DETERMINATION OF FLOOD HYDROGRAPHS FOR STREAMS IN SOUTH CAROLINA:

VOLUME 1. SIMULATION OF FLOOD HYDROGRAPHS FOR RURAL WATERSHEDS IN SOUTH CAROLINA

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CONVERSION FACTORS AND ABBREVIATIONS OF UNITS

The inch-pound units used in this report may be converted to metric (International System) units by the following factors.

Multiply inch-pound unit	By	To obtain metric unit
inch (in.)	2.54	centimeter (cm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	2.590	square kilometer (km^2)
cubic foot per second (ft^3/s)	0.028317	cubic meter per second (m^3/s)

DETERMINATION OF FLOOD HYDROGRAPHS FOR STREAMS IN SOUTH CAROLINA: VOLUME 1. SIMULATION OF FLOOD HYDROGRAPHS FOR RURAL WATERSHEDS IN SOUTH CAROLINA

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ABSTRACT

A typical (average) flood hydrograph corresponding to a peak discharge of specific recurrence interval can be simulated for ungaged rural basins having drainage areas less than 500 square miles in South Carolina. Three dimensionless hydrographs were developed on the basis of data collected during 188 storm events at 49 stations representing a wide range of drainage area sizes and basin conditions. The design peak discharge and a volume-adjusted average basin lagtime are required to apply the technique. The standard errors of estimate for simulated hydrograph widths at 50 and 75 percent, respectively, of observed stormflow were ±14.1 and ±18.3 percent for basins in the Blue Ridge physiographic province, ±29.2 and ±36.2 percent for basins in the Piedmont province, and ±17.8 and ±22.8 percent for basins in the Upper and Lower Coastal Plain subprovinces.

Multiple-regression analyses were used to develop equations for estimating average basin lagtime. At the 95-percent confidence level, drainage area was determined to be the only significant explanatory variable needed to estimate the average lagtime for basins in each physiographic province. The standard error of estimate of regression relations developed for estimating lagtimes for the Blue Ridge and Piedmont provinces, and the Upper Coastal Plain and Lower Coastal Plain subprovinces were ±7.3, ±25.6, ±34.3, and ±25.6 percent, respectively.

A regression equation that provides runoff volume in inches also was developed. The explanatory variables used in the equation for estimating runoff volume are peak discharge, average basin lagtime, and drainage area; the standard errors of estimate of equations applicable in the Blue Ridge, Piedmont, Upper Coastal Plain, and Lower Coastal Plain were ±10.3, ±21.1, ±13.6, and ±15.1 percent respectively. The regression equations for estimating runoff volume are the basis of an adjustment to average basin lagtime, which is required to simulate flood hydrographs by use of the dimensionless hydrographs.

The simulation techniques and regression equations may be useful engineering tools for estimation where time of inundation or storage of floodwater is a part of the flood prevention or structure design criteria.

INTRODUCTION

The hydraulic design of highway drainage structures involves an evaluation of the flood hazard to the highway and of the effect of the proposed structures on the hazard to lives, property, and stream stability. Risk analysis is a useful tool in evaluating these hazards. The application of risk analysis to the design of drainage structures allows the engineer to select the design that will provide the least expected cost to the public (Corry and others, 1980). To fully evaluate these risks, a runoff hydrograph with a peak discharge of specific recurrence interval may be needed to estimate the length of time that specific features, such as roads and bridges, will be inundated. In basins where little or no systematic streamflow data are available, it may be necessary to simulate a typical or design hydrograph by using one or more hydrograph estimation techniques.

Most traditional approaches rely on the unit hydrograph method whereby design hydrographs are computed by convolution of the unit hydrograph with rainfall excess. This requires rainfall totals and actual or synthetic storm distributions, as well as the evaluation of a number of parameters needed to specify rainfall-runoff relations (determination of infiltration and other abstractions). The recurrence interval of the rainfall amount is used for design purposes in these methods; however, the resulting peak discharge, volume, and hydrograph may or may not have the same recurrence interval.

A need exists for an easy-to-apply, direct method of estimating the flood hydrograph, volume, and width associated with a peak discharge of specific recurrence interval. In a nationwide study, Stricker and Sauer (1982) developed a dimensionless hydrograph for use in estimating flood hydrographs on ungaged urban watersheds. Recently, a similar method was developed for rural and urban streams in Georgia (Inman, 1986) and was successfully applied to streams in central Tennessee (Robbins, 1986). The method involves direct computation of a design hydrograph and requires only two parameters, the design peak discharge and basin lagtime. In this method, a recurrence interval is assigned to the peak discharge and a typical or average hydrograph associated with that peak is computed. The resulting hydrograph or volume may or may not have the same recurrence interval.

The purpose of this report (Volume 1), which is the first of two reports, is to describe a technique for simulating flood hydrographs (shape, volume, width) for ungaged rural basins draining areas less than 500 mi^2 (square miles) in South Carolina. The hydrograph simulation procedure developed in the Georgia study is used in this report, except that three dimensionless hydrographs and a lagtime that has been adjusted to achieve a closer fit of observed volumes are required for South Carolina basins. Equations for estimating the adjusted lagtime, average basin lagtime, and runoff volume also are included in this report. The second report will discuss techniques for estimating urban flood magnitude and frequency and for simulating flood hydrographs for small urban watersheds in South Carolina (Determination of Flood Hydrographs for Streams in South Carolina: Volume 2. Estimation of Peak Discharges and Simulation of Flood Hydrographs for Urban Watersheds in South Carolina, U.S. Geological Survey, written commun., 1989).

This study was conducted by the U.S. Geological Survey in cooperation with the South Carolina Department of Highways and Public Transportation and the Federal Highway Administration. The guidance and technical assistance of C. Lamar Sanders, Hydrologist, U.S. Geological Survey, in the regionalization and statistical analyses in this study are recognized and greatly appreciated.

DATA BASE

The data base used in this study consisted of 188 flood events observed at 49 stations throughout South Carolina (plate 1). Only simple (or noncompound) discharge hydrographs resulting from uniform, relatively short-duration rainfall events were selected. Concurrent rainfall and discharge unit values were available for 24 basins from earlier model calibration studies (Whetstone, 1975). Rain gages for these basins were located at the watershed outlets near the stream stage recorders. Areal uniformity of the storms was evaluated by comparing precipitation totals to those recorded at nearby National Weather Service daily rainfall stations. Stage-discharge relations for these gaging stations were reviewed and the unit values adjusted where necessary.

The data for the remaining 25 basins came from Geological Survey long-term, continuous-record discharge stations and from National Weather Service hourly rainfall records. Uniformity of the storms was determined by examining daily rainfall amounts at two to six rainfall stations in or near the basin. Average basin rainfall totals were computed by using the Theissen polygon method. The time distribution for the average basin rainfall was obtained by using either one National Weather Service hourly rainfall station located within the basin or the weighted time distribution of two hourly rainfall stations near the basin. Discharge was obtained by applying the proper stage-discharge relation to the gage heights.

FLOOD HYDROGRAPH SIMULATION PROCEDURE

A dimensionless hydrograph may be defined as a representative hydrograph shape for which the discharge is expressed as the ratio of discharge to peak discharge and the time as the ratio of time to lagtime. It is developed by averaging typical hydrographs from a variety of watersheds. Three dimensionless hydrographs were developed in this study to simulate design flood hydrographs within the Blue Ridge, Piedmont, and Coastal Plain physiographic provinces in South Carolina. Estimates of the two principal parameters, peak discharge and basin lagtime, are required for simulations. A U.S. Geological Survey report by Whetstone (1982) provides equations for estimating peak discharge of specific recurrence interval using basin drainage area as the sole explanatory variable. Equations for estimating peak discharges that update those presented by Whetstone have been developed for estimating peak discharges (Techniques for Estimating Magnitude and Frequency of Floods in South Carolina, 1990, U.S. Geological Survey, written commun., 1989). A recurrence interval may be assigned to the peak discharge of the design hydrographs simulated by using methods found in this report only if the peak discharge has been estimated on the basis of the latest U.S. Geological Survey regression equations or some other method consistent with the report by the Water Resources Council (1982).

Methods for estimating average basin lagtime (LT) are presented in another section of this report. Unlike the previous studies by Stricker and Sauer (1982), Inman (1986), and Robbins (1986), an adjustment to average basin lagtime was needed to achieve the best fit of observed hydrograph volumes and widths in South Carolina. The adjusted lagtimes (LT_A) should be applied whenever a dimensionless hydrograph is used to simulate a design flood hydrograph. Average basin lagtimes (LT) may be used to estimate runoff volumes on the basis of equations presented later in this report.

The remainder of this section explains the development and regionalization of dimensionless hydrographs and the procedure for obtaining the adjusted basin lagtime (LT_A).

Development of Dimensionless Hydrographs

Three dimensionless hydrographs were developed for use in South Carolina for rural basins having drainage areas less than 500 mi^2. Peak discharge and a basin lagtime adjusted to produce correct runoff volume are required to convert the dimensionless hydrograph to a simulated hydrograph for a given basin.

The dimensionless hydrographs were based on data from 151 observed floods at 39 gaging stations. The remaining 37 floods from 10 stations in the data base were used to verify the results. A series of computer programs (S.E. Ryan, U.S. Geological Survey, written commun., 1986) were used as an aid in developing the dimensionless hydrograph shapes (steps 3 through 7, below) and to perform subsequent statistical analyses. Following is a description of the steps in the dimensionless hydrograph development process, which is based in part on information in the report by Inman (1986):

(1) A discharge hydrograph is plotted on semilogarithmic paper for 3 to 5 floods at each of the 39 gaging stations. The end of direct runoff is estimated to be the point in time when a straight-line recession began. A unit hydrograph with a rainfall excess duration equal to one recording interval is then computed using the unit hydrograph method described by O'Donnell (1960). This method assumes base flow to be equal to the first and last discharges supplied by the user and is interpolated in between. These amounts are then subtracted from the discharge ordinates to obtain the direct runoff hydrograph from which the unit hydrograph is derived. The lagtime of each unit hydrograph is computed concurrently. Lagtime, in these calculations, is defined as the time difference between the centroid of the rainfall excess and the centroid of the direct runoff hydrograph. A typical event is illustrated in figure 1, and the corresponding unit hydrograph is shown in figure 2.

(2) The unit hydrographs with inconsistent shapes are eliminated and unit hydrographs from additional storms are computed if needed.

(3) An average unit hydrograph with a duration equal to the computation (recording) interval is computed by aligning the

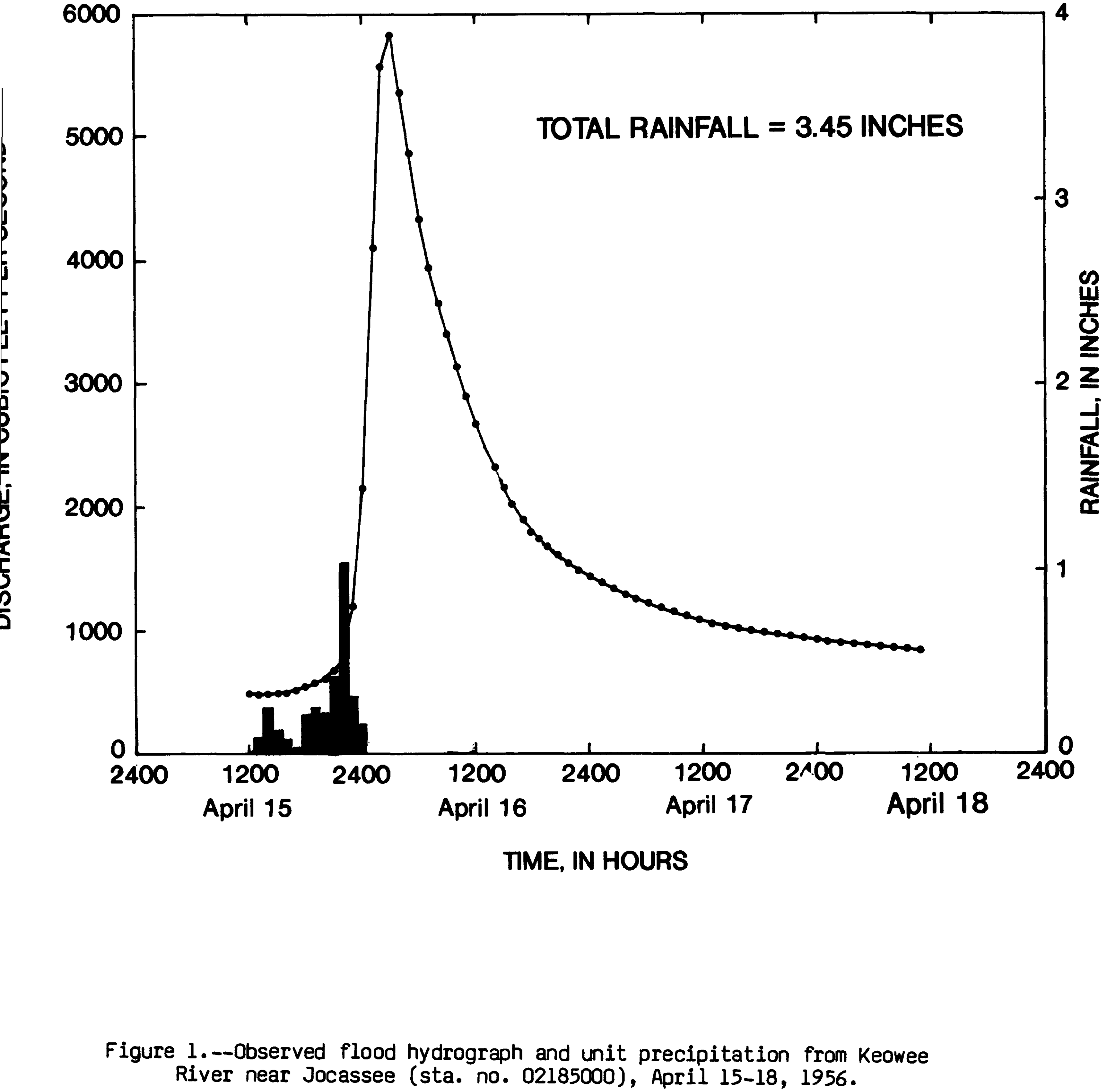

Figure 1.--Observed flood hydrograph and unit precipitation from Keowee River near Jocassee (sta. no. 02185000), April 15-18, 1956.

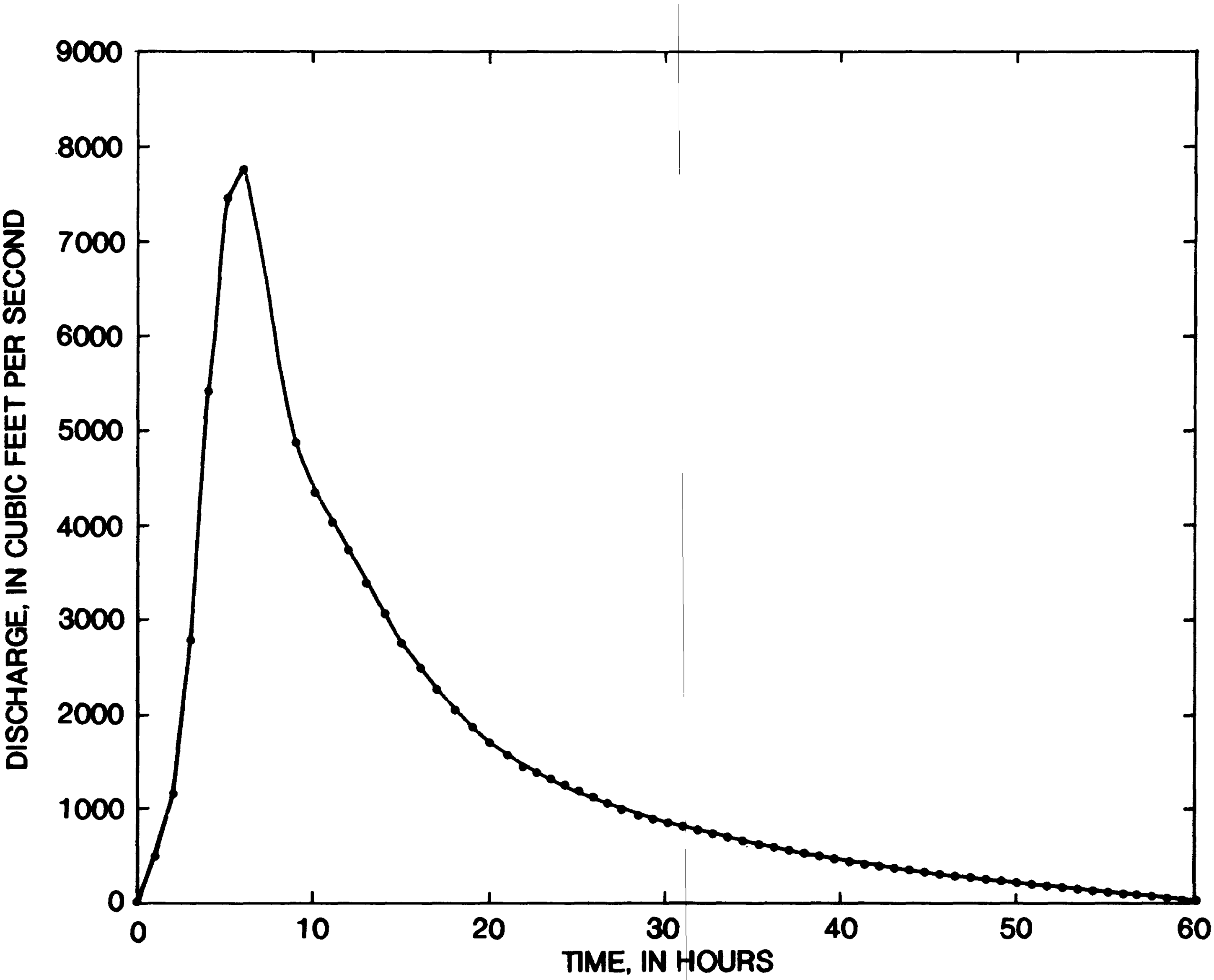

Figure 2.--Unit hydrograph computed from observed data in figure 1, with precipitation excess of 1.00 inch and lagtime of 14.2 hours.

peaks and averaging the discharge ordinates of the final selection of unit hydrographs (table 1 and fig. 3). The correct timing of the average unit hydrograph is obtained by averaging the time of the center of mass of the individual unit hydrographs and plotting the average center of mass at this average time. The computed lagtimes for each event are also averaged to provide a mean basin lagtime.

