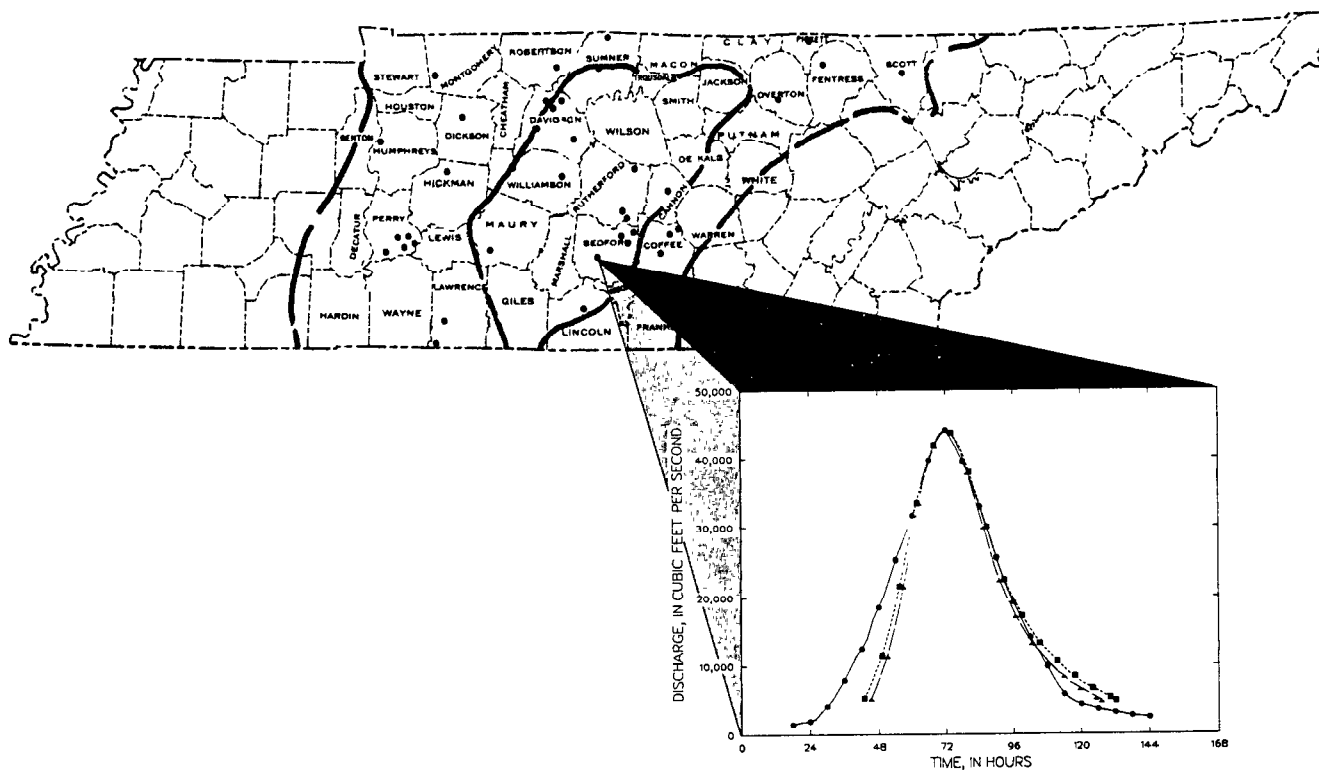


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# TECHNIQUES FOR SIMULATING FLOOD HYDROGRAPHS AND ESTIMATING FLOOD VOLUMES FOR UNGAGED BASINS IN CENTRAL TENNESSEE



U.S. GEOLOGICAL SURVEY

Prepared in cooperation with the  
TENNESSEE DEPARTMENT OF TRANSPORTATION

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Clarence H. Robbins

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U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 86-4192

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Nashville, Tennessee  
1986

UNITED STATES DEPARTMENT OF THE INTERIOR

DONALD PAUL HODEL, Secretary

GEOLOGICAL SURVEY

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## CONVERSION FACTORS

For readers who may prefer to use the International System of Units (SI) rather than the inch-pound units used herein, the conversion factors are listed below:

<u>Multiply</u>	<u>By</u>	<u>To obtain</u>
inch (in.)	2.540	centimeter (cm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi <sup>2</sup> )	2.590	square kilometer (km <sup>2</sup> )
cubic foot per second (ft <sup>3</sup> /s)	0.02832	cubic meter per second (m <sup>3</sup> /s)

# TECHNIQUES FOR SIMULATING FLOOD HYDROGRAPHS AND ESTIMATING FLOOD VOLUMES FOR UNGAGED BASINS IN CENTRAL TENNESSEE

Clarence H. Robbins

## ABSTRACT

A dimensionless hydrograph developed for a variety of basin conditions in Georgia was tested for its applicability to central Tennessee streams by comparing it to a similar dimensionless hydrograph developed for central Tennessee streams. Statistical analyses were performed by comparing simulated (or computed) hydrographs, derived by application of the Georgia-study dimensionless hydrograph and the central Tennessee dimensionless hydrograph, with 163 observed hydrographs from 38 stations having a wide range of drainage area sizes and basin conditions, at 50 and 75 percent of their peak flow widths. Test results indicate the two dimensionless hydrographs are essentially the same. Using the Georgia-study dimensionless hydrograph, the standard error of estimate was + 21.2 percent at the 50 percent of peak flow width and + 24.8 percent at the 75 percent of peak flow width. Study results indicate the Georgia dimensionless hydrograph is applicable to central Tennessee streams.

Equations for rural and urban basin lagtime were derived from multiple-regression analyses that relate lagtime to physical basin characteristics. At the 95-percent confidence level, channel length was significant for the rural basin equation and channel length and percentage of impervious area were significant for the urban basin equation. A regression equation that related flood volumes to drainage area size, peak discharge, and basin lagtime also was developed. The flood-hydrograph and flood-volume techniques are useful for estimating a typical (average) flood hydrograph and volume for any specified recurrence interval peak discharge at any ungaged stream site draining areas less than 500 square miles in central Tennessee.

## INTRODUCTION

Flood hydrographs and flood volumes are often needed for the design of highway drainage structures and embankments or where storage of floodwater or flood prevention is part of the design. Additionally, hydrographs may be necessary to estimate the length of time of inundation of specific features, for example, roads and bridges.

A flood of sufficient magnitude for structural design analysis may have occurred before systematic streamflow data collection began, or the site of interest is ungaged and hydrograph data are not available. Under these conditions, a typical or design hydrograph may be simulated using one, or a combination of several, traditional hydrograph estimation methods. Each of the traditional methods have inherent characteristics, data requirements, and basin characteristics or coefficients which must be estimated or calculated. Most methods rely on the unit hydrograph whereby design hydrographs are computed by

convolution of the unit hydrograph with rainfall excess. Therefore, rainfall data and methods for estimating rainfall excess are necessary for use of any of the unit hydrograph methods.

A need exists for a simple, direct-approach method to estimate the flood hydrograph, volume, and width associated with a peak discharge of specific recurrence interval (a design discharge). Recently, a direct-approach method was developed for streams in Georgia (Inman, 1986). However, the applicability of this direct-approach method for the Georgia streams to other areas was unknown. This report describes the results of a study to determine the applicability of Inman's method to central Tennessee streams and provides techniques for estimating flood hydrographs (shape, volume, and width) for ungaged basins draining areas less than 500 mi<sup>2</sup> in central Tennessee. Future studies will determine the applicability of Inman's method to west and east Tennessee streams. This study was conducted in cooperation with the Tennessee Department of Transportation.

### INMAN'S HYDROGRAPH SIMULATION METHOD

Inman (1986) used 355 actual (observed) streamflow hydrographs from 80 basins 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<sup>2</sup>. 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 hydrograph 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. The drainage areas of these stations ranged from 20 to 500 mi<sup>2</sup>. Inman (1986) reported that at the 50 percent of peak flow width, the standard error of estimate was  $\pm 39.5$  percent; and at the 75 percent of peak flow width, the standard error of estimate was  $\pm 43.6$  percent.

Inman (1986) performed a second verification to assess the total or accumulative 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. Inman (1986) reported standard errors of prediction of  $\pm 51.7$  and  $\pm 57.1$  percent at the 50 and 75 percent of peak flow widths, respectively.

On the basis that Inman's basic dimensionless hydrograph was developed and tested for a variety of conditions (including urban, rural, mountainous, coastal plain, and small and large drainage basins), it was theorized that it may be applicable to streams in central Tennessee. The time and discharge coordinates of Inman's (1986) dimensionless hydrograph are listed in table 1.

### TESTING INMAN'S DIMENSIONLESS HYDROGRAPH ON CENTRAL TENNESSEE STREAMS

Inman's dimensionless hydrograph was tested by comparing it to a similar dimensionless hydrograph developed for central Tennessee streams. The test involved several phases and is described in detail in this section of the report.

A total of 245 observed hydrographs from 38 basins across central Tennessee (fig. 1) with drainage areas of 0.17 to 481 mi<sup>2</sup>, representing both rural and urban streamflow characteristics, were available for use in the test. However, only 163 observed hydrographs had matching rainfall data and were selected for use in this test. The 38 basins are located within hydrologic areas 2 and 3 in Tennessee, as defined by Randolph and Gamble (1976). Of the 38 basins, 21 (81 observed hydrographs) are in hydrologic area 2, and 17 (82 observed hydrographs) are in hydrologic area 3. A computer program (S.E. Ryan, U.S. Geological Survey, written commun., 1986) used by Inman (1986) for development of the dimensionless hydrograph and subsequent statistical analyses also was used for this analysis.

Unit hydrographs and lagtime were computed from the 163 observed hydrographs, and an average unit hydrograph and an average lagtime were computed for each of the 38 basins. These average unit hydrographs were 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. The 38 dimensionless hydrographs were then separated into their respective hydrologic areas (21 in area 2, and 17 in area 3). Representative dimensionless hydrographs from eight randomly selected basins in each area were combined to generate one typical (average) dimensionless hydrograph for each of the four generalized durations in each hydrologic area.

