational formula is applicable to watersheds of the types and sizes considered in this study, provided the rational runoff coefficient is calibrated properly.

The rainfall intensity in the rational formula is usually averaged over a duration equal to the watershed's time of concentration, which can be approximated as five-thirds of the lag time. In urban hydrology, the time of concentration is generally 60 minutes or less. The annual exceedance probability (AEP) for bankfull or capacity flow is typically 50% or greater for natural channels and between 4% and 20% for engineered channels and enclosed conduits.

Figure 3.1 examines the relationship between rainfall intensity and duration for durations from 5 to 60 minutes and AEPs from 4% to 50% for downtown Kansas City. The data plotted in this figure were obtained from Atlas 14 of the National Oceanic and Atmospheric Administration (NOAA; Perica et al., 2013). Figure 3.2 examines how the 50%-chance rainfall intensity varies with duration for four U.S. cities with different hydroclimates. The dashed lines in these figures are equations of the form $i = c \cdot D^x$, in which D is duration and c and x are constants fitted to the Atlas 14 data by least-squares regression.

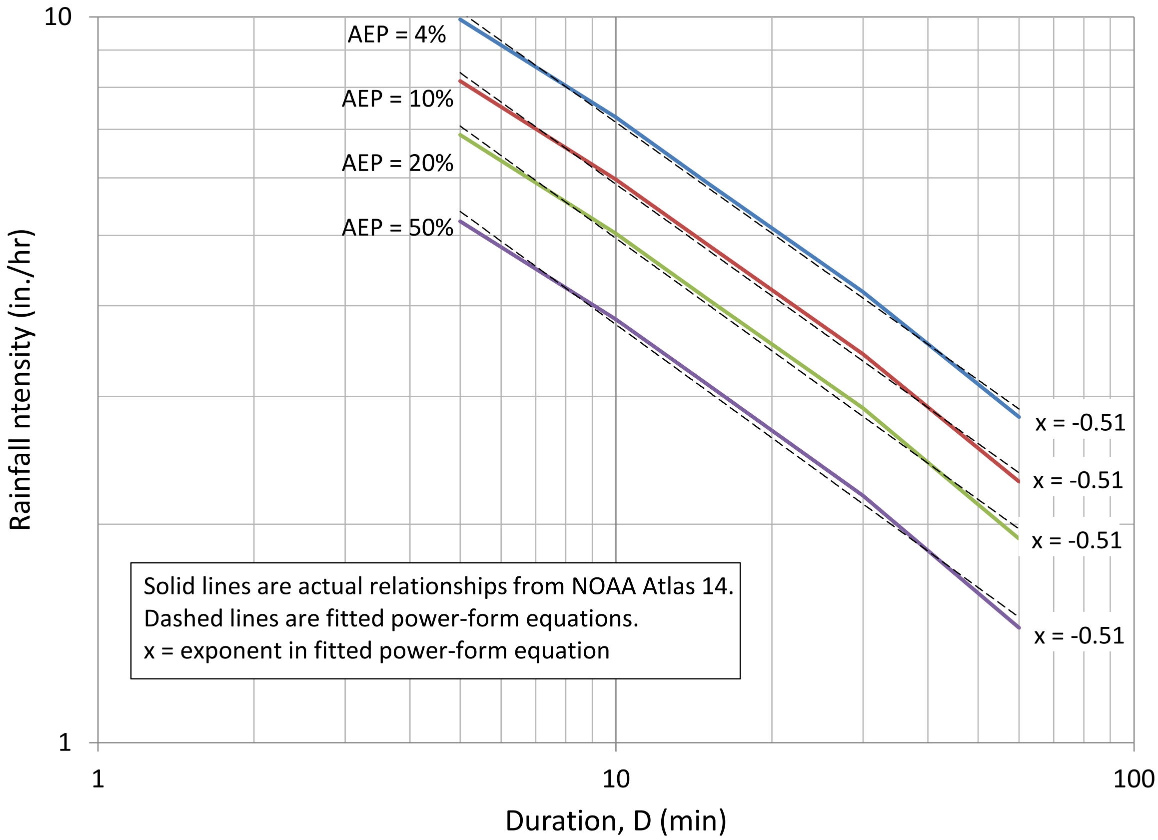

Figure 3.1: Rainfall Intensity-Duration Relationship for Kansas City

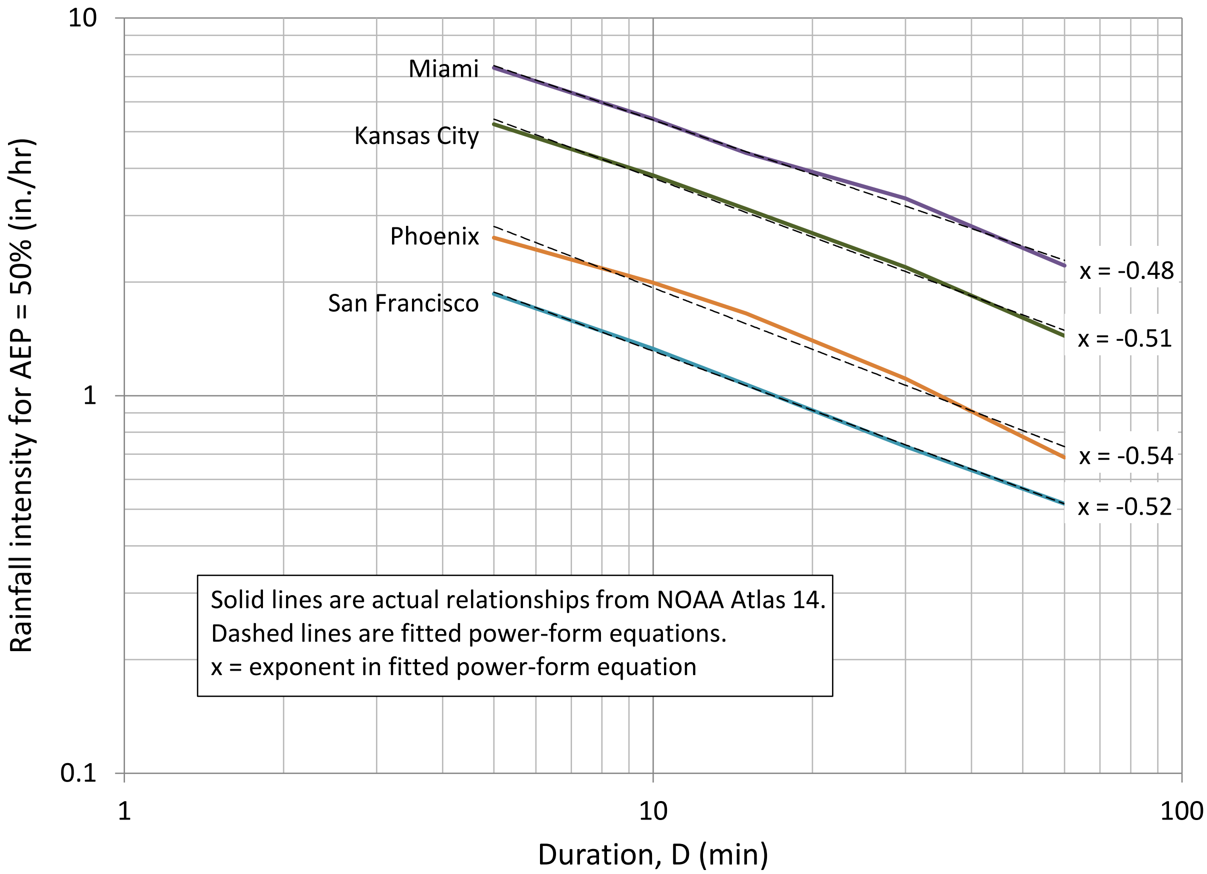

Figure 3.2: 50%-Chance Rainfall Intensity-Duration Relationships for Four Cities

The fitted power-form equations in Figures 3.1 and 3.2 approximate the i-D relationships reasonably well. These figures show the value of the exponent x for each fitted equation. Rounded to one decimal place, these exponents are all -0.5. Therefore, as a reasonable first approximation, we consider rainfall intensity for a given AEP to vary with the inverse square root of duration over the range of interest, i.e.:

$$i = i_o \left(\frac{D}{D_o}\right)^{-1/2} \qquad \textbf{Equation 3.12}$$

In which:

i_o is the rainfall intensity for reference duration D_o

Substituting this relationship for rainfall intensity in the rational formula, setting the rainfall duration equal to time of concentration, and approximating time of concentration as five-thirds of lag time leads to the following proportionality:

$$Q \propto \bar{C}\, i_o\, T_L^{-1/2}\, A \qquad \textbf{Equation 3.13}$$

In which:

$\bar{C}$ represents the average runoff coefficient for the watershed

The right-hand side of Equation 3.14 can be substituted for Q in Equation 3.10, and the resulting relation can be solved for T_L to obtain the result:

$$T_L \propto \left(\frac{L\,\bar{n}}{\sqrt{S}}\right)^{0.87} (\bar{C}\, i_o W)^{-0.26} \qquad \textbf{Equation 3.14}$$