(4) The average unit hydrographs computed in step 3 are transformed to hydrographs having durations of one-fourth, one-third, one-half, and three-fourths of the average lagtime computed in step 3. This transformation is necessary because the unit hydrographs have been computed using 15-, 30-, or 60-minute time intervals. To convert the average unit hydrograph to a more realistic duration, fractions of lagtime were used. The fractional lagtimes are further adjusted to the nearest multiple of the original duration (recording interval). For example, if the original duration is 5 minutes and the average lagtime is 0.7 hours (42 minutes), then one-fourth lagtime is 10.5 minutes, which would be rounded to 10 minutes. One-third lagtime is 14 minutes, which would be rounded to 15 minutes. One-half lagtime is 21 minutes, which would be rounded to 20 minutes. Three-fourths lagtime is 31.5 minutes, rounded to 30 minutes. The transformed unit hydrographs will have durations of 2 times, 3 times, 4 times, and 6 times the duration of the original unit hydrograph. The transformation of a short duration unit hydrograph to a long duration unit hydrograph (for instance, a 5-minute duration to a 20-minute duration) can be accomplished through the use of the following equations:

$D/\Delta t$	EQUATION
2	$TUHD(t)=1/2[TUH(t)+TUH(t-1)]$
3	$TUHD(t)=1/3[TUH(t)+TUH(t-1)+TUH(t-2)]$
4	$TUHD(t)=1/4[TUH(t)+TUH(t-1)+TUH(t-2)+TUH(t-3)]$
n	$TUHD(t)=1/n[TUH(t)+TUH(t-1) \ldots TUH(t-n-1)]$, (1)

where Δt is computation interval, (original unit hydrograph has a duration equal to Δt);

D is design duration of the unit hydrograph, (must be a multiple of Δt);

TUHD(t) is ordinate of the design unit hydrograph at time t; and

TUH(t), TUH(t-1), and so forth, are ordinates of the original unit hydrograph at times t, t-1, t-2, and so forth.

Table 1.--Listing of discharges at 1-hour intervals with peaks aligned for five unit hydrographs with date of occurrence and average hydrograph computed for Keowee River near Jocassee (sta. no. 02185000)

Hydrographs (discharge in cubic feet per second)					Average unit hydrograph
(04-15-56)	(02-04-60)	(01-24-64)	(04-12-59)	(03-05-63)	
0	0	0	0	0	0
0	0	0	98	0	19
498	0	0	490	500	298
1,160	599	0	1,077	1,352	837
2,796	2,721	4,317	3,545	3,699	3,416
5,421	5,387	6,484	6,309	6,213	5,963
7,473	7,361	8,022	8,048	7,802	7,741
7,773	8,045	8,347	8,260	8,117	8,108
6,825	7,417	7,629	7,568	7,619	7,412
5,694	6,234	6,575	6,530	6,640	6,334
4,857	5,540	5,495	5,572	5,680	5,429
4,359	4,946	4,746	5,000	4,956	4,802
4,038	4,478	4,437	4,560	4,357	4,374
3,732	4,176	4,012	4,111	3,851	3,977
3,398	3,683	3,390	3,671	3,434	3,515
3,059	3,403	2,848	3,178	2,980	3,093
2,756	3,031	2,689	2,735	2,623	2,767
2,495	2,609	2,551	2,375	2,354	2,477
2,271	2,405	2,144	2,085	2,118	2,204
2,063	2,110	1,770	1,918	1,877	1,948
1,869	1,953	1,677	1,786	1,704	1,798
1,696	1,791	1,716	1,627	1,547	1,675
1,559	1,605	1,545	1,486	1,438	1,527
1,448	1,544	1,224	1,324	1,333	1,375
1,354	1,385	1,133	1,197	1,228	1,259
1,264	1,306	1,249	1,176	1,111	1,221
1,183	1,219	1,179	1,150	1,044	1,155
1,109	1,099	900	1,087	973	1,033
1,046	1,074	779	991	901	958
983	954	899	827	813	895
919	894	924	746	764	850
866	865	694	734	725	777
815	730	508	702	687	688
769	704	610	705	625	683
729	636	722	643	571	660
687	561	575	534	542	580
645	545	359	492	525	513
601	462	394	456	466	476
565	437	551	468	412	487
529	400	494	500	365	458
489	351	238	405	357	368
461	346	169	315	339	326
436	274	342	262	266	316
412	248	360	208	220	290
381	208	139	271	235	247
349	167	15	294	224	210
328	176	152	209	168	207
306	97	269	139	85	179

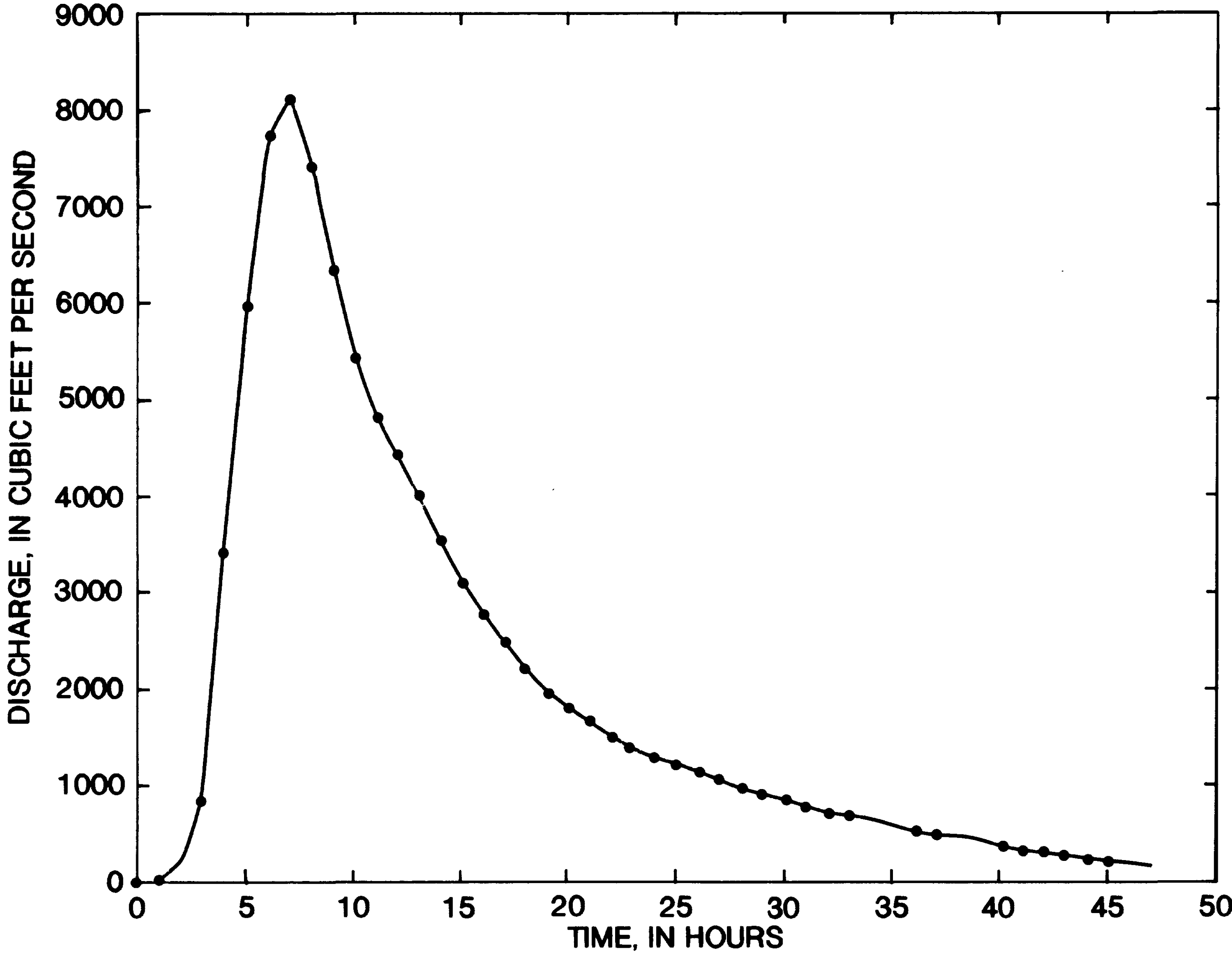

Figure 3.--Average unit hydrograph for Keowee River near Jocassee, (sta. no. 02185000).

Actual duration of rainfall excess for a storm may be defined as the time during which precipitation falls at a rate greater than the existing infiltration capacity. A design duration is used in this study, rather than actual duration, because the actual duration of rainfall excess is highly variable. The design duration is expressed as a fractional part of lagtime, such as one-fourth, one-third, one-half, or three-fourths of the average lagtime computed in step 3. As discussed later in this report, the design duration for each dimensionless hydrograph that most closely reproduced the observed hydrographs in each region was chosen.

(5) The one-fourth, one-third, one-half, and three-fourths lagtime hydrographs are reduced to dimensionless terms by dividing the time coordinates of the unit hydrographs by lagtime and the discharge coordinates by peak discharge. The results of this step for one basin are illustrated in figure 4.

(6) An average dimensionless hydrograph is computed by averaging the dimensionless hydrographs at the stations in one or more regions. The average hydrographs were computed by aligning the peaks and averaging each ordinate of the discharge ratio, Q/Qp. The average one-half-lagtime duration dimensionless hydrograph in the Piedmont province and the range of the data from the 19 stations from which it was computed is illustrated in figure 5.

(7) The best hydrograph shape for each region is then determined by computing the standard error of hydrograph widths simulated by using various regional dimensionless hydrographs with durations of one-fourth, one-third, one-half, and three-fourths lagtime. The standard error of the estimate of the width comparisons is based on the mean-square difference between the observed and estimated hydrograph widths at 50 and 75 percent of peak stormflow. An example of these comparisons is shown in figure 6.

(8) Further investigation (discussed later) showed that some improvement in the results of (7) could be obtained by making a correction to lagtime based on a regression analysis of runoff volume. The correction factor is calculated as the ratio of the runoff volume predicted by the volume regression equations to the runoff volume simulated by using the appropriate dimensionless hydrograph and unadjusted lagtimes. The average basin lagtime equation and the volume correction factor equation were then combined and simplified algebraically into one equation for an "adjusted" lagtime.

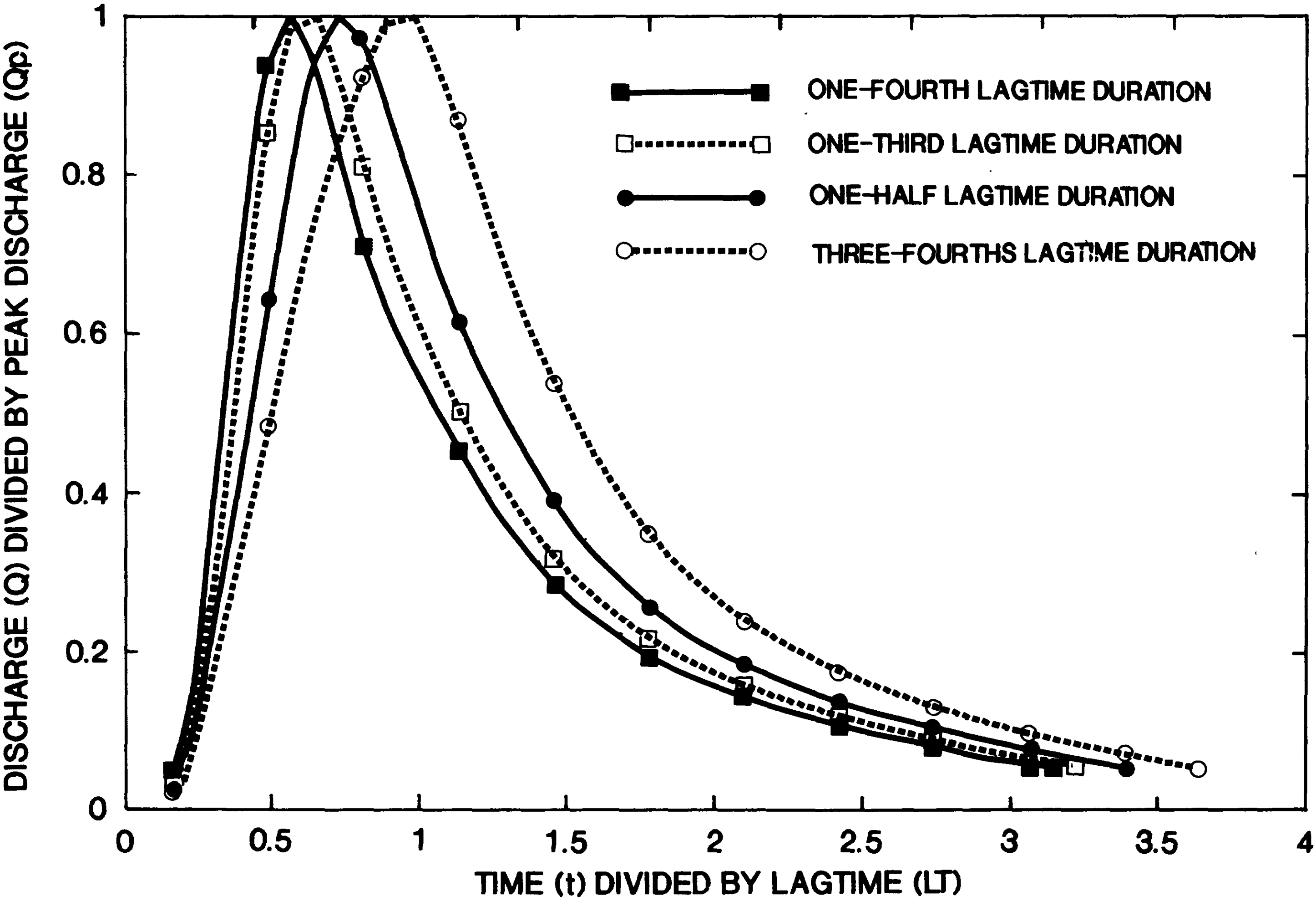

Figure 4.--One-fourth-, one-third-, one-half-, and three-fourths-lagtime duration dimensionless hydrographs for Keowee River near Jocassee (sta. no. 02185000)

