TECHNIQUES FOR ESTIMATING FLOOD HYDROGRAPHS FOR UNGAGED URBAN WATERSHEDS

By V. A. Stricker and V. B. Sauer

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JAMES G. WATT, Secretary

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Dallas L. Peck, Director

For additional information write to:

U.S. Geological Survey, WRD
Suite B
6481 Peachtree Industrial Boulevard
Doraville, Georgia 30360

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FACTORS FOR CONVERTING INCH-POUND UNITS TO INTERNATIONAL SYSTEM (SI) UNITS

The following factors may be used to convert the inch-pound units published herein to the International System of Units (SI):

Multiply inch-pound	By	To obtain SI units
	Length	
inches (in)	25.4	millimeters (mm)
	0.0254	meters (m)
feet (ft)	0.3048	meters (m)
miles (mi)	1.609	kilometers (km)
	Area	
square miles (mi^2)	2.590	square kilometers (km^2)
	Flow	
cubic feet per second (ft^3/s)	0.02832	cubic meters per second (m^3/s)

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ABSTRACT

The Clark Method, modified slightly, was used to develop a synthetic, dimensionless hydrograph that can be used to estimate flood hydrographs for ungaged urban watersheds. Application of the technique results in a typical (average) flood hydrograph for a given peak discharge. Input necessary to apply the technique is an estimate of basin lagtime and the recurrence interval peak discharge. Equations for this purpose were obtained from a recent nationwide study on flood frequency in urban watersheds. A regression equation was developed which relates flood volumes to drainage area size, basin lagtime, and peak discharge. This equation is useful where storage of floodwater may be a part of design or flood prevention.

INTRODUCTION

The design of highway bridges and embankments requires an evaluation of the flood-related risk both to the structures and to the surrounding property. Risk analyses of alternate designs are necessary to determine the design with the least total expected cost (Corry and others, 1980). As part of these analyses, runoff hydrographs may be necessary to estimate the length of time of occurrence of inundation of specific features, for example, road overflow. Many times site hydrograph data is not available; therefore, there is a need for a simple method to estimate the flood hydrograph associated with a peak discharge of specific recurrence interval (a design discharge). The objective of this study is to define techniques for estimating flood hydrographs (shape and volume) in ungaged urban areas for watersheds without significant in-channel storage. This report is prepared in cooperation with the Federal Highway Administration, Office of Research.

DATA BASE

The data base used in this study consisted of 62 stations: 3 in Georgia, 2 in Pennsylvania, 3 in Tennessee, 1 in Colorado, 25 in Missouri, 2 in Oklahoma, 21 in Oregon, and 5 in Texas. These stations were selected from the data base developed by Sauer and others (1981). They were chosen because a rainfall-runoff model has been calibrated for each watershed and it was possible to easily obtain volumes and peaks for known storms, and hydrograph time characteristics for unit-hydrograph derivations. Various subsets of these stations were used in the development and testing of the hydrograph estimating procedure, and the development and testing of the hydrograph-volume relations. Some stations were used in more than one subset.

HYDROGRAPH ESTIMATING PROCEDURE

A simplified procedure is described for estimating a hydrograph in an ungaged urban watershed for a flood of selected recurrence interval. The procedure uses methods developed by Sauer and others (1981) to estimate peak discharges of specified recurrence intervals, and basin lagtime. The Clark Method (1945) is used to derive the hydrograph estimating procedure.

Dimensionless Hydrograph

A dimensionless hydrograph was developed similar to those proposed by Mitchell (1972) and the U.S. Department of Agriculture, Soil Conservation Service (1972). Derivation of the dimensionless hydrograph began by using the Clark Method (1945) to derive T-hour unit hydrographs for 19 watersheds representing a variety of basin sizes and different time characteristics. The Clark Method was modified by the use of an isosceles triangle for the time-area-histogram. Duration of rainfall excess for each basin was set at approximately one-third of the lagtime for that basin after a study of various durations indicated only minor variations in the dimensionless hydrographs. Each unit hydrograph was transformed to dimensionless terms by dividing the elapsed time by the hydrograph lagtime, and by dividing the discharge at any given point on the hydrograph by the peak discharge. The resulting dimensionless hydrographs were similar, particularly the upper one-third of the hydrographs, when compared by alining the peaks. A representative dimensionless hydrograph was selected from the 19 watersheds; the coordinates are listed in table 1, and plotted in figure 1. To use these values to derive a hydrograph at a site, it is necessary to know the basin lagtime and the peak discharge at the site. Both of these may be estimated by using methods described by Sauer and others (1981). These methods are briefly described in the next section.

Estimating Basin Lagtime

Basin response time, or lagtime, was used as the principal time factor in the dimensionless hydrograph. Lagtime is generally considered to be constant for a basin and is defined as the time elapsed from the centroid of rainfall excess to the centroid of the resultant runoff hydrograph. This time characteristic of a basin is a principal factor in determining the relative shape of a hydrograph, such as a broad flat-crested hydrograph or a narrow sharp-crested hydrograph. Since lagtime is often not known for a basin, it is usually estimated from other basin characteristics. In this report, the simplified equation previously developed by Sauer and others (1981) is used to estimate lagtime. The equation is as follows:

$$LT = 0.85\ (L/\sqrt{SL})^{0.62}\ (13\text{-}BDF)^{0.47} \quad \text{(standard error of regression} = \pm\ 76\text{ percent)} \qquad (1)$$

Table 1.--Time and discharge ratios of the dimensionless hydrograph

Time ratio (t/LT)	Discharge ratio (Q_t/Q_p)
0.45	0.27
.50	.37
.55	.46
.60	.56
.65	.67
.70	.76
.75	.86
.80	.92
.85	.97
.90	1.00
.95	1.00
1.00	.98
1.05	.95
1.10	.90
1.15	.84
1.20	.78
1.25	.71
1.30	.65
1.35	.59
1.40	.54
1.45	.48
1.50	.44
1.55	.39
1.60	.36
1.65	.32
1.70	.30

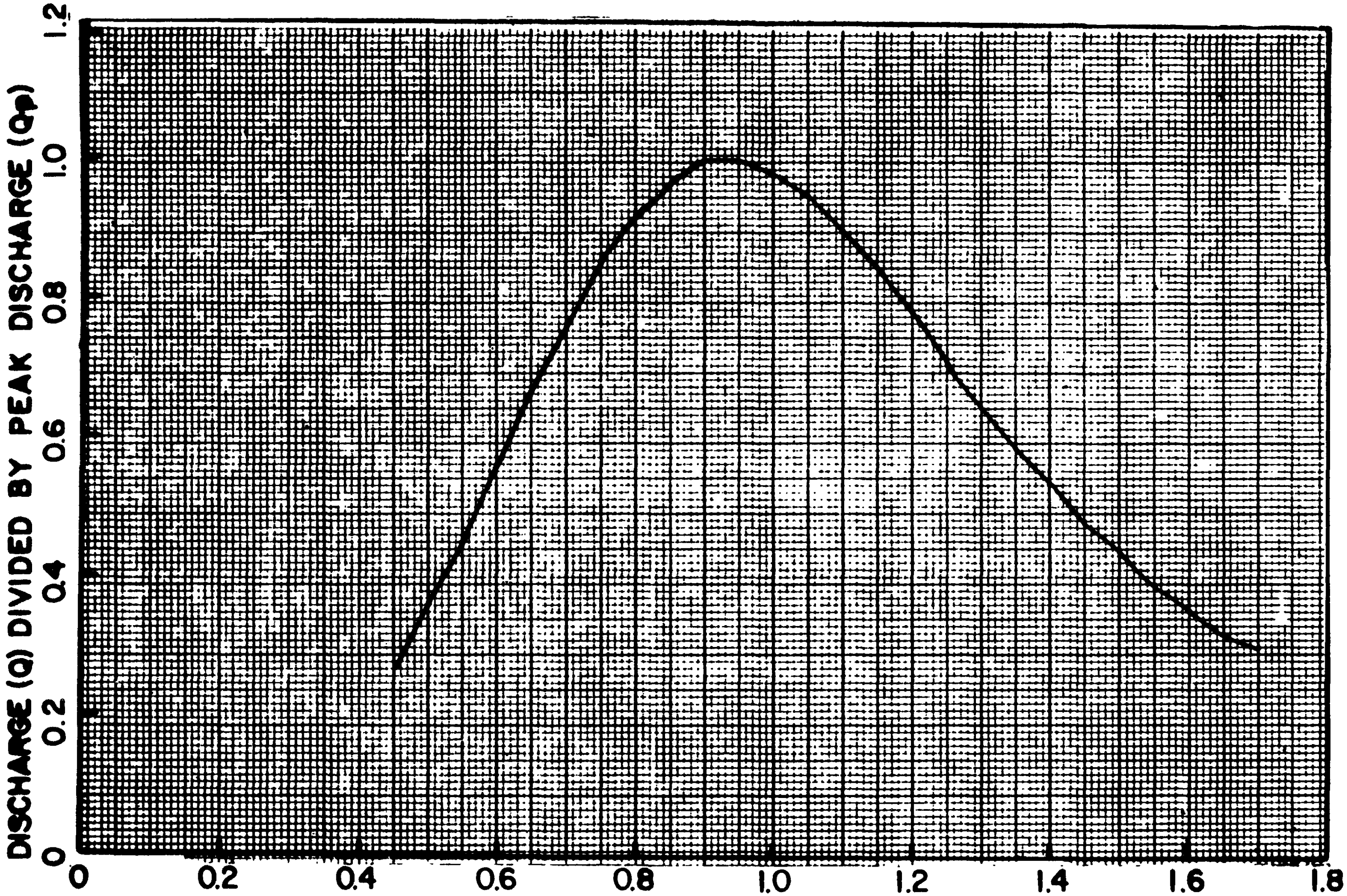

Figure 1. Dimensionless Hydrograph.

where LT = lagtime, in hours, for the urban watershed,
L = basin length, in miles, measured on topographic maps along the main channel from the gaging station to the basin divide,
SL = the main channel slope, in feet per mile (ft/mi), measured between points which are 10 percent and 85 percent of the main channel length upstream from the study site. For sites where SL is greater than 70 ft/mi, use 70 ft/mi in the equation, as documented in Sauer and others (1981), and
BDF = basin development factor, as determined using methods described by Sauer and others (1981). The basin development factor will range from 0 to 12.