In each hydrologic area, the four generalized duration dimensionless hydrographs, average basin lagtimes, and peak discharges from the 163 observed hydrographs were used to generate simulated hydrographs for the corresponding observed hydrographs. The simulated hydrograph widths were compared with the widths of the observed hydrographs at 50 and 75 percent of peak flow. The dimensionless hydrograph based on the one-half lagtime duration provided the best fit of the observed data in both hydrologic areas. At the 50 and 75 percent of peak flow widths, standard errors of estimate were  $\pm 21.4$  and  $\pm 24.4$  percent, respectively, in hydrologic area 2 and  $\pm 22.2$  and  $\pm 26.0$  percent, respectively, in hydrologic area 3.

Inman's dimensionless hydrograph was then used with the average basin lagtimes, and peak discharges from the 163 observed hydrographs to generate simulated hydrographs for the observed hydrographs in each hydrologic area. The simulated hydrographs and the observed hydrographs were compared at 50 and 75 percent of their peak flow widths. At the 50 and 75 percent of peak flow widths, standard errors of estimate were  $\pm 20.8$  and  $\pm 24.1$  percent, respectively, in hydrologic area 2 and  $\pm 22.1$  and  $\pm 26.0$  percent, respectively, in hydrologic area 3. Based on the standard errors of estimate, the simulated



Table 1.--Time and discharge ratios of Inman's dimensionless hydrograph

Time ratio ( $t/LT$ )	Discharge ratio ( $Q_t/Q_p$ )
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

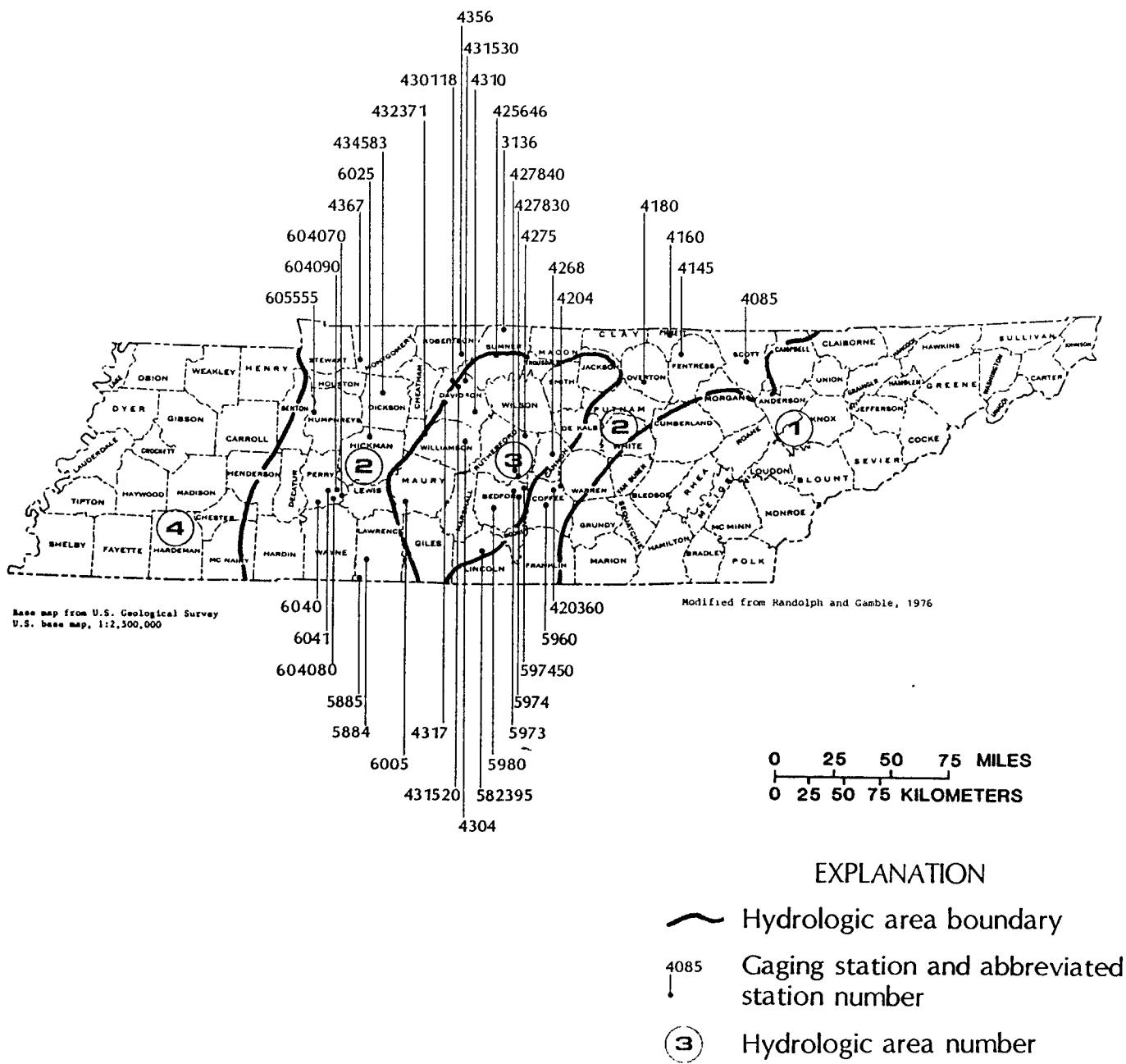


Figure 1.--Location of study area and gaging stations used in dimensionless hydrograph analysis.

hydrographs generated from Inman's dimensionless hydrograph had a slightly better fit of the observed data than the simulated hydrographs generated from the average dimensionless hydrograph for each hydrologic area.

The eight randomly selected dimensionless hydrographs from each hydrologic area were then combined to generate one typical (average) dimensionless hydrograph (based on 16 dimensionless hydrographs) for each of the four generalized durations for the entire area (hydrologic areas 2 and 3). Using the four generalized duration dimensionless hydrographs, average basin lagtimes, and peak discharges from the 163 observed hydrographs, simulated hydrographs were generated for the corresponding observed hydrographs and their widths were compared with the widths of the observed hydrographs at 50 and 75 percent of peak flow. As in the preceding test, the dimensionless hydrograph based on the one-half lagtime duration provided the best fit of the observed data. At the 50 and 75 percent of peak flow widths, standard errors of estimate were  $\pm 21.7$  and  $\pm 25.4$  percent, respectively.

Finally, Inman's dimensionless hydrograph, average basin lagtimes, and peak discharges from the 163 observed hydrographs were used to generate simulated hydrographs for the corresponding observed hydrographs. The simulated hydrographs and the observed hydrographs were compared at 50 and 75 percent of their peak flow widths. Standard errors of estimate were  $\pm 21.2$  and  $\pm 24.8$  percent at the 50 and 75 percent of peak flow widths, respectively. Based on the standard errors of estimate, the simulated hydrographs generated from Inman's dimensionless hydrograph had a slightly better fit of the observed data than the simulated hydrographs generated from the average dimensionless hydrograph developed from Tennessee data.

Based on the preceding tests, Inman's dimensionless hydrograph is applicable to streams in central Tennessee, and the average dimensionless hydrograph derived from 16 of the 38 central Tennessee basins is essentially the same as the dimensionless hydrograph derived from Georgia basins. Therefore, Inman's dimensionless hydrograph is the preferred one to use for simulating hydrographs in central Tennessee. A comparison of the two dimensionless hydrographs is shown in figure 2. At 50 and 75 percent of the ratio  $Q_t/Q_p$  widths, the Inman dimensionless hydrograph is 5.4 and 4.8 percent wider, respectively, than the one derived for central Tennessee. The difference between the two dimensionless hydrographs is probably due to the number of basins used to develop each dimensionless hydrograph.

#### Verification Of Dimensionless Hydrograph

A test was performed to assess the total or accumulative prediction error for large floods through the combined use of the Inman dimensionless hydrograph, estimated basin lagtimes, and discharges derived from regional flood-frequency equations. Randolph and Gamble (1976) provide a technique for estimating the peak discharge of a selected recurrence interval for rural streams in Tennessee, and Robbins (1984) provides a technique for estimating the peak discharge of a selected recurrence interval for urban basins draining areas less than 25 mi<sup>2</sup> in Tennessee. Equations for estimating basin lagtime were developed for central Tennessee streams as part of this study and are presented in a supplement at the end of this report.

This verification test used 36 observed hydrographs (not previously used) from 36 of the original 38 basins. These hydrographs were selected from those having the highest