In which:

W is the watershed's average width, defined as A/L

This proportionality can be written as an equation by inserting a calibration constant, k, on the right-hand side:

$$T_L = k\left(\frac{L\,\bar{n}}{\sqrt{S}}\right)^{0.87} (\bar{C}\, i_o W)^{-0.26} \qquad \textbf{Equation 3.15}$$

With sufficient reliable data from gaged watersheds, the numerical value of k could be determined. Properly calibrated, Equation 3.15 would be applicable anywhere.

It is interesting to note that, on the right-hand side of Equation 3.15, the variables in the first group are all channel characteristics and the variables in the second group are all watershed characteristics. The relative magnitudes of the exponents on the groups of terms indicate that lag time is much more sensitive to the channel characteristics than to the watershed characteristics.

3.2 Lag-Time Relationships for Regional Calibration

When applied to watersheds with similar rainfall, geology, and soils, the general lag-time equation can be simplified by incorporating certain quantities into the calibration constant.

However, if the general lag-time Equation 3.15 is to be applied to both urban and rural watersheds within the region, it must account for development-related differences in $\bar{n}$ and $\bar{C}$.

The average Manning n value can be considered a length-weighted average of typical Manning n values for the natural and paved/enclosed segments of the main channel:

$$\bar{n} = n_p R_c + n_n (1 - R_c) \qquad \textbf{Equation 3.16}$$

In which:

R_c is the fraction of the main-channel length that is paved or enclosed, and

n_n and n_p are representative n values for natural and paved/enclosed conditions.

This relationship can be written as:

$$\bar{n} = n_n(1 - \beta_c R_c) \qquad \textbf{Equation 3.17}$$

In which:

$$\beta_c = 1 - \frac{n_p}{n_n} \qquad \textbf{Equation 3.18}$$

The rational runoff coefficient, $\bar{C}$, depends on climate, soil characteristics, vegetation, and land use. Within a limited geographic region, land use is the most important factor. In a region with significant urbanization, the average runoff coefficient for a watershed depends largely on the fraction of the surface area that is impervious. In this case, $\bar{C}$ can be considered the area-weighted average of separate C values for the pervious and impervious parts of the watershed:

$$\bar{C} = C_i R_i + C_p(1 - R_i) \qquad \textbf{Equation 3.19}$$

In which:

C_p and C_i are the rational runoff coefficients for the pervious and impervious portions of the watershed, and

R_i is the fraction of watershed area that is impervious.

