ESTIMATING FLOOD HYDROGRAPHS FOR ARKANSAS STREAMS

By Braxtel L. Neely, Jr.

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 89-4109

Prepared in cooperation with the

ARKANSAS STATE HIGHWAY AND TRANSPORTATION DEPARTMENT

Little Rock, Arkansas

1989

DEPARTMENT OF THE INTERIOR

MANUEL LUJAN, JR., Secretary

U.S. GEOLOGICAL SURVEY

Dallas L. Peck, Director

For additional information write to:

District Chief
U.S. Geological Survey
Water Resources Division
2301 Federal Office Building
Little Rock, Arkansas 72201

Copies of this report can be purchased from:

U.S. Geological Survey
Books and Open-File Reports
Federal Center, Building 810
Box 25425
Denver, Colorado 80225

CONTENTS

ILLUSTRATIONS

TABLES

CONVERSION FACTORS

For use of readers who prefer to use metric (International System) units, rather than the inch-pound units used in this report, the following conversion factors may be used:

Multiply inch-pound unit	By	To obtain metric unit
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
inch (in.)	25.4	millimeter (mm)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)

ESTIMATING FLOOD HYDROGRAPHS FOR ARKANSAS STREAMS

By Braxtel L. Neely, Jr.

ABSTRACT

Flood hydrographs are needed for the design of many highway drainage structures and embankments and floodwater storage structures. A method for estimating these flood hydrographs at ungaged sites in Arkansas is presented in this report.

Dimensionless hydrographs are presented that can be used with equivalent lagtime and peak discharge to produce a typical hydrograph for streams in Arkansas with drainage areas less than 600 square miles. A hydrograph-width relation is presented for those instances when it is only necessary to know the period of time that a specific discharge will be exceeded.

Multiple regression analysis was used to define relations between equivalent lagtime and basin characteristics. Data collected on 450 storms at 49 gaging stations were used in the analysis. The regression analysis indicated that drainage area and 100-year discharge are significant parameters for estimating equivalent lagtime.

A method is presented for computing the volume of runoff for a flood when the peak discharge, equivalent lagtime, and drainage basin size are known.

page 3 follows

INTRODUCTION

The design of many highway structures such as bridges, culverts, and road embankments and floodwater storage structures on streams requires a determination of the flood hydrograph for the design flood event. At ungaged sites the hydrograph of a flood of a given peak discharge commonly needs to be estimated.

The Arkansas State Highway and Transportation Department recognized the need for adequate flood hydrograph data to more efficiently design drainage structures in Arkansas. Because of this need, the Arkansas State Highway and Transportation Department entered into a cooperative agreement with the U.S. Geological Survey to providé a method for estimating flood hydrographs.

The purpose of this report is to provide a method for estimating the flood hydrograph associated with a design discharge at ungaged sites on streams in Arkansas. Because lagtime, the time from the centroid of rainfall excess to the centroid of runoff, is needed in estimating the flood hydrographs, equations for estimating lagtime on streams in Arkansas with drainage areas less than 600 square miles (mi^2) were developed using multiple regression techniques.

The flood hydrograph data used in this study were collected over the period of record at 49 stream-gaging stations in rural basins in Arkansas (fig. 1 and table 1) with drainage areas ranging from 0.1 to 576 mi^2. The flood data were collected by the U.S. Geological Survey on streams that were free of substantial regulation.

A summary of the distribution of data among drainage area size classes follows:

Drainage area, in square miles	Number of stations
Less than 1	9
1 to 5	4
5 to 10	1
10 to 50	3
50 to 100	7
100 to 300	13
300 to 576	12

The gaging stations used in the study are listed in table 1 with data on drainage area, precipitation, slope, length, 100-year discharge, peak discharge, and equivalent lagtime for each station. Lagtime also is shown for 17 stations.