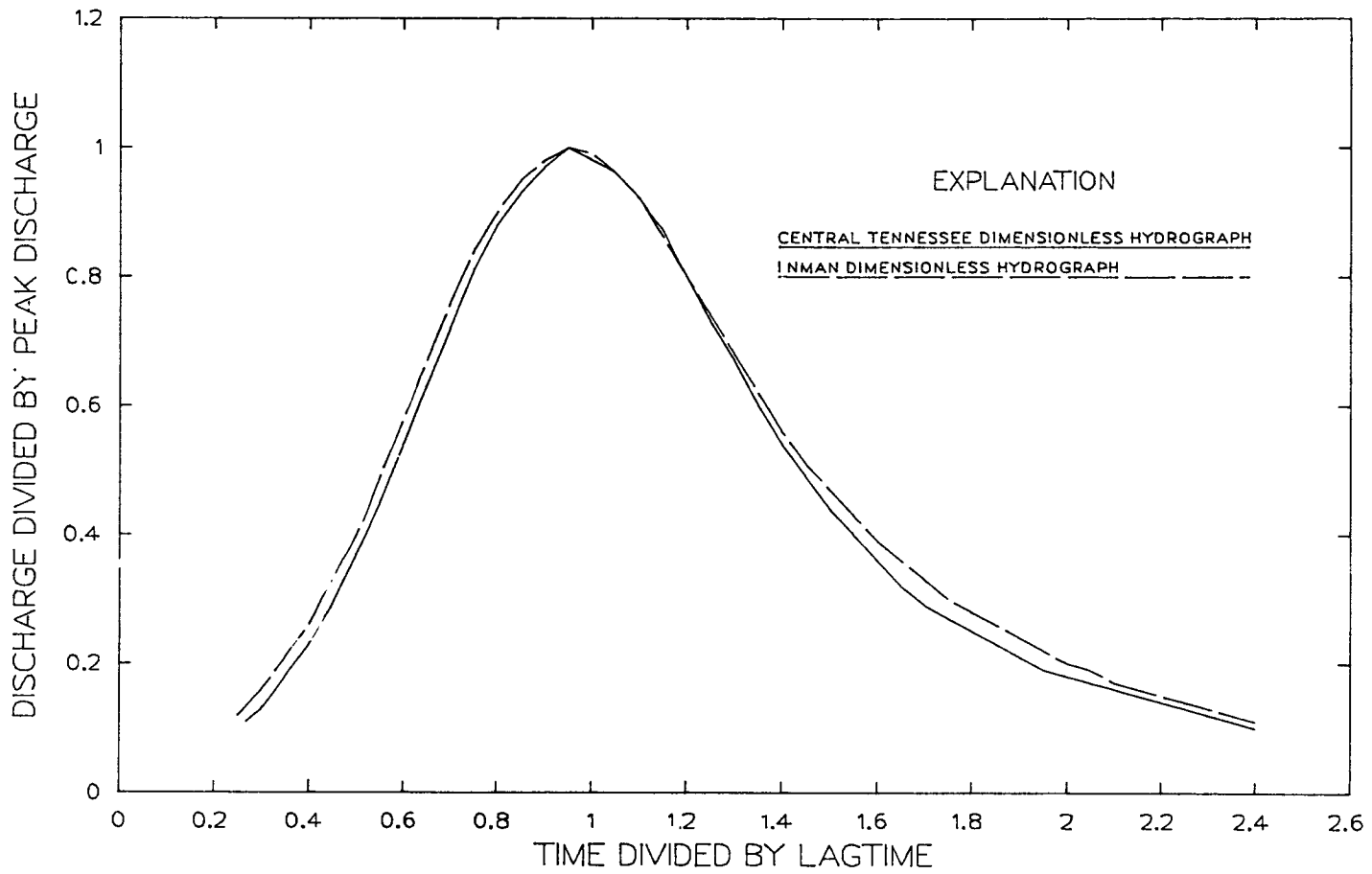


Figure 2.-- Comparison of Inman and central Tennessee dimensionless hydrographs.

peaks at each station, and where a station flood-frequency curve was available. The test was conducted as follows. The recurrence interval of each observed peak discharge was determined from its station-frequency curve. The appropriate regional flood-frequency equation, from Randolph and Gamble (1976) or Robbins (1984), was then used to estimate the corresponding peak discharge for this recurrence interval. The average recurrence interval of the 36 peaks was approximately 25 years. For each station, a basin lagtime was estimated from the appropriate regional basin lagtime equation (presented in a later section of this report). The estimated peak discharge, the estimated basin lagtime for each basin, and Inman's dimensionless hydrograph were then used to generate simulated flood hydrographs. A comparison of the simulated and observed hydrograph widths at 50 and 75 percent of peak flow showed standard errors of prediction of  $\pm 43.9$  and  $\pm 39.2$  percent, respectively.

Example comparisons between observed hydrographs and simulated hydrographs based on observed peak discharge and measured basin lagtime, and regression discharge and regression basin lagtime are shown in figures 3, 4, and 5. A comparison between observed and simulated hydrographs for an urban basin in the Nashville, Tenn., area is shown in figure 3. Similar comparisons for much larger basins in the rolling hills of central Tennessee are shown in figures 4 and 5. All three comparisons show good agreement between the observed and simulated hydrographs. Peak discharges of the simulated hydrographs based on regression (estimated) discharge ( $Q_p$ ) and regression (estimated) lagtime (LT) may not coincide with the observed peak discharges because the simulated hydrographs incorporate the error inherent in the regional flood-frequency relations. These errors are representative of the total error that might occur at an ungaged site.

### Bias

Two tests for bias were conducted, one for simulated versus observed hydrograph width, and the other for geographical bias. The width-bias test was conducted using the residuals (in percent) at 50 and 75 percent of peak flow for the 36 stations in the previous verification test. The mean error indicated a small positive error (simulated greater than observed) in the hydrograph widths at 50 percent of peak flow and a small negative error (simulated less than observed) in the hydrograph widths at 75 percent of peak flow. Additionally, there was a small positive error (regression greater than observed) in the comparison of peak discharge from regional regression equations and peak discharge from station frequency curves. The student's t-test indicated these errors were not statistically significant at the 0.01 level of significance, and therefore, the simulated hydrograph widths and regression discharges are not biased.

In the geographical-bias test, residual differences in simulated and observed hydrograph widths (in percent) at 50 and 75 percent of peak flow at each station were plotted on a map to evaluate the bias of the simulated hydrographs. Although the residual differences in widths varied considerably between some stations, no specific geographic trends could be detected. Results of both these tests indicate no bias in the results using the outlined procedures and support results reported by Inman (1986).

## REGIONALIZATION OF BASIN LAGTIME AND FLOOD VOLUME

### Estimating Basin Lagtime

Average basin lagtime is used as the principal time factor in the dimensionless hydrograph. Lagtime (LT) is generally considered to be constant for a basin (as long as basin

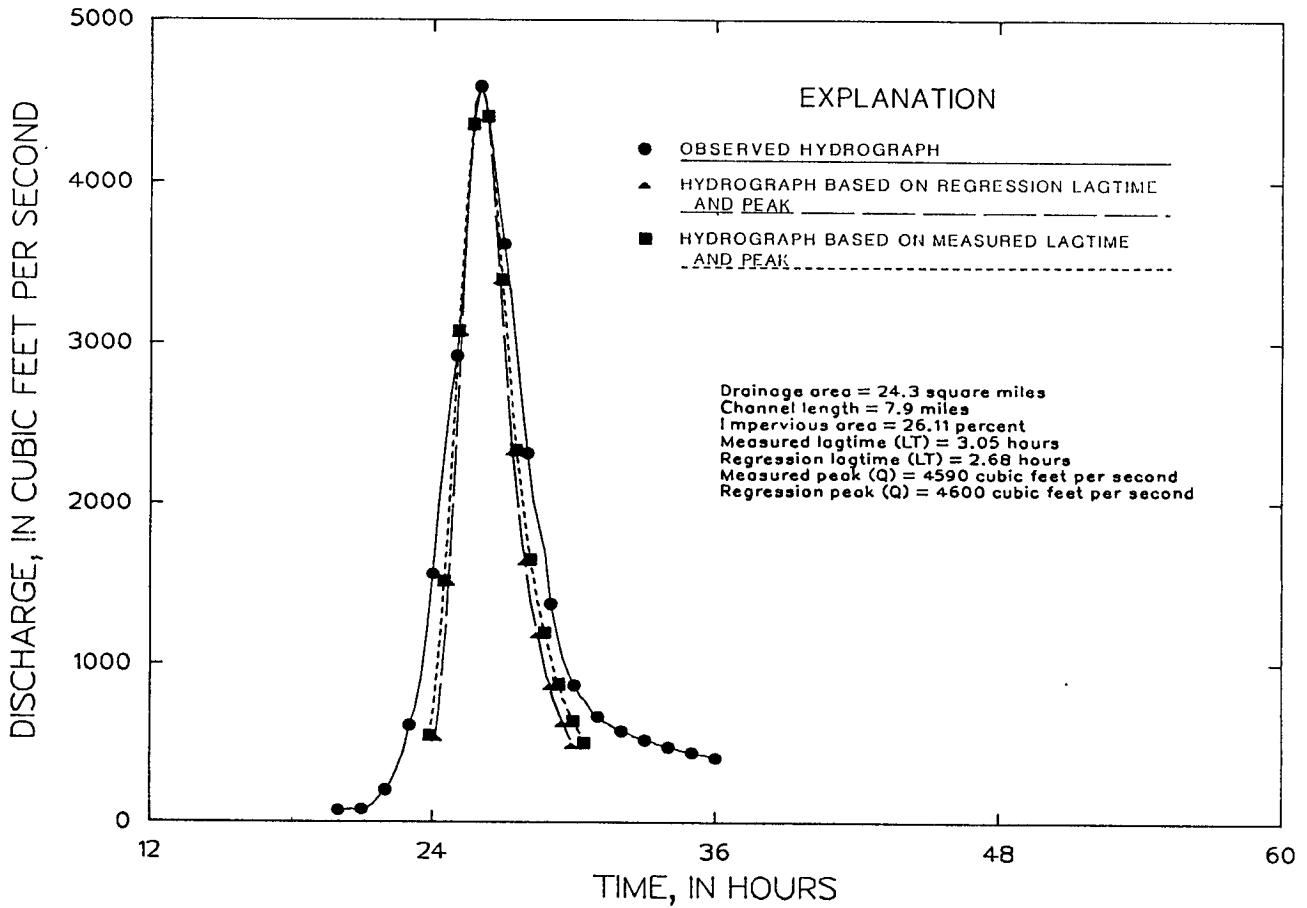


Figure 3.—Comparison of observed and simulated hydrographs for Richland Creek at Charlotte Avenue at Nashville, Tenn. (03431700), for storm of May 3, 1979.