This relationship can also be expressed as:

$$\bar{C} = C_p(1 + \beta_i \cdot R_i) \qquad \textbf{Equation 3.20}$$

In which:

$$\beta_i = \frac{C_i}{C_p} - 1 \qquad \textbf{Equation 3.21}$$

R_i = fraction of watershed area that is impervious

β_i = a dimensionless coefficient, dependent on climate and soil conditions

Equations 3.17 and 3.20 can be incorporated into Equation 3.15 as follows:

$$T_L = k\left[\frac{L\, n_n(1 - \beta_c R_c)}{\sqrt{S}}\right]^{0.87} \left[i_o W C_p(1 + \beta_i R_i)\right]^{-0.26} \qquad \textbf{Equation 3.22}$$

When applied to watersheds with similar climatic and geologic characteristics, the lag-time equation can be simplified by incorporating the variables i_o, n_n, and C_p into the calibration constant, k:

$$T_L = k\left[\frac{L\,(1 - \beta_c R_c)}{\sqrt{S}}\right]^{0.87} [W\,(1 + \beta_i R_i)]^{-0.26} \qquad \textbf{Equation 3.23}$$

If the lag-time equation is to be applied only to rural watersheds, the terms $\beta_c R_c$ and $\beta_i R_i$ can be omitted:

$$T_L = k\left(\frac{L}{\sqrt{S}}\right)^{0.87} W^{-0.26} \qquad \textbf{Equation 3.24}$$

In summary, we recommend Equations 3.23 and 3.24 for calibration and use within a limited geographic region. If the region includes urban areas, Equation 3.23 should be used. If the watersheds of interest are all rural, Equation 3.24 can be used.

Chapter 4: Calibrated T_L and T_C Equations for the Kansas City Area

4.1 Calibration of the Regional Lag-Time Equation

The physical characteristics and calibrated lag times for the 30 gaged watersheds (Tables 2.1 and 2.3) were used to calibrate the general lag-time Equation 3.1 for the Kansas City area. In fitting this equation to the Kansas City data, the values of the constants β_c and β_i were assigned on the basis of local observations and judgment, and then the value of the coefficient k was determined by least-squares regression. Equation 3.18 relates β_c to the ratio n_p/n_n, and Equation 3.21 relates β_i to the ratio C_i/C_p. Our comparison of estimated Manning n values for the natural channels and the paved/enclosed components indicated that, on average, $n_p/n_n \approx 1/4$, which led us to set $\beta_c = 0.75$. In the KC-APWA (2011) Section 5600 design guidance, the recommended rational runoff coefficients for pervious and impervious surfaces are $C_p = 0.30$ and $C_i = 0.90$. These values, which are supported by our recent study of the Wilshire Woods watershed in Overland Park (McEnroe, Young, & Gamarra, 2015), led us to set $\beta_i = 2.0$.

A logarithmic transformation was applied to Equation 3.23 prior to calibration of the coefficient k to make the variance in error more uniform. The log-transformed equation, with $\beta_c = 0.75$ and $\beta_i = 2.0$, is:

$$\ln(T_L) = \ln(X) + \ln(k) \qquad \textbf{Equation 4.1}$$

In which:

$$X = \left[\frac{L\,(1 - 0.75\,R_c)}{\sqrt{S}}\right]^{0.87} [W\,(1 + 2.0\,R_i)]^{-0.26} \qquad \textbf{Equation 4.2}$$

We fit Equation 4.1 to Kansas City dataset by solving for the value of k that minimized the sum of the squares of the errors in $\ln(T_L)$. The best-fit value of k was found to be 0.0112. Therefore the calibrated lag-time equation for the Kansas City area is:

$$T_L = 0.0112\left[\frac{L\,(1 - 0.75\,R_c)}{\sqrt{S}}\right]^{0.87} [W\,(1 + 2.0\,R_i)]^{-0.26} \qquad \textbf{Equation 4.3}$$

Applying the NRCS-recommended approximation $T_C = 5/3\ T_L$, the recommended time-of-concentration equation for the Kansas City area is:

$$T_C = 0.0187 \left[\frac{L\,(1 - 0.75\,R_c)}{\sqrt{S}}\right]^{0.87} [W\,(1 + 2.0\,R_i)]^{-0.26} \qquad \textbf{Equation 4.4}$$

Equation 4.3 has a coefficient of determination (R^2) of 0.910 and a standard error of 0.269 in natural-log units or (+31%, -24%) in percentage terms. Figure 4.1 confirms the fit of the equation to the data. Table 4.1 compares the observed (median) lag times and the lag times estimated with Equation 4.3 for the 30 gage sites.