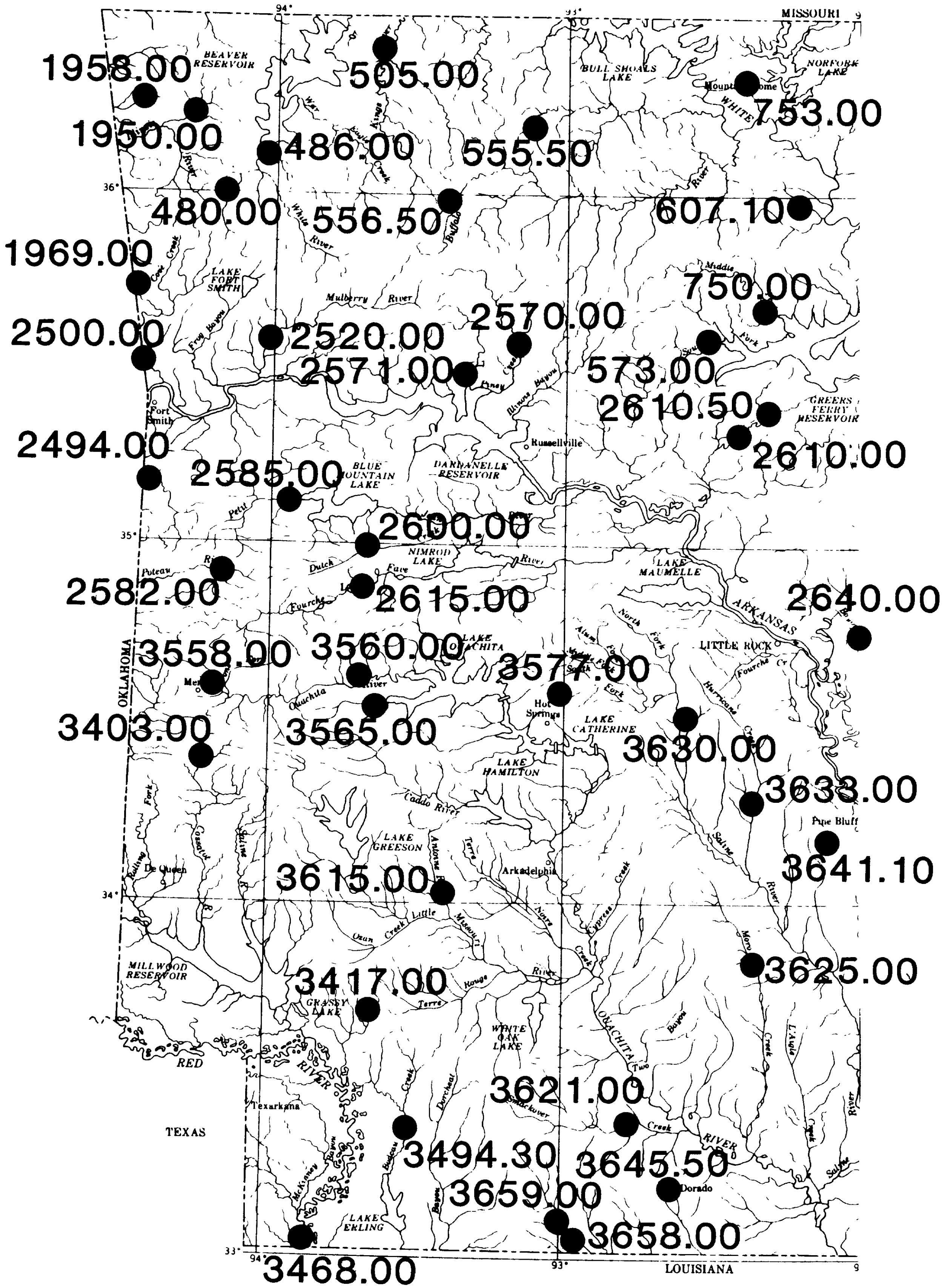
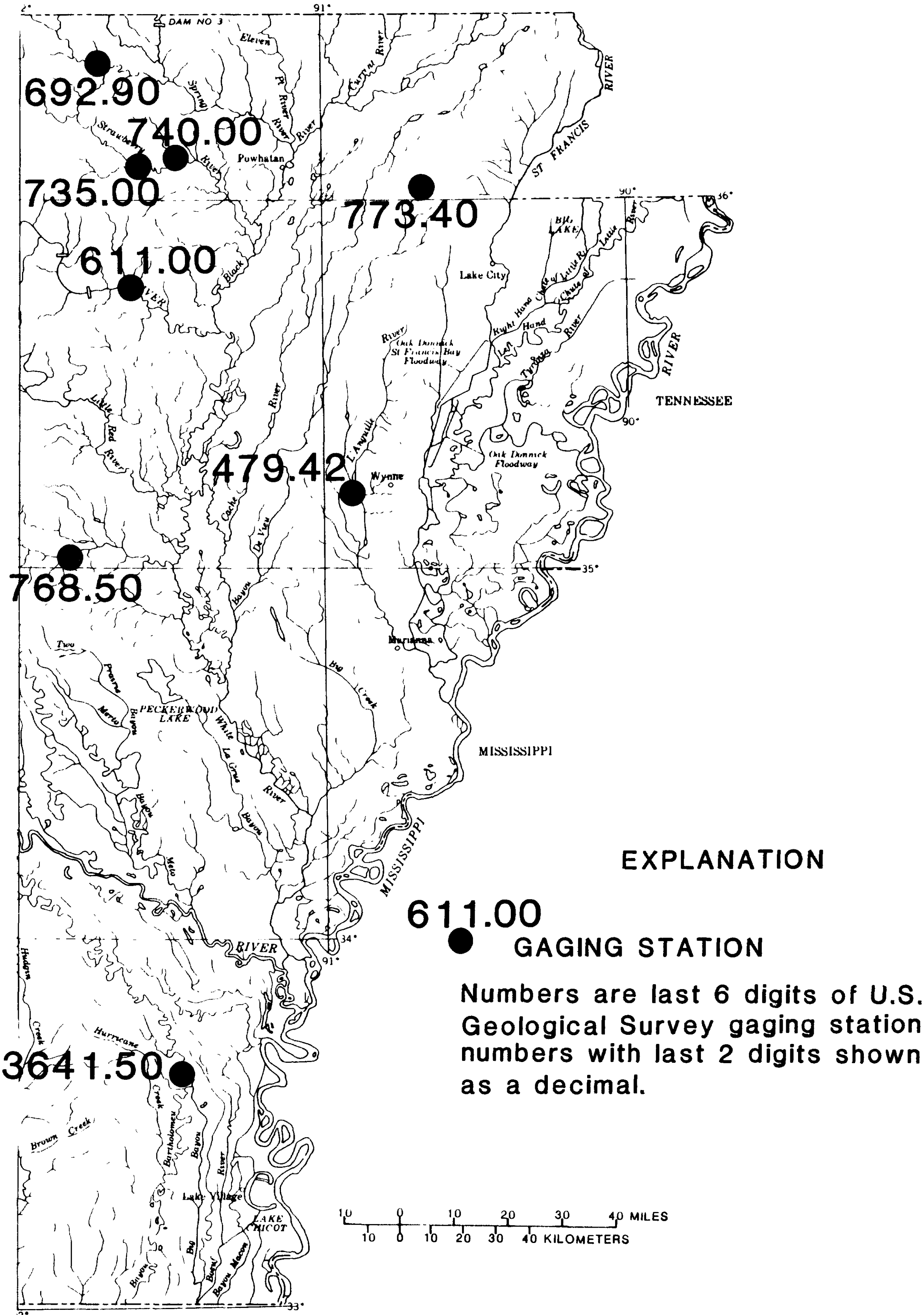


Figure 1.--Location of study area



and gaging stations.

Table 1.-- Basin and hydrologic characteristics for gaging stations used in this study

[A, drainage area; P, precipitation; S, slope; L, length; Q100, 100-year discharge; Qp, peak discharge; ELT, equivalent lagtime; LT, lagtime; NE, number of storm events; --, no data available]