The simplified equation rather than the more complex equation given by Sauer and others (1981) was used because it utilizes variables employed by previous investigators and because it contains a definitive measure of basin development.

Estimating Peak Discharge

The peak discharge for various recurrence intervals, in years, can be estimated for ungaged urban watersheds in the United States by use of the following three-parameter equations previously developed by Sauer and others (1981).

Equation	Standard error of regression, in percent	
$UQ2 = 13.2\ A^{0.21}(13-BDF)^{-0.43}RQ2^{0.73}$	$\pm$ 43	(2)
$UQ5 = 10.6\ A^{0.17}(13-BDF)^{-0.39}RQ5^{0.78}$	$\pm$ 40	(3)
$UQ10 = 9.51\ A^{0.16}(13-BDF)^{-0.36}RQ10^{0.79}$	$\pm$ 41	(4)
$UQ25 = 8.68\ A^{0.15}(13-BDF)^{-0.34}RQ25^{0.80}$	$\pm$ 43	(5)
$UQ50 = 8.04\ A^{0.15}(13-BDF)^{-0.32}RQ50^{0.81}$	$\pm$ 44	(6)
$UQ100 = 7.70\ A^{0.15}(13-BDF)^{-0.32}RQ100^{0.82}$	$\pm$ 46	(7)
$UQ500 = 7.47\ A^{0.16}(13-BDF)^{-0.30}RQ500^{0.82}$	$\pm$ 52	(8)

where UQx = peak discharge, for recurrence interval x in years, in cubic feet per second (ft^3/s), for the urban watershed,
A = contributing drainage area, in square miles (mi^2),
BDF = basin development factor, as defined by Sauer and others (1981),
and RQx = peak discharge, for recurrence interval x in years, in cubic feet per second (ft^3/s), for an equivalent rural watershed in the same hydrologic area as the urban watershed.

Estimating Design Flood Hydrographs

A typical hydrograph for a specified recurrence interval (a design hydrograph) can be estimated from the dimensionless hydrograph presented in table 1 (or fig. 1), where ordinate values are expressed in a dimensionless ratio of Q_t/Q_p, and abscissa values are expressed in a dimensionless ratio of t/LT. Q_t is discharge, in cubic feet per second, at time t, in hours.

Q_p is the peak discharge of the hydrograph, in cubic feet per second, and LT is basin lagtime, in hours, as previously defined. The basin lagtime can be estimated using equation 1, and the desired design peak discharge from the appropriate equation selected from equations 2 through 8. The time scale for the hydrograph is computed by multiplying LT times each of the t/LT ratios in table 1. The corresponding discharges are computed by multiplying the peak discharge, Q_p, times each of the Q_t/Q_p ratios in table 1. The resulting hydrograph will have a peak discharge equal to the design peak discharge, and is assumed to be a typical (or average) flood hydrograph for the selected recurrence interval.

Comparison of Estimated and Observed Hydrographs

The dimensionless hydrograph method was tested by applying the method at 14 gaging stations where the results could be compared to observed hydrographs. In these tests, two estimates of lagtime, LT, were used. One estimate of LT was derived from an average of measured lagtimes for several observed flood hydrographs at each station. This should produce the best estimate of LT. However, to provide a more realistic comparison of the method at ungaged sites, a second estimate of LT was made using equation 1. Drainage basin characteristics and the two estimates for lagtime for each station are shown in table 2. Hydrographs were estimated using each of the two lagtimes, and the observed peak discharge for Q_p. The hydrograph comparisons are presented in figures 2 through 16. For comparative purposes, the hydrographs have been plotted so that the peak discharges coincide.

HYDROGRAPH-WIDTH RELATION

For some problems it is only necessary to know the period of time that a specific discharge will be exceeded, therefore a complete hydrograph is not needed. For this case, the dimensionless hydrograph in table 1 or figure 1 was used to define a dimensionless hydrograph-width relation. Hydrograph width is denoted as W, in hours, and the width ratio W/LT was determined by subtracting the value of t/LT on the rising limb of the dimensionless hydrograph from the value of t/LT on the falling limb of the hydrograph at the same Q/Q_p discharge ratio (fig. 1). This relation is shown in table 3 and figure 17. 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/LT ratio by the estimated lagtime, LT.

To test this method, hydrograph widths were computed at a Q/Q_p ratio of 0.75 (75 percent of the peak discharge) for moderate to large floods at 14 gaging stations. Lagtime, LT, for these sites was estimated using equation 1 to make the test indicative of results to be expected at ungaged sites. The computed hydrograph widths are compared to observed hydrograph widths in table 4. The average standard error or estimate is $\pm$ 89 percent, which is slightly more than the standard error of estimate of lagtime reported by Sauer and others (1981). The small sample size does not make this a good

Table 2.—Drainage basin characteristics

Station No.	Station name	Drainage area, A (mi^2)	Basin length, L (mi)	Basin slope, SL (ft/mi)	Basin development factor, BDF	Measured lagtime, LT (hr)	Estimated lagtime from equation 1 LT (hr)
01475530	Cobbs Creek at U.S. Hwy. 1, at Philadelphia, Pa.	4.78	4.27	62.50	6	1.71	1.45
02203884	Conley Creek near Forest Park, Ga.	1.88	2.25	75.10*	3	1.22	1.11
02203870	Cobbs Creek near Atlanta, Ga.	3.68	3.84	45.80	3	2.12	1.76
02336102	North Fork Peachtree Creek tributary near Atlanta, Ga.	2.19	2.46	63.60	3	1.30	1.21
06935800	Shotwell Creek at State Hwy. 340, near Ellisville, Mo.	.81	1.10	84.80*	5	.53	.64
06936380	Paddock Creek at Lindbergh Blvd., at St. Louis, Mo.	2.64	2.56	29.30	9	.90	1.02
06936460	Coldwater Creek at Old Halls Ferry Rd., near St. Louis, Mo.	38.90	14.40	8.67	9	3.64	4.36
07002000	Watkins Creek at Cold Bank Rd., near St. Louis, Mo.	6.17	5.30	24.70	7	1.40	2.05
07242200	Deep Fork at Portland Ave., at Oklahoma City, Okla.	2.93	3.00	44.00	12	.95	.52
08055600	Joes Creek at Dallas, Tex.	7.51	6.42	31.00	11	3.56	1.29
08055700	Bachman Branch at Dallas, Tex.	10.00	6.32	31.60	6	2.06	2.28
08056500	Turtle Creek at Dallas, Tex.	7.98	5.30	36.30	9	1.50	1.51
08057200	White Rock Creek abv. Greenville Ave., at Dallas, Tex.	66.40	21.90	12.00	3	4.02	7.87
14206900	Fanno Creek at Portland, Oreg.	2.37	2.50	200.00*	7	1.87	.93

* Use equation limit of 70.00, as documented in Sauer and others (1981).

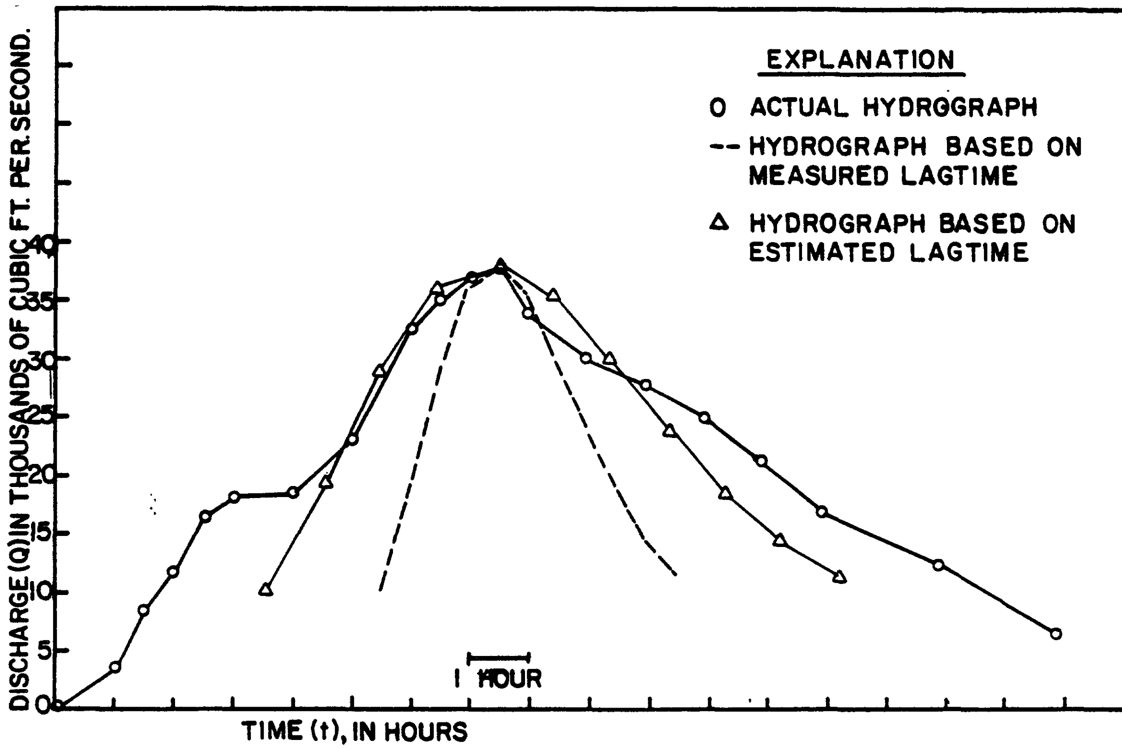

Figure 2. White Rock Creek above Greenville Ave., at Dallas Tex. (08057200), for storm of September 21, 1964.