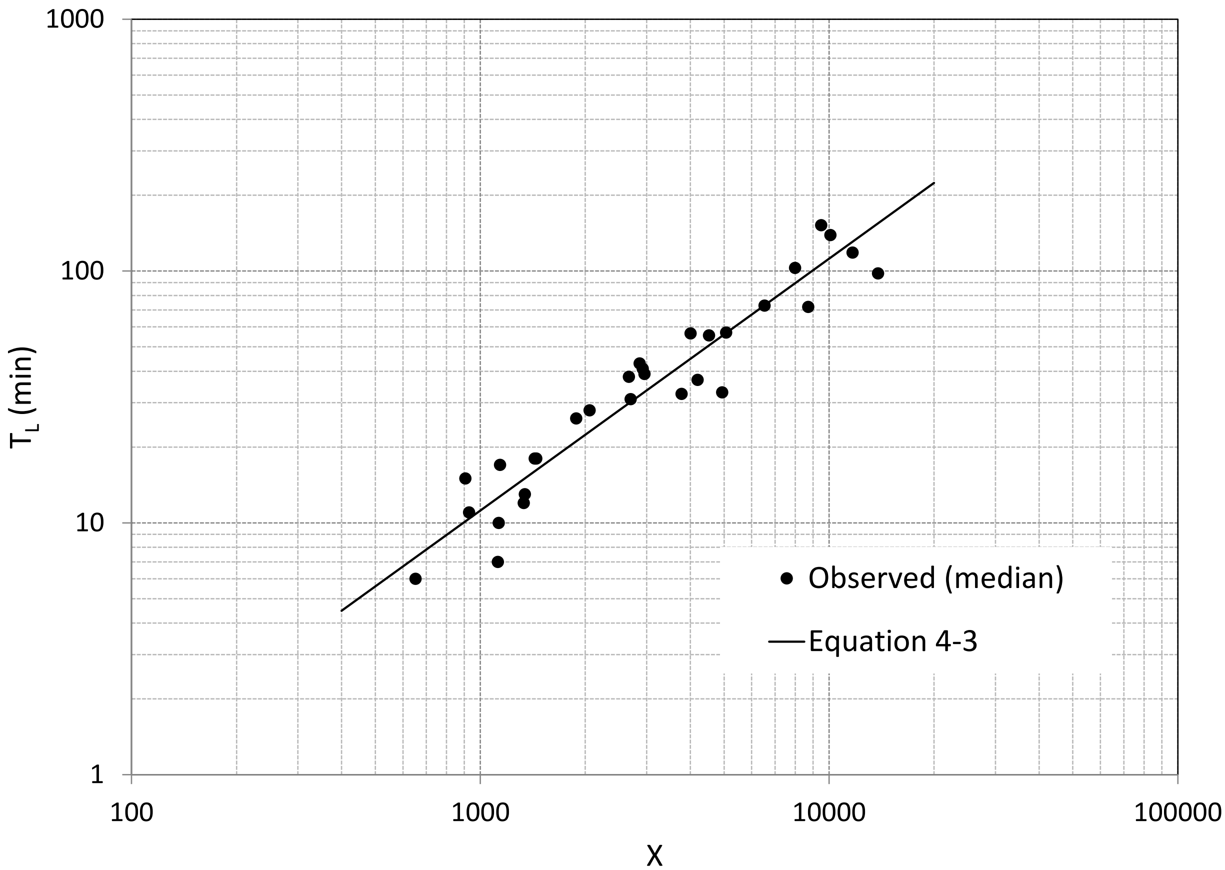

Figure 4.1: Equation 4.3 Fitted to Data for Gaged Watersheds in Kansas City Area

Table 4.1: Comparison of Observed Lag Times and Lag Times Estimated with Equation 4.3

	T_L in minutes	
Site ID	Observed (median)	Equation 4-3
1140	32.5	42
1400	41	33
1450	18	16
1650	26	21
1680	6	7
2090	73	73
2220	152	106
2540	57	57
2600	15	10
2640	11	10
2700	18	16
2720	7	13
2730	10	13
3020	13	15
3160	38	30
3170	98	155
3250	72	98
3310	118.5	131
3350	139	113
3660	103	90
3690	39	33
3720	43	32
3840	55.5	51
3900	37	47
3940	33	55
3980	56.5	45
4080	31	30
4150	17	13
5050	28	23
5700	12	15

4.2 Comparison with 2001 Lag-Time Equation for Johnson County

A 2001 study of the lag times of urban and developing watersheds in Johnson County (McEnroe & Zhao, 2001) yielded the following equation:

$$T_L = 0.0087\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.74} e^{-3.5R_i}$$ **Equation 4.5**

In which:

$S_{10\text{-}85}$ is the average slope between the 10% and 85% points on the longest flow path in ft/ft and all other variables are as defined previously.