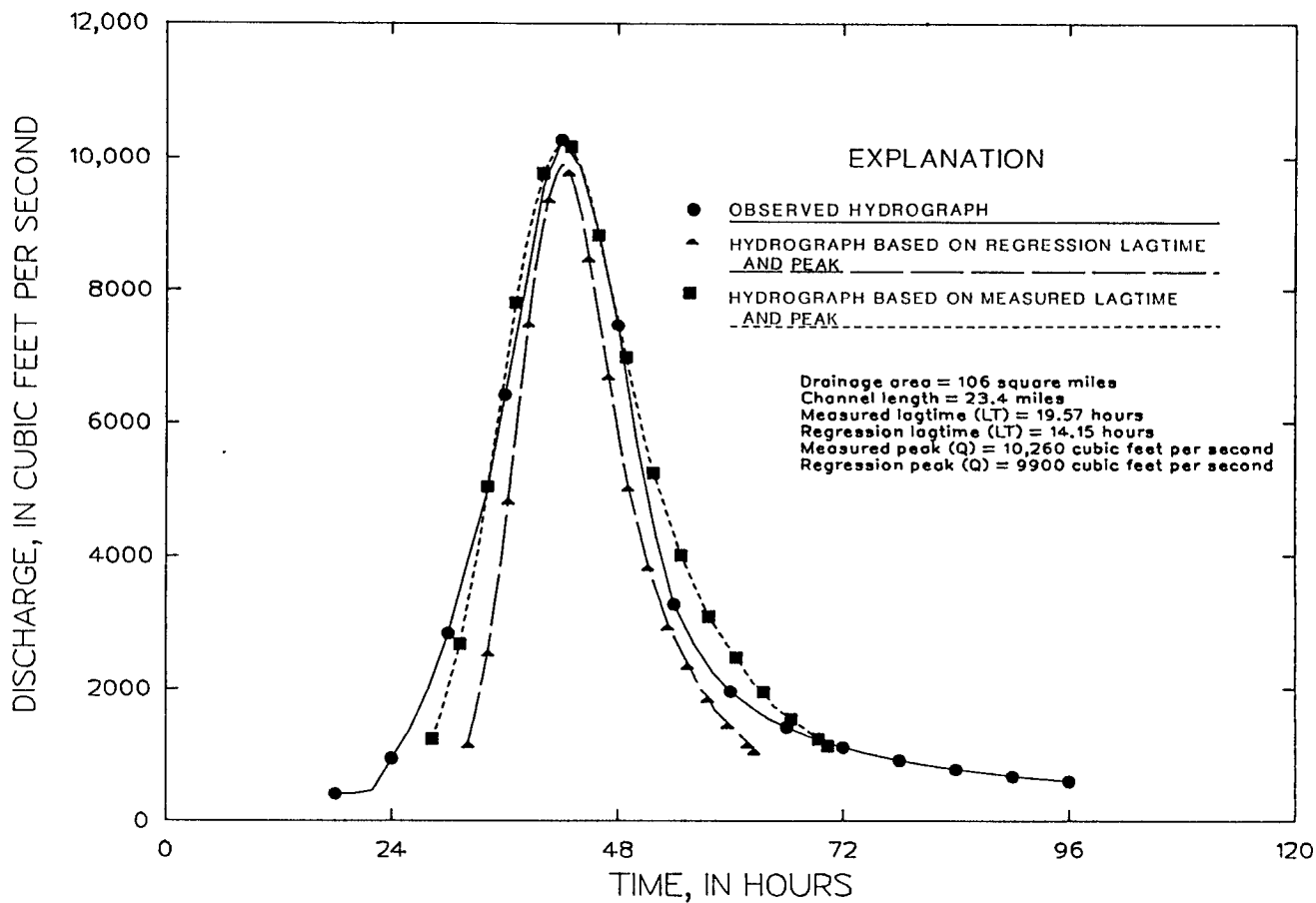


Figure 4.— Comparison of observed and simulated hydrographs for Wolf River near Byrdstown, Tenn. (03416000), for storm of April 3, 1977.

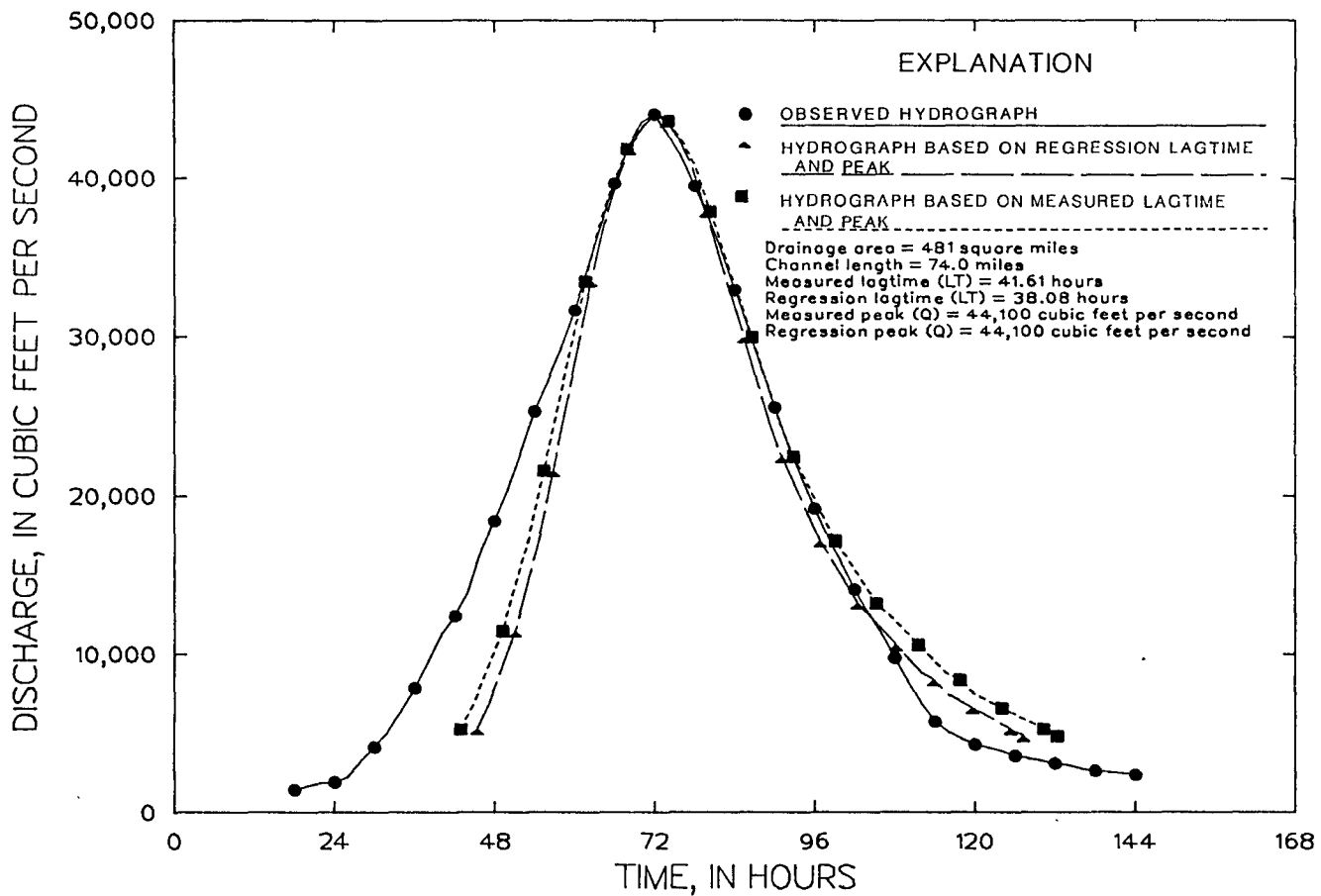


Figure 5.—Comparison of observed and simulated hydrographs for Duck River near Shelbyville, Tenn. (03598000), for storm of March 14, 1973.



conditions remain the same) and is defined as the elapsed time from the centroid of rainfall excess to the centroid of the resultant runoff hydrograph (Stricker and Sauer, 1982). The lagtime of a basin is the principal factor in determining the relative shape of a hydrograph from that basin. For example, a long lagtime will produce a broad flat-crested hydrograph and a short lagtime will produce a narrow sharp-crested hydrograph. Since lagtime is usually not known for a basin, it is often estimated from basin characteristics.

To provide a method of estimating lagtime for ungaged basins in central Tennessee, the 38 average basin lagtimes obtained from the dimensionless hydrograph development procedure and 7 basins with measured lagtimes from a rainfall-runoff modeling study by Wibben (1976) were related to their basin characteristics. Rural and urban basins were analyzed separately because of the effects of urbanization on lagtime. Standard multiple linear regression techniques were used to develop equations for estimating rural and urban basin lagtimes from five basin characteristics. All five characteristics defined below were used in the regression analyses; however, only those characteristics statistically significant at the 95-percent confidence level are included in the final equations. Definitions of these five basin characteristics are as follows:

Drainage area (DA) is the contributing drainage area of the basin, in square miles.

Channel slope (CS) is the slope, in feet per mile, of the main channel determined from the difference in elevation at points 10 and 85 percent of the distance along the main channel from the discharge site to the drainage-basin divide.

Channel length (CL) is the distance, in miles, from the discharge site to the drainage-basin divide, measured along the main water course.

$CL/\sqrt{CS}$  is a ratio, where CL and CS are as previously defined.

Percentage of impervious area (IA) is the percentage of the contributing drainage area that is impervious to infiltration of rainfall. This parameter was measured using the grid method on recent aerial photographs. IA can also be measured from topographic maps or from population and industrial density reports.

All of the basins and their characteristics used in the regression analyses are listed in table 2.

### Regression Analyses

Stepwise regression techniques were used with the five basin characteristics of 31 rural basins and 14 urban basins to derive equations for estimating basin lagtime during the initial regression analyses. Drainage area, channel slope,  $CL/\sqrt{CS}$ , and percentage of impervious area were insignificant for rural lagtime and were deleted from successive regression analyses. All of these basin characteristics, with the exception of percentage of impervious area, were insignificant for urban lagtime also, and were deleted from successive regression analyses.

The final regression analyses were performed using channel length for rural basin lagtime, and channel length and percentage of impervious area for urban basin lagtime. These selected variables are easily obtainable and are of practical use in estimating rural and urban basin lagtimes in central Tennessee. Drainage area size of the basins used in the rural lagtime regression analyses ranged from 0.17 to 481 mi<sup>2</sup>, and in the urban lagtime regression analyses, from 0.47 to 64 mi<sup>2</sup>. The following tables summarize the distribution of drainage area size for the basins used.