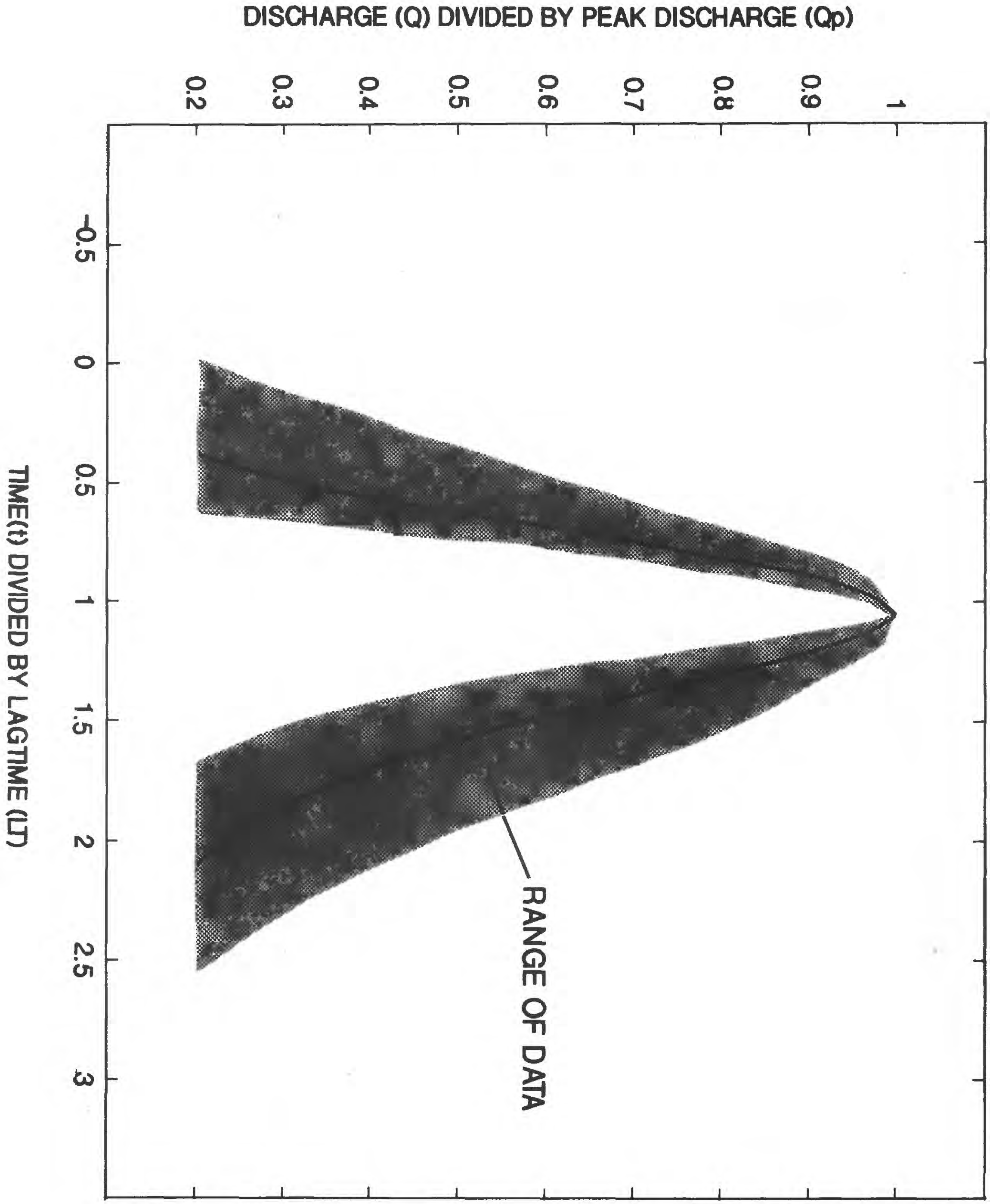

Figure 5.--Average one-half-lagtime duration dimensionless hydrograph in the Piedmont and the range of the data from the 19 stations from which it was computed.

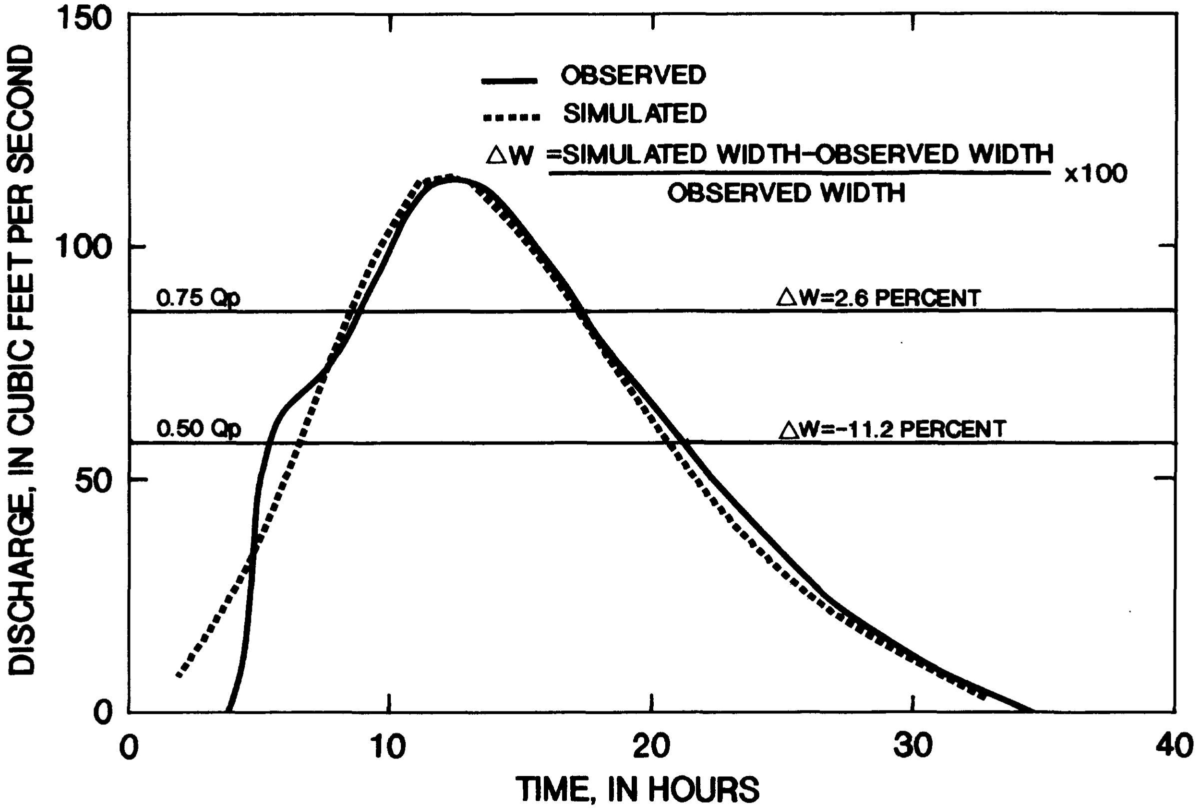

Figure 6.--Simulated runoff hydrograph, using observed peak flow and average basin lagtime, and the observed runoff hydrograph showing width comparisons at 50 and 75 percent of peak discharge for Two Mile Branch near Lake City (sta. no. 02132100), September 20, 1970.

Regionalization of Dimensionless Hydrograph Shape

Regionalization is the process by which records may be extended in space. In this process, the flow characteristics of gaged sites are related to measurable basin parameters so that estimates of those flow characteristics can be made at ungaged sites. The regional analysis of hydrograph shapes is summarized in table 2 and discussed in the remaining paragraphs of this section.

Inman (1986) found regionalization unnecessary and that only one dimensionless hydrograph was required to adequately predict flood hydrographs, both rural and urban, in Georgia. A central Tennessee study (Robbins, 1986) reported that the Georgia dimensionless hydrograph could also be used to predict urban and rural runoff hydrographs in that region. Therefore, hydrographs simulated by using Inman's dimensionless hydrograph were compared to the 151 observed stormflow hydrographs that were used to develop dimensionless hydrographs for South Carolina to test its applicablility to South Carolina's basins. Runs 1-3, table 2, show the standard error to be in an acceptable range; however, run 3 indicates that the number of hydrograph widths at 0.75 Qp that were underpredicted (observed width greater than simulated width) was much greater than the number of overpredicted hydrographs in the coastal areas of South Carolina.

Next, one dimensionless hydrograph was developed using the 151 flood events in South Carolina (run 4). Only slightly smaller standard errors than for run 1 resulted. Runs 5 through 8 revealed that hydrograph widths in the mountainous region of the State (Blue Ridge province) were overpredicted by 16 events to 1 and widths in the coastal regions were underpredicted by 50 events to 18.

Average dimensionless hydrographs were then developed by using the data separated by physiographic province. Previous investigations by Whetstone (1982) and Bloxham (1976, 1981) used four hydrologic areas consisting of two physiographic provinces and two subprovinces (plate 1) to regionalize streamflow characteristics in South Carolina:

1. Blue Ridge.--This province, located in the northwestern section of the State, is characterized by rugged mountains with great relief and narrow valleys and occupies about 2 percent of the State's area.

2. Piedmont.--The Piedmont province composes about 35 percent of the State and is a region of moderate relief and gently rolling slopes.

3. Upper Coastal Plain.--The Fall Line marks the transition from the Piedmont to the Upper Coastal Plain and where clayey residual soils of crystalline bedrock give way to the sand, gravel, and clay of the coastal sediments. The Upper Coastal Plain, sometimes referred to as the Carolina and Georgia Sandhills, occupies about 19 percent of the State and has gradual slopes and rounded summits, although there are several areas of intensely irregular terrain.

4. Lower Coastal Plain.--The Lower Coastal Plain, which comprises about 44 percent of the State, is characterized by gentle slopes and extensive swamplands. The boundary between the Upper and Lower Coastal Plain subprovinces generally coincides with the Citronelle Escarpment (Doering, 1960), which marks the innermost sea-cut terraces of the Coastal Plain.

Table 2.--Statistical summary of dimensionless hydrograph regional analysis

Run number	Stations used to develop the average dimensionless hydrograph	Stations compared to the average dimensionless hydrograph	Standard error of estimate, in percent, of hydrograph widths at 50 and 75 percent of observed peak discharge (Qp)		Lagtime duration	Number of hydrograph widths overestimated (+) or underestimated (-) at 0.75 Qp	
			0.50 Qp	0.75 Qp			
1	GEORGIA[1]	ALLSC[2]	33.1	37.7	1/2	51+	100-
2	GEORGIA	BRP[3]	32.3	37.5	1/2	40+	49-
3	GEORGIA	CP[4]	34.3	38.4	1/2	11+	51-
4	ALLSC	ALLSC	29.7	34.8	1/2	73+	78-
5	ALLSC	BLUER[5]	25.0	27.2	1/2	16+	1-
6	ALLSC	PIED[6]	35.5	40.7	1/2	40+	32-
7	ALLSC	UCP[7]	22.8	32.1	1/2	8+	20-
8	ALLSC	LCP[8]	23.6	27.0	1/2	10+	30-
9	ALLSC	BRPSL>50DA<5[9]	65.1	69.9	1/2	10+	3-
10	BLUER	BLUER	17.2	20.4	1/4	3+	14-
			15.3	16.3	1/3	7+	10-
11	BLUER	VBLUER[10]	12.8	19.9	1/4	6+	1-
			13.3	24.0	1/3	6+	1-
12	PIED	PIED	36.5	43.7	1/3	25+	47-
			34.6	40.5	1/2	32+	40-
13	PIED	VPIED[11]	37.7	38.5	1/3	5+	4-
			39.6	41.0	1/2	5+	4-
14	PIED	PIEDSL>50DA<5[12]	52.8	58.4	1/2	13+	4-
15	UCP	UCP	20.3	27.8	1/2	13+	15-
16	LCP	LCP	18.6	22.4	1/2	17+	23-
17	BRP	BRP	32.0	37.3	1/2	43+	46-
18	BRP	BLUER	15.9	17.7	1/4	4+	13-
			15.0	15.7	1/3	8+	9-
			17.4	19.4	1/2	12+	5-
19	BRP	PIED	36.9	44.0	1/3	24+	48-
			34.7	40.6	1/2	31+	41-
20	BRP	BRPSL>50DA<5	54.4	59.9	1/2	10+	3-
21	BRP	BRPREST[13]	27.3	32.9	1/2	33+	43-
22	BRPL>50DA<5	BRPSL>50DA<5	32.7	38.7	1/3	8+	5-
23	BRPL>50DA<5	VBRPSL>50DA<5	41.9	43.7	1/3	3+	4-
24	BRP	VBLUER	30.7	33.3	1/3	7+	1-
			37.0	43.8	1/2	7+	1-
25	BRP	VPIED	37.7	38.5	1/3	5+	4-
			39.4	40.6	1/2	5+	4-
26	CP	CP	19.4	25.0	1/2	28+	34-
27	CP	UCP	20.6	27.8	1/2	13+	15-
28	CP	LCP	18.6	22.3	1/2	17+	23-
29	CP	VUCP[14]	15.9	26.5	1/2	0+	8-
30	CP	VLCP[15]	18.2	28.5	1/2	5+	7-

[1]GEORGIA--------Georgia statewide hydrograph (Inman, 1986)
[2]ALLSC----------All South Carolina stations
[3]BRP------------Stations in the Blue Ridge and Piedmont physiographic provinces
[4]CP-------------Stations in the Upper and Lower Coastal Plain physiographic subprovinces
[5]BLUER----------Stations in the Blue Ridge physiographic province
[6]PIED-----------Stations in the Piedmont physiographic province
[7]UCP------------Stations in the Upper Coastal Plain physiographic subprovince
[8]LCP------------Stations in the Lower Coastal Plain physiographic subprovince
[9]BRPSL>50DA<5---Stations in the Blue Ridge and Piedmont provinces with slopes greater than 50 feet per mile and drainage areas less than 5 square miles (does not include verification stations)
[10]VBLUER---------Verification stations in the Blue Ridge physiographic province
[11]VPIED----------Verification stations in the Piedmont physiographic province
[12]PIEDSL>50DA<5--Stations in the Piedmont physiographic province with slopes greater than 50 feet per mile and drainage areas less than 5 square miles (includes verification station)
[13]BRPREST--------Stations in the Blue Ridge and Piedmont provinces except those in BRPSL>50DA<5
[14]VUCP-----------Verification stations in the Upper Coastal Plain physiographic subprovince
[15]VLCP-----------Verification stations in the Lower Coastal Plain physiographic subprovince

Comparisons made (runs 10, 12, 15, and 16) between observed data and simulations using the appropriate dimensionless hydrograph for the respective provinces showed a significant improvement and no geographical bias (imbalance of over- and under-predicted widths) as in the previous runs. Plots of the appropriate duration dimensionless hydrographs for all provinces are shown in figure 7.

Runs were then made to see if one hydrograph for north of the Fall Line (Blue Ridge and Piedmont physiographic provinces) and one for south of the Fall Line (Upper and Lower Coastal Plain subprovinces) would adequately predict flood hydrographs for the state. Runs 15 and 16 and runs 27 and 28 for basins in the Upper and Lower Coastal Plain subprovinces have virtually equal error with no change in bias between them. This indicates that one dimensionless hydrograph will perform as well as two for the Coastal Plain. The standard errors of estimate for simulated hydrograph widths, using the average dimensionless hydrograph developed for basins in the Coastal Plain, were ±19.4 and ±25.0 percent for hydrograph widths at 50 and 75 percent of observed peak stormflow, respectively. The Coastal Plain dimensionless hydrograph coordinates are shown in figure 8 and table 3.

In the Piedmont and Blue Ridge provinces, however, there appears to be some bias present if only one dimensionless hydrograph is used. Run 10 (one-third-lagtime duration) and run 18 (one-half-lagtime duration) show that enough improvement in bias and accuracy in the Blue Ridge is made by separation of the provinces to warrant individual dimensionless hydrographs. Because of the scarcity of data in the Blue Ridge, the standard error of the verification stations (discussed in a later section) also was used in deciding that a separate hydrograph was needed (run 11, one-third-lagtime duration and run 24, one-half-lagtime duration). The storm duration that provided the best fit of observed data in the Blue Ridge was the one-third-lagtime duration hydrograph. The standard errors of estimate at 50 and 75 percent of observed stormflow were ±15.3 and ±16.3 percent, respectively, for the Blue Ridge dimensionless hydrograph. The Blue Ridge dimensionless hydrograph is shown in figure 9 and table 3.

The one-half-lagtime duration dimensionless hydrograph gave the best results for the Piedmont province. The standard errors of estimate at 50 and 75 percent of observed stormflow for the Piedmont dimensionless hydrograph were ±34.6 and ±40.5 percent. The coordinates for the Piedmont dimensionless hydrograph are found in figure 10 and table 3.