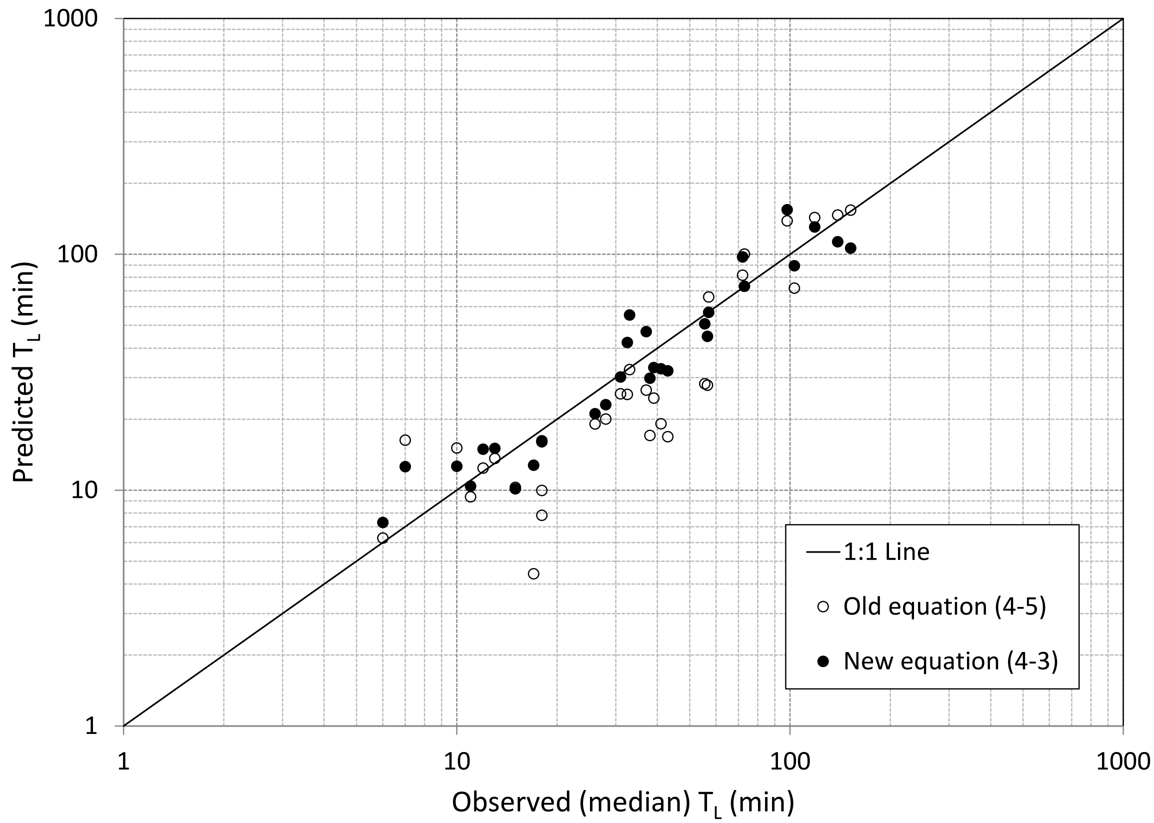

Figure 4.2: Comparison of Lag Times Estimated with Old and New Equations

Equation 4.5 was developed from an analysis of ALERT-system data through 1999 for 14 watersheds in Johnson County. In Figure 4.2, lag times estimated with Equations 4.3 and 4.5 are plotted versus the observed (median) lag time for the 30 watersheds. The new equation provides much better estimates of lag time. The main advantage of the new equation is that it accounts for the more rapid flow through the enclosed and concrete-lined sections of the main channel. The old equation accounts for the overall development within the watershed through the impervious area ratio, but it does not explicitly account for channel modifications.

4.3 Limitations of the Calibrated Equations for T_L and T_C

Equations 4.3 and 4.4 are applicable to watersheds with characteristics that do not vary greatly from those of the watersheds in our dataset. Table 4.2 shows the ranges of values for the five inputs to the new equations.

Because the new equations have a solid theoretical foundation, they should give reasonable results for watersheds that are smaller or more densely developed than those in our dataset. These equations should be applied only to watersheds that do not contain impoundments of a size that would alter the flood hydrograph to a significant degree. Impoundments increase the response time of a watershed; the new equations would tend to underestimate the lag times of watersheds with significant impoundments. Equations 4.3 and 4.4 do not account for the spatial distribution of development within the watershed. A watershed with development concentrated at the lower end would have a shorter lag time than the same watershed with development concentrated at the upper end. Reasonable judgement should be used when applying these equations to such watersheds. Because the values of the constants in Equations 4.3 and 4.4 incorporate local information on natural channel conditions, soil characteristics, and rainfall intensities, these calibrated equations may not perform as well in locations where geologic or climatic conditions differ significantly. Ideally, Equation 3.11 could be calibrated with a large dataset representing a wide range of geologic and climatic conditions. However, the data that would be needed are not readily available.

Table 4.2: Ranges of variables in dataset used to calibrate Equation 4.3

Variable	Range
L	0.9 – 11 miles
S	0.4% – 2%
W	0.2 – 1.4 miles
R_c	0 – 0.75
R_i	0.01 – 0.50

4.4 Recommendation for Local Governments in the Kansas City Area

We recommend that Equations 4.3 and 4.4 be incorporated into the Kansas City Metropolitan Chapter of the American Public Works Association (2011) Section 5600 design guidance as the preferred methods for estimation of lag time and time of concentration.

4.5 Recommendations for KDOT

KDOT's *Design Manual*, Volume 1 (Part C) Road Section (KDOT, 2011), presents three pairs of equations for lag time and time of concentration, the choice of which depends on the watershed's impervious area ratio, R_i. The three lag-time equations are:

$$T_L = 0.0221\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.66}$$ for $R_i \leq 0.03$ **Equation 4.6**

$$T_L = 0.0087\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.74} e^{-3.5R_i}$$ for $0.03 < R_i < 0.40$ **Equation 4.7**

$$T_L = 0.0021\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.74}$$ for $R_i \geq 0.40$ **Equation 4.8**

In which all variables are as defined previously.

The corresponding equations for time of concentration are:

$$T_C = 0.0368\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.66}$$ for $R_i \leq 0.03$ **Equation 4.9**

$$T_C = 0.0145\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.74} e^{-3.5R_i}$$ for $0.03 < R_i < 0.40$ **Equation 4.10**

$$T_C = 0.0036\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.74}$$ for $R_i \geq 0.40$ **Equation 4.11**

Equation 4.6 was derived from an analysis of USGS gaging data for 19 rural watersheds across Kansas (McEnroe and Zhao, 1999). Equation 4.7 (same as 4.5) is the equation derived from the 2001 analysis of ALERT-system data for 14 sites in Johnson County (McEnroe & Zhao, 2001). Because none of the watersheds in the 2001 study had an impervious area ratio over 0.40, the applicability of Equation 4.7 is capped at $R_i = 0.40$. Equation 4.8 is equivalent to Equation 4.7 with R_i set to 0.40.