Station number	Station name	A	P	S	L	Q100	Qp	ELT	LT	NE
07047942	L'Anguille River near Colt	535	49	0.90	58.70	16,900	9,360	106	105	3
07048000	West Fork White River at Greenland	83.1	45	27.50	21.00	46,600	12,700	8.01	--	13
07048600	White River near Fayetteville	400	45	14.40	45.60	97,400	22,200	20.7	--	14
07050500	Kings River near Berryville	527	43	6.80	82.50	79,100	20,300	29.8	--	9
07055550	Crooked Creek tributary near Dogpatch	4.36	45	56.10	3.80	3,840	752	2.28	1.92	10
07055650	Smith Creek near Boxley	8.35	47	137.00	6.24	10,300	1,470	2.68	2.88	6
07057300	Dodd Creek tributary near Mountain Home	.76	43	118.00	1.82	969	292	1.07	1.54	7
07060710	North Sylamore Creek near Fifty Six	58.1	45	15.40	16.10	35,500	6,840	6.38	--	11
07061100	Gibbs Creek at Sulphur Rock	3.90	47	46.20	3.66	3,890	452	1.93	1.88	12
07069290	Miller Creek near Salem	2.28	44	77.50	2.37	2,650	344	1.96	1.88	7
07073500	Piney Creek near Evening Shade	99.2	45	7.60	33.60	27,300	11,400	16.4	--	8
07074000	Strawberry River near Poughkeepsie	473	45	6.00	77.40	84,200	12,700	18.5	--	7
07075000	Middle Fork Little Red River at Shirley	302	45	13.90	63.40	126,000	19,800	13.1	--	7
07075300	South Fork Little Red River at Clinton	148	50	21.10	47.30	65,300	9,840	14.3	--	7
07076850	Cypress Bayou near Beebe	166	50	2.60	35.60	21,600	6,280	55.1	--	6
07077340	Sugar Creek tributary near Walcott	.68	48	91.30	1.45	1,000	241	.69	.90	8
07195000	Osage Creek near Elm Springs	130	43	16.90	18.20	36,300	10,000	8.48	--	8
07195800	Flint Creek at Springtown	14.2	43	22.70	6.20	13,000	1,040	5.38	6.50	12
07196900	Barren Fork at Dutch Mills	46.0	46	40.20	11.60	33,600	9,920	3.40	--	13
07249400	James Fork near Hackett	147	43	14.20	26.90	37,200	7,080	23.9	--	14
07250000	Lee Creek near Van Buren	426	45	17.40	53.20	114,000	26,100	19.8	--	18
07252000	Mulberry Creek near Mulberry	373	48	18.10	52.40	89,800	33,300	17.6	--	13
07257000	Big Piney Creek near Dover	274	49	17.00	41.80	105,000	26,100	12.2	--	10
07257100	Minnow Creek tributary near Hagarville	.20	49	311.00	1.20	335	107	.91	.88	4
07258200	Pack Saddle Creek tributary near Waldron	.92	43	58.60	2.10	947	145	2.26	1.00	6
07258500	Petit Jean River near Booneville	241	44	9.80	34.30	48,100	12,300	28.4	--	13
07260000	Dutch Creek at Waltreak	81.4	47	19.40	28.90	28,400	7,600	14.9	--	8
07261000	Cadron Creek near Guy	169	50	7.40	36.00	31,800	10,000	27.5	--	15
07261050	Pine Mountain Creek tributary near Damascus	.29	50	88.20	0.91	534	68	1.87	1.57	9
07261500	Fourche LaFave River near Gravelly	410	46	11.00	59.90	114,000	29,800	21.5	--	13
07264000	Bayou Meto near Lonoke	207	50	1.30	52.40	5,580	2,680	255	--	11
07340300	Cossatot River near Vandervoort	89.6	53	29.90	18.40	66,000	13,800	9.27	--	12
07341700	Caney Creek near Hope	12.9	51	17.50	5.88	8,600	2,170	6.86	6.00	10
07346800	East Fork Kelly Bayou tributary at Kiblah	.13	46	109.00	0.56	164	30	2.40	.75	2
07349430	Bodcau Creek at Stamps	234	50	3.60	29.00	22,600	5,190	41.8	--	2
07355800	Lewis Creek tributary near Mena	.64	51	159.00	2.04	819	109	2.03	1.88	12
07356000	Ouachita River near Mount Ida	414	52	7.80	63.00	93,500	19,800	31.7	--	10
07356500	South Fork Ouachita River at Mount Ida	64.0	53	15.40	20.40	27,700	8,300	8.18	--	7
07357700	Glazypeau Creek at Mountain Valley	3.82	56	72.10	3.43	3,980	440	3.20	2.60	7
07361500	Antoine River at Antoine	178	52	8.40	35.00	41,700	12,400	17.0	--	12
07362100	Smackover Creek near Smackover	385	50	4.00	37.90	39,000	10,900	72.6	--	12
07362500	Moro Creek near Fordyce	240	51	5.60	31.80	28,100	6,450	71.1	--	11
07363000	Saline River at Benton	550	54	12.40	55.40	116,000	31,400	28.8	--	10
07363300	Hurricane Creek near Sheridan	204	53	6.90	41.20	42,400	8,370	31.4	--	14
07364110	Nevins Creek tributary near Pine Bluff	.75	51	36.60	1.30	819	144	3.14	3.75	3
07364150	Bayou Bartholomew near McGehee	576	52	0.50	167.00	6,860	3,560	586	--	3
07364550	Caney Creek tributary near El Dorado	.10	49	222.00	0.30	357	82	1.23	.75	6
07365800	Cornie Bayou near Three Creeks	180	48	5.10	25.90	41,500	10,200	41.0	--	13
07365900	Three Creeks near Three Creeks	50.3	48	6.20	14.00	17,000	10,800	19.6	--	3

DIMENSIONLESS HYDROGRAPH METHOD

Inman (1986) used 355 actual (observed) streamflow hydrographs from 80 basins in Georgia, and harmonic analysis as described by O'Donnell (1960), to develop unit hydrographs. The 80 basins represented both urban and rural streamflow characteristics and had drainage areas less than 20 mi^2. An average unit hydrograph and an average lagtime were computed for each basin. These average unit hydrographs were then transformed to unit hydrographs having generalized durations of one-fourth, one-third, one-half, and three-fourths lagtime, then reduced to dimensionless terms by dividing the time by lagtime and the discharge by peak discharge. Representative dimensionless hydrographs developed for each basin were combined to generate one typical (average) dimensionless hydrograph for each of the four generalized durations. Using the four generalized duration dimensionless hydrographs, average basin lagtime, and peak discharge for each observed hydrograph, simulated hydrographs were generated for each of the 355 observed hydrographs, and their widths were compared with the widths of the observed hydrographs at 50 and 75 percent of peak flow. Inman (1986) concluded that the dimensionless hydrographs based on the one-half lagtime duration provided the best fit of the observed data. At the 50 percent of peak flow width, the standard error of estimate was $\pm$ 31.8 percent; and at the 75 percent of peak flow width, the standard error of estimate was $\pm$ 35.9 percent.

For verification, the one-half lagtime duration dimensionless hydrograph was applied to 138 hydrographs from 37 Georgia stations that were not used in its development. Drainage areas of these stations ranged from 20 to 500 mi^2. Inman (1986) reported that at 50 percent of peak flow, the standard error of estimate of the width was $\pm$ 39.5 percent and at 75 percent of peak flow, the standard error of estimate of the width was $\pm$ 43.6 percent.