Inspection of the standard errors during the analysis of the north-of-Fall Line group seemed to show that the largest errors were associated with the basins whose main channel slopes were greater than 50 ft/mi (feet per mile) and whose drainage areas were less than 5 mi^2. In these cases the hydrograph widths were usually overestimated. Pursuant to this, a dimensionless hydrograph was developed from the four development stations that fell into this category. The standard error improved significantly (runs 20 and 22), but with such a small number of basins it was difficult to defend yet another group with its own unique dimensionless hydrograph. In an attempt to justify one, data for 21 events from 5 Piedmont basins used in the Inman (1986) study that have slopes and drainage areas falling into this category were tested by applying the high-slope, small-drainage-area

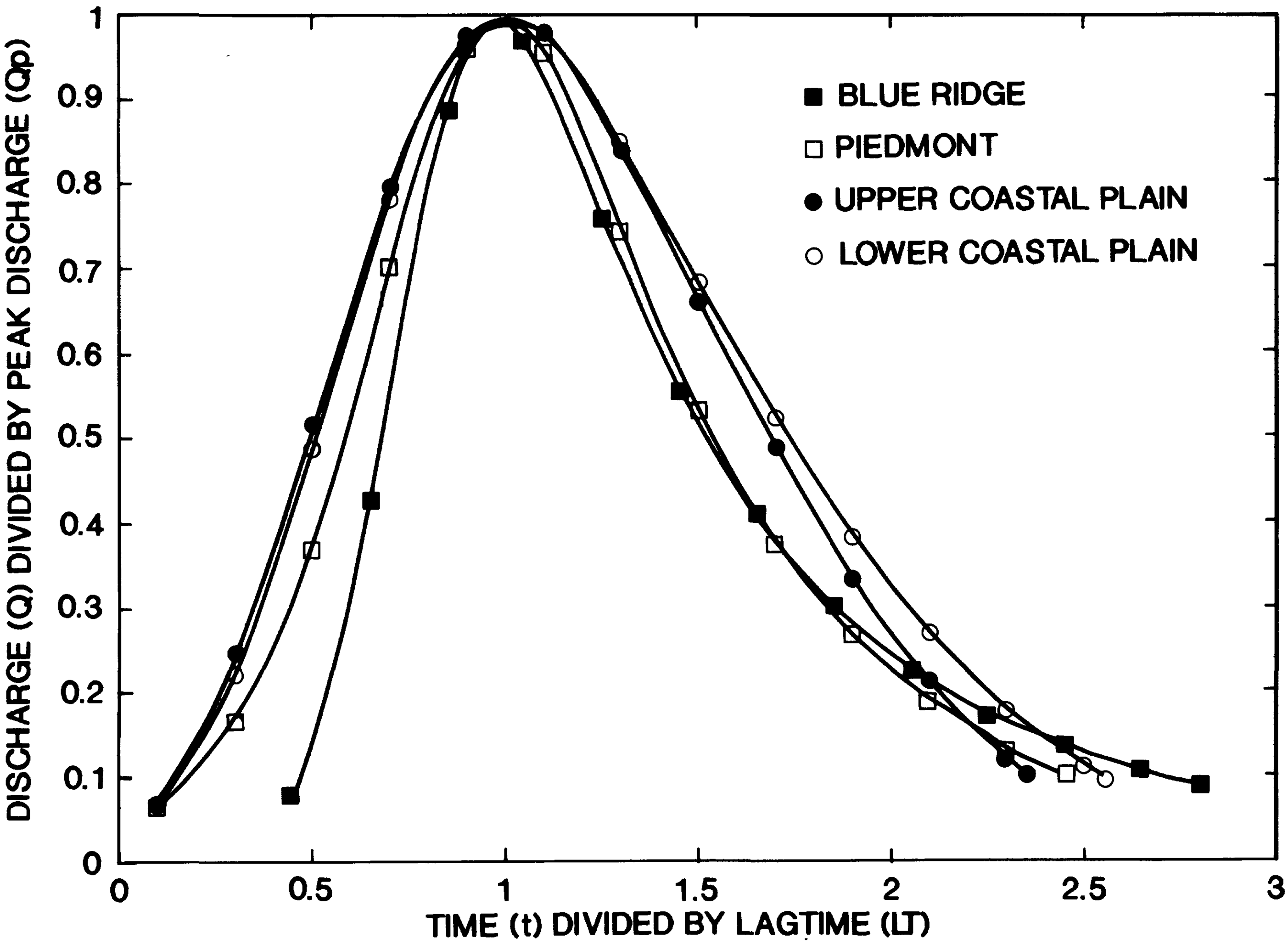

Figure 7.--Average dimensionless hydrographs for the Blue Ridge (one-third-lagtime duration) and the Piedmont, Upper Coastal Plain, and Lower Coastal Plain (one-half-lagtime-duration) with peaks aligned.

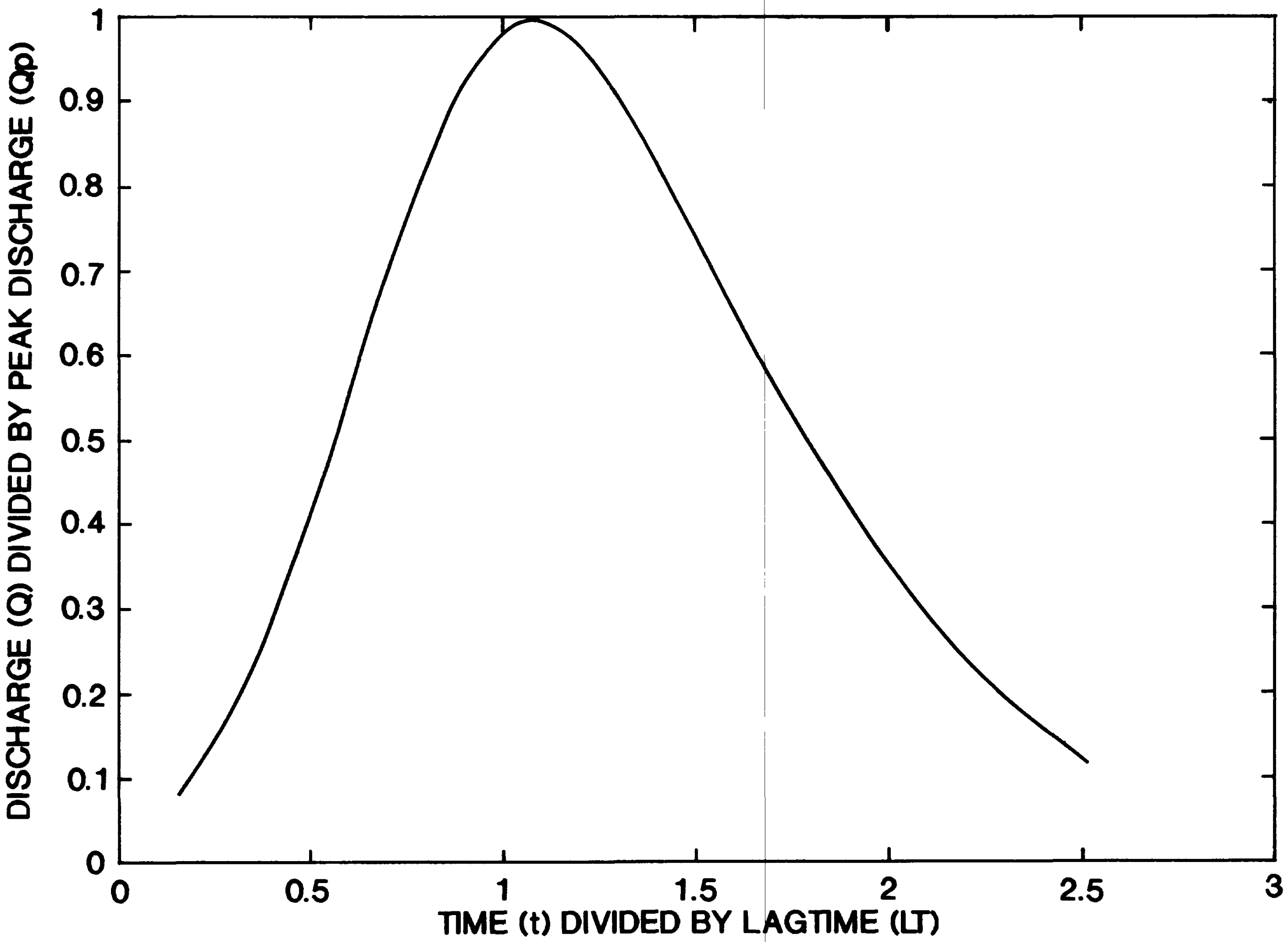

Figure 8.--Dimensionless hydrograph for the Upper Coastal Plain and Lower Coastal Plain subprovinces.

Table 3.--Time and discharge ratios of the dimensionless hydrographs for the indicated regions

Time ratio (t/LT_A)	Discharge ratio (Q/Qp) Blue Ridge province	Piedmont province	Coastal Plain province
0.15	0.08	0.07	.07
.20	.14	.09	.10
.25	.22	.11	.14
.30	.31	.14	.18
.35	.43	.17	.23
.40	.56	.21	.29
.45	.69	.25	.35
.50	.80	.30	.42
.55	.89	.37	.50
.60	.96	.44	.57
.65	.99	.53	.64
.70	1.00	.61	.71
.75	.97	.70	.78
.80	.93	.78	.85
.85	.88	.86	.90
.90	.82	.92	.94
.95	.76	.96	.97
1.00	.71	.99	.99
1.05	.65	1.00	1.00
1.10	.60	.98	.99
1.15	.56	.96	.98
1.20	.51	.91	.95
1.25	.47	.86	.92
1.30	.44	.80	.88
1.35	.41	.74	.84
1.40	.38	.69	.80
1.45	.35	.63	.76
1.50	.33	.58	.72
1.55	.30	.53	.68
1.60	.28	.49	.63
1.65	.26	.44	.59
1.70	.24	.41	.55
1.75	.23	.37	.51
1.80	.21	.34	.48
1.85	.20	.32	.44
1.90	.19	.29	.40
1.95	.17	.27	.37
2.00	.16	.25	.34
2.05	.15	.23	.31
2.10	.14	.21	.28
2.15	.14	.19	.25
2.20	.13	.18	.23
2.25	.12	.16	.20
2.30	.12	.15	.18
2.35	.11	.13	.17
2.40	.10	.12	.15
2.45	.10	.11	.13
2.50	.09	.10	.11

Note: t = time
LT_A = lagtime adjusted for correct runoff volume
Q = discharge
Qp = peak discharge

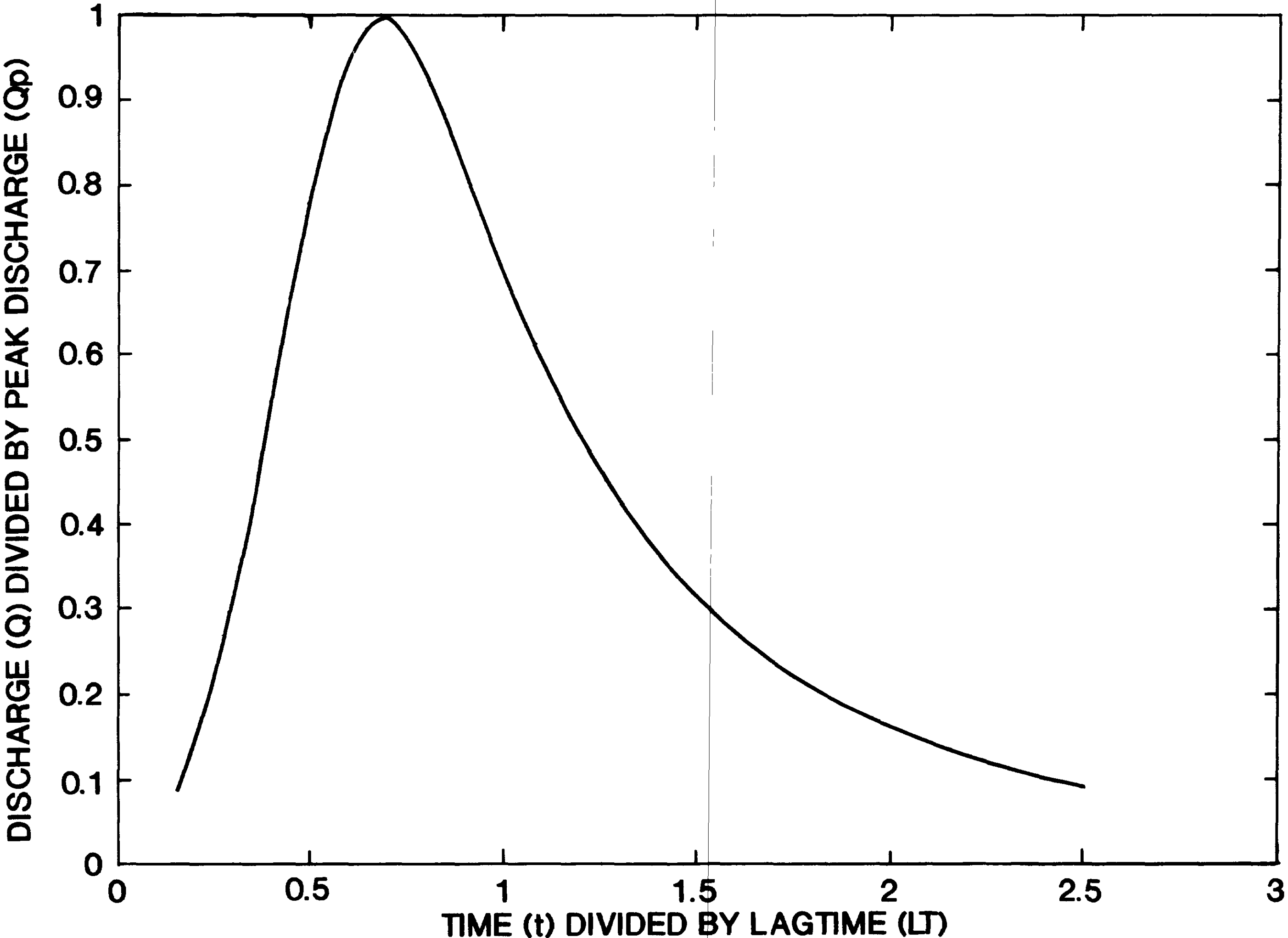

Figure 9.--Dimensionless hydrograph for the Blue Ridge province.

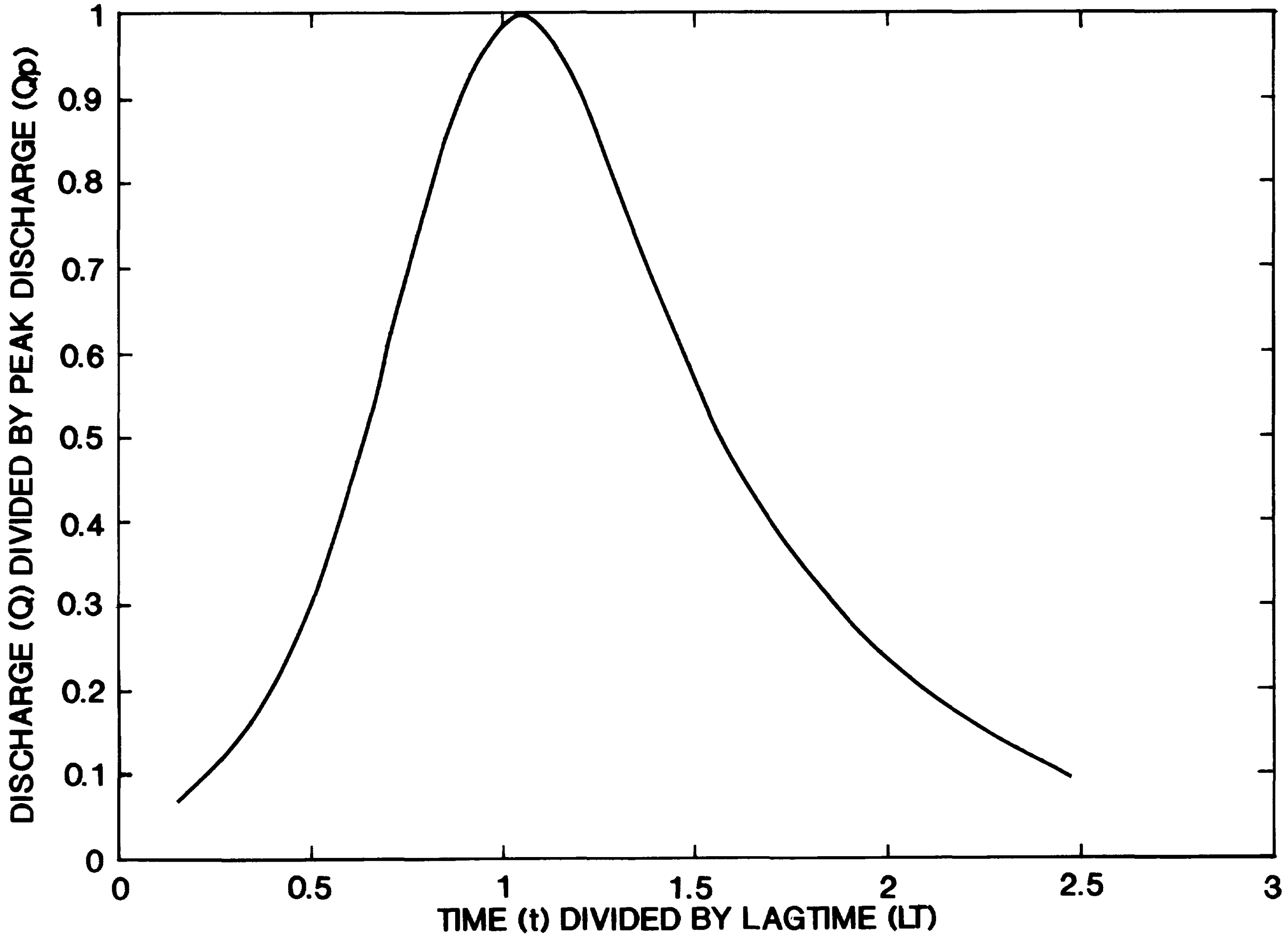

Figure 10.--Dimensionless hydrograph for the Piedmont province.

dimensionless hydrograph derived from the four stations in South Carolina. These results did not favor the use of a separate dimensionless hydrograph for basins with these characteristics. It was later found that results for these basins could be significantly improved by applying a correction factor to lagtime that was based on volume regression analyses (see section entitled "Adjusting basin lagtime for correct runoff volume").