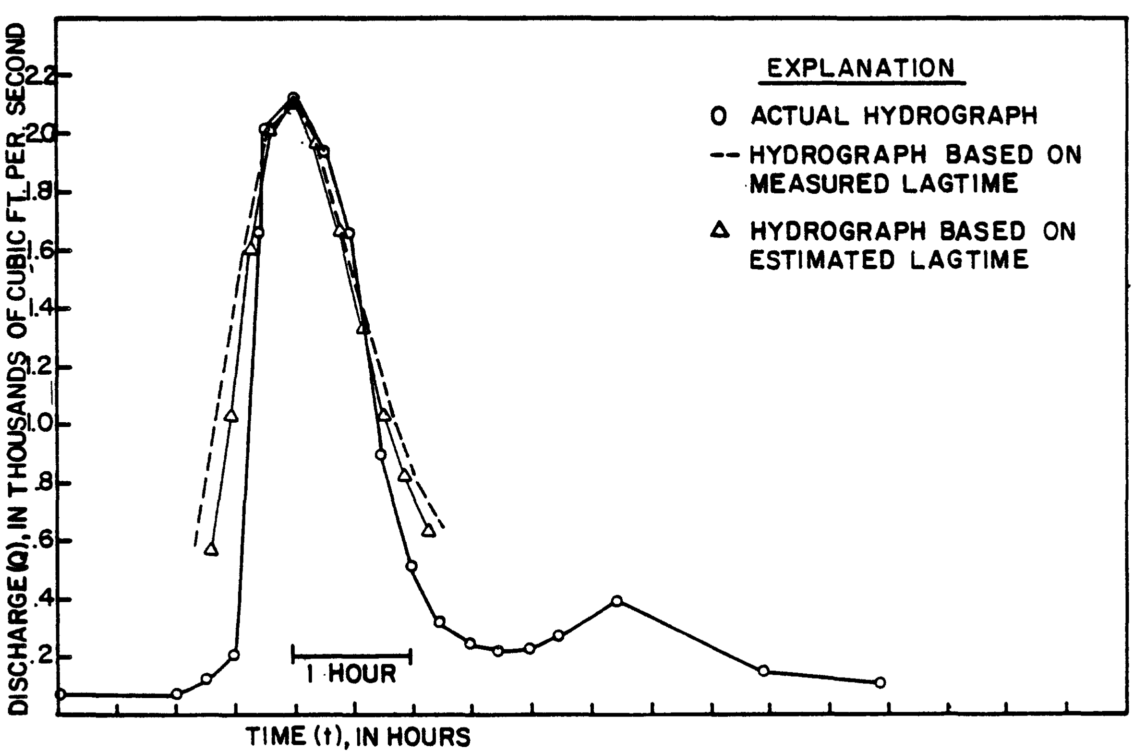

Figure 3. Cobbs Creek at U.S. Hwy. 1, at Philadelphia Pa. (01475530), for storm of July 13, 1975.

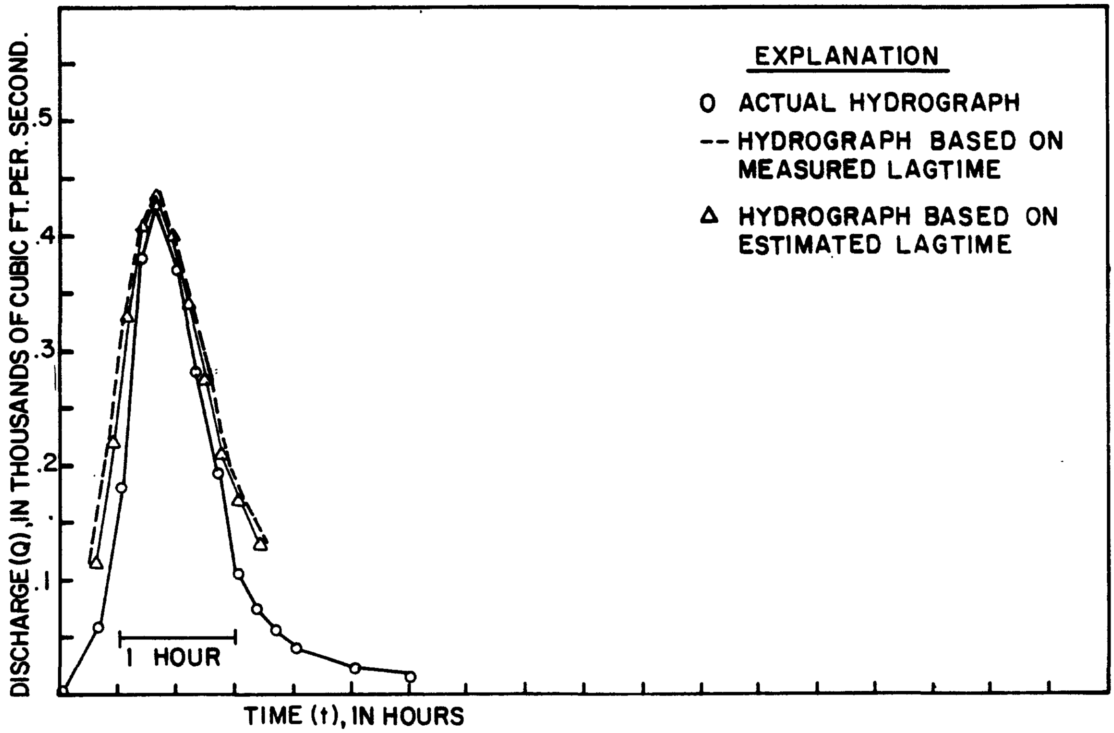

Figure. 4. Conley Creek near Forest Park, Ga. (02203884), for storm of July 2, 1974.

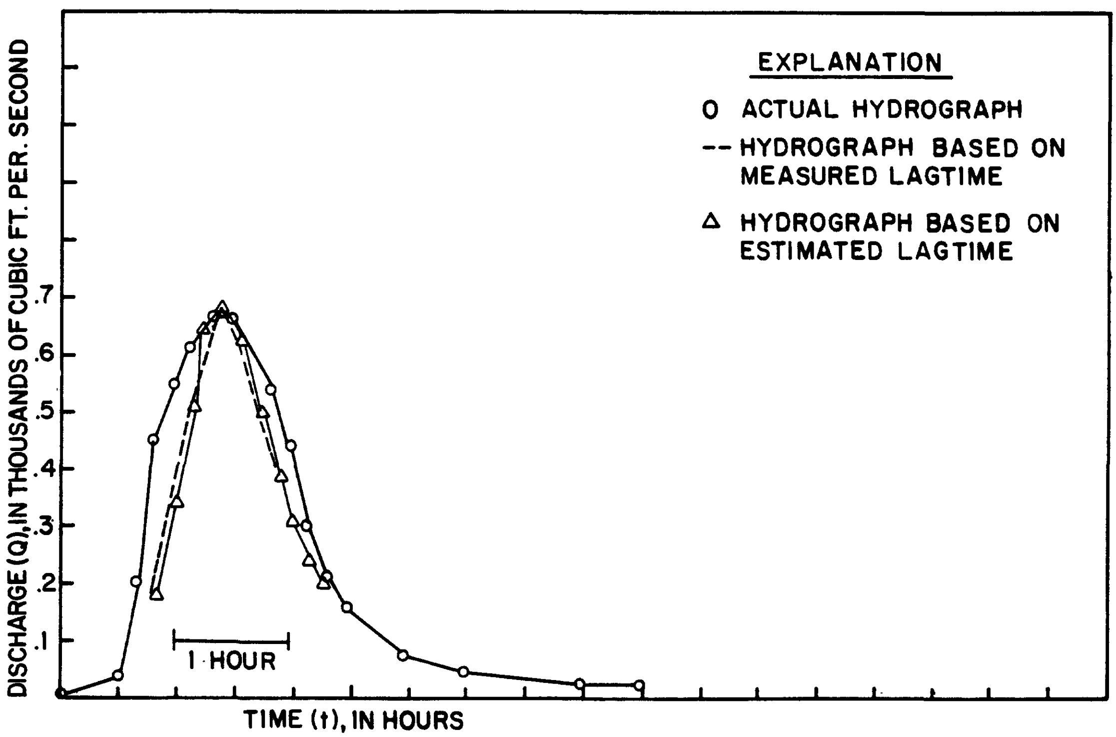

Figure 5. Conley Creek near Forest Park, Ga. (02203884), for storm of January 10, 1975.