Table 2.--Stations and drainage basin characteristics used in lagtime regression analyses

[\*, Basin draining urban area. Used in urban lagtime regression analysis; \*\*, Urban basin having a channel length less than 2 miles. Use rural lagtime equation to compute estimated lagtime]

Station No.	Station name	Drainage area, DA (mi <sup>2</sup> )	Percentage of impervious area, IA (percent)	Channel length, CL (mi)	Channel slope, CS (ft/mi)	Average basin lagtime, LT (h)	Estimated lagtime from equations, LT (h)
03313600	West Fork Drakes Creek Trib. near Fountain Head, Tenn.	0.95	0	1.55	73.92	1.27	1.37
03408500	New River at New River, Tenn.	382	0	46.20	7.06	27.94	25.39
03414500	East Fork Obey River near Jamestown, Tenn.	196	0	37.60	37.00	16.69	21.27
03416000	Wolf River near Brydstown, Tenn.	106	0	23.40	12.30	19.57	14.15
03418000	Roaring River near Hilham, Tenn.	51.4	0	20.10	14.60	20.09	12.41
03420360	Mud Creek Trib. #2 near Summitville, Tenn.	2.28	0	2.46	35.38	3.14	2.04
03420400	Mud Creek near Summitville, Tenn.	7.30	0	4.05	30.62	5.15	3.13
03425646	Town Creek at Maple Street, at Gallatin, Tenn.*	5.00	9.34	5.07	37.60	2.59	2.54
03426800	East Fork Stones River at Woodbury, Tenn.	39.1	0	9.00	31.85	12.92	6.22
03427500	East Fork Stones River near Lascassas, Tenn.	262	0	39.20	6.36	23.78	22.05
03427830	Short Creek Trib., near Christiana, Tenn.	.17	0	.56	100.32	.72	.57
03427840	Short Creek near Christiana, Tenn.	3.54	0	4.58	73.92	2.12	3.48
03430118	McCrary Creek at Ironwood Drive, at Donelson, Tenn.*	7.31	5.74	4.48	26.80	2.86	2.59
03430400	Mill Creek at Nolensville, Tenn.	12.0	0	4.34	30.58	2.20	3.32
03430600	Mill Creek at Hobson Pike, near Antioch, Tenn.	43.0	0	10.71	16.11	4.16	7.22
03431000	Mill Creek near Antioch, Tenn.*	64.0	4.2	17.0	11.40	5.39	5.22
03431340	Browns Creek at Factory Street at Nashville, Tenn.*	13.2	31.50	6.51	42.60	1.92	2.36
03431490	Pages Branch at Avondale, Tenn.*	2.01	14.97	2.27	97.10	1.48	1.58
03431520	Claylick Creek at Lickton, Tenn.*	4.13	8.20	3.01	69.26	1.87	2.01
03431580	Ewing Creek at Knight Road, near Bordeaux, Tenn.*	13.3	14.20	4.50	46.70	2.00	2.24
03431600	Whites Creek at Tucker Road, near Bordeaux, Tenn.*	51.6	8.0	11.13	21.49	3.48	3.83

Table 2.--Stations and drainage basin characteristics used in lagtime regression analyses--Continued

Station No.	Station name	Drainage area, DA (mi <sup>2</sup> )	Percentage of impervious area, IA (percent)	Channel length, CL (mi)	Channel slope, CS (ft/mi)	Average basin lagtime, LT (h)	Estimated lagtime from equations, LT (h)
03431630	Richland Creek at Lynnwood Blvd., at Belle Meade, Tenn.*	2.21	11.7	1.96	119.0	1.33	1.54
03431700	Richland Creek at Charlotte Ave., at Nashville, Tenn.*	24.3	26.11	7.90	33.00	3.05	2.68
03432371	Harpeth River Trib. at Franklin, Tenn.*	1.81	4.68	2.49	74.90	2.17	2.00
03434583	Jones Creek Trib. at Dickson, Tenn.*	2.29	14.79	1.98	68.90	1.48	1.49
03435020	Red River near New Deal, Tenn.	9.32	0	4.00	46.46	1.92	3.10
03435030	Red River near Portland, Tenn.	15.1	0	6.70	27.98	3.52	4.83
03436700	Yellow Creek near Shiloh, Tenn.	124	0	25.70	12.30	19.16	15.33
03582395	Tanyard Branch at Fayetteville, Tenn.**	.47	48.30	.65	184.00	.69	.65
03588400	Chisholm Creek at Westpoint, Tenn.	43.0	0	18.94	15.13	11.76	11.79
03588500	Shoal Creek at Iron City, Tenn.	348	0	44.10	8.16	22.47	24.40
03595520	Grindstone Hollow Creek at Manchester, Tenn.*	2.11	28.20	2.60	28.20	2.24	1.54
03596000	Duck River below Manchester, Tenn.	107	0	24.80	13.90	16.57	14.87
03597300	Wartrace Creek above Bell Buckle, Tenn.	4.99	0	4.20	49.63	1.99	3.23
03597400	Wartrace Creek near Bell Buckle, Tenn.	9.59	0	6.08	31.68	3.02	4.44
03597450	Kelly Creek Trib. near Bell Buckle, Tenn.	.73	0	1.70	132.00	.88	1.48
03598000	Duck River near Shelbyville, Tenn.	481	0	74.00	6.11	41.61	38.08
03600500	Big Bigby Creek at Sandy Hook, Tenn.	17.5	0	7.10	40.40	5.82	5.07
03602500	Piney River at Vernon, Tenn.	202	0	24.60	11.50	18.80	14.77
03604000	Buffalo River near Flatwoods, Tenn.	447	0	71.10	5.08	30.49	36.79
03604070	Coon Creek Trib. near Hohenwald, Tenn.	.51	0	.87	200.48	1.76	.83
03604080	Hugh Hollow Branch near Hohenwald, Tenn.	1.52	0	2.16	105.60	2.51	1.82
03604090	Coon Creek above Chop Hollow, near Hohenwald, Tenn.	6.02	0	3.14	73.92	2.19	2.51
03604100	Coon Creek near Hohenwald, Tenn.	10.1	0	5.23	49.63	2.46	3.90
03605555	Trace Creek above Denver, Tenn.	31.9	0	20.30	19.76	10.62	12.52

Rural	
Range in drainage area size (mi <sup>2</sup> )	Number of basins in analysis
0.17 - 0.75	3
.76 - 5.00	5
5.01 - 15.0	6
15.1 - 50.0	6
50.1 - 200	5
201 - 350	3
351 - 481	3
Total	<u>31</u>

Urban	
Range in drainage area size (mi <sup>2</sup> )	Number of basins in analysis
0.47 - 2.00	2
2.01 - 4.00	4
4.01 - 13.2	4
13.3 - 25.0	2
25.1 - 64.0	2
Total	<u>14</u>

Channel length ranged from 0.56 to 74.0 miles for the rural lagtime regression analysis and from 0.65 to 17.0 miles for the urban lagtime regression analyses. The following tables summarize the distribution of channel length for the basins used.

Rural	
Range in channel length (mi)	Number of basins in analysis
0.56 - 2.00	4
2.01 - 4.20	6
4.21 - 9.00	7
9.01 - 25.0	7
25.1 - 45.0	4
45.1 - 74.0	3
Total	<u>31</u>

Urban	
Range in channel length (mi)	Number of basins in analysis
0.65 - 2.00	3
2.01 - 3.00	3
3.01 - 6.50	4
6.51 - 10.0	2
10.1 - 17.0	2
Total	<u>14</u>

In the urban lagtime analysis, percentage of impervious area ranged from 4.20 to 48.3 percent. The following table summarizes the distribution of impervious area for the basins used.

Urban	
Range in impervious area (percent)	Number of stations in analysis
4.20 - 6.00	3
6.01 - 10.0	3
10.1 - 15.0	4
15.1 - 30.0	2
30.1 - 48.3	2
Total	<u>14</u>

A correlation matrix indicated that drainage area, channel slope, and channel length were highly correlated. In addition, the one-variable equations containing drainage area or channel slope had higher standard errors of estimate and lower  $R^2$  (coefficient of determination) values than the equations containing channel length. Therefore, the following equations may be used for estimating rural basin lagtimes and urban basin lagtimes for ungedged basins in central Tennessee. It should be noted that the standard error of estimate for the urban basin lagtime equation may be unusually low because of the limited data base (14 stations).

Rural basin lagtime

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	Standard error of estimate, in percent	Coefficient of determination $R^2$	
$RLT = 0.94 (CL)^{0.86}$	$\pm 39.2$	0.90	(1)

where RLT is estimated rural basin lagtime, in hours; and  
CL is channel length, in miles.

Urban basin lagtime

---

	Standard error of estimate, in percent	Coefficient of determination $R^2$	
$ULT = 1.64 (CL)^{0.49} (IA)^{-0.16}$	$\pm 15.9$	0.91	(2)

where ULT is estimated urban basin lagtime, in hours;  
CL is channel length, in miles; and  
IA is the percentage of the contributing drainage basin occupied by impervious surface.

The log-linear form of the estimating equations was checked with graphical plots. Plots of regression residuals versus observed lagtime, residuals versus channel length, and residuals versus percentage of impervious area were included on each graph. The scatter of plotting points on each graph appeared to be random with no apparent bias. Therefore, the form of the estimating equation is assumed to be appropriate.

It should be noted that the urban basin lagtime equation will sometimes predict a longer lagtime than the rural basin lagtime equation. Conceptually, this situation should not exist because increasing imperviousness should decrease lagtime. Therefore, when estimating lagtime for urbanized basins, lagtime should be calculated from both equations, and the smallest value should be used. Additionally, the estimates of lagtime for urban basins with channel lengths less than 2 miles may have errors higher than the reported standard error of estimate.

Station residuals were plotted on a map to evaluate geographic bias of estimates from the rural and urban basin lagtime equations. Although the residuals varied considerably between some stations, no specific geographic trends could be detected. Due to the limited number of stations available for the rural (31) and urban (14) lagtime regression analysis, verification of the regression equations was not possible.

A partial analysis of the sensitivity of the rural and urban basin lagtime equations to channel length (CL) and percentage of impervious area (IA) was performed. Results of sensitivity of the regression equations are shown graphically in figure 6. For the rural basin lagtime equation, for example, an error of 40 percent in computing channel length results in about a 33-percent difference in lagtime. For the urban basin lagtime equation, an error of 40 percent in computing channel length results in about an 18-percent difference in lagtime, and an error of 60 percent in computing the percentage of impervious area results in about a 7-percent difference in lagtime.

### Estimating Flood Volume

Storage of floodwater or flood prevention may often be part of a particular structure's design. In such cases, it is important to know the volume associated with the design flood. Therefore, an equation for estimating flood volumes for selected recurrence interval floods on central Tennessee streams was developed. The equation relates flood volumes to drainage area size, flood peak discharge, and basin lagtime. Observed flood volumes (in inches of runoff) from 245 storms from the 38 basins used in the dimensionless hydrograph analysis were used in this analysis. Flood volumes were obtained as part of the unit hydrograph computations discussed earlier. The flood-volume analysis was conducted using a split-sample technique in which half of the data set (123 storms) was used to develop the regression equation and the other half (122 storms) was used for verification and to determine prediction error.