Inman (1986) performed a second verification to assess the total or cumulative prediction error for large floods through the combined use of the dimensionless hydrograph, estimated lagtimes from regional lagtime equations, and peak discharges from regional flood-frequency equations. He found standard errors of prediction of $\pm$ 51.7 and $\pm$ 57.1 percent for peak flow widths at 50 percent and 75 percent of peak flow, repectively.

On the basis that Inman's dimensionless hydrograph was developed and tested for a variety of conditions (small and large drainage basins in urban, rural, mountainous, and coastal plain areas) and has been shown by Robbins (1986) to be applicable to central Tennessee, it was theorized that it may be applicable to streams in Arkansas (C.R. Gamble, U.S. Geological Survey, written comm., 1989).

TESTING THE DIMENSIONLESS HYDROGRAPH FOR ARKANSAS STREAMS

A dimensionless hydrograph was developed for each of 17 gaging stations in Arkansas using data from 6 to 12 flood events at each site. The dimensionless hydrograph for each flood event was developed by dividing the discharge ordinate (Q) by the peak discharge (Q_p) and by dividing the time (t) abscissa by the lagtime (LT). Zero time for flood events was the beginning of rainfall. The average dimensionless hydrograph for each station was constructed by aligning the peaks of each hydrograph and averaging the ordinate of discharge. The statewide dimensionless hydrograph was developed by the same averaging procedure using the dimensionless hydrographs for the 17 stations. The dimensionless hydrograph for Arkansas streams was similar to those developed for streams in Georgia (Inman, 1986) and in Memphis, Tennessee (Neely, 1984) as shown in figure 2. The time scale of the Memphis hydrograph is about 10 percent less than the time scale of the Arkansas and Georgia hydrographs but if the peaks on all the hydrographs are aligned, the hydrographs are very similar in shape.

The Arkansas dimensionless hydrograph was developed using actual hydrographs that included base flow whereas the Georgia dimensionless hydrograph was developed from several unit hydrographs that excluded base flow. For this reason the leading and trailing edges of the Arkansas dimensionless hydrograph are higher than those of the Georgia hydrograph. Also, the storm duration was assumed to be equal to one-half of the lagtime for the Georgia hydrographs whereas the duration of the actual storms was used to prepare the dimensionless hydrograph for streams in Arkansas.

The Georgia dimensionless hydrograph was developed using more stations and is smoother in shape than the Arkansas hydrograph. For this reason, the dimensionless hydrograph used in this study is the one developed for Georgia streams by Inman (1986). The exclusion of base flow in the Georgia hydrograph is not a problem because this low part of the hydrograph corresponds to streamflow conditions below bankfull stage on most streams. The dimensionless hydrograph developed by Inman (1986) was verified by data from 17 stations in Arkansas. The Georgia dimensionless hydrograph was based on data for rural and urban streams, however, the 17 Arkansas stations used to verify the hydrograph were rural streams. Although no urban Arkansas streams were available for verification, the dimensionless hydrograph presented in this report can be used for both rural and urban streams in Arkansas. A dimensionless hydrograph developed using 27 urban stations in the adjacent Memphis, Tennessee, area is shown on figure 2 for comparison. The hydrograph for Memphis streams was developed by using the dimensionless unit hydrograph (Neely, 1984) and by assuming that duration was equal to one-half the lagtime.

The dimensionless hydrograph used for this study (Inman, 1986) has the shape of a typical hydrograph on streams in Arkansas. The dimensionless hydrograph is shown graphically on figure 2, and data are compiled in table 2. These data can be used to compute a typical hydrograph for a selected peak discharge by multiplying each discharge ordinate of the dimensionless hydrograph by the peak discharge and multiplying each time abscissa by the lagtime.