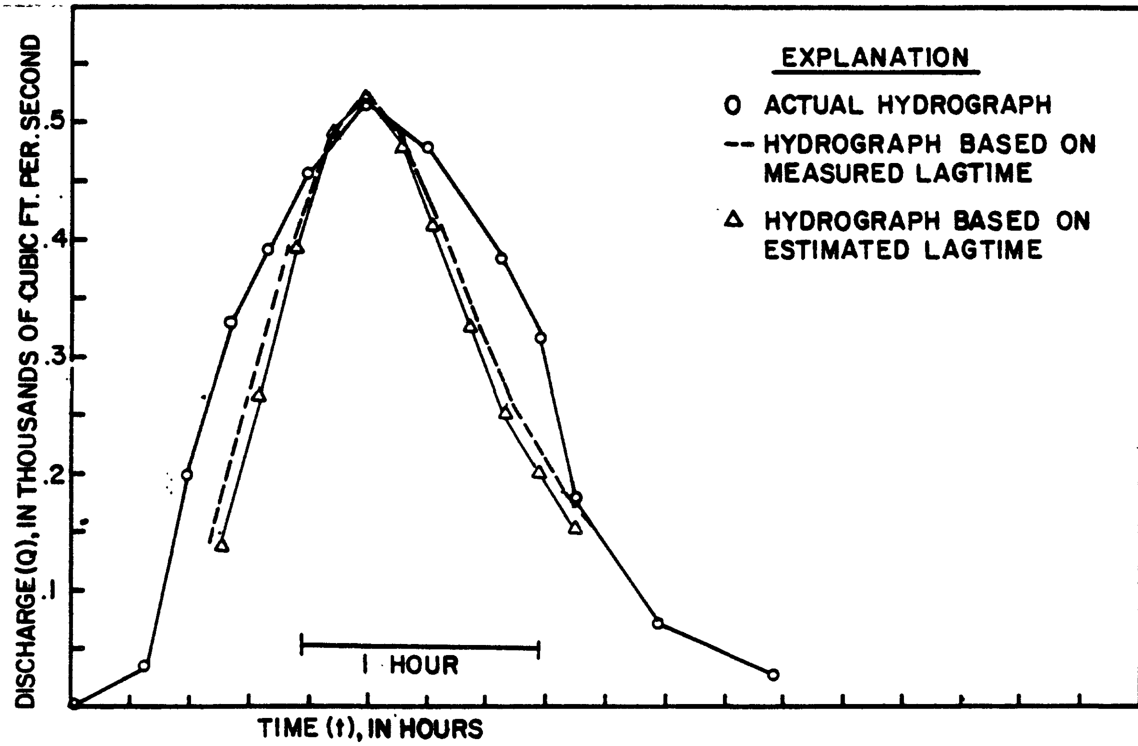

Figure 6. North Fork Peachtree Creek tributary near Atlanta, Ga. (02336102), for storm of June 20, 1973.

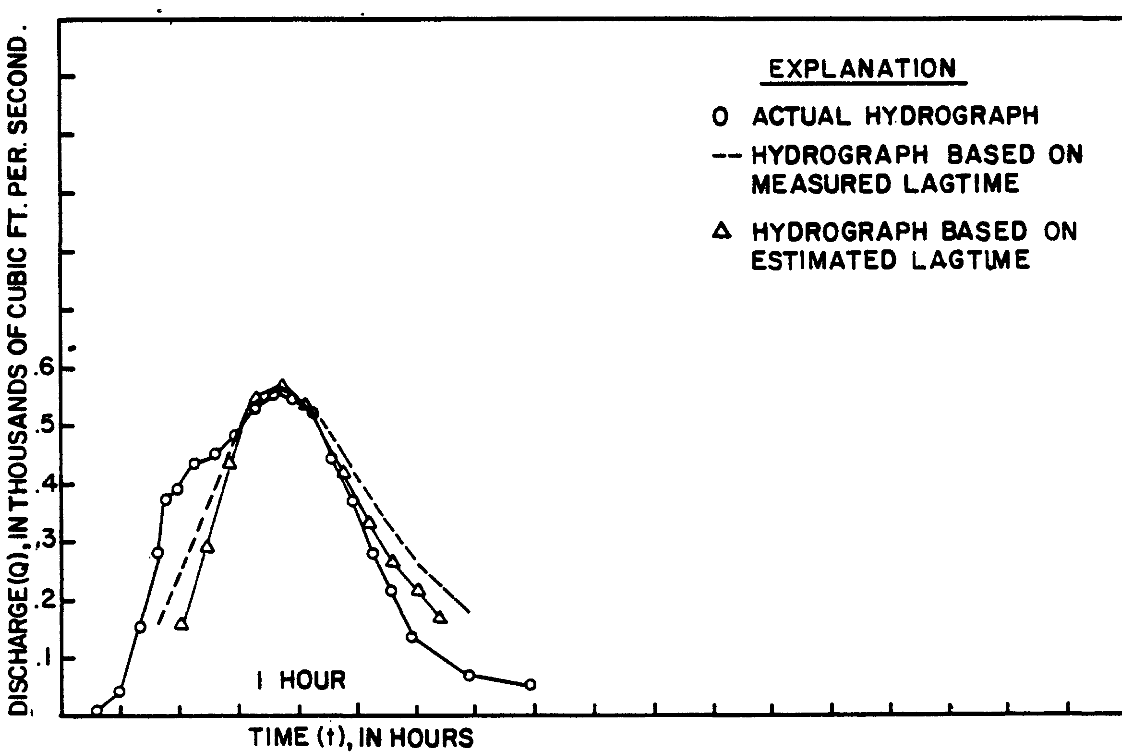

Figure 7. Cobbs Creek near Atlanta, Ga. (02203870), for storm of June 19, 1975.

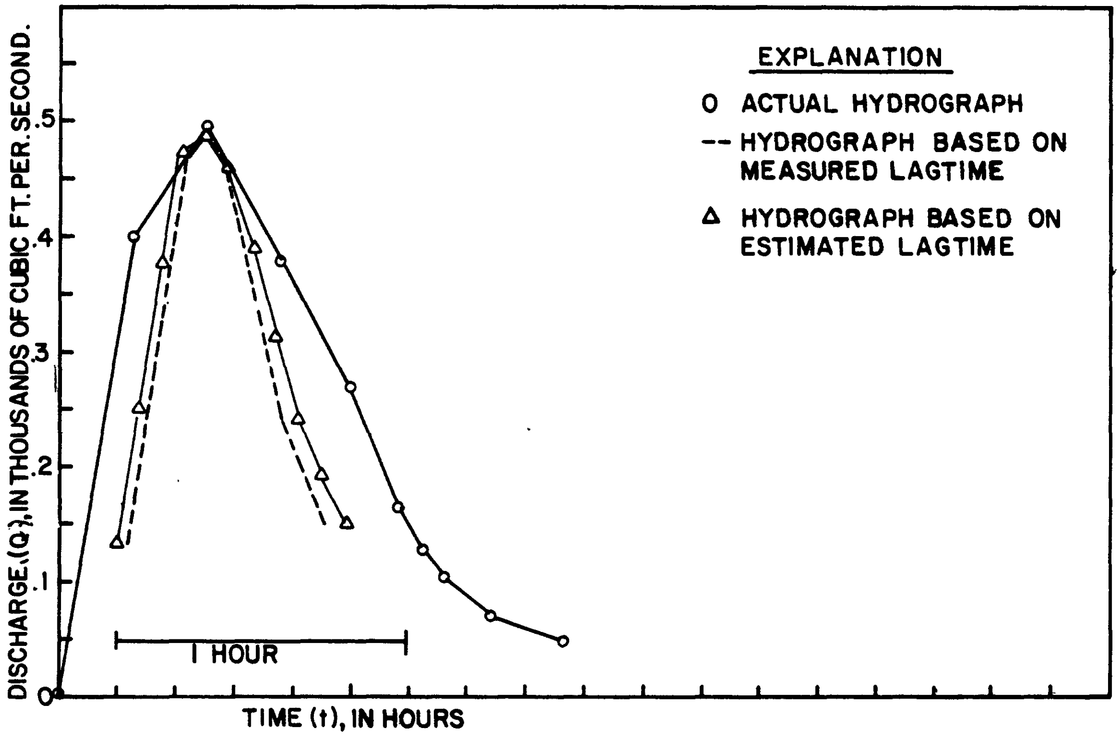

Figure 8. Shotwall Creek at Hwy. 304, near Ellisville, Mo. (06935800), for storm of July 23, 1973.