### Regression Analyses

Stepwise regression techniques were used with three basin characteristics to derive the equation for estimating flood volumes. The three basin characteristics, drainage area size, flood peak discharge, and basin lagtime, were all statistically significant at the 95-percent confidence level. Drainage area sizes used in the regression analysis had the same range and distribution as that used in the rural and urban basin lagtime analysis (0.17 to 481 mi<sup>2</sup>). Flood peak discharges ranged from 24 to 45,200 ft<sup>3</sup>/s. The following table summarizes the distribution of flood peak discharges for the storms used.

Range in flood peak discharge (ft <sup>3</sup> /s)	Number of storms in analysis
24 - 100	14
101 - 1000	42
1,001 - 5000	35
5,001 - 20,000	17
20,001 - 45,200	15
Total	<u>123</u>

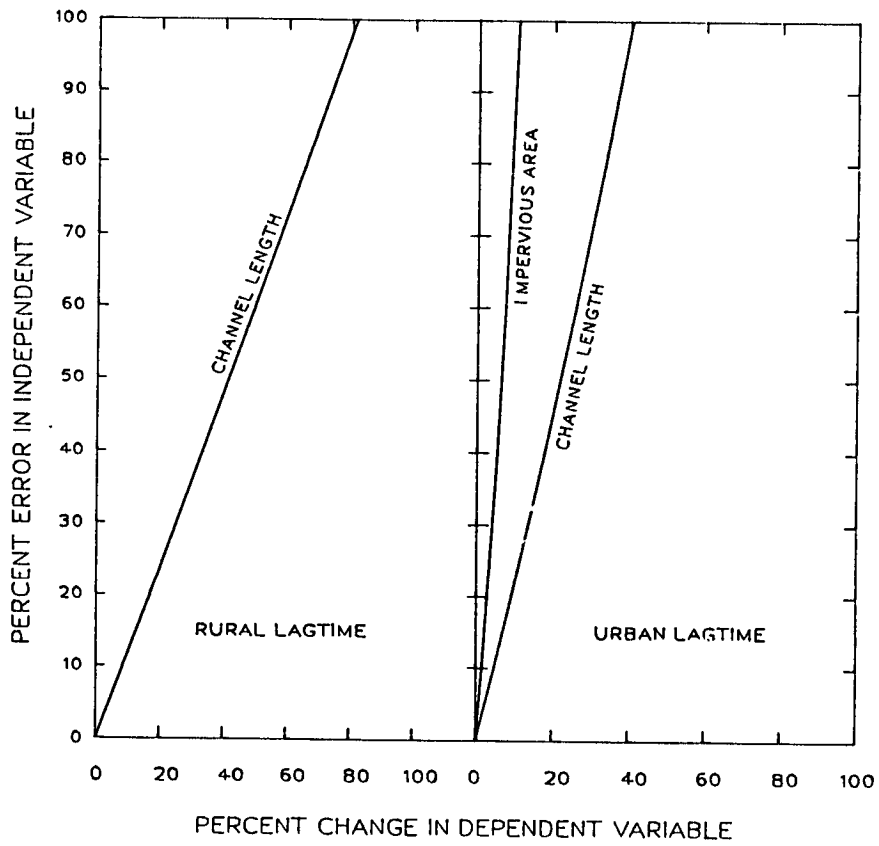


Figure 6.— Percent change in rural lagtime and urban lagtime resulting from errors in computing channel length and percentage of impervious area.

Measured basin lagtimes ranged from 0.69 to 41.6 hours. The following table summarizes the distribution of lagtimes for the basins from which the storms were obtained.

Range in basin lagtime (h)	Number of basins in analysis
0.69 - 1.00	3
1.01 - 2.50	12
2.51 - 15.0	12
15.1 - 30.0	9
30.1 - 41.6	2
Total	38

The following equation may be used for estimating flood volumes associated with a T-year peak discharge for ungaged streams in central Tennessee. Flood volume, for a given T-year peak discharge, also can be obtained by summing the ordinates of the estimated flood hydrograph.

	Standard error of estimate, in percent	Standard error of prediction, in percent	Coefficient of determination, R <sup>2</sup>	
$V = 1.3 \times 10^{-3} (DA)^{-1.06} (Q)^{1.05} (LT)^{1.03}$	+32.3	+47.2	0.94	(3)

where V is estimated flood volume, in inches;  
 DA is drainage area, in square miles;  
 Q is flood peak discharge, in cubic feet per second; and  
 LT is basin lagtime, in hours.

A second regression analysis was performed using the other half of the data set. This regression analysis produced a volume equation that was not significantly different from the first equation. A third analysis was performed to determine the prediction error associated with using the volume equation at an ungaged site. This was accomplished by substituting regression basin lagtime for measured basin lagtime, and regression discharge associated with the observed recurrence interval flood for observed discharge. The volume equation was then used to predict flood volume for each of the observed storms in the second half of the data set. Results of this analysis produced an estimate of the standard error of prediction (listed above) which is a measure of how well the regression equation will estimate the dependent variable at sites other than those used to derive the equation.

The log-linear form of the estimating equation was checked with graphical plots. Plots of regression residuals versus drainage area, residuals versus flood peak discharge, and residuals versus basin lagtime were included on each graph. The scatter of plotting points on each graph appeared to be random with no apparent bias. Therefore, the form of the estimating equation is assumed to be appropriate.

Station residuals were plotted on a map to evaluate geographic bias of estimates from the flood-volume equation. Although the residuals varied between stations, no geographic trends could be detected.



A partial analysis of the sensitivity of the volume equation to drainage area (DA), flood peak discharge ( $Q_p$ ), and basin lagtime (LT) was performed, and the results are shown graphically in figure 7. Results of the sensitivity analysis indicate that an error of 40 percent in computing drainage area, for example, results in about a 30-percent difference in flood volume. An error of 20 percent in computing flood peak discharge and an error of 10 percent in computing basin lagtime results in differences in flood volumes of 21 percent and 11 percent, respectively.

### HYDROGRAPH-WIDTH RELATION

For some hydraulic analyses, it is only necessary to estimate the period of time that a specific discharge will be exceeded, therefore a complete flood hydrograph is not needed. In order to estimate this time period, a hydrograph-width relation was defined by Inman (1986) for the dimensionless hydrograph in table 1. Hydrograph-width ratios were 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 discharge ratio ( $Q_t/Q_p$ ) over the full range of the dimensionless hydrograph. The resulting hydrograph-width relations are listed in table 3 and are shown graphically in figure 8. The simulated hydrograph width ( $W$ ) in hours can be estimated for a specified discharge ( $Q_t$ ) by first computing the ratio  $Q_t/Q_p$  and then multiplying the corresponding  $W/LT$  ratio in table 3 by the estimated basin lagtime (LT). The resulting hydrograph width is the period of time a specified discharge will be exceeded.

Table 3.--Discharge and hydrograph-width ratios for Inman's dimensionless hydrograph

[Modified from Inman (1986)]

Discharge ratios	Width ratios
$Q_t/Q_p$	$W/LT$
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

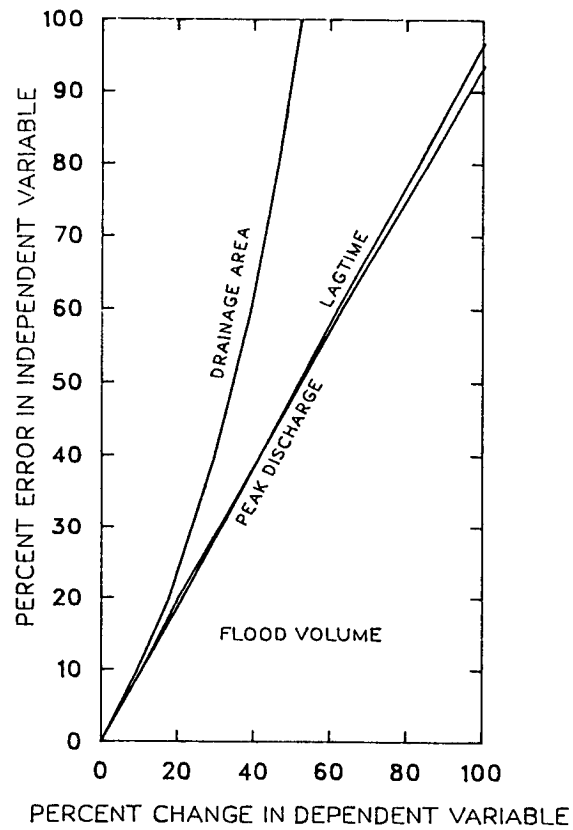


Figure 7.—Percent change in flood volume resulting from errors in computing drainage area, lagtime, and peak discharge.

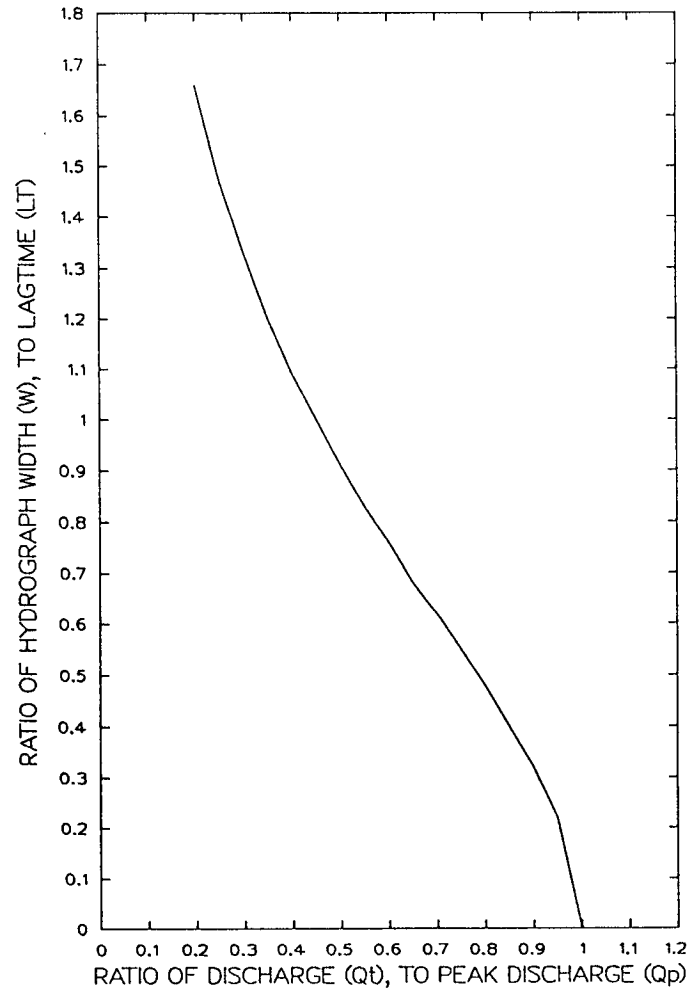


Figure 8.—Hydrograph—width relation for Inman's dimensionless hydrograph.