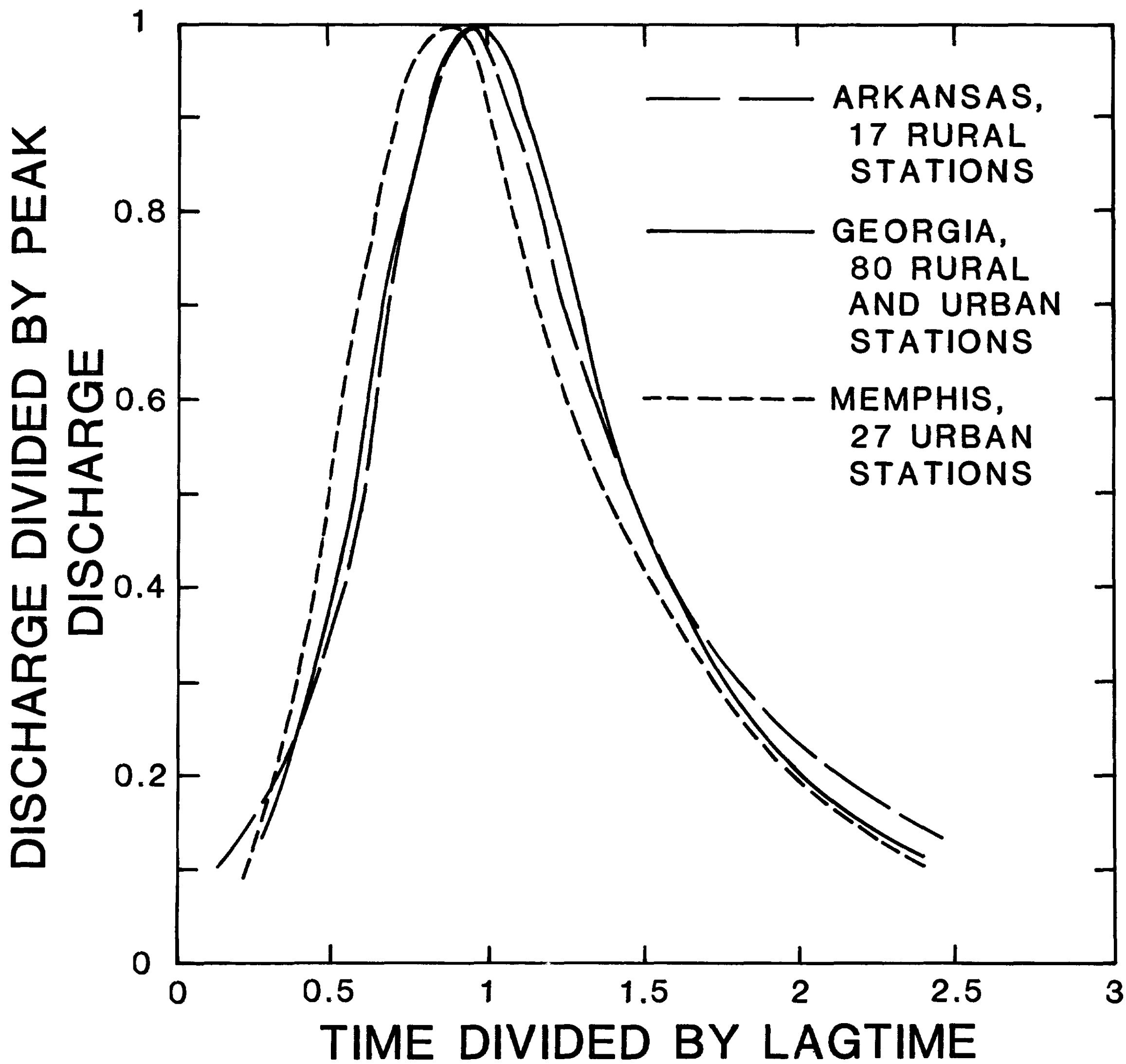


Figure 2.--Dimensionless hydrographs for streams in Arkansas, Georgia, and the Memphis, Tennessee area. Georgia hydrograph from Inman, 1986. Memphis hydrograph from Neely, 1984.

Table 2.--Time and discharge ratios for the dimensionless hydrograph

[t, time; ELT, equivalent lagtime; Q, discharge; Q_p, peak discharge]

Time ratio (t/ELT)	Discharge ratio (Q/Q_p)
0.25	0.12
.30	.16
.35	.21
.40	.26
.45	.33
.50	.40
.55	.49
.60	.58
.65	.67
.70	.76
.75	.84
.80	.90
.85	.95
.90	.98
.95	1.00
1.00	.99
1.05	.96
1.10	.92
1.15	.86
1.20	.80
1.25	.74
1.30	.68
1.35	.62
1.40	.56
1.45	.51
1.50	.47
1.55	.43
1.60	.39
1.65	.36
1.70	.33
1.75	.30
1.80	.28
1.85	.26
1.90	.24
1.95	.22
2.00	.20
2.05	.19
2.10	.17
2.15	.16
2.20	.15
2.25	.14
2.30	.13
2.35	.12
2.40	.11

ESTIMATING EQUIVALENT LAGTIME

Lagtime is defined as the time between the centroid of rainfall excess and the centroid of runoff. Lagtime (LT) for 17 stations in Arkansas (table 1) was computed using the equation, $LT = KSW + 0.5\ T_c$ where KSW is the recession coefficient and T_c is the time base of the hydrograph as computed by the rainfall-runoff model developed by Dawdy and others (1972) and modified by Carrigan (1973). If lagtime, peak discharge, and a dimensionless hydrograph are known, a hydrograph can be estimated for an ungaged site.

It is difficult to accurately determine the lagtime at some sites in the State for the following reasons. The time when the centroid of rainfall excess occurs may not be accurately defined because data from National Weather Service rain gages are usually recorded at 1-hour intervals. In addition, the rainfall measured at the rain gage may not be representative of the rainfall in the basin due to spatial variability of precipitation. Therefore, if data are insufficient for computation of the lagtime, an equivalent lagtime (ELT) can be estimated by the method described below.

Equivalent lagtime (ELT) is computed using data from an actual discharge hydrograph with the dimensionless hydrograph (fig. 2). Because it is impossible to reproduce an actual discharge hydrograph in its entirety using one value of equivalent lagtime, it was decided to reproduce it for only a few points. The three selected points chosen and the appropriate equations for computing ELT at each point are as follows:

1. Width (in time) of the hydrograph at 75 percent of peak discharge.
$$ELT = width/0.55 \tag{1}$$
2. Width (in time) of the hydrograph at 50 percent of peak discharge.
$$ELT = width/0.91 \tag{2}$$
3. Width (in time) of the hydrograph between 50 percent of peak discharge (rising stage) and 75 percent of peak discharge (falling stage).
$$ELT = width/0.69 \tag{3}$$

From an actual discharge hydrograph, a value of ELT is computed using each of the three equations. If the values of ELT computed using the three equations are about the same, the arithmetic average can be used. If they differ, it is probably because the leading or trailing edge of the hydrograph is not typical and the ELT computed by equation 1 should be used.

The constants in the three equations were computed from the dimensionless hydrograph in table 2. Each constant was determined by subtracting the value of t/ELT on the rising limb of the dimensionless hydrograph from the value of t/ELT on the falling limb of the hydrograph at the appropriate discharge ratio, Q/Q_p.

The computation of ELT in this manner includes the different durations of the flood events used. In order to determine the average or typical hydrograph, the ELT computed from several hydrographs at a station are averaged.

The ELT that is computed for a given discharge hydrograph may not be the actual lagtime, but it is the value that can be used to reproduce the actual discharge hydrograph from the dimensionless hydrograph. The lagtime and the ELT for the 17 stations in Arkansas used in defining the dimensionless hydrograph are plotted on figure 3. The plot shows some scattering of the points but a relation does exist. Thus, ELT can be used to estimate hydrographs for selected peak discharge recurrence intervals.

At each gaging station listed in table 1, ELT was determined for each of several storm events. A total of 450 storm events was used for the 49 stations. Because some stations had many storm events while others had only a few, it was decided to compute an average ELT for each station rather than including all 450 storm events that could bias the results toward stations that had more storm events. At each station an average peak discharge of the storm events and an average ELT were determined and are shown in table 1. The average peak discharge of the storm events is the arithmetic average of the peak discharges of each storm event. The average ELT is the arithmetic average of the ELTs of each storm event.

Regression Analysis

The equivalent lagtime determined for the gaging stations used in the analysis was related to basin, climatic, and hydrologic parameters using linear multiple-regression techniques. The regression equation has the form:

$$Y = aA^{b1}S^{b2}L^{b3}---, \qquad (4)$$

where, Y = equivalent lagtime,

A, S, and L are basin, climatic, and hydrologic characteristics, and

a, b1, b2, b3 are constants and coefficients obtained by regression analysis.

Regression analysis of the data indicates that drainage area and 100-year discharge are statistically significant characteristics (at the 95 percent confidence limit) for estimating ELT. The following equation was developed by the multiple regression technique using data from 49 gaging stations in Arkansas. The 100-year discharge is used so that one equation can be used throughout the State even though Arkansas has considerable variation in topography. The equation for computing ELT is shown below with the standard error of estimate.

$$ELT = 3{,}480\ A^{1.15}Q_{100}^{-1.04} \pm 38 \text{ percent} \qquad (5)$$

where, ELT = the equivalent lagtime, in hours, used with the dimensionless hydrograph to reproduce an average discharge hydrograph.