We recommend that KDOT replace Equations 4.7 and 4.8 with the new equation for T_L and Equations 4.10 and 4.11 with Equation 4.4. The new equations for T_L and T_C should be applied to any watershed in which R_i or R_c exceeds 0.03:

$$T_L = 0.0112\left[\frac{L\,(1 - 0.75\,R_c)}{\sqrt{S}}\right]^{0.87}\,[W\,(1 + 2.0\,R_i)]^{-0.26} \quad \text{for } R_i > 0.03 \text{ or } R_c > 0.03$$

Equation 4.12

$$T_C = 0.0187\left[\frac{L\,(1 - 0.75\,R_c)}{\sqrt{S}}\right]^{0.87}\,[W\,(1 + 2.0\,R_i)]^{-0.26} \quad \text{for } R_i > 0.03 \text{ or } R_c > 0.03$$

Equation 4.13

4.6 Example Application of New Equations for T_L and T_C

In this example, Equations 4.3 and 4.4 are used to estimate the lag time and time of concentration for a watershed with the following characteristics:

- Drainage area = 711 acres, of which 149 acres is impervious
- Length of longest flow path = 10,440 feet, of which 1,120 feet is paved or enclosed
- Flowline elevation at watershed outlet = 865 feet
- Flowline elevation at upper end of longest flow path = 934 ft

The required inputs to Equations 4.3 and 4.4 are computed from these characteristics.

$$\mathrm{S} = \frac{934\ \mathrm{ft} - 865\ \mathrm{ft}}{10{,}440\ \mathrm{ft}} = 0.0066\ \mathrm{ft/ft}$$ **Equation 4.14**

$$\mathrm{W} = \frac{711\ \mathrm{ac}\ \times 43{,}560\ \mathrm{ft}^2/\mathrm{ac}}{10{,}440\ \mathrm{ft}} = 2967\ \mathrm{ft}$$ **Equation 4.15**

$$\mathrm{R_c} = \frac{1120\ \mathrm{ft}}{10{,}440\ \mathrm{ft}} = 0.107$$ **Equation 4.16**

$$\mathrm{R_i} = \frac{149\ \mathrm{ac}}{711\ \mathrm{ac}} = 0.210$$ **Equation 4.17**

These values are inserted into Equations 4.3 and 4.4 to obtain:

$$\mathrm{T_L} = 0.0112 \left[\frac{10{,}440\ (1 - 0.75 \times 0.107)}{\sqrt{0.0066}}\right]^{0.87} [2967\ (1 + 2.0 \times 0.210)]^{-0.26}$$
$$= 33\ \text{minutes}$$

Equation 4.18

$$\mathrm{T_C} = 0.0187 \left[\frac{10{,}440\ (1 - 0.75 \times 0.107)}{\sqrt{0.0066}}\right]^{0.87} [2967\ (1 + 2.0 \times 0.210)]^{-0.26}$$
$$= 55\ \text{minutes}$$

Equation 4.19

Chapter 5: Conclusions

From the work presented in this report, we draw these conclusions:

1. A watershed's lag time is strongly related to the length, slope, and roughness of the main channel and less strongly related to other watershed characteristics that affect the bankfull discharge in the channel.
2. A general relationship for lag time can be derived from the Manning equation for frictional resistance, the rational formula for bankfull discharge, and a generalized rainfall intensity-duration relationship. The inputs to the resulting equation are the length, average slope, and average Manning n for the longest flow path and three watershed characteristics: average width (defined as drainage area over channel length), a rational runoff coefficient, and a reference rainfall intensity.
3. When applied to watersheds with similar climatic and geologic characteristics, the general lag-time equation can be simplified by incorporating certain channel and watershed characteristics into the calibration coefficient. The regional form of the lag-time equation accounts for the impacts of urbanization through two inputs: a channel development ratio and an impervious area ratio. The channel development ratio is defined as the fraction of the longest flow path that is paved or enclosed. The impervious area ratio is the fraction of the drainage area covered by impervious surfaces.
4. The regional lag-time equation was effectively calibrated for the Kansas City area with observed lag times and channel and watershed characteristics for 30 gage sites in the ALERT system. Average lag times for the gaged watersheds were determined from archived rainfall and water-level records for significant runoff events.

5. The new lag-time equation for the Kansas City area performs much better than the 2001 equation developed from a study of 14 gaged watersheds in Johnson County (McEnroe & Zhao, 2001). The main advantage of the new lag-time equation is that it accounts directly for the higher velocities in the paved and enclosed segments of the flow path. The new equation's stronger theoretical foundation is another advantage.
6. Because time of concentration cannot be determined from gaging data, direct calibration of an equation for T_C is not possible. However, time of concentration and lag time are closely related, so T_C can be estimated from T_L. The NRCS recommends the approximation $T_C = 5/3\ T_L$. Equation 4.4, the recommended time-of-concentration equation for the Kansas City area, incorporates this approximation.

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Appendix: Calibration Results for Gaged Watersheds

Gage 1140				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	03/09/09	23	1.06	7.61
2	05/15/09	29	1.50	6.89
3	06/16/10	39	0.95	8.43
4	07/11/10[A]	30	0.79	8.39
5	09/23/10	45	0.83	5.88
6	05/21/11	43	0.67	7.50
7	06/19/11	31	1.38	9.49
8	08/19/11	28	1.65	7.62
9	05/31/13	34	2.80	13.01
10	10/02/14	38	0.83	4.60
Median		32.5		

[A] First event of the day.