## APPLICATION OF HYDROGRAPH SIMULATION TECHNIQUE

A step-by-step procedure is described below to assist the user in applying the techniques for simulating flood hydrographs and estimating flood volumes and hydrograph widths as presented in this report. In addition, an example is given to demonstrate these techniques. The procedure is as follows:

1. Determine the drainage area and main-channel length of the basin from the best available topographic maps.
2. Compute the peak discharge for the desired recurrence-interval flood from the applicable flood-frequency report (flood-frequency equations included in the supplement).
3. Estimate percentage of impervious area if the basin is urbanized.
4. Compute the basin lagtime from the appropriate equation (1 or 2, page 16).
5. Compute the coordinates of the flood hydrograph by multiplying the value of lagtime by the time ratios and the value of peak discharge by the discharge ratios (table 1, page 4).
6. Compute the volume for the selected recurrence-interval flood using equation 3 (page 19).
7. Compute the period of time a specific discharge will be exceeded using the dimensionless hydrograph-width relation (table 3, page 20; or figure 8, page 22).

### Example Problem

The following example illustrates the procedure for computing the simulated hydrograph associated with the 100-year discharge estimate in a hypothetical rural basin in hydrologic area 3 in central Tennessee.

1. The drainage area (DA) is determined as 393 mi<sup>2</sup> and the main-channel length is determined to be 60.6 miles.
2. The peak discharge ( $Q_{100}$ ) for the 100-year recurrence-interval flood is 82,500 ft<sup>3</sup>/s (Randolph and Gamble, 1976--in supplement).
3. Using equation 1, rural basin lagtime (RLT) is estimated to be:

$$\begin{aligned} \text{RLT} &= 0.94 (\text{CL})^{0.86} \\ &= 0.94 (60.6)^{0.86} \\ &= 32.1 \text{ h.} \end{aligned}$$

4. The coordinates of the simulated flood hydrograph are listed below and are shown graphically in figure 9:

t/LT (from table 1)	X	LT (from step 3)	= time (h)	Q <sub>t</sub> /Q <sub>p</sub> (from table 1)	X	Q <sub>p</sub> (from step 2)	= Discharge (ft <sup>3</sup> /s)
0.25		32.1	8.0	0.12		82,500	9,900
.30		32.1	9.6	.16		82,500	13,200
.35		32.1	11.2	.21		82,500	17,320
.40		32.1	12.8	.26		82,500	21,450
.45		32.1	14.4	.33		82,500	27,220
.50		32.1	16.0	.40		82,500	33,000
.55		32.1	17.7	.49		82,500	40,420
.60		32.1	19.3	.58		82,500	47,850
.65		32.1	20.9	.67		82,500	55,270
.70		32.1	22.5	.76		82,500	62,700
.75		32.1	24.1	.84		82,500	69,300
.80		32.1	25.7	.90		82,500	74,250
.85		32.1	27.3	.95		82,500	78,370
.90		32.1	28.9	.98		82,500	80,850
.95		32.1	30.5	1.00		82,500	82,500
1.00		32.1	32.1	.99		82,500	81,670
1.05		32.1	33.7	.96		82,500	79,200
1.10		32.1	35.3	.92		82,500	75,900
1.15		32.1	36.9	.86		82,500	70,950
1.20		32.1	38.5	.80		82,500	66,000
1.25		32.1	40.1	.74		82,500	61,050
1.30		32.1	41.7	.68		82,500	56,100
1.35		32.1	43.3	.62		82,500	51,150
1.40		32.1	44.9	.56		82,500	46,200
1.45		32.1	46.5	.51		82,500	42,070
1.50		32.1	48.1	.47		82,500	38,770
1.55		32.1	49.8	.43		82,500	35,470
1.60		32.1	51.4	.39		82,500	32,170
1.65		32.1	53.0	.36		82,500	29,700
1.70		32.1	54.6	.33		82,500	27,220
1.75		32.1	56.2	.30		82,500	24,750
1.80		32.1	57.8	.28		82,500	23,100
1.85		32.1	59.4	.26		82,500	21,450
1.90		32.1	61.0	.24		82,500	19,800
1.95		32.1	62.6	.22		82,500	18,150
2.00		32.1	64.2	.20		82,500	16,500
2.05		32.1	65.8	.19		82,500	15,670
2.10		32.1	67.4	.17		82,500	14,020
2.15		32.1	69.0	.16		82,500	13,200
2.20		32.1	70.6	.15		82,500	12,370
2.25		32.1	72.2	.14		82,500	11,550
2.30		32.1	73.8	.13		82,500	10,720
2.35		32.1	75.4	.12		82,500	9,900
2.40		32.1	77.0	.11		82,500	9,070

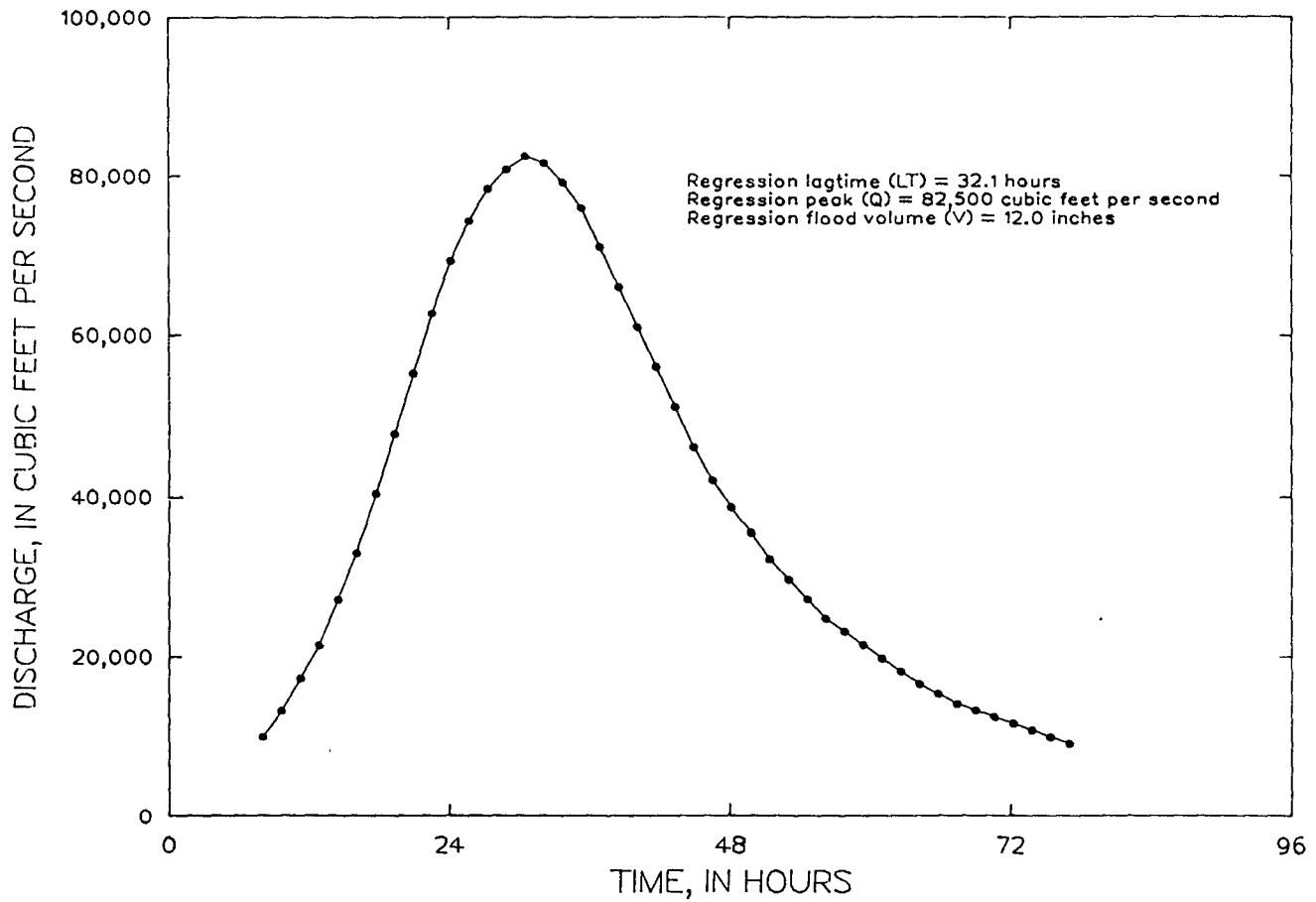


Figure 9.— Plot of example simulated 100—year flood hydrograph for a hypothetical river in central Tennessee.

5. Using equation 3, flood volume (V) is estimated to be:

$$\begin{aligned} V &= 1.3 \times 10^{-3} (DA)^{-1.06} (Q_p)^{1.05} (LT)^{1.03} \\ &= 1.3 \times 10^{-3} (393)^{-1.06} (82,500)^{1.05} (32.1)^{1.03} \\ &= 12.0 \text{ in.} \end{aligned}$$

6. If an estimate were needed for the period of time road overflow (beginning at a discharge of 60,000 ft<sup>3</sup>/s) would occur, compute it as follows:

- a.  $Q_t/Q_p = 60,000/82,500 = 0.73$
- b. from figure 8,  $W/LT = 0.58$
- c. rural basin lagtime,  $RLT = 32.1 \text{ h}$ , from step 3
- d. road overflow time =  $(W/LT)(RLT)$   
=  $(0.58)(32.1)$   
= 18.6 h.