A = the contributing drainage area of the basin, in mi^2, and

Q_{100} = the discharge, in ft^3/s for the 100-year flood (Neely, 1987).

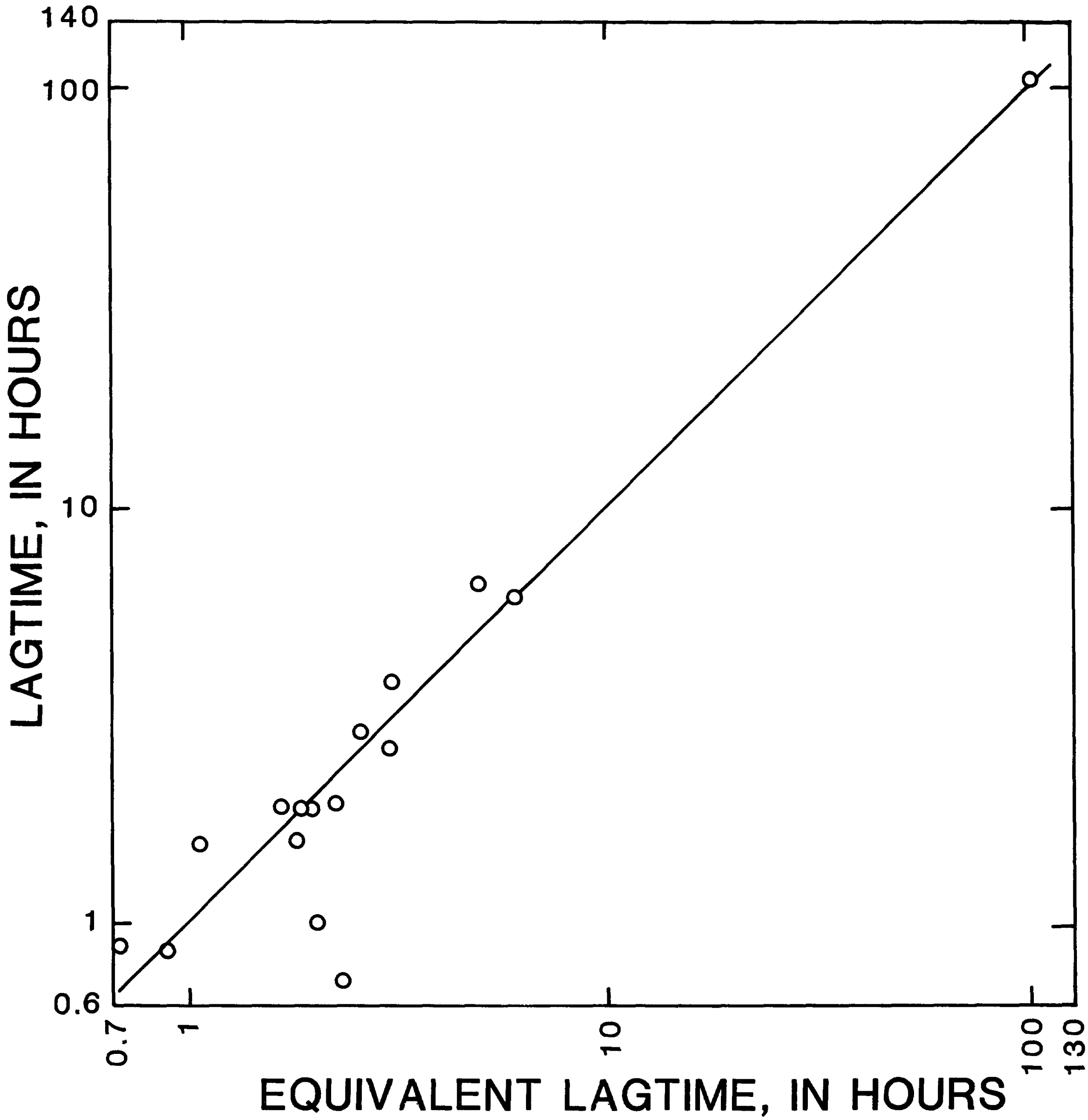


Figure 3.--Relation between aquivalent lagtime and lagtime for selected stations in Arkansas.

Testing

The accuracy of the equation for estimating ELT is determined by computing the difference between the ELT values based on average station data (table 1) and the ELT values estimated by the regression equation. The accuracy, in percent, referred to as standard error, is the range of error that can be expected about two-thirds of the time. The standard error of regression of equation 5 is $\pm$ 38 percent.

A test also was made to measure the accuracy of the ELT values obtained from the regression equation for large flood events. In this test, the largest flood of record was selected at each gaging station. The ELT was computed from the actual flood hydrographs by determination of the hydrograph widths at 50 and 75 percent of the peak discharges. The widths were used in equations 1 and 2 to compute the appropriate ELT values. These values of ELT were then compared with the ELT values computed using the regression equation. The standard errors at 50 and 75 percent of peak discharge were 41 and 39 percent, respectively.

The regression equation for equivalent lagtime also was tested for geographical bias and for variable bias. Geographical bias was tested by plotting the residuals on a State map. The residual is the computed ELT from station data divided by the computed ELT using the regression equation. The residuals were uniformly scattered on the plot and no geographical bias was observed. Each variable in the regression equation also was checked by residual analysis for bias. This was done by solving equation 5 for the particular variable being tested. The computed value was plotted on log-log paper against the variable being tested. If the variable being tested is unbiased, the plotted points should define a straight line with a slope equal to the exponent of the variable being tested. No bias was observed for any of the variables.

Sensitivity analyses were performed on the regression equation to measure the effect of errors in the independent variables (A, Q_{100}) on the dependent variable (ELT). All parameters were assumed to be constant except the one being tested for sensitivity; that parameter was assumed to contain an error ranging from +50 percent to -50 percent. The sensitivity of the regression equation to error in basin and hydrologic characteristics is shown below.

Percent Error in Computed ELT

Percent error in independent variable	A	Q_{100}
50	59	-34
30	35	-24
10	12	-9
-10	-11	12
-30	-34	45
-50	-55	106

For example, assume that the drainage area (A) for a particular site contains an error of +30 percent. That error would result in a 35 percent error in the computed ELT for the site using the regression equation. The percent error table shows that the equation is most sensitive to errors in drainage area (A) and 100-year discharge (Q_{100}).