Gage 1400				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/05/01	43	2.50	14.82
2	08/25/01	41	1.34	10.62
3	05/24/02	34	2.21	9.83
4	06/22/03	39	2.37	12.92
5	07/11/06	65	3.43	11.49
6	07/09/07	41	2.13	13.67
7	08/08/07	42	2.72	12.49
8	04/27/14	48	1.08	10.18
9	07/07/14	40	1.02	5.33
10	09/17/14	34	1.18	4.96
Median		41		

Gage 1450				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/05/08	22	1.42	6.80
2	06/10/09	18	1.62	7.76
3	07/12/09	19	0.71	5.98
4	07/11/10	18	1.05	8.92
5	07/20/10	18	1.38	7.39
6	05/31/13	31	1.73	6.96
7	07/07/14	15	1.06	3.87
8	08/07/14	15	1.38	3.80
Median		18		

Gage 1650				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/27/06	19	4.41	11.34
2	10/15/08	30	1.10	12.02
3	06/19/11	19	1.38	7.89
4	08/22/11	23	2.17	9.97
5	05/06/12	16	2.88	10.25
6	05/31/13	15	2.92	12.05
7	04/27/14	29	1.24	8.47
8	07/08/14	33	0.99	5.33
9	09/17/14	30	0.95	6.18
10	10/02/14	31	0.83	5.66
Median		26		

Gage 1680				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	10/04/98	6	0.71	3.47
2	06/05/01	6	1.69	4.49
3	08/26/05	5	1.35	3.45
4	07/11/06	9	4.1	5.93
5	08/27/06	6	3.94	4.37
6	06/03/08	6	1.26	3.12
7	07/29/08	6	6.70	4.77
8	06/14/10	6	2.40	4.15
9	05/31/13	10	2.68	5.9
10	06/15/13	4	2.64	4.86
Median		6		

Gage 2090				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/04/01	70	1.18	6.97
2	05/24/02	80	1.66	8.22
3	06/10/04	84	1.00	7.00
4	06/04/05	82	3.75	8.83
5	08/27/06	76	4.14	8.18
6	06/04/08	70	3.47	10.41
7	06/12/10	65	2.56	9.79
8	07/20/10	69	1.42	8.30
9	06/19/11	61	1.62	7.66
10	05/31/13	96	1.58	8.90
Median		73		

Gage 2220				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/26/99	150	1.32	9.48
2	06/04/01	195	1.42	10.88
3	03/04/04	125	4.62	10.47
4	05/19/04	152	3.00	10.81
5	08/24/04	232	2.49	10.32
6	06/04/05	173	2.92	10.27
7	06/03/08	104	3.83	12.05
Median		152		

Gage 2540				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	03/10/10	81	0.75	7.83
2	04/06/10	39	1.77	14.56
3	04/09/10	55	2.32	10.95
4	04/22/11	82	1.26	8.59
5	05/28/11	54	0.98	10.26
6	05/30/13	57	1.29	9.87
7	04/27/14[A]	85	0.51	10.13
Median		57		

[A] First event of the day.

Gage 2600				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/21/11	18	1.62	2.92
2	05/24/11[A]	14	0.91	1.85
3	05/24/11[B]	14	0.75	1.66
4	05/25/11	16	1.10	2.42
5	08/18/11	16	1.08	2.12
6	02/28/12	9	1.38	2.76
7	06/15/13	13	1.57	2.58
8	02/19/14[B]	15	0.81	2.13
9	04/27/14	17	1.05	2.52
Median		15		

[A] First event of the day. [B] Second event of the day

Gage 2640				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	07/06/11	12	1.42	3.04
2	08/18/11	10	1.57	3.28
3	09/19/13	11	2.11	3.42
Median		11		

Gage 2700				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/16/11	22	1.42	0.92
2	05/06/12[A]	14	1.20	0.79
3	05/31/13	18	1.85	1.10
Median		18		

[A] First event of the day.

Gage 2720				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/19/11	6	1.16	2.40
2	05/27/13	14	2.36	1.19
3	05/31/13	7	1.41	1.52
Median		7		

Gage 2730				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	05/25/11	9	0.98	2.58
2	08/20/11	11	1.50	2.97
3	05/06/12	10	1.15	2.16
4	04/08/13	13	0.61	2.40
5	05/31/13	14	1.20	3.27
6	07/03/13	7	1.20	2.02
7	09/18/13	9	2.09	2.01
Median		10		

Gage 3020				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/07/00	13	1.15	5.74
2	08/13/05	7	1.67	7.65
3	08/08/07	21	3.02	6.57
4	06/03/08	13	1.32	6.33
5	06/24/09	14	1.53	5.70
6	04/04/10	18	1.03	5.93
7	08/06/14	13	1.14	2.26
Median		13		

Gage 3160				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/09/07	48	2.96	8.78
2	07/29/08	30	2.57	7.80
3	09/12/08	33	2.35	8.84
4	06/10/09	39	1.62	8.20
5	06/13/10	44	3.39	9.94
6	07/28/10	38	3.68	7.83
7	07/07/14	34	1.18	3.39
8	08/06/14[A]	35	1.50	3.52
9	08/06/14[B]	39	0.95	3.68
Median		38		

[A] First event of the day. [B] Second event of the day.