### Limitations

The techniques for simulating flood hydrographs and estimating flood volumes described in this report are limited to streams in hydrologic areas 2 and 3 in central Tennessee. In deriving the rural and urban lagtime equations, basin size ranged from 0.17 to 481 mi<sup>2</sup> and from 0.47 to 64.0 mi<sup>2</sup>, respectively; channel length ranged from 0.56 to 74.0 miles and from 0.65 to 17.0 miles, respectively; and impervious area in urban basins ranged from 4.20 to 48.3 percent. For the flood-volume equation, flood peak discharge ranged from 24 to 45,200 ft<sup>3</sup>/s and basin lagtime ranged from 0.69 to 41.6 hours. Use of the hydrograph simulation technique and regression equations should be limited to these ranges because the techniques presented have not been tested beyond the indicated ranges in values. If values outside these ranges are used, the standard error may be considerably higher than for sites where all variables are within the specified ranges. In addition, these techniques should not be applied to streams where temporary in-channel storage or over-bank detention storage is significant unless suitable estimates of peak discharge and lagtime are available which account for these effects.

### CONCLUSIONS

A dimensionless hydrograph developed for Georgia streams was tested for its applicability to central Tennessee streams by comparing it to a similar dimensionless hydrograph developed for central Tennessee streams. Test results indicate the two dimensionless hydrographs are essentially the same. Therefore, the Georgia dimensionless hydrograph can be used to simulate flood hydrographs at ungaged sites for both rural and urban streams in central Tennessee. A total of 163 observed flood hydrographs from 38 basins in central Tennessee were used in the test of the Georgia dimensionless hydrograph and an additional 36 flood hydrographs were used for a verification test.

Multiple-regression techniques were used to develop relations between basin lagtime and selected basin characteristics, of which channel length was significant for the rural basins, and channel length and percentage of impervious area were significant for urban basins. Tests indicated no variable or geographical bias in either the rural or urban equations.

An equation for estimating flood volumes was also developed using multiple-regression techniques. Drainage area, flood peak discharge, and basin lagtime were the significant variables in the volume equation. Tests indicated no variable or geographic bias in the volume equation.

A simulated flood hydrograph can be computed by applying lagtime, obtained from the appropriate regression equation, and peak discharge of a specific recurrence interval, to the dimensionless hydrograph time and discharge ratios in table 1. The coordinates of the simulated flood hydrograph are computed by multiplying lagtime by the time ratios and peak discharge by the discharge ratios. The volume of the simulated flood hydrograph can be estimated from the volume regression equation.

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SYMBOLS, DEFINITIONS, AND UNITS

<u>Symbol</u>	<u>Definition</u>	<u>Unit</u>
CL	Channel length measured along main water course	mi
CS	Main channel slope	ft/mi
$CL/\sqrt{CS}$	Ratio of channel length to the square root of channel slope	---
DA	Contributing drainage area of a basin	mi <sup>2</sup>
IA	Impervious area	percent
LT	Basin lagtime	h
P <sub>2</sub> P <sub>4</sub>	2-year 24-hour rainfall amount	in.
Q <sub>p</sub>	Flood peak discharge	ft <sup>3</sup> /s
Q <sub>t</sub>	Discharge occurring at time t	ft <sup>3</sup> /s
Q <sub>t</sub> /Q <sub>p</sub>	Ratio of discharge occurring at time t to flood peak discharge	---
Q <sub>2,5,10,25,50,100</sub>	Rural basin flood-frequency discharge for recurrence intervals of 2 through 100-years, respectively	ft <sup>3</sup> /s
Q <sub>(u)2,5,10,25,50,100</sub>	Urban basin flood-frequency discharge for recurrence intervals of 2 through 100-years, respectively	ft <sup>3</sup> /s
R <sup>2</sup>	Coefficient of determination	---
RLT	Rural basin lagtime	h
t/LT	Ratio of instantaneous time to basin lagtime	---
ULT	Urban basin lagtime	h
V	Flood volume	in.
W	Hydrograph width	h
W/LT	Ratio of hydrograph width to basin lagtime	---

SUPPLEMENT

REGIONAL FLOOD-FREQUENCY EQUATIONS  
FOR RURAL BASINS IN TENNESSEE

The following is a list of the rural basin flood-frequency equations from Randolph and Gamble (1976) for hydrologic areas 2 and 3.

Hydrologic Area 2

$$\begin{aligned}Q_2 &= 199 A^{0.744} \\Q_5 &= 352 A^{0.729} \\Q_{10} &= 465 A^{0.723} \\Q_{25} &= 614 A^{0.722} \\Q_{50} &= 738 A^{0.719} \\Q_{100} &= 867 A^{0.718}\end{aligned}$$

Hydrologic Area 3

$$\begin{aligned}Q_2 &= 319 A^{0.733} \\Q_5 &= 512 A^{0.725} \\Q_{10} &= 651 A^{0.723} \\Q_{25} &= 836 A^{0.720} \\Q_{50} &= 977 A^{0.720} \\Q_{100} &= 1,125 A^{0.719}\end{aligned}$$

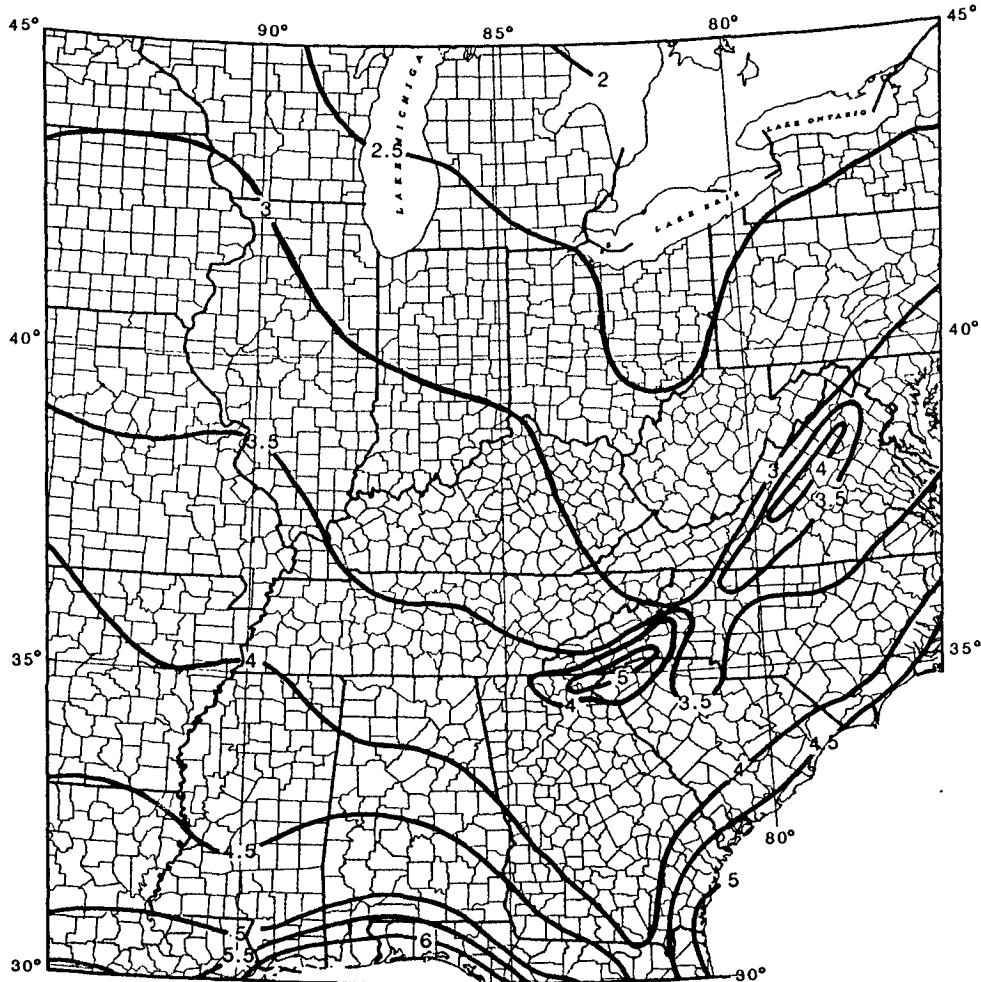
where  $Q_{25}$  is the 25-year recurrence-interval flood, in cubic feet per second; and  
A is contributing drainage area, in square miles.

REGIONAL FLOOD-FREQUENCY EQUATIONS  
FOR URBAN BASINS IN TENNESSEE

The following is a list of the urban basin flood-frequency equations from Robbins (1984) which are applicable statewide. The precipitation factor (P2\_24) used in each equation can be determined from figure 10.

$$\begin{aligned}
 Q(u)_2 &= 1.76(A)^{0.74} (IA)^{0.48} (P2\_24)^{3.01} \\
 Q(u)_5 &= 5.55(A)^{0.75} (IA)^{0.44} (P2\_24)^{2.53} \\
 Q(u)_{10} &= 11.8(A)^{0.75} (IA)^{0.43} (P2\_24)^{2.12} \\
 Q(u)_{25} &= 21.9(A)^{0.75} (IA)^{0.39} (P2\_24)^{1.89} \\
 Q(u)_{50} &= 44.9(A)^{0.75} (IA)^{0.40} (P2\_24)^{1.42} \\
 Q(u)_{100} &= \overset{77.0}{\cancel{44.9}}(A)^{0.75} (IA)^{0.40} (P2\_24)^{1.10}
 \end{aligned}$$

Where  $Q(u)_{25}$  is the 25-year recurrence-interval flood, in cubic feet per second;  
 $A$  is contributing drainage area, in square miles;  
 $IA$  is percentage of the contributing drainage basin occupied by impervious surface; and  
 $P2\_24$  is the 2-year, 24-hour rainfall amount, in inches.



EXPLANATION

—5— Line of equal rainfall shows amount of 2-year 24-hour rainfall in inches. Interval 0.5 inch

Figure 10.--2-year 24-hour rainfall, in inches (U.S. Department of Commerce, 1961).