ESTIMATING FLOOD VOLUME

In some instances it is necessary to know the volume of runoff in the design of hydraulic structures. The following equation derived from this study can be used to determine the runoff volume for any typical flood event:

$$V = \frac{0.00169\ Q_p ELT}{A} \tag{6}$$

where V is runoff volume, in inches, not including base flow;

Q_p is peak discharge, in ft^3/s;

ELT is equivalent lagtime, in hours; and

A is drainage area, in mi^2.

The constant in equation 6 was determined by summing the discharge ratio values in table 2 and adding estimated values for the leading and trailing edges of the hydrograph. This value was then multiplied by 3,600 (the number of seconds in an hour) x 12 (the number of inches in a foot) x 0.05 (the time increment used in table 2) and divided by $(5,280)^2$ (5,280 = the number of feet in a mile).

HYDROGRAPH-WIDTH RELATION

For some design purposes, it is necessary to know only the period of time that a specific discharge will be exceeded. To provide this information, a hydrograph-width relation based on the dimensionless hydrograph was developed and is presented in table 3. The width ratio (W/ELT) was determined by subtracting the value of t/ELT on the rising limb of the dimensionless hydrograph from the value of t/ELT on the falling limb of the hydrograph at the same discharge ratio, Q/Q_p. The hydrograph width, W, can be estimated for a specified discharge, Q, by first computing the ratio Q/Q_p and then multiplying the corresponding W/ELT ratio by the equivalent lagtime, ELT.

APPLICATION OF HYDROGRAPH ESTIMATING TECHNIQUE

Suppose the shape of a typical hydrograph is needed for a site on Example Creek for the 25-year flood. The drainage area of the site is 22.4 mi^2, and the discharges of the 25- and 100-year floods have been determined (Neely, 1987) to be 11,700 ft^3/s and 18,000 ft^3/s, respectively. From equation 5, a value of ELT is computed as:

$$ELT = 3,480\ (22.4)^{1.15}(18,000)^{-1.04} = 4.67 \text{ hours}$$

Table 3.--Hydrograph-width ratios for selected discharge-peak discharge ratios

[Q, discharge; Q_p, peak discharge; W, width; ELT, equivalent lagtime]

Discharge ratios Q/Q_p	Hydrograph width ratios W/ELT
1.00	0
.95	.22
.90	.32
.85	.40
.80	.48
.75	.55
.70	.62
.65	.68
.60	.76
.55	.83
.50	.91
.45	1.00
.40	1.09
.35	1.20
.30	1.33
.25	1.47
.20	1.66

The values for ELT and the 25-year discharge are used with the dimensionless hydrograph in table 2 to compute the typical simulated hydrograph. The simulated coordinates of the flood hydrograph are shown in table 4 and on figure 4. Each value of t/ELT is multiplied by 4.67 to determine the time, and each value of Q/Q_p is multiplied by 11,700 to determine the discharge.

The amount of time that the flow of Example Creek will be above bankfull stage during a 25-year flood event can be determined from data in table 3. The discharge at bankfull stage is 3,010 ft^3/s, therefore the ratio of bankfull discharge to peak discharge (Q/Q_p = 3,010/11,700) is equal to 0.26. From table 3, a Q/Q_p ratio of 0.26 corresponds to a ratio of 1.45 for W/ELT. The hydrograph width (W) is equal to 1.45 times the ELT of 4.67 or 6.77 hours. Water would be above bankfull stage about 7 hours during a 25-year flood event. This value could also be scaled directly from the computed hydrograph shown in figure 4.

Table 4.--Simulated coordinates of the flood hydrograph for Example Creek

[t, time; ELT, equivalent lagtime; Q, discharge; Q_p, peak discharge, ft^3/s, cubic feet per second]

t/ELT (from table 2)	x ELT (hours)	= Time (hours)	Q/Q_p (from table 2)	x Q_p (ft^3/s)	= Discharge, (ft^3/s)
0.25	4.67	1.16	0.12	11,700	1,400
.30	4.67	1.40	.16	11,700	1,870
.35	4.67	1.63	.21	11,700	2,460
.40	4.67	1.87	.26	11,700	3,040
.45	4.67	2.10	.33	11,700	3,860
.50	4.67	2.33	.40	11,700	4,680
.55	4.67	2.57	.49	11,700	5,730
.60	4.67	2.80	.58	11,700	6,790
.65	4.67	3.04	.67	11,700	7,840
.70	4.67	3.27	.76	11,700	8,890
.75	4.67	3.50	.84	11,700	9,830
.80	4.67	3.74	.90	11,700	10,500
.85	4.67	3.97	.95	11,700	11,100
.90	4.67	4.20	.98	11,700	11,500
.95	4.67	4.44	1.00	11,700	11,700
1.00	4.67	4.67	.99	11,700	11,600
1.05	4.67	4.90	.96	11,700	11,200
1.10	4.67	5.14	.92	11,700	10,800
1.15	4.67	5.37	.86	11,700	10,100
1.20	4.67	5.60	.80	11,700	9,360
1.25	4.67	5.84	.74	11,700	8,660
1.30	4.67	6.07	.68	11,700	7,960
1.35	4.67	6.30	.62	11,700	7,250
1.40	4.67	6.54	.56	11,700	6,550
1.45	4.67	6.77	.51	11,700	5,970
1.50	4.67	7.00	.47	11,700	5,500
1.55	4.67	7.24	.43	11,700	5,030
1.60	4.67	7.47	.39	11,700	4,560
1.65	4.67	7.71	.36	11,700	4,210
1.70	4.67	7.94	.33	11,700	3,860
1.75	4.67	8.17	.30	11,700	3,510
1.80	4.67	8.41	.28	11,700	3,280
1.85	4.67	8.64	.26	11,700	3,040
1.90	4.67	8.87	.24	11,700	2,810
1.95	4.67	9.11	.22	11,700	2,570
2.00	4.67	9.34	.20	11,700	2,340
2.05	4.67	9.57	.19	11,700	2,220
2.10	4.67	9.81	.17	11,700	1,990
2.15	4.67	10.04	.16	11,700	1,870
2.20	4.67	10.27	.15	11,700	1,760
2.25	4.67	10.51	.14	11,700	1,640
2.30	4.67	10.74	.13	11,700	1,520
2.35	4.67	10.97	.12	11,700	1,400
2.40	4.67	11.21	.11	11,700	1,290