Gage 3170				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	07/06/04	111	2.87	15.68
2	08/20/05	98	2.64	14.22
3	08/09/07	59	4.49	14.83
4	04/06/10	83	1.34	14.13
5	06/14/10	99	2.64	15.65
Median		98		

Gage 3250				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/20/05	72	3.25	11.60
2	06/11/09	63	1.67	11.06
3	06/15/09	72	1.20	11.35
4	06/14/10	63	2.46	13.19
5	06/19/11	68	1.14	10.96
6	08/22/11	88	1.22	10.89
7	05/06/12	115	1.30	11.10
8	05/31/13	72	1.72	11.64
Median		72		

Gage 3310				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/25/05	105	1.84	9.82
2	05/06/07	107	1.54	9.35
3	06/02/08	130	1.85	9.99
4	06/03/08	132	1.49	11.95
5	06/14/10	107	1.81	10.65
6	09/02/14	170	1.69	7.17
Median		118.5		

Gage 3350				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/20/05	139	3.23	9.23
2	08/26/05	109	1.38	9.74
3	05/06/07	123	1.93	9.22
4	06/02/08	155	2.48	10.88
5	06/04/08	162	1.46	11.33
6	04/06/10	139	1.34	9.79
7	05/25/11	121	1.26	9.14
8	05/31/13	163	2.09	11.18
Median		139		

Gage 3660				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/21/10[A]	104	0.55	6.01
2	04/22/11	103	1.18	6.94
3	05/25/11[A]	98	1.14	3.73
4	05/25/11[B]	82	0.75	4.78
5	08/19/11	87	1.26	5.44
6	08/20/11	99	1.30	6.04
7	02/28/12	115	0.87	4.57
8	05/27/13	110	1.81	6.27
9	05/30/13	144	0.98	5.63
10	06/28/13	116	0.91	4.85
11	09/19/13	94	2.13	5.50
12	10/04/13	90	1.30	5.76
13	06/12/14	154	0.87	5.56
Median		103		

[A] First event of the day. [B] Second event of the day.

Gage 3690				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/22/11	39	1.38	3.40
2	05/24/11[A]	43	0.83	2.07
3	06/26/11	43	1.65	2.23
4	12/19/11	42	1.22	2.09
5	02/28/12	32	1.50	3.58
6	06/15/13	37	1.34	2.87
7	04/27/14	33	1.30	3.18
Median		39		

[A] First event of the day.

Gage 3720				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/18/10	41	0.79	1.11
2	09/22/10[A]	57	0.87	0.79
3	09/23/10	43	0.83	1.07
Median		43		

Gage 3840				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	07/12/11	56	1.30	1.53
2	08/19/11	49	0.79	2.03
3	05/31/13	58	1.50	1.44
4	06/15/13	57	1.18	1.09
5	09/19/13	53	2.09	1.00
6	04/24/14	64	0.83	1.13
7	04/27/14	54	1.58	1.64
8	06/02/14	55	1.02	1.27
Median		55.5		

Gage 3900				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/01/10	37	2.40	3.81
2	07/12/11	40	1.65	4.07
3	08/19/11	35	1.69	4.34
4	02/28/12	41	1.30	3.98
5	05/31/13	35	1.89	5.23
6	06/15/13	44	1.50	4.11
7	04/24/14	45	1.38	3.86
8	04/27/14	33	1.58	3.87
9	06/01/14	37	1.30	3.51
Median		37		

Gage 3940				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	09/18/10	15	3.82	6.41
2	09/21/10	36	0.98	3.05
3	07/07/11	32	1.26	3.34
4	08/18/11	33	0.63	3.37
5	05/30/13	61	1.97	5.37
6	05/31/13	32	1.81	7.05
7	06/15/13	38	1.38	5.20
8	04/24/14	67	1.02	4.11
9	06/02/14	30	1.22	3.21
Median		33		

Gage 3980				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/22/11	61	1.22	0.75
2	09/19/13	52	3.23	1.43
Median		56.5		

Gage 4080				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	05/31/13	32	0.98	3.00
2	09/19/13	18	2.60	2.19
3	05/27/14	27	2.17	1.95
4	06/02/14	31	0.75	1.30
5	06/12/14	32	1.10	1.22
Median		31		

Gage 4150				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	04/14/11	23	1.14	1.14
2	04/22/11	18	1.34	2.91
3	05/20/11	21	0.63	1.02
4	05/24/11	17	0.67	1.67
5	05/25/11	17	0.83	1.97
6	08/18/11	17	1.58	1.00
7	06/27/13	12	1.58	1.00
Median		17		

Gage 5050				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	06/04/08[A]	20	1.46	7.70
2	06/04/08[B]	28	1.73	8.45
3	06/05/08	35	1.10	7.39
4	06/09/09	28	2.21	7.83
5	06/16/10	34	0.87	7.00
6	08/20/10	18	2.25	7.56
7	08/20/11	23	1.50	7.44
8	05/06/12	29	2.21	7.54
9	05/31/13	28	1.50	8.97
10	07/07/14	21	1.22	6.94
11	08/06/14	23	3.15	9.45
12	10/01/14	34	0.61	5.93
Median		28		

[A] First event of the day. [B] Second event of the day.

5700				
Event	Date	Lag Time (min.)	Depth (in)	Peak stage (ft)
1	08/13/05[C]	12	2.13	7.30
2	04/25/07	7	1.58	7.08
3	09/07/07	13	1.34	9.64
4	09/25/07	9	0.83	9.52
5	09/30/07	15	0.55	6.70
Median		12		

[C] Third event of the day.

Discharge is the product of cross-sectional area and average velocity, so Equation 3.2 can also be written as:

$$Q \propto \frac{A\,R^{2/3} S^{1/2}}{\bar{n}} \qquad \textbf{Equation 3.3}$$

In which:

Q is a representative discharge and

A is the cross-sectional area corresponding to the representative hydraulic radius R.

$$T_L = 0.0021\left(\frac{L}{\sqrt{S_{10\text{-}85}}}\right)^{0.74} \qquad \text{for } 0 \leq R_i \leq 0.4 \qquad \textbf{Equation 4.6}$$


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