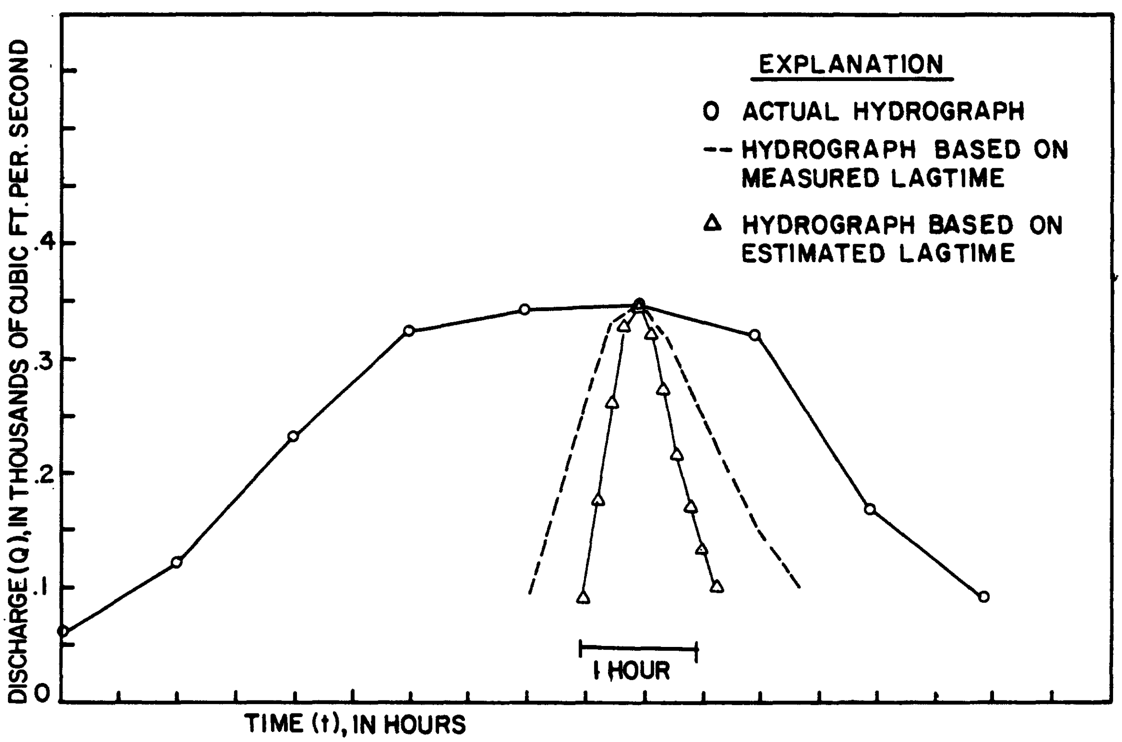

Figure 9. Fanno Creek at Portland, Oreg. (14206900), for storm of December 13, 1977.

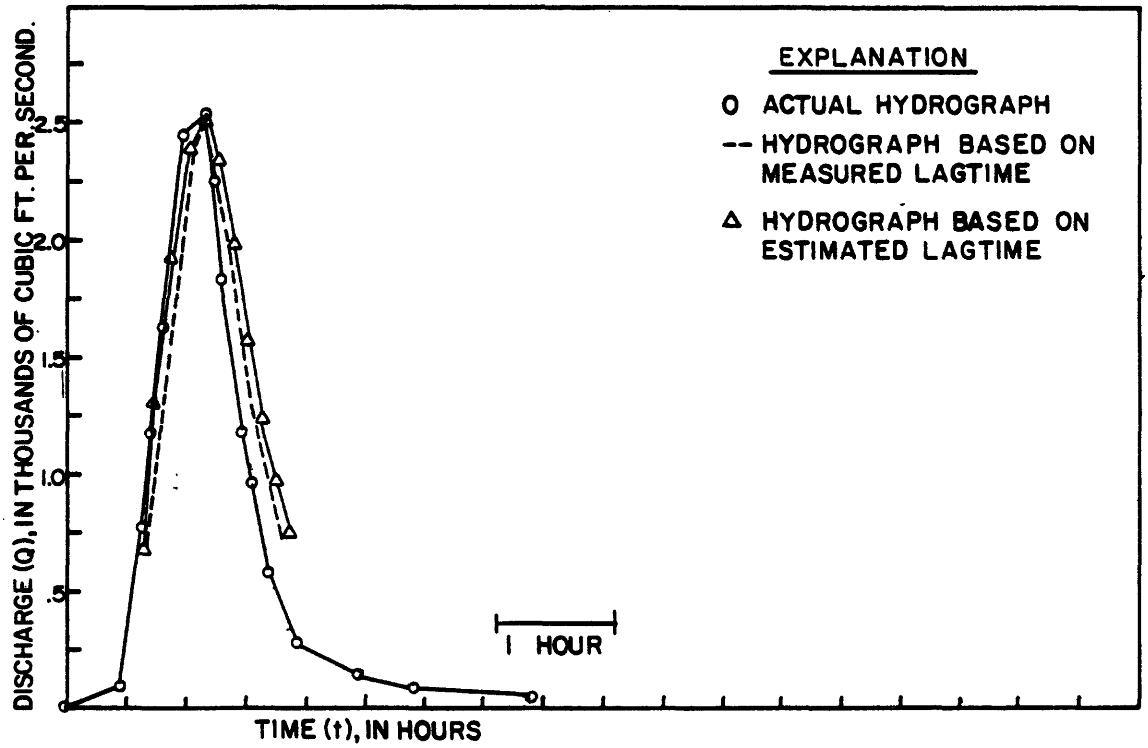

Figure 10. Paddock Creek at Lindbergh Blvd., at St. Louis, Mo. (06936380), for storm of July 23, 1973.

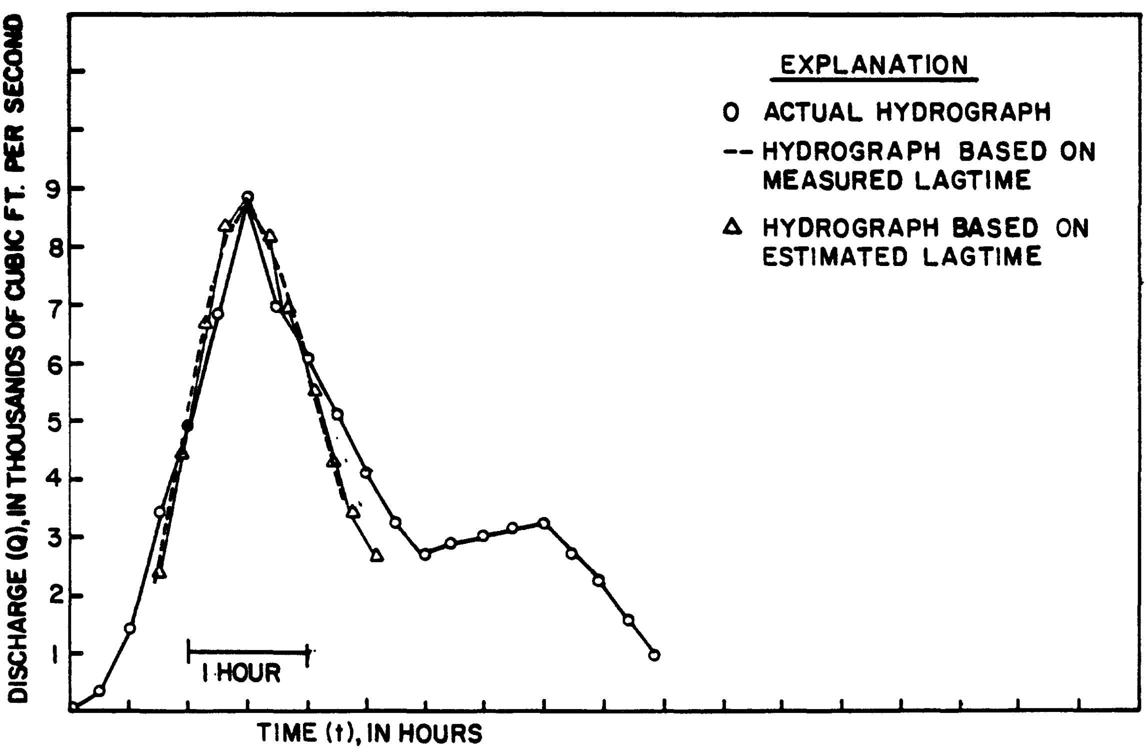

Figure 11. Turtle Creek at Dallas, Tex. (08056500), for storm of May 6-7, 1969.

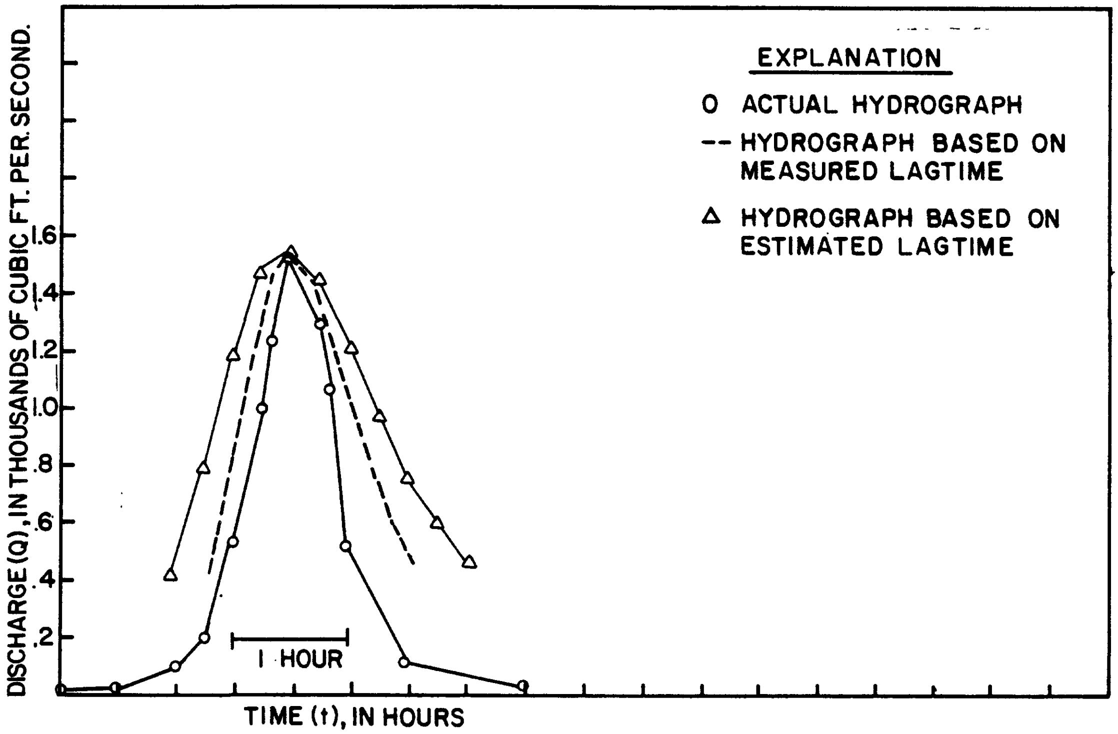

Figure 12. Watkins Creek at Coal Bank Road near St. Louis, Mo. (07002000), for storm of April 21, 1972