Runs 9, 14, 17, 19, and 25 of table 2 have not been discussed but may be of some interest to the reader.

A comparison of the dimensionless hydrographs developed in this study, with that of the U.S. Soil Conservation Service (SCS) is illustrated in figure 11. Details on the development of the SCS dimensionless hydrograph were described by the U.S. Department of Agriculture (1972). The SCS hydrograph is similar in shape to the Coastal Plain dimensionless hydrograph, and both are appreciably wider than the Blue Ridge and Piedmont dimensionless hydrographs. Only one Coastal Plain station with a channel slope of less than 10 ft/mi was used in the Inman study to develop his dimensionless hydrograph and his verification stations were all located in a more upland physiography. Most of the Coastal Plain stations in the South Carolina study are in a more lowland physiography (15 of 21 stations had channel slopes less than 10 ft/mi). The Inman (1986) and Stricker and Sauer (1982) dimensionless hydrographs (not illustrated) are almost identical in shape to the South Carolina Piedmont one-half-lagtime duration hydrograph.

Adjusting Basin Lagtime for Correct Runoff Volume

The volume of runoff associated with each of the dimensionless hydrographs can be estimated by equations of the form:

$$V = (K)(Qp)^{1.0}(LT)^{1.0}(A)^{-1.0} \quad , \qquad (2)$$

where V is runoff volume, in inches;
K is a conversion constant;
Qp is peak discharge, in cubic feet per second;
LT is lagtime, in hours; and
A is drainage area, in square miles.

The constant (K) is calculated by first extrapolating the rising and falling limbs of each dimensionless hydrograph to a discharge ratio of zero. The discharge-ratio ordinates are then summed at time-ratio intervals of 0.05. This sum is then multiplied by time and drainage area conversion constants in order to provide volume in watershed inches. The K values from equation 2 for the three dimensionless hydrographs resulting from this study are:

$$K_{\text{Blue Ridge}} = 0.00166$$

$$K_{\text{Piedmont}} = 0.00176$$

$$K_{\text{Coastal Plain}} = 0.00202$$

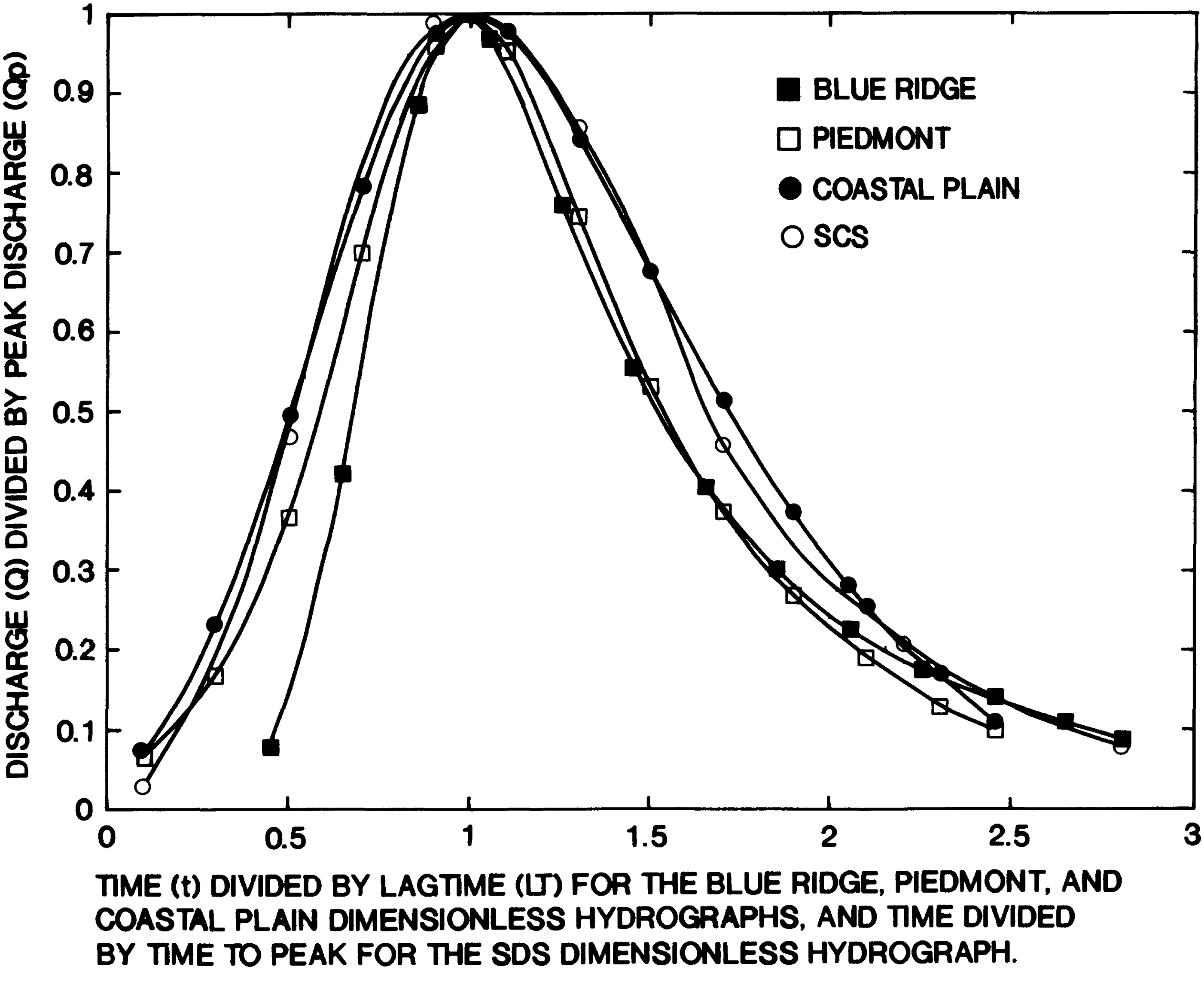

Figure 11.--Dimensionless hydrographs for the Blue Ridge, Piedmont, and Coastal Plain provinces, and the U.S. Soil Conservation Service dimensionless hydrograph.

The volume was computed for each of the 188 events used to develop the dimensionless hydrographs by using equation 2 and the appropriate regional K value. The standard error of volume was computed for each region. Plots of residuals versus independent variables were made to detect trends (variable bias), and plots of observed versus predicted runoff volumes were prepared for each region. The standard errors were low and, with the plotting scales used, there was no conspicuous variable bias.

Regression equations for estimating runoff volume on the basis of the same three explanatory variables (lagtime, drainage area, and peak discharge) did not yield runoff estimates similar to those estimated from the dimensionless hydrographs in many instances. This was further complicated by the fact that the regression residuals also showed no bias and the standard error of estimate, although slightly less, was quite close to that of the dimensionless hydrograph.

After careful examination it was concluded that there was some bias in the runoff volumes derived from the dimensionless hydrograph when the method was applied to smaller peaks and drainage areas that the multiple regression method could discern and adjust for. One possible explanation for this inconsistency is that many more large basins were used than small ones, especially in the Piedmont. Because the dimensionless hydrograph is an arithmetic mean of the dimensionless hydrographs for many individual basins, a bias may result which favors the more numerous larger basins. Recall that very poor results for small basins were obtained in the Piedmont dimensionless hydrograph shape analysis.

This led to the development of a volume adjustment factor to be applied to the average basin lagtime prior to simulation using the dimensionless hydrograph. Thus the basic shape of the hydrograph is preserved while using volume as a normalizing variable. The correction factor (F) may be computed by calculating the ratio of regression to dimensionless volumes in each case. This computation was reduced to single equations applicable in the indicated regions (see plate 1):

$$F_{\text{Blue Ridge}} = 2.277\, A^{0.089}\, Qp^{-0.112}\, LT^{-0.121} \quad , \tag{3}$$

$$F_{\text{Piedmont}} = 1.374\, A^{0.202}\, Qp^{-0.120}\, LT^{-0.104} \quad , \tag{4}$$

$$F_{\text{Upper Coastal Plain}} = 1.908\, A^{0.074}\, Qp^{-0.010}\, LT^{-0.279} \quad , \tag{5}$$

$$F_{\text{Lower Coastal Plain, region 1}} = 1.313\, A^{0.047}\, Qp^{-0.022}\, LT^{-0.118} \quad , \tag{6}$$

$$F_{\text{Lower Coastal Plain, region 2}} = 1.422\, A^{0.047}\, Qp^{-0.022}\, LT^{-0.118} \quad , \tag{7}$$

where F is volume correction factor;
A is basin drainage area, in square miles;
Qp is peak discharge, in cubic feet per second; and
LT is average basin lagtime, in hours.

An "adjusted" basin lagtime (LT_A) required for simulating flood hydrographs can be computed as follows:

(1) Compute the average basin lagtime (LT) from the appropriate equation in table 11 (derivation of LT equations will be discussed in the section entitled "Estimating Average Basin Lagtime").

(2) Compute the lagtime correction factor (F) from the appropriate equation (3-7) above.

(3) Multiply the results of steps (1) and (2) to obtain the adjusted lagtime.

Steps (1), (2), and (3) can be combined algebraically into one equation for each physiographic province, subprovince or region. Equations 8-12 can be substituted for the three-step process described above.

Province or subdivision	Equation	
Blue Ridge	$LT_A = 7.21\ A^{0.322}\ Q_p^{-0.112}$	(8)
Piedmont	$LTA = 3.30\ A^{0.614}\ Q_p^{-0.120}$	(9)
Upper Coastal Plain	$LT_A = 7.03\ A^{0.375}\ Q_p^{-0.010}$	(10)
Lower Coastal Plain		
Region 1	$LT_A = 6.95\ A^{0.348}\ Q_p^{-0.022}$	(11)
Region 2	$LT_A = 11.7\ A^{0.348}\ Q_p^{-0.022}$	(12)

Where LT_A is basin lagtime adjusted to achieve the correct volume, in hours;
A is the basin drainage area, in square miles; and
Q_p is the peak discharge, in cubic feet per second.

By use of equations 8 through 12, many of the statistical analyses shown in table 2 were repeated for comparison. These results are shown in table 4. The results indicate that the use of the adjusted basin lagtimes (LT_A) lowered the standard errors of estimate and, therefore, should be used in place of average basin lagtime (LT) before simulating a hydrograph with the dimensionless hydrograph technique.

HYDROGRAPH-WIDTH RELATIONS

Some hydraulic analyses require only an estimate of the period of time during which a specific discharge will be exceeded for a given flood. In these cases a complete hydrograph is not needed and the hydrograph widths can be determined from the hydrograph-width relations, shown graphically in figure 12 and tabulated in table 5. The hydrograph-width ratios were determined by subtracting the value of t/LT on the rising limb of the

Table 4.--Statistical summary of dimensionless hydrograph regional analysis after applying the volume-adjustment factor to the average station lagtimes (Note: station group names and run numbers correspond to those in table 2 for purposes of comparison)

Run number	Stations used to develop the average dimensionless hydrograph	Stations compared to the average dimensionless hydrograph	Standard error of estimate, in percent, of hydrograph widths at 50 and 75 percent of observed peak discharge (Qp)		Lagtime duration	Number of hydrograph widths overestimated (+) or underestimated (-) at 0.75 Qp	
			0.50 Qp	0.75 Qp			
10	BLUER	BLUER	14.1	18.3	1/3	9+	8-
11	BLUER	VBLUER	19.9	29.8	1/3	6+	1-
12	PIED	PIED	29.2	36.2	1/2	34+	38-
13	PIED	VPIED	30.9	31.1	1/2	4+	5-
14	PIED	PIEDSL>50DA<5	35.3	42.5	1/2	11+	6-
26	CP	CP	17.8	22.8	1/2	25+	37-
27	CP	UCP	16.7	24.0	1/2	11+	17-
28	CP	LCP	18.6	21.5	1/2	16+	24-
--	CP	VCP	15.1	23.1	1/2	6+	14-
29	CP	VUCP	9.1	12.0	1/2	1+	7-
30	CP	VLCP	18.5	28.9	1/2	5+	7-

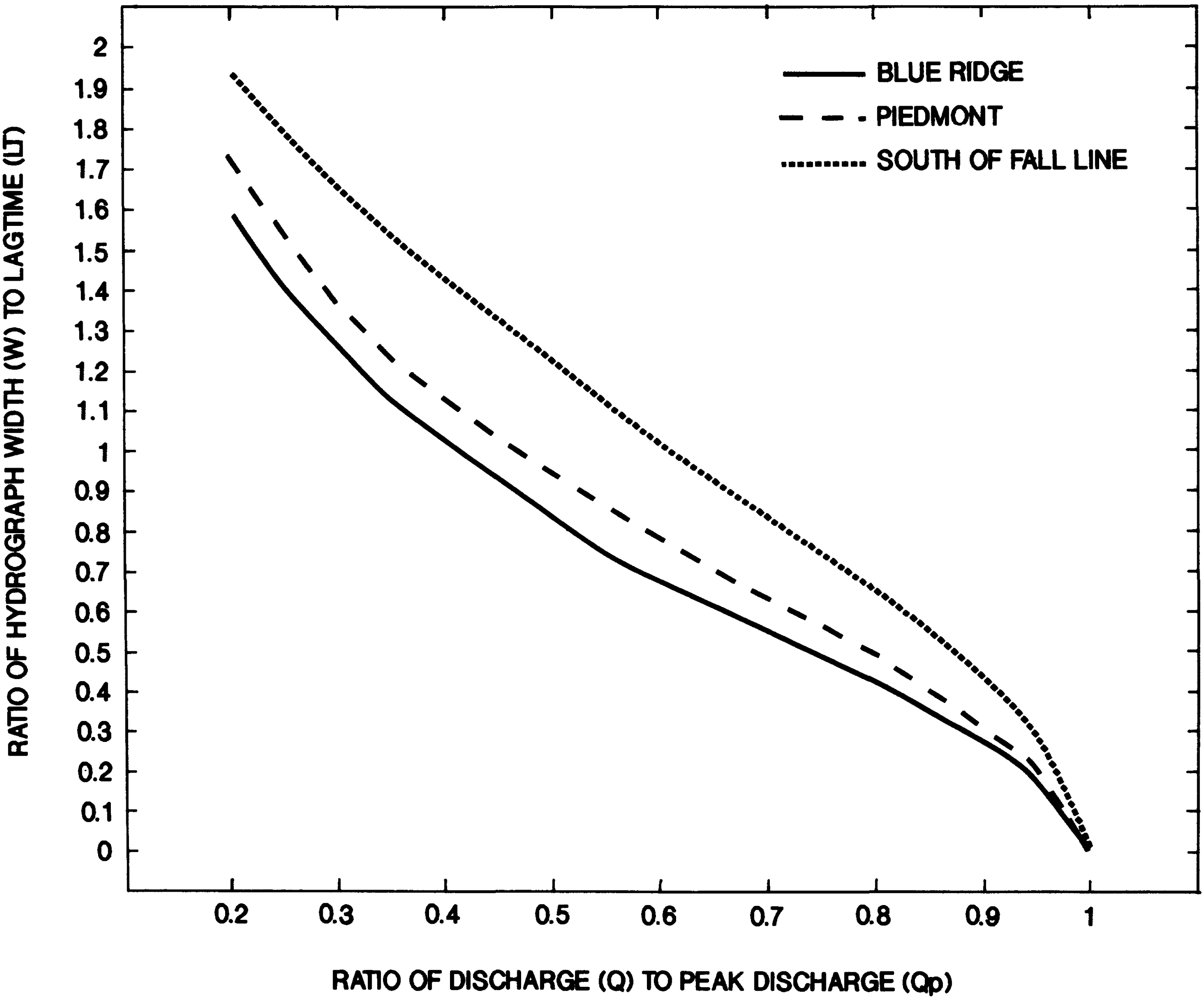

Figure 12.--Hydrograph-width relations for the indicated dimensionless hydrographs.