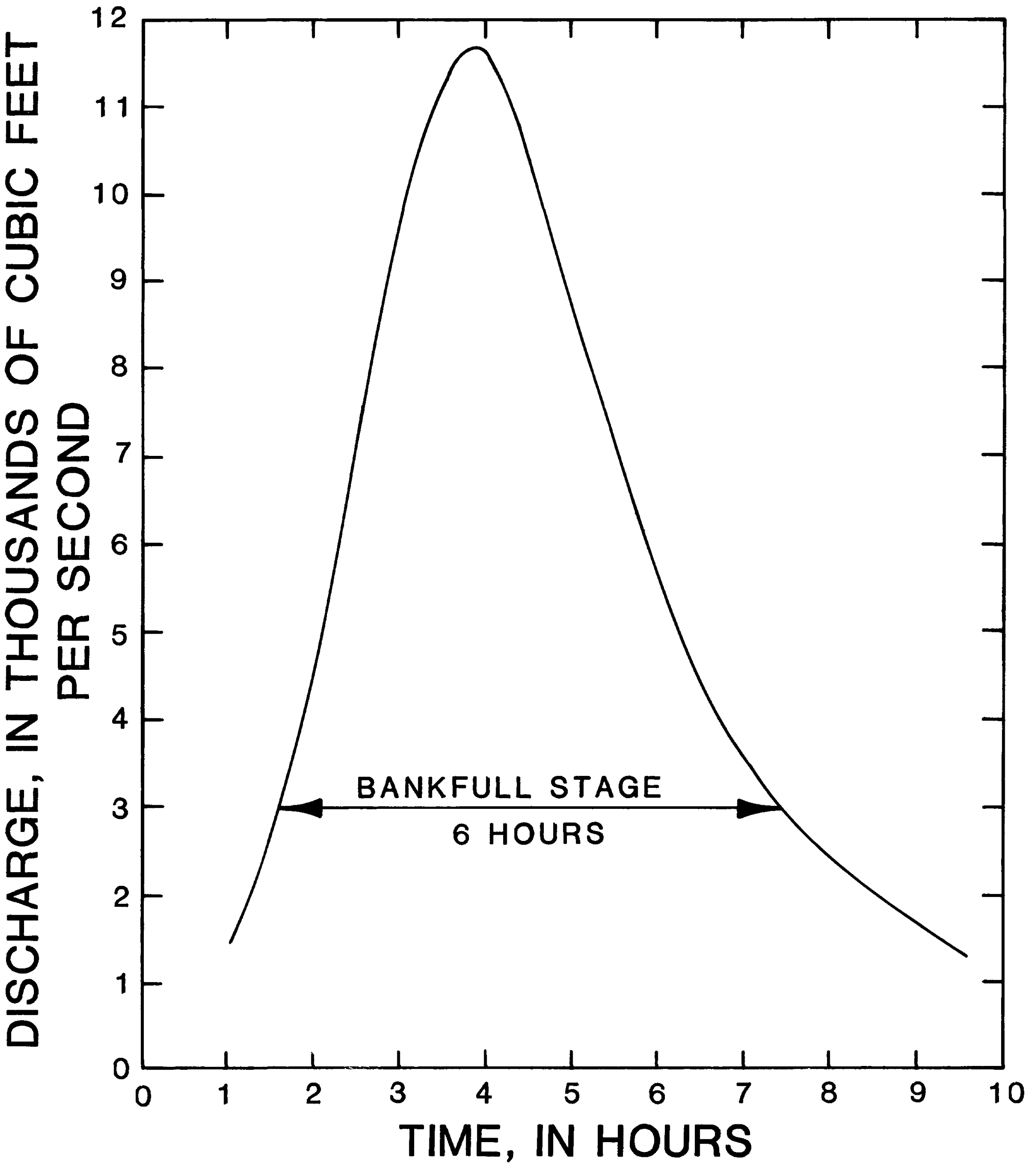


Figure 4.--Simulated flood hydrograph for Example Creek.

SUMMARY

A dimensionless hydrograph is presented for Arkansas streams having drainage areas less than about 600 mi^2. This dimensionless hydrograph can be used with peak discharge and equivalent lagtime to determine flood hydrographs at ungaged sites on rural and urban streams in Arkansas.

Multiple regression analysis was used to define relations between equivalent lagtime and basin, climatic, and hydrologic characteristics. Data collected on 450 storms at 49 gaging stations were used in the analysis. The regression analysis indicated that drainage area and 100-year discharge are significant parameters for estimating equivalent lagtime. The standard error of the regression equation is $\pm$ 38 percent. The equation was tested for accuracy, bias, and sensitivity.

An equation is presented for computing the volume of flood runoff when the peak discharge, equivalent lagtime, and drainage area are known. In addition, a hydrograph-width relation is presented for estimating the length of time that a specific discharge will be exceeded.

REFERENCES

Carrigan, P.H., Jr., 1973, Calibration of U.S. Geological Survey rainfall-runoff model for peak flow synthesis--natural basins: U.S. Geological Survey open-file report, 109 p.

Carrigan, P.H., Dempster, G.R., and Bower, D.E., 1977, User's guide for U.S. Geological Survey rainfall-runoff model--revision of Open-File Report 74-33: U.S. Geological Survey Open-File Report 77-884, 273 p.

Dawdy, D.R., Lichty, R.W., and Bergmann, J.M., 1972, A rainfall-runoff simulation model for estimation of flood peaks for small drainage basins: U.S. Geological Survey Professional Paper 506-B, 28 p.

Inman, E.J., 1986, Simulation of flood hydrographs for Georgia streams: U.S. Geological Survey Water-Resources Investigations Report 86-4004, 41 p.

Neely, B.L., Jr., 1984, Flood frequency and storm runoff of urban areas of Memphis and Shelby County, Tennessee: U.S. Geological Survey Water-Resources Investigations Report 84-4110, 51 p.

----- 1987, Magnitude and frequency of floods in Arkansas: U.S. Geological Survey Water-Resources Investigations Report 86-4335, 51 p.

O'Donnell, Terrance, 1960, Instantaneous unit hydrograph derivation by harmonic analysis: Commission of Surface Waters, Publication 51, International Association of Scientific Hydrology, p. 546-557.

Robbins, C.H., 1986, Techniques for simulating flood hydrographs and estimating flood volumes for ungaged basins in central Tennessee: U.S. Geological Survey Water-Resources Investigations Report 86-4192, 32 p.

☆ U.S. GOVERNMENT PRINTING OFFICE; 1989 — 648-137/00014