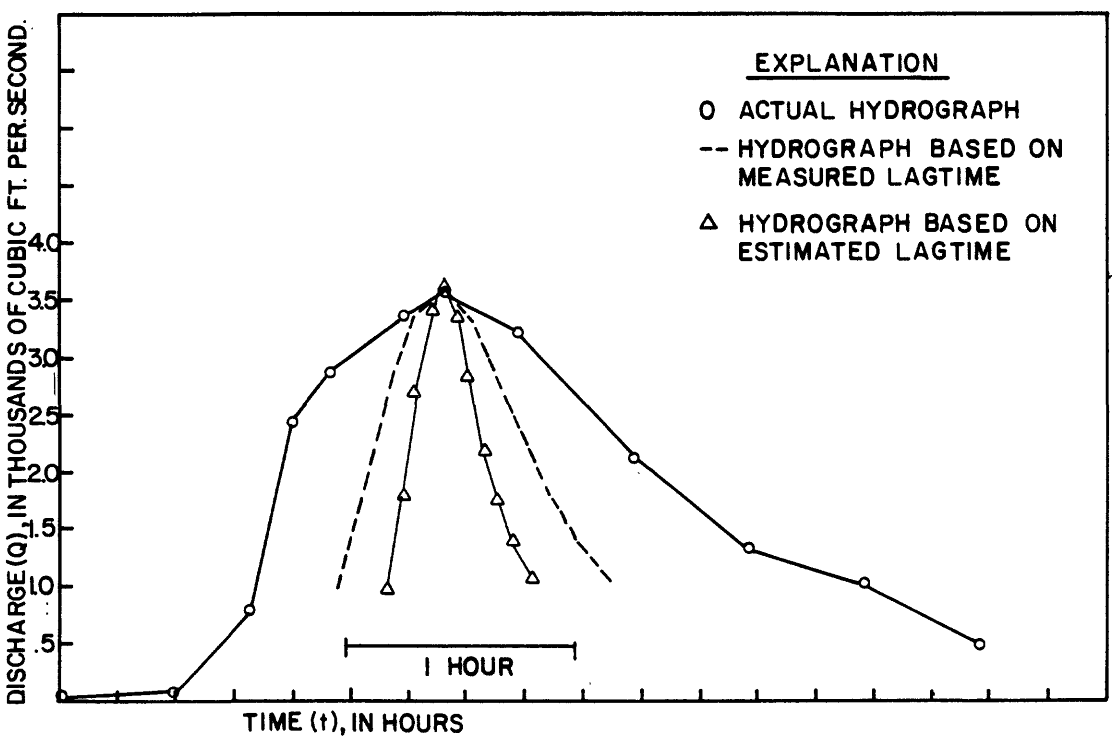

Figure 13. Deep Fork at Portland Ave., at Oklahoma City, Okla. (07242200), for storm of November 2, 1974.

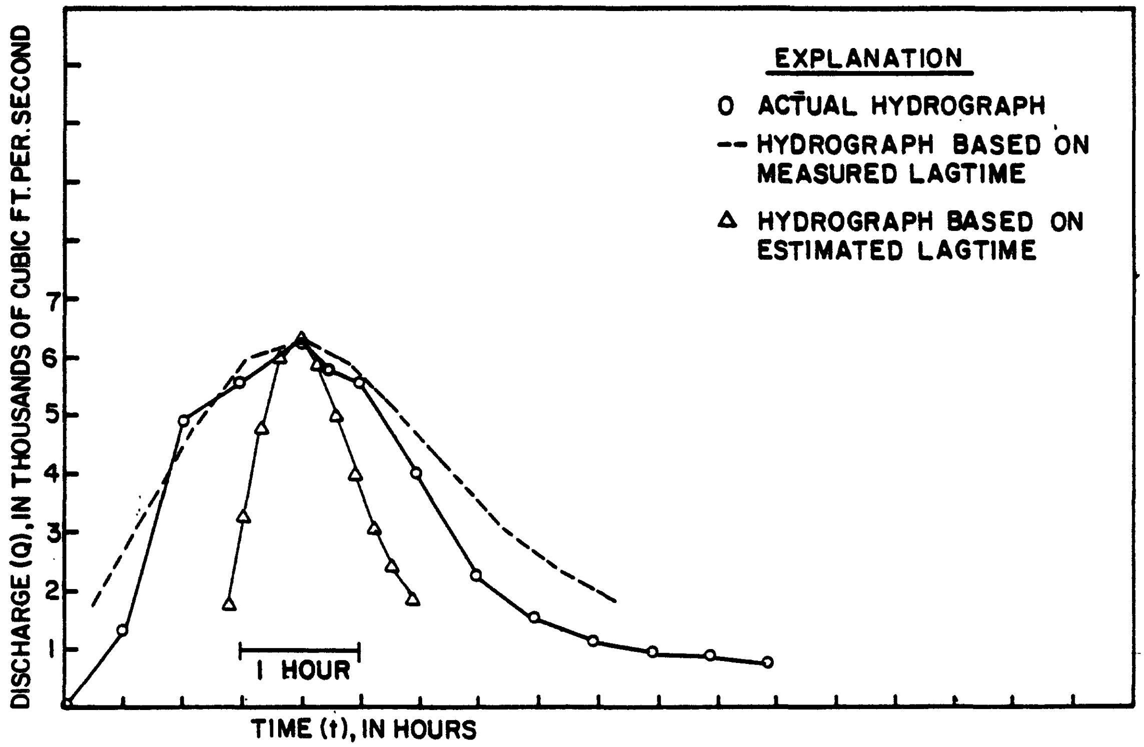

Figure 14. Joes Creek at Dallas, Tex.(08055600), for storm of April 28,1966.

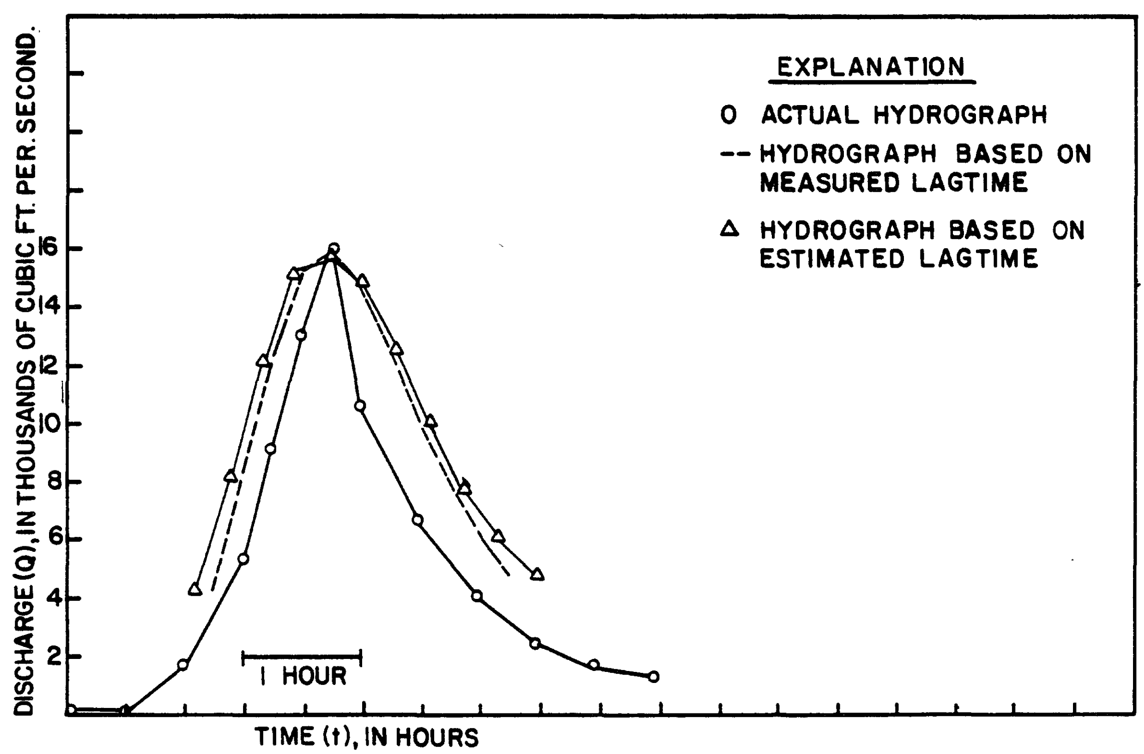

Figure 15. Bachman Branch at Dallas, Tex.(08055700), for storm of April 28,1966.