Table 5.--Relation of discharge ratios to hydrograph width ratios for drainage basins in the indicated regions

Discharge ratio (Q/Qp)	Width ratio (W/LT)		
	Blue Ridge province	Piedmont province	Coastal Plain province
1.00	0.00	0.00	0.00
.95	.18	.22	.30
.90	.27	.32	.43
.85	.34	.41	.55
.80	42	.50	.65
.75	.48	.57	.74
.70	.54	.64	.83
.65	.61	.71	.92
.60	.68	.79	1.02
.55	.74	.87	1.11
.50	.84	.95	1.22
.45	.92	1.04	1.32
.40	1.02	1.14	1.43
.35	1.12	1.24	1.53
.30	1.26	1.38	1.65
.25	1.41	1.55	1.79
.20	1.60	1.74	1.94

Note: Q = Discharge
Qp = Peak discharge
W = Hydrograph width
LT = Lagtime

dimensionless hydrographs from the value of t/LT on the falling limb of the hydrograph at the same discharge ratio (Q/Qp) over the full range of each dimensionless hydrograph. The simulated hydrograph width (W) in hours can be estimated for a desired discharge (Q) by first computing the ratio Q/Qp and then multiplying the corresponding W/LT ratio in table 5 by the estimated basin lagtime that has been corrected for volume (LT_A). The resulting hydrograph width is the period of time a particular discharge will be exceeded.

TESTING OF DIMENSIONLESS HYDROGRAPHS

Several tests were made to evaluate the validity of the average dimensionless hydrograph models. The standard error of estimate of simulated hydrograph widths was the first test and was explained in the section covering the development of the dimensionless hydrographs. The other tests were for verification, bias, and sensitivity.

Verification

The standard error of estimate is a measure of how well a model performs at the sites used to develop it. The standard error of prediction, on the other hand, is a measure of how well the model works at stations other than those used in the development of the model (Sauer and others, 1983). The dimensionless hydrographs were verified with 37 flood events from 10 basins not used in their development. These basins were selected prior to development of the dimensionless hydrograph to represent a wide range of basin characteristics throughout the State. Average basin lagtimes (LT, not adjusted for volume) were determined by unit hydrograph computations, as in the development phase, and used with observed peak runoff discharge to simulate flood hydrographs. Predicted and observed hydrograph widths at 50 and 75 percent of peak stormflow were then compared in the same manner as shown in figure 6 and are reported in table 2 (runs 11, 13, 29, and 30). Using the adjusted lagtimes (LT_A), as described earlier, the standard errors of prediction of the simulated hydrograph widths at 50 and 75 percent of peak stormflow for verification basins in the Coastal Plain province were ±15.1 and ±23.1 percent, respectively. Verification basins in the Piedmont province had standard errors of prediction of ±30.9 and ±31.1 percent for hydrograph widths at 50 and 75 percent of observed peak stormflow. Blue Ridge verification stations had standard errors of prediction of ±19.9 and ±29.8 percent. These results are presented in table 4.

Nearly all the events used to develop and verify the hydrographs to this point have been relatively small in magnitude (less than annual floods). Also, measured average basin lagtimes have been used in the simulation comparisons. An additional verification test was made which may be a better measure of the simulation procedure accuracy within the recurrence interval range in which it will be used. In this test, the peak-of-record floods (simple or compound hydrographs) from 20 gaging stations with long-term records and station frequency curves as defined by the Water Resources Council (1982) were selected. The average recurrence interval for these events was approximately 30 years. The group of stations consists of 12 Piedmont basins, 4 Blue Ridge basins, and 4 Coastal Plain basins.

A flood hydrograph was simulated by using the observed peak discharge, the regression average basin lagtime, the volume adjustment factor, and the proper average dimensionless hydrograph. The observed peak was used to provide an improved estimate of the error associated with the methods described in this report alone. There usually is some difference between discharge frequency curves developed from long-term gaging station records and those estimated from regression relations. Unless there is some bias in the regression flood frequency relations, one would expect that the use of the corresponding regression-estimated peak discharge rather than the observed peak would produce less accurate results in some cases and improve the results in other cases in the additional verification.

Adjusted lagtimes were calculated from equations 8-12. The lagtimes were weighted according to the percentage of the basin located in each physiographic province. Only one dimensionless hydrograph (representative of the majority of the basin) was used in each case.

A comparison of the simulated and observed hydrograph widths at 50 and 75 percent of peak discharge yielded standard errors of prediction of ±31.7 and ±37.1 percent, respectively. An example of this comparison is depicted graphically in figure 13. A tabulation of the results of this additional verification step is in table 6.

Bias

Two tests for width bias were made, using the 20 events in the additional verification tests. An average-bias test involved simply computing the mean residuals (in percent) at 50 and 75 percent of peak flow. The mean errors were negative (simulated less than observed), but the students t-test indicated the bias to be not statistically significant at the 0.01 level of significance. The simulated hydrograph widths are, therefore, not considered biased.

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 presence, if any, of geographical bias in the simulated hydrographs. Although the residual differences in widths varied considerably between some stations, no specific geographic trends could be detected.

Sensitivity

Peak discharge and basin lagtime for ungaged basins are usually computed from regional regression equations or other methods that use basin characteristics measured from maps and are therefore subject to errors in measurement and judgment. To illustrate the effect of such errors in application of the dimensionlesss hydrograph, a sensitivity test was made by holding one of the two independent variables constant and varying the other by +10 and +20 percent and then comparing the hydrograph widths corresponding to 50 and 75 percent of peak flow in each case. When peak discharge was varied, the hydrograph widths did not change at 50 and 75 percent of that varied peak discharge. When lagtime was varied, the hydrograph widths varied by an equal percentage.

ESTIMATING AVERAGE BASIN LAGTIME

Basin lagtime, the principal time factor used in applying the dimensionless hydrograph, locates the hydrograph's position relative to the causative storm pattern. It has been defined as the time from center-of-mass of rainfall excess to center-of-mass of the resultant runoff hydrograph (Stricker and Sauer, 1982). The lagtime value used in expanding the dimensionless hydrograph will determine whether the simulated hydrograph shape is sharp-crested (short lagtime) or broad-crested (long lagtime). Lagtime may generally be considered constant as long as land use and other basin conditions remain the same; however, a study by Horner and Flynt (1956) led to the inference that lagtime was a variable, its value being determined more by rainfall characteristics than by such physical attributes

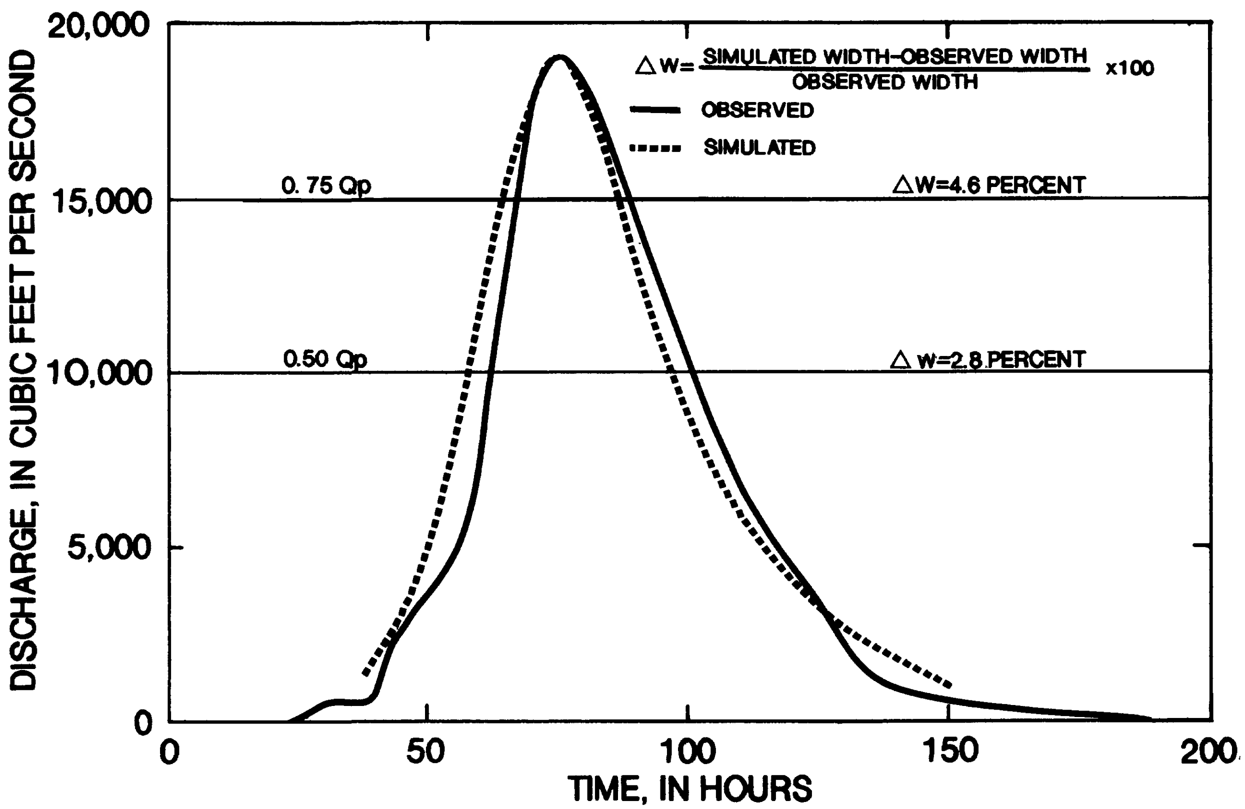

Figure 13.--Observed hydrograph and hydrograph simulated by using observed peak discharge and regression lagtime for Enoree River at Whitmire (sta. no. 02160700), October 7, 1976.

Table 6.--Standard error and the individual and mean percentage differences between estimated and observed runoff volumes and hydrograph widths at 50 and 75 percent of observed peak flow for selected peak-of-record events

Station number	Estimated hydrograph width at 50 percent of peak flow (hours)	Observed hydrograph width at 50 percent of peak flow (hours)	Percentage difference	Estimated hydrograph width at 75 percent of peak flow (hours)	Observed hydrograph width at 75 percent of peak flow (hours)	Percentage difference	Estimated[1] runoff volume (inches)	Observed runoff volume (inches)	Percentage difference
02131150	23.09	35.02	-34.1	14.02	21.61	-35.1	1.15	2.31	-50.1
02148300	32.24	27.14	18.8	19.59	10.25	91.0	0.54	0.42	28.6
02154500	15.77	14.94	5.6	9.39	9.32	0.0	3.11	3.18	-2.2
02157500	15.07	23.55	-36.0	8.97	13.52	-33.7	1.96	2.91	-32.6
02158000	23.06	24.95	-7.6	13.73	14.00	-1.9	3.20	3.40	-5.9
02159000	25.03	30.01	-16.6	14.90	17.73	-16.0	2.31	2.74	-15.7
02160000	26.41	50.17	-47.4	15.72	28.20	-44.3	1.96	3.26	-39.9
02160700	40.61	39.50	2.8	24.17	23.10	4.6	3.26	3.10	5.2
02162010	12.39	8.84	40.0	7.37	6.19	19.2	2.23	1.54	44.8
02162500	22.00	35.73	-38.4	13.09	22.43	-41.6	1.22	1.82	-33.0
02163000	26.93	33.74	-20.2	16.03	17.75	-9.7	1.57	1.84	-14.7
02165000	29.52	24.30	21.5	17.57	11.70	50.1	2.48	1.86	33.3
02165200	9.33	17.04	-45.3	5.55	11.84	-53.1	2.34	3.79	-38.3
02169550	47.67	44.62	6.8	28.94	28.53	1.4	0.86	0.83	4.8
02171680	32.93	30.97	6.3	19.97	18.70	6.8	2.77	3.02	-8.3
02184500	7.89	7.74	1.9	4.53	4.48	1.0	2.08	3.24	-35.8
02185000	9.93	11.76	-15.6	5.70	6.12	-6.9	2.63	3.77	-30.2
02185200	9.59	9.65	-0.6	5.51	4.77	15.4	3.14	2.69	16.7
02185500	21.53	27.83	-22.7	12.36	16.85	-26.6	2.21	2.27	-2.6
02186000	19.90	26.92	-26.1	11.84	17.64	-32.9	1.66	1.92	-13.5
MEAN PERCENTAGE ERROR			-10.3			-5.6			-9.5
STANDARD ERROR			31.7			37.1			36.7

[1]Estimated by using regression equations rather than dimensionless hydrograph equations.

of the watershed as size, slope, shape, and storage capacity. An attempt was made to relate rainfall amount and intensity to lagtime, employing the same data base used to develop and verify the dimensionless hydrographs. The greatest intensity for 60 percent of rainfall and the total rainfall amount were analyzed for their relation to individual event lagtimes (not average basin lagtimes). In no case did the inclusion of either rainfall variable increase the accuracy of the estimating equations. The fact that no relation was found is not conclusive evidence that rainfall patterns have no influence on lagtimes. Instead, these results merely point out the need for better definition of rainfall characteristics in future studies. In this study, storm distributions and amounts for the large basins were estimated rather than observed; and the smaller basins had only one rain gage but were of sufficient size to warrant multiple rain gages.

In this study average basin lagtime was related to various physical and geometric basin characteristics by linear, multiple regression techniques as described by Riggs (1968). Average basin lagtime, the dependent variable, was computed for 48 basins by the O'Donnell (1960) method, which was used to compute unit hydrographs in the dimensionless hydrograph development phase. The following paragraphs define lagtime and the independent variables whose relation to lagtime was examined in the regression analyses. Average basin lagtime and selected independent variables for the basins used in these analyses are listed in tables 7-10.

Average basin lagtime (LT).--The average time, in hours, from the centroid of the rainfall excess to the centroid of the resultant runoff hydrograph.

Drainage area (A).--Area of the basin, in square miles, planimetered from U.S. Geological Survey 7.5- and 15-minute topographic maps.

Main channel slope (S).--The slope of the main channel, in feet per mile, measured from a topographic map between points 10 percent and 85 percent of the main channel length upstream from the gaging site.

Main channel length (L).--Length of the main channel, in miles, from the gaging station to the most distant point on the basin divide.

Lagtime index (LI).--A ratio, $L/S^{0.5}$, where L and S have been previously defined.

Sinuousity (SIN).--A measure of stream sinuousity defined as the ratio of the main channel length to the length measured by 1-mile chords, less 1.0.

Basin storage (ST).--The percentage of the basin occupied by lakes, reservoirs, swamps, and wetlands. In-channel storage of a temporary nature, resulting from detention ponds or roadway embankments, is not included in the computation of ST.

Shape factor (SH).--A dimensionless measure of shape defined as the drainage area divided by the squared length of the basin (Chow, 1964, p. 4.51).

Length to center of gravity (LCG).--Distance, in miles, from the gaging station to a point on the main channel opposite the center of gravity (centroid) of the total drainage area (Chow, 1964, p. 4-47).

Storage indicator (SI).--An index of overbank storage (C.L. Sanders, written communication, 1987) computed by first determining the top width of flow for a depth of 10 feet at the main channel, measured from topographic maps at points located at 10, 30, and 50 percent of the main channel length upstream of the gaging station. These widths are then divided into 10 and averaged for the final SI value.

Drainage density (DD).--Total length of all channels in the drainage system divided by the drainage area.

Table 7.--Selected physical characteristics of basins of the Blue Ridge physiographic province

Station number	Lag-time (LT) (hours)	Channel slope (S) (feet per mile)	Drainage area (A) (square miles)	Channel length (L) (miles)
02184500	11.51	250.0	48.50	14.00
02185000	12.41	111.8	148.00	28.40
02185020	8.77	115.3	30.20	12.00
02185200	11.78	38.4	72.00	18.20
02185500	19.64	64.8	455.00	50.30
02185600	4.88	87.3	2.83	2.20

Table 8.--Selected physical characteristics of basins in the Piedmont physiographic province

Station number	Lagtime (LT) (hours)	Channel slope (S) (feet per mile)	Drainage area (A) (square miles)	Channel length (L) (mile)
02131309	16.38	32.4	24.30	10.20
02147500	23.25	8.2	194.00	31.50
02147600	4.16	64.3	4.61	3.50
02154500	28.09	30.9	116.00	32.70
02157500	26.35	10.9	68.30	24.30
02158000	37.77	10.3	162.00	45.30
02158500	20.49	8.8	106.00	33.90
02159000	29.18	9.0	174.00	53.00
02159500	35.88	8.8	351.00	54.00
02160000	28.62	9.3	183.00	41.70
02160700	51.98	8.0	444.00	83.90
02162005	2.85	97.8	1.13	1.80
02162010	10.42	14.8	48.90	15.80
02162500	32.63	12.1	295.00	46.40
02163000	32.56	6.4	405.00	64.90
02165000	36.81	10.1	236.00	58.50
02165200	19.34	15.1	29.90	13.10
02167200	2.39	113.2	0.62	1.10
02167750	2.30	98.7	0.52	1.00
02186000	20.16	11.6	106.00	21.60
02192500	27.48	7.8	217.00	41.70
02195660	1.92	103.0	1.26	1.20

Table 9.--Selected physical characteristics of basins in the Upper Coastal Plain physiographic subprovince

Station number	Lag-time (LT) (hours)	Channel slope (S) (feet per mile)	Drain-age area (A) (square miles)	Channel length (L) (miles)
02130500	49.70	9.1	64.00	17.50
02135500	56.47	3.8	401.00	41.70
02147900	13.42	71.8	2.92	2.60
02148300	25.90	13.7	40.20	11.00
02169550	41.43	15.6	122.00	17.90
02169630	9.88	50.5	10.10	3.70
02175500	95.46	4.3	341.00	44.30
02197410	13.40	28.0	7.82	4.00

Table 10.--Selected physical characteristics of basins in the Lower Coastal Plain physiographic subprovince

Station number	Lag-time (LT) (hours)	Channel slope (S) (feet per mile)	Drain-age area (A) (square miles)	Channel length (L) (miles)	Region
02110700	13.02	5.6	14.00	4.80	1
02131150	21.07	4.5	27.70	12.80	1
02131990	13.35	9.4	8.40	4.40	1
02132100	11.66	4.5	18.40	9.20	1
02135050	22.83	5.9	10.40	7.20	1
02135500	56.47	3.8	401.00	41.70	1
02136010	18.71	4.1	14.60	6.20	1
02171680	26.94	6.0	17.40	5.10	2
02174250	29.84	6.3	23.40	12.00	2
02174300	31.22	7.0	11.90	7.00	2
02175450	19.42	11.3	12.40	6.50	2
02175500	95.46	4.3	341.00	44.30	2
02176100	27.06	14.1	7.67	5.20	2
02176500	55.20	7.1	203.00	22.50	2

Stepwise regression analyses were made using P-STAT (P-STAT, Inc., 1986), a file management, data modification, and statistical analysis computer system. All variables were transformed into logarithms before analysis to : (1) obtain a linear regression model, and (2) achieve equal variance about the regression line throughout the range (Riggs, 1968, p. 10). A 95-percent confidence limit was specified to select the significant independent variables.

Accuracy of linear multiple-regression techniques can be expressed by two standard statistical measures, the coefficient of determination (R^2) and the standard error of regression (or estimate). The R^2 statistic indicates the proportion of the total variation of the dependent variable that is explained by the independent variables. For example, an R^2 of 0.93 would indicate that 93 percent of the variation in the dependent variable is accounted for by the independent variables. The standard error of regression is, by definition, the standard deviation of the residuals of the regression equation and contains about two-thirds of the data within this range at the 95-percent confidence level. Conversely, about one-third of the data will fall outside of the standard error of regression.

Initial tests were made to determine if one equation would adequately predict lagtime for all of South Carolina. A distinct geographical bias was evidenced by consistent overprediction of lagtime in the areas north of the Fall Line and consistent underpredictions south of the Fall Line.

Next, regression runs were made for each of the four physiographic provinces and subprovinces and for the two composite regions north and south of the Fall Line. During this part of the analysis, many combinations of independent variables were tested. The correlation matrices showed that, depending on the region, either the lagtime index (LI) or channel length (L) was most closely related to lagtime and, in both cases, was closely followed by drainage area. A volume problem arises, however, if any basin characteristic other than drainage area is used, because regional flood frequency relations for South Carolina are dependent solely on drainage area. Consider, for example, two basins with the same drainage area; one might be long and narrow while the other is more circular in shape. The lagtime for the narrow basin would probably be longer and, because it has the same regression peak discharge, simulation using the dimensionless hydrograph would suggest that the long narrow basin also produces a larger volume of runoff. If length had been incorporated into the peak discharge regressions, the longer lagtime might have been offset by the smaller peak expected in narrow basins and the runoff volume for the two basins would have been nearly alike. Similar examples can be found for other basin characteristics, such as slope and ground cover. Until a broader range of data is collected (especially for small basins), the influence of basin characteristics other than drainage area on peak discharge and lagtime cannot be defined properly. In order to avoid possible volume discrepancies and because the use of LI or L provided predictions that were only a few percent better than DA alone, it was decided that DA would be the only independent variable in the lagtime estimating equations. In no case did the use of parameters other than DA improve the standard error by more than 4 percent.

When all stations north of the Fall Line were included in one group, a bias was detected in plots of observed versus predicted lagtime by province. Therefore, separate equations were developed for the Blue Ridge and Piedmont provinces. It should be noted that the standard error of estimate for the Blue Ridge equation may be low because of the limited data base (6 stations). The final Blue Ridge and Piedmont estimating equations for average basin lagtime and the corresponding statistical measures are shown in table 11.

Table 11.--Summary of average basin lagtime estimating equations

Province or subprovince	Number of observa-tions	Equation	Standard error of regression (percent)	Coefficient of determination, (R^2)
Blue Ridge	6	$3.71\ A^{0.265}$	±7.3	0.97
Piedmont	22	$2.66\ A^{0.460}$	±25.6	.96
Upper Coastal Plain	8	$6.10\ A^{0.417}$	±34.3	.85
Lower Coastal Plain	14		±25.6	.85
Region 1		$6.62\ A^{0.341}$		
Region 2		$10.88\ A^{0.341}$		

When data from the two physiographic provinces south of the Fall Line were combined, large standard errors resulted and a geographical bias was evident. Even after the Upper and Lower Coastal Plains were separated, an areal bias was noted in the Lower Coastal Plain. Only 14 stations were available for analyses in the Lower Coastal Plain. To include as wide a range of data as possible, all 14 stations were used in the regression with a qualitative location variable that classified lagtimes according to region of the Lower Coastal Plain. All stations east of the Santee River were assigned a location value of 0, and stations west of the Santee River were given location values of 1. The regression was repeated treating location as a basin characteristic without transformation into log units. The resulting model had two constants that differentiated the two regions and made a significant improvement in the results. The final estimating equations for average basin lagtime in the Upper Coastal Plain and the two regions in the Lower Coastal Plain and their statistical measures are presented in table 11.

TESTING OF AVERAGE BASIN LAGTIME ESTIMATING EQUATIONS

The average basin lagtime regression equations were tested for accuracy (standard error of the estimate), bias, and sensitivity. According to Myers (1986), there were not enough observations in each of the final regions to permit the use of split-sample verification techniques.

Bias

Two tests for bias were made, one for variable bias and the other for geographical bias. The variable-bias tests were done by plotting regression residuals (in percent) versus observed lagtimes and residuals versus drainage area. 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.

Residuals for each station were also plotted on a State map to ascertain the presence of any geographic bias in the estimating equations. Although the residuals varied considerably between stations, no specific geographic trends could be detected.

Sensitivity

To illustrate the resultant effect in average basin lagtime of error in the estimation of drainage area, a sensitivity test of the four regional equations was made. This was done by introducing errors of a specified percentage in the value of the drainage area and computing the consequential error in estimating basin lagtime, the dependent variable. The results are shown in table 12.

Table 12.--Sensitivity of computed average basin lagtime to errors in the independent variable, drainage area

Percent error in drainage area	Percent error in computed lagtime for indicated provinces and subprovinces			
	Blue Ridge	Piedmont	Upper Coastal Plain	Lower Coastal Plain (Regions 1 and 2)
-50	-16.8	-27.3	-25.1	-21.1
-25	-7.3	-12.4	-11.3	-9.3
-10	-2.8	-4.7	-4.3	-3.5
+10	2.6	4.5	4.1	3.3
+25	6.1	10.8	9.8	7.9
+50	11.3	20.5	18.4	14.8

ESTIMATING RUNOFF VOLUME

Floodwater detention storage is an important consideration in the economic design of some hydraulic structures. In such cases, the volume of runoff associated with a design flood must be estimated. It is important to realize that it cannot be assumed that any given flood event will have a peak discharge and runoff volume of the same recurrence interval. Therefore it must be emphasized that the runoff volumes estimated by using procedures in this study represent only the average volumes that would occur with the associated peak discharges and recurrence interval. Thus, for a 100-year peak discharge, the hydrograph and volumes simulated by the methods described herein can be expected to occur on the average. The term "on the average" is fundamentally significant because it has been demonstrated (Sauer, 1964) that a wide variation may exist between the recurrence intervals computed for storm runoff and corresponding peak discharge. This variation was explained by several factors that affect the relation between storm runoff and peak discharge. Sauer explained that two storms having identical total runoff may have different peaks because of different storm durations or a different distribution of rainfall over the basin. Other factors given by Sauer that may cause variations in the peak discharge-storm runoff relation are direction of storm movement and the flow in the channel at the time of storm runoff, which may include only base flow or base flow plus flow from the recession of a previous storm.

This is not to say that there is little or no relation between peak discharge and runoff volume. On the contrary, a literature search revealed that many studies, such as those by Rogers and Zia (1982) and Singh and Aminian (1986), have demonstrated the linearity that exists between peak discharge and runoff volume. The South Carolina data used in this report also indicated a strong relation between peak discharge and runoff volumes. Plots of peak discharge versus runoff volume at each station for the 188 events used in the South Carolina study were made. A generally linear relation was observed, especially at stations that had events covering a wide range of discharges. A simple regression analysis was done using the same data set with volume as the dependent variable and only peak discharge and drainage area as independent variables. The coefficient of determination, R^2, was very high; 0.97 for the Blue Ridge province, 0.93 for the Piedmont province, 0.90 for the Upper Coastal Plain, and 0.77 for the Lower Coastal Plain. Finally, recall the form of equation 2 for volume of the dimensionless hydrographs. The drainage area and K are constants and, assuming a linear basin in which LT remains essentially constant, the volume will vary directly with the peak discharge.

To summarize this short discussion of peak discharge and runoff volume, it is sufficient to say that although the two parameters are highly correlated, their frequency relation is a complex one. Just as there are many possible volumes that can be associated with a peak discharge of specific recurrence interval, there are likewise many possible peaks for a storm runoff of given recurrence interval. Efforts to develop a separate dimensionless hydrograph that corresponds to volumes of specific recurrence interval would therefore provide little additional information.

A study to regionalize volume frequency characteristics was not made, because of the difficulties involved in defining duration limits and because of time and financial constraints. The dimensionless hydrograph or regression model volumes in this report should not be construed as having the same recurrence interval as the peak discharge. The dimensionless hydrograph, width relations, and volumes may be considered to be averages associated with a peak discharge of specific recurrence interval. Any differences between actual hydrographs and those simulated by using this report simply represent the variation of actual hydrographs from the average or typical hydrograph for the given peak discharge.

Multiple regression analyses of volumes used the same 188-event data base employed to develop and verify the dimensionless hydrographs. In addition to the variables present in the dimensionless hydrograph volume equations (A, Qp, LT), the rainfall amount and intensity were included to see if storm characteristics might be significant in the prediction of runoff volume. Initial results showed that in the Piedmont, for instance, intensity was not significant at the 95-percent confidence level and that amount of rainfall decreased the standard error by only 1 percent and increased the R^2 value by only 0.003. Runs were then made with A, Qp, and LT, first for the entire State and then using various province groupings as in other analyses. A logical progression using smaller and smaller regions was employed until no geographic bias could be detected. One equation was chosen to represent each province except in the Lower Coastal Plain, which again was divided into two regions. The standard errors of estimate for volumes in the Blue Ridge, Piedmont, Upper Coastal Plain, and Lower Coastal Plain were +10.3, +21.1, +13.6, and +15.1 percent, respectively. The equations and their statistical parameters are shown in table 13. Simple average basin lagtime (LT) should be used in the volume equations and should not be confused with the adjusted lagtimes (LT_A) that are used with the dimensionless hydrograph only.

Table 13.--Summary of runoff volume estimating equations

Province or subprovince	Number of observa-tions	Equation	Standard error of regression (percent)	Coefficient of determination, (R^2)
Blue Ridge	22	$0.003780\ A^{-0.911}Qp^{0.888}LT^{0.879}$	±10.3	0.98
Piedmont	81	$0.002418\ A^{-0.798}Qp^{0.880}LT^{0.896}$	±21.1	.97
Upper Coastal Plain	30	$0.003854\ A^{-0.926}Qp^{0.990}LT^{0.721}$	±13.6	.98
Lower Coastal Plain	52		±15.1	.95
Region 1		$0.002652\ A^{-0.953}Qp^{0.978}LT^{0.882}$		
Region 2		$0.002872\ A^{-0.953}Qp^{0.978}LT^{0.882}$		

TESTING OF RUNOFF VOLUME EQUATIONS

Verification

The percentage difference between observed and estimated runoff volumes for several floods at a station usually were of the same sign and magnitude; therefore, splitting the data within a station would have resulted in a false (but successful) verification of the volume equations. Further, the small number of stations in each region also precludes the ability to verify by using split-sample techniques among stations.

Verification of the volume equations was similar to the additional verification step in the dimensionless hydrograph tests. Observed peak runoff, drainage area, and weighted regression lagtime were used to compute volume for the peak-of-record events shown in table 6. The average of the standard errors of estimate for the 20 events listed was $\pm$36.7 percent.

Bias

Variable bias was checked with graphical plots in percentage versus each independent variable. The scatter of plotting points appeared to be random. Geographical bias was checked by plotting the mean percent difference at each station on a State map. No trends were noted, although the numbers varied considerably from site to site. Although the average volume was underestimated by 9.5 percent, the students t-test indicated no bias at the 99-percent level of significance.

Sensitivity

A sensitivity test of the four equations was made by introducing errors of a specified range in the independent variables and computing the consequential error in estimating runoff volume. The results are shown in table 14.

LIMITATIONS

Use of the hydrograph simulation technique should be limited to rural basins with drainage areas less than 500 mi^2 in South Carolina. The extremes for the independent variables used in the regression analyses of basin lagtime and runoff volume are listed in table 15. The expected errors are unknown for watersheds with characteristics outside these specified ranges. In addition, these methods are not applicable to streams where regulation, urbanization, temporary in-channel storage, or overbank detention storage is significant unless suitable estimates of peak discharge and lagtime are available to account for these effects.