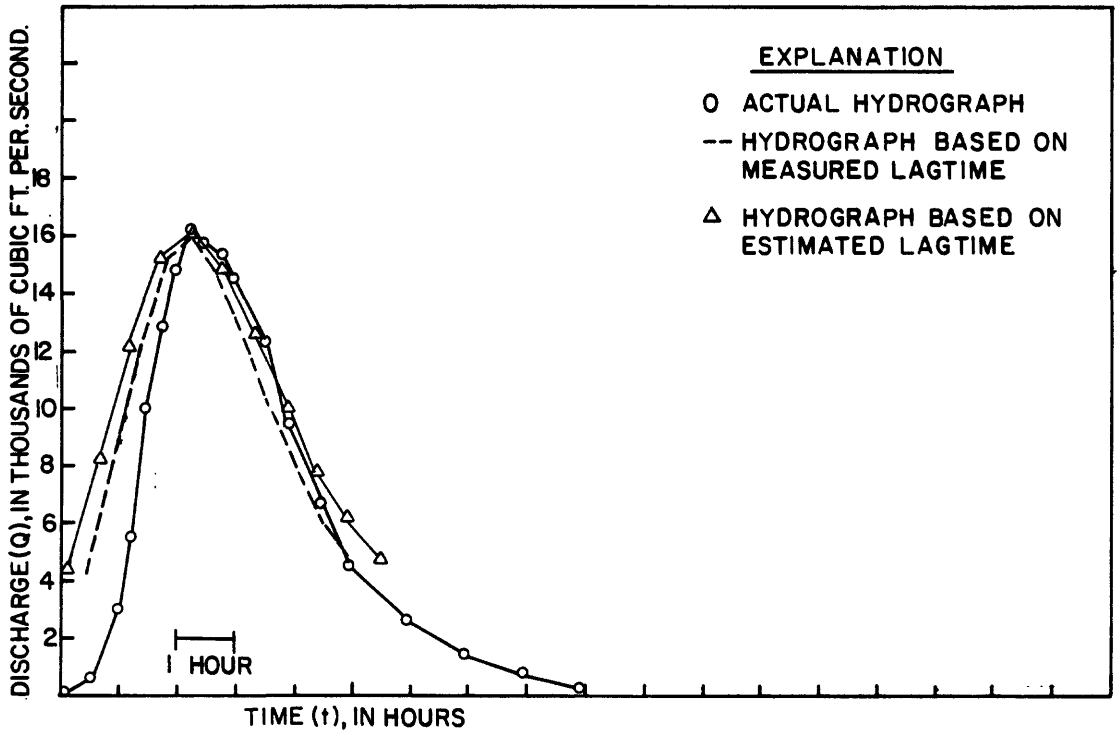

Figure. 16. Coldwater Creek at Old Halls Ferry Rd. near St. Louis, Mo. (06936460), for storm of September 7, 1972.

Table 3.—Relation of discharge ratios to hydrograph width ratios

Discharge ratios Q/Q_p	Width ratios W/LT
1.0	0
.9	.32
.8	.46
.7	.59
.6	.72
.5	.86
.4	1.01
.3	1.23

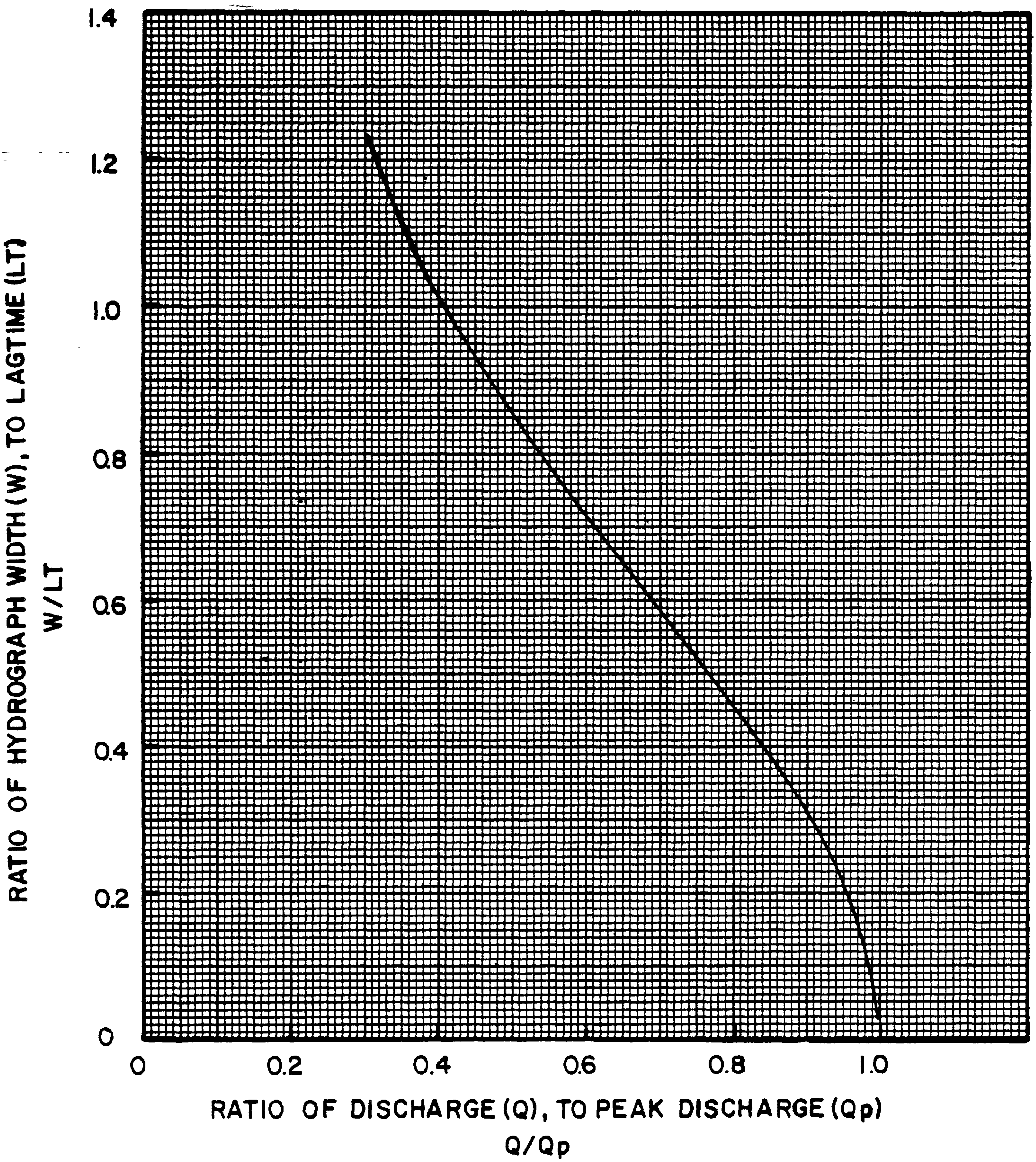

Figure 17. Hydrograph-width relation for dimensionless hydrograph.

Table 4.—Hydrograph-width comparisons

Location and date of storm	Peak Discharge Q_p (ft^3/s)	Width of observed hydrograph, at $Q = 0.75\ Q_p$ (hr)	Width of computed hydrograph, at $Q = 0.75\ Q_p$ (hr)	Difference in width (percent)
01475530 Cobbs Creek at U.S. Hwy. 1, at Philadelphia, Pa. 07-13-75	2,110	0.82	0.77	-6
02203884 Conley Creek near Forest Park, Ga. 07-02-74	429	.44	.61	+39
02203884 Conley Creek near Forest Park, Ga. 01-10-75	673	.97	.58	-40
02203870 Cobbs Creek near Atlanta, Ga. 06-19-75	566	1.26	.97	-23
02336102 North Fork Peachtree Creek tributary near Atlanta, Ga. 06-20-73	522	.98	.64	-35
06935800 Shotwell Creek at Hwy. 340, near Ellisville, Mo. 07-23-73	489	.53	.35	-34
06936380 Paddock Creek at Lindbergh Blvd., at St. Louis, Mo. 07-23-73	2,540	.43	.54	+26
06936460 Coldwater Creek at Old Halls Ferry Rd., near St. Louis, Mo. 09-07-72	16,000	1.88	2.32	+23

Table 4.—Hydrograph-width comparisons—Continued

Location and date of storm	Peak Discharge Q_p (ft^3/s)	Width of observed hydrograph, at $Q = 0.75\ Q_p$ (hr)	Width of computed hydrograph, at $Q = 0.75\ Q_p$ (hr)	Difference in width (percent)
07002000 Watkins Creek at Coal Bank Rd., near St. Louis, Mo. 04-21-72	1,540	.48	1.11	+131
07242200 Deep Fork at Portland Ave., at Oklahoma City, Okla. 11-02-74	3,600	1.14	.28	-75
08055600 Joes Creek at Dallas, Tex. 04-28-66	6,350	1.80	.70	-61
08055700 Bachman Branch at Dallas, Tex. 04-28-66	16,000	.48	1.20	+150
08056500 Turtle Creek at Dallas, Tex. 05-6&7-69	8,800	.64	.79	+23
08057200 White Rock Creek above Greenville Ave. at Dallas, Tex. 09-21-64	38,100	4.08	4.15	+2
14206900 Fanno Creek at Portland, Oreg. 12-13-77	351	4.08	.50	-88

indication of the true standard error. The errors are about equally divided between plus and minus, thus indicating little or no bias. Hydrograph width errors are comparable to errors in estimating basin lagtime.