Table 14.--Sensitivity of computed runoff volume to errors in the independent variables

Percent error in indicated variable	Percent error in computed runoff volume for indicated provinces and subprovinces			
	Blue Ridge	Piedmont	Upper Coastal Plain	Lower Coastal Plain (Regions 1 and 2)
DRAINAGE AREA				
-50	88.0	73.9	90.0	93.6
-25	30.0	25.8	30.5	31.5
-10	10.1	8.8	10.2	10.6
+10	-8.3	-7.3	-8.4	-8.7
+25	-18.4	-16.3	-18.7	-19.2
+50	-30.9	-27.6	-31.3	-32.1
PEAK DISCHARGE				
-50	-46.0	-45.7	-49.7	-49.2
-25	-22.5	-22.4	-24.8	-24.5
-10	-8.9	-8.8	-9.9	-9.8
+10	8.8	8.7	9.9	9.8
+25	21.9	21.7	24.7	24.4
+50	43.3	42.9	49.4	48.7
AVERAGE BASIN LAGTIME				
-50	-45.6	-46.3	-39.3	-45.7
-25	-22.3	-22.7	-18.7	-22.7
-10	-8.8	-9.0	-7.3	-8.9
+10	8.7	8.9	7.3	8.8
+25	21.7	22.1	17.5	21.8
+50	42.8	43.8	34.0	43.0

Table 15.--Range of independent variables used in the average basin lagtime and runoff volume regression analyses

[mi², square miles; ft³/s, cubic feet per second]

Province	Variable	Minimum	Maximum	Units
AVERAGE BASIN LAGTIME				
Blue Ridge	A	2.83	455.	mi^2
Piedmont	A	0.52	444.	mi^2
Inner Coastal Plain	A	2.92	401.	mi^2
Lower Coastal Plain (Regions 1 and 2)	A	7.67	401.	mi^2
RUNOFF VOLUME				
Blue Ridge	A	30.2	455.	mi^2
	Qp	231.	12800.	ft^3/s
	LT	8.77	19.6	hours
Piedmont	A	0.52	444.	mi^2
	Qp	2.94	16400.	ft^3/s
	LT	1.92	52.0	hours
Inner Coastal Plain	A	2.92	122.	mi^2
	Qp	10.4	625.	ft^3/s
	LT	9.88	49.7	hours
Lower Coastal Plain (Regions 1 and 2)	A	7.67	401.	mi^2
	Qp	16.7	2560.	ft^3/s
	LT	11.7	95.5	hours

Note: A = Drainage area
Qp = Peak discharge
LT = Lagtime

COMPARISON OF U.S. GEOLOGICAL SURVEY AND U.S. SOIL CONSERVATION SERVICE METHODOLOGIES

As mentioned in the introduction, the hydrograph simulation technique presented in this report is quite different from traditional unit hydrograph methods such as the Soil Conservation Service method. These differences make direct comparison between the methods impossible. It is important here to point out certain inherent features or basin conditions that might assist the reader in choosing the methodology most appropriate for the situation.

The most important difference between the two methods is the basis for assigning a frequency (recurrence interval) to a flood event. The U.S. Geological Survey method attaches a frequency to the peak discharge. The resulting hydrograph and volume may or may not have the same frequency. The Soil Conservation Service method uses rainfall frequency as its design basis. All three parameters (peak discharge, volume, and hydrograph) resulting from these computations may or may not have the same recurrence interval. If the purpose of any particular study is risk assessment, or if the design requirements specify a recurrence interval peak, the U.S. Geological Survey method is the most applicable.

An equally fundamental difference between the two methods is that the U.S. Geological Survey procedure provides estimates of the accuracy that can be expected. Furthermore, the results are consistent among users. In contrast, it is very difficult to estimate accuracy when using the Soil Conservation Service procedure, and results are not always consistent among users.

Basin homogeneity is an extremely important consideration. The U.S. Geological Survey method is best suited for homogeneous watersheds typical of the region in which it was developed. The Soil Conservation Service method may be applied to either homogeneous basins or basins with major land use and soil variations. Although the errors in the actual frequency of the resultant peak discharges are unknown, the Soil Conservation Service method can be used for comparing the relative effects of subbasin land use changes on runoff characteristics.

The Soil Conservation Service method assumes that rainfall is uniformly distributed over the entire basin and is based on data sets from smaller basins (less than about 10 mi^2) where this assumption is more likely to exist. Larger basins should be subdivided, and the streamflow hydrographs should be routed and accumulated if the Soil Conservation Service method is to be used.

Neither method reproduces double peaks that occur naturally, although the Soil Conservation Service method could use subdividing and routing steps to account for this characteristic. Double peaks generally are the result of tributary flow entering the main channel just upstream of a basin outlet or from substantial changes in land use, soils, or physiography between the upstream and downstream areas of the watershed. If the double peak is the result of a complex storm distribution, the Soil Conservation Service method is capable of simulating the resulting double-peak hydrograph.

Because the Soil Conservation Service method uses the convolution of rainfall excess to generate a hydrograph, it is sensitive to variations in the distribution and duration of storms and to antecedent conditions. A curve number, representative of land use, soils, and antecedent conditions must be selected in the Soil Conservation Service method. The probabilistic basis of the curve number leads to uncertainty in assigning a recurrence interval to the simulated peak discharge. Additionally, the procedure for the Soil Conservation Service method is to use the design recurrence interval for the 24-hour rainfall total distributed according to a standard distribution curve. This procedure can overestimate peak rates in many instances (Sanders, 1987). The duration and intensity of rainfall needed to produce a peak discharge of specific recurrence interval varies with basin size, but this is not taken into account with the standard 24-hour rainfall distribution used in the Soil Conservation Service technique.

Actual storm hydrographs may be more nearly reproduced by using the Soil Conservation Service unit hydrograph method. The U.S. Geological Survey dimensionless hydrograph is not intended for this purpose; it yields only an average flood hydrograph for a specified peak discharge.

Finally, the overall level of effort required for each method may be of interest where accuracy requirements and level of effort can be weighed against each other. The U.S. Geological Survey technique can be completed in a few minutes with a hand-held calculator. The Soil Conservation Service method requires the evaluation of several abstraction parameters (infiltration, antecedent conditions, and so on), computation of a unit hydrograph, and convolution of the rainfall excess.

APPLICATION OF TECHNIQUE

The following example illustrates the procedures to be used when computing a simulated hydrograph at an ungaged site. The hypothetical basin for which the 100-year hydrograph is desired has a drainage area lying in both the Blue Ridge and Piedmont physiographic provinces.

(1) Locate the site on the best available topographic maps. Delineate and planimeter the drainage area (A). In this example, A=50.0 mi^2.

(2) Determine from plate 1 which hydrologic regions are involved and the percentage of the basin in each. The drainage area in the Blue Ridge in this example is 10 mi^2, or 20 percent of the total area. The remaining 80 percent is situated in the Piedmont Province.

(3) The 100-year discharge for a 50-mi^2 basin in the Blue Ridge province is computed to be 11,200 ft^3/s, using the most current flood frequency report (Whetstone, 1982). The 100-year discharge for a 50-mi^2 basin in the Piedmont province is computed to be 7,710 ft^3/s.

(4) The discharges computed in step 3 are prorated by using the percentages determined in step 2, as follows:

Blue Ridge: (11,200 ft^3/s)(20 percent)= 2,240 ft^3/s

Piedmont: (7,710 ft^3/s)(80 percent)= 6,170 ft^3/s

SUM = 8,410 ft^3/s

(5) The adjusted lagtime (LT_A) for a Blue Ridge event of 8,410 cubic feet per second at this station is computed to be 9.24 hrs (hours) by use of equation 8. The Piedmont equation (equation 9) gives an adjusted lagtime of 12.3 hrs.

(6) The adjusted lagtimes are prorated in the same manner as the peak discharges in step (4):

Blue Ridge: (9.24 hrs)(20 percent) = 1.85 hrs

Piedmont: (12.3 hrs)(80 percent) = 9.84 hrs

Sum = 11.7 hrs

If only a simulated hydrograph is desired at this point, you may skip to step 10. If runoff volume is desired, continue with step 7.

(7) The average basin lagtime (LT) is computed to be 10.5 hrs by use of the Blue Ridge equation in table 11. The Piedmont equation (table 11) gives a 16.1-hr average basin lagtime.

(8) The lagtimes are prorated in the same manner as the peak discharges in step (4):

Blue Ridge: (10.5 hrs)(20 percent)= 2.1 hrs

Piedmont: (16.1 hrs)(80 percent)= 12.9 hrs

SUM = 15.0 hrs

(9) Volume, if desired, is calculated by using the weighted peak discharge and lagtime computed in steps (4) and (8) above and the equations in table 13. The volumes computed for basins in the Blue Ridge and Piedmont provinces (3.54 and 3.43 inches, respectively) are then weighted:

Blue Ridge: (3.54 in)(20 percent)= 0.71 in
Piedmont: (3.43 in)(80 percent)= 2.74 in

SUM = 3.45 in

(10) Because a majority of the basin lies in the Piedmont province, the coordinates for that dimensionless hydrograph will be used. The hydrograph can be simulated at this point using the

weighted Qp, the weighted LT_A, and the dimensionless hydrograph from table 3 for basins in the Piedmont province. The technique is illustrated in table 16, and figure 14 shows the product hydrograph. If a basin appears to be situated in more than one province, as in this example, the dominant regional hydrograph may be used or the hydrograph ordinates may be averaged after aligning the peaks. For basins located near the hydrologic boundaries in plate 1, consult more detailed soils maps to determine the appropriate equations and dimensionless hydrographs to use.

SUMMARY

Three dimensionless hydrographs were developed for use in simulating flood hydrographs at ungaged rural sites draining less than 500 mi^2 in South Carolina. The dimensionless hydrographs are based on data from 151 floods at 39 sites throughout the State. The dimensionless hydrographs were verified by using 37 floods observed at 10 sites not used in their development. A simulated flood hydrograph can be computed by applying a volume-adjusted lagtime (LT_A, equations 8-12) and peak discharge of a specific recurrence interval to the appropriate dimensionless hydrograph. The coordinates of the runoff hydrograph are calculated by multiplying the volume-adjusted lagtime by selected time ratios and peak discharge by selected discharge ratios.

Multiple-regression analyses were used to develop equations for estimating average basin lagtime. Five equations for estimating average basin lagtime were developed for sites in the Blue Ridge province, Piedmont province, Upper Coastal Plain subprovince, and two regions in the Lower Coastal Plain subprovince on the basis of data from 48 of the stations used to derive and verify the dimensionless hydrographs. The only significant explanatory parameter in the equations for estimating lagtime was drainage area. Analysis of residuals (differences between estimated and observed values) indicated no variable or geographical bias in the equations.

Five equations are given for estimating flood volumes. Drainage area, peak discharge, and average basin lagtime were the explanatory variables used in the volume equations. These equations for estimating runoff volume are the basis of the adjusted lagtimes (LT_A, equations 8-12) that must be estimated prior to simulation with the dimensionless hydrographs. No bias was evident in the final equations for estimating runoff volume. The volumes computed by use of these equations cannot be assumed to have the same recurrence interval as the peak discharge.

Table 16.--Computation of the simulated coordinates of the flood hydrograph in the example application problem

t/LT_A (from table 4)	x LT_A	= time (hrs)	Q/Qp (from table 4)	x Qp	= discharge (cubic feet per second)
0.15	11.7	1.76	0.07	8,410	589
.20	11.7	2.34	.09	8,410	757
.25	11.7	2.93	.11	8,410	925
.30	11.7	3.51	.14	8,410	1,180
.35	11.7	4.10	.17	8,410	1,430
.40	11.7	4.68	.21	8,410	1,770
.45	11.7	5.27	.25	8,410	2,100
.50	11.7	5.85	.30	8,410	2,520
.55	11.7	6.44	.37	8,410	3,110
.60	11.7	7.02	.44	8,410	3,700
.65	11.7	7.61	.53	8,410	4,460
.70	11.7	8.19	.61	8,410	5,130
.75	11.7	8.78	.70	8,410	5,890
.80	11.7	9.36	.78	8,410	6,560
.85	11.7	9.95	.86	8,410	7,230
.90	11.7	10.53	.92	8,410	7,740
.95	11.7	11.12	.96	8,410	8,070
1.00	11.7	11.70	.99	8,410	8,330
1.05	11.7	12.29	1.00	8,410	8,410
1.10	11.7	12.87	.98	8,410	8,240
1.15	11.7	13.46	.96	8,410	8,070
1.20	11.7	14.04	.91	8,410	7,650
1.25	11.7	14.63	.86	8,410	7,230
1.30	11.7	15.21	.80	8,410	6,730
1.35	11.7	15.80	.74	8,410	6,220
1.40	11.7	16.38	.69	8,410	5,800
1.45	11.7	16.97	.63	8,410	5,300
1.50	11.7	17.55	.58	8,410	4,880
1.55	11.7	18.14	.53	8,410	4,460
1.60	11.7	18.79	.49	8,410	4,120
1.65	11.7	19.31	.44	8,410	3,700
1.70	11.7	19.89	.41	8,410	3,450
1.75	11.7	20.48	.37	8,410	3,110
1.80	11.7	21.06	.34	8,410	2,860
1.85	11.7	21.65	.32	8,410	2,690
1.90	11.7	22.23	.29	8,410	2,440
1.95	11.7	22.82	.27	8,410	2,270
2.00	11.7	23.40	.25	8,410	2,100
2.05	11.7	23.99	.23	8,410	1,930
2.10	11.7	24.57	.21	8,410	1,770
2.15	11.7	25.16	.19	8,410	1,600
2.20	11.7	25.74	.18	8,410	1,510
2.25	11.7	26.33	.16	8,410	1,350
2.30	11.7	26.91	.15	8,410	1,260
2.35	11.7	27.50	.13	8,410	1,090
2.40	11.7	28.08	.12	8,410	1,010
2.45	11.7	28.67	.11	8,410	925
2.50	11.7	29.25	.10	8,410	841

Note: t = time
LT_A = lagtime adjusted for correct runoff volume
Q = discharge
Qp = peak discharge

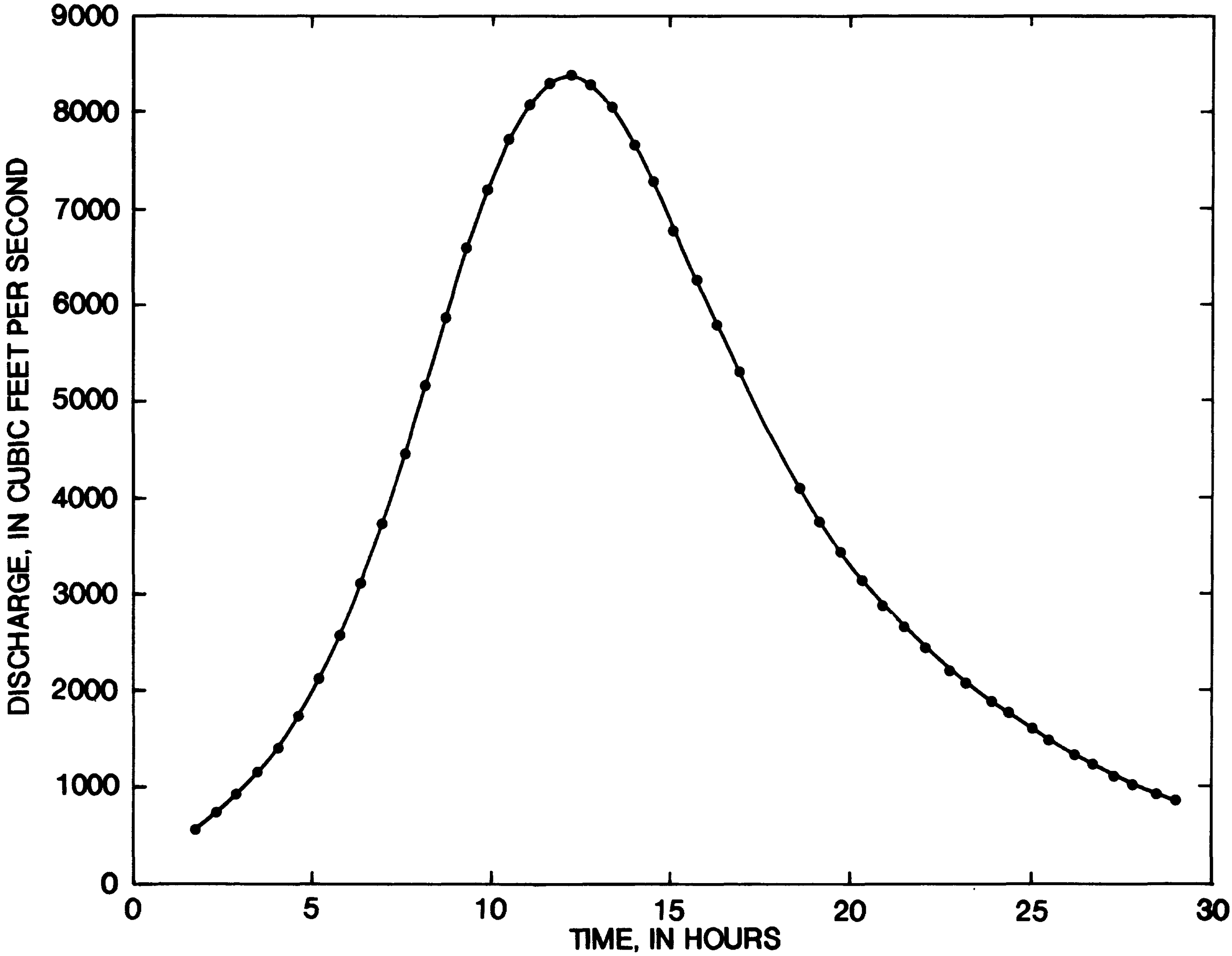

Figure 14.--Simulated 100-year flood hydrograph for a hypothetical river in the Blue Ridge and Piedmont provinces in South Carolina.

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