HYDROGRAPH-VOLUME RELATION

Estimating Flood Volume

As part of this study, runoff volumes were related to flood peaks and drainage basin characteristics by use of linear multiple-regression techniques. Two hundred and seventy-one (271) storms from 55 stations located in Pennsylvania, Missouri, Oregon, Texas, and Oklahoma were used in the regression analysis. The watershed drainage area, lagtime, and flood peak discharge were included as independent variables and the following equation was derived:

$$V = 0.0142(A)^{-0.75}(LT)^{0.63}(Q_p)^{0.72} \quad \text{(standard error of regression} = \pm\ 62.8 \text{ percent)} \qquad (9)$$

where V = runoff volume, in inches (in.),
A = contribution drainage area, in square miles (mi^2),
LT = lagtime, in hours (hr), and
Q_p = peak discharge, in cubic feet per second (ft^3/s).

Equation 9 is useful for estimating flood volumes associated with peak discharges of selected recurrence intervals. These can be used where storage may be part of the design criteria.

Runoff volumes can also be estimated by computing the volume of the synthesized hydrographs described in previous parts of this report. This calculation requires that the rising and falling limbs of the hydrograph be extrapolated to zero discharge. The volume is then computed by summing the discharge ordinates at a given time interval, and converting the sum to runoff in inches.

Comparison of Estimated and Observed Volumes

Runoff volumes for estimated (synthesized) hydrographs and for observed hydrographs for the 14 sample basins used earlier in this report are compared in table 5. Also included in table 5 are volumes estimated from equation 9, using lagtime, LT, estimated from equation 1. This is a small sample for comparison purposes, but is typical of the results to be expected. From an analysis of the hydrographs, inaccuracies in lagtime probably cause the most error in estimating the hydrograph volumes. Measured lagtime provides the best estimate of runoff volumes and, conversely, lagtime estimated from basin characteristics is subject to greater error.

Table 5.—Runoff volume comparisons

Location and date of storm	Observed volume (in.)	Volume of hydrograph based on measured LT (in.)	Percent different from observed	Volume of hydrograph based on estimated LT (in.)	Percent different from observed	Volume estimated from equation 9 (in.)	Percent different from observed
01475530 Cobbs Creek at U.S. Hwy. 1, at Philadelphia, Pa. 01-13-75	1.46	1.17	-20	0.99	-32	1.37	-6
02203884 Conley Creek near Forest Park, Ga. 07-02-74	.33	.43	+30	.39	+18	.74	+124
02203884 Conley Creek near Forest Park, Ga. 01-10-75	.91	.68	-25	.62	-32	1.03	+13
02203870 Cobbs Creek near Atlanta, Ga. 06-19-75	.47	.50	+6	.42	-11	.73	+56
02336102 North Fork Peachtree Creek tributary near Atlanta, Ga. 06-20-73	.59	.48	-19	.45	-24	.80	+36
06935800 Shotwell Creek at State Hwy. 340, near Ellisville, Mo. 07-23-73	1.09	.50	-54	.60	-45	1.08	-1
06936380 Paddock Creek at Lindbergh Blvd., at St. Louis, Mo. 07-23-73	1.35	1.34	-1	1.52	+13	1.96	+45
06936460 Coldwater Creek at Old Halls Ferry Rd., near St. Louis, Mo. 09-07-72	2.05	2.32	+13	2.78	+36	2.45	+20
07002000 Watkins Creek at Coal Bank Rd., near St. Louis, Mo. 04-21-72	.35	.54	+54	.79	+126	1.12	+221
07242200 Deep Fork at Portland Ave., at Oklahoma City, Okla. 11-02-74	3.89	1.81	-54	.99	-75	1.53	-61
08055600 Joes Creek at Dallas, Tex. 04-28-66	4.09	4.66	+14	1.69	-59	2.01	-51
08055700 Bachman Branch at Dallas, Tex. 04-28-66	4.41	5.11	+16	5.65	+28	4.52	+2
08056500 Turtle Creek at Dallas, Tex. 05-6&7-69	4.44	2.56	-42	2.58	-42	2.68	-40
08057200 White Rock Creek abv. Greenville, Ave., at Dallas, Tex. 09-21-64	8.24	3.58	-57	7.00	-15	4.45	-46
14206900 Fanno Creek at Portland, Oreg. 12-13-77	1.89	.43	-77	.21	-89	.48	-75

APPLICATION OF TECHNIQUE

Stepwise Procedure

A step-by-step procedure described below assists the user in applying the techniques for estimating flood hydrograph properties as presented in this report. In addition, an example is given to demonstrate the technique. The stepwise procedure is as follows:

1. From the best available topographic maps, determine the drainage area, main-channel length, and main-channel slope of the basin.

2. Compute the equivalent rural peak discharge from the applicable U.S. Geological Survey flood-frequency report.

3. Compute the basin development factor. This parameter, defined by Sauer and others (1981), can be easily determined using drainage maps and making field inspections of the drainage basin.

4. Compute the urban peak discharge using the appropriate equation (2-8), given in this report.

5. Compute the lagtime from equation 1, given in this report.

6. For some situations an entire hydrograph may not be needed. An estimate of the width of the hydrograph for a specific discharge, Q, may be enough to estimate the time that flow will inundate a specific feature, such as a road embankment. This time, W, can be determined by calculating the ratio Q/Q_p; using Q/Q_p to determine a value of W/LT from figure 3, in this report, and multiplying the lagtime, LT, by the ratio W/LT to obtain the hydrograph width or time that flow is greater than the specified Q. The recurrence interval corresponds to the recurrence interval of Q_p.

7. The coordinates of the runoff hydrograph can be computed by multiplying the value of lagtime by the time ratios and the value of peak discharge by the discharge ratios presented in table 1 of this report.

Example Problem

The procedure is illustrated in an example to compute a hydrograph associated with the 100-year discharge estimated for Little Sugar Creek at Charlotte, N.C.

1. The drainage area (A) is determined as 41 mi^2 and the basin length (L) and slope (SL) are determined to be 11 mi and 13.10 ft/mi, respectively.

2. The equivalent rural peak discharge (RQ100) for the 100-year recurrence-interval flood is 7,460 ft^3/s (Jackson, 1976).

3. The basin development factor (BDF) is computed to be 9.

4. Using equation 7, the urban peak discharge for the 100-year recurrence-interval flood (UQ100) is estimated to be:

$$UQ100 = 7.70A^{0.15}(13-BDF)^{-0.32}RQ100^{0.82}$$
$$= (7.70)(41)^{0.15}(13-9)^{-0.32}(7460)^{0.82}$$
$$= 12{,}900 \text{ ft}^3/\text{s}.$$

5. Using equation 1, lagtime (LT) is estimated to be:

$$LT = 0.85(L/\sqrt{SL})^{0.62}(13-BDF)^{0.47}$$
$$= (0.85)(11/\sqrt{13.1})^{0.62}(13-9)^{0.47}$$
$$= 3.2 \text{ hr.}$$

6. If an estimate were needed for a time of road overflow for a discharge of 9,000 ft^3/s, compute it as follows:
   a. $Q/Q_p = 9{,}000/12{,}900 = 0.70$
   b. from figure 17, $W/LT = 0.59$
   c. lagtime, LT = 3.2 hr., from step 5.
   d. road overflow time $= (W/LT)(LT)$
      $= (0.59)(3.2)$
      $= 1.9$ hr.

7. The coordinates of the runoff hydrograph are shown below:

t/LT (from table 1)	xLT (from step 5)	= time (hr)	Q_t/Q_p (from table 1)	xQ_p (from step 4)	= Discharge (ft /s)
0.45	3.2	1.4	0.27	12,900	3,500
.50	3.2	1.6	.37	12,900	4,800
.55	3.2	1.8	.46	12,900	5,900
.60	3.2	1.9	.56	12,900	7,200
.65	3.2	2.1	.67	12,900	8,600
.70	3.2	2.2	.76	12,900	9,800
.75	3.2	2.4	.86	12,900	11,100
.80	3.2	2.6	.92	12,900	11,900
.85	3.2	2.7	.97	12,900	12,500
.90	3.2	2.9	1.00	12,900	12,900
.95	3.2	3.0	1.00	12,900	12,900
1.00	3.2	3.2	.98	12,900	12,600
1.05	3.2	3.4	.95	12,900	12,200
1.10	3.2	3.5	.90	12,900	11,600
1.15	3.2	3.7	.84	12,900	10,800
1.20	3.2	3.8	.78	12,900	10,100
1.25	3.2	4.0	.71	12,900	9,200
1.30	3.2	4.2	.65	12,900	8,400
1.35	3.2	4.3	.59	12,900	7,600
1.40	3.2	4.5	.54	12,900	7,000
1.45	3.2	4.6	.48	12,900	6,200
1.50	3.2	4.8	.44	12,900	5,700
1.55	3.2	5.0	.39	12,900	5,000
1.60	3.2	5.1	.36	12,900	4,600
1.65	3.2	5.3	.32	12,900	4,100
1.70	3.2	5.4	.30	12,900	3,900

Effects of In-Channel Storage

The equations for peak discharge and lagtime, developed by Sauer and others (1981) and used in this report, were estimated from watersheds without significant in-channel storage. These equations are not applicable to sites where in-channel storage is significant. However, the dimensionless hydrograph presented in this report can be used for sites with in-channel storage, provided suitable estimates of peak discharge and lagtime are available which account for the effects of in-channel storage.

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