SIMULATION OF FLOOD HYDROGRAPHS FOR GEORGIA STREAMS

U.S. GEOLOGICAL SURVEY

WATER-RESOURCES INVESTIGATIONS REPORT 86-4004

PREPARED IN COOPERATION WITH THE
STATE OF GEORGIA
DEPARTMENT OF TRANSPORTATION

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By Ernest J. Inman

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SIMULATION OF FLOOD HYDROGRAPHS FOR GEORGIA STREAMS

By E. J. Inman

ABSTRACT

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

The O'Donnell method was used to compute unit hydrographs and lagtimes for 355 floods at 80 gaging stations. An average unit hydrograph and an average lagtime were computed for each station. These average unit hydrographs were transformed to unit hydrographs having 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. Hydrographs were simulated for these 355 floods and their widths were compared with the widths of the observed hydrographs at 50 and 75 percent of peak flow. The dimensionless hydrograph based on one-half lagtime duration provided the best fit of the observed data.

Multiple-regression analysis was then used to define relations between lagtime and certain physical basin characteristics, of which drainage area and slope were found to be significant for the rural-stream equations, and drainage area, slope, and impervious area were found to be significant for the Atlanta urban-stream equation.

A hydrograph can be simulated from the dimensionless hydrograph, the peak discharge of a specific recurrence interval, and the lagtime obtained from regression equations for any site with less than a 500 square mile drainage area in Georgia.

For simulating hydrographs at sites with basins larger than 500 square miles, the U.S. Geological Survey computer model CONROUT can be used. This model routes streamflow from an upstream channel location to a user-defined location downstream. The product of CONROUT is a simulated discharge hydrograph for the downstream site, which has a peak discharge of a specific recurrence interval. The diffusion-analogy routing method with single linearization was used in this study.

INTRODUCTION

The design of many highway drainage structures and embankments requires an evaluation of the flood-related risk to the structures and to the surrounding property. Risk analyses of alternate designs are necessary to determine the design with the least total expected cost (Corry and others, 1980). To fully evaluate these risks, a runoff hydrograph with a peak discharge of specific recurrence interval may be necessary to estimate the length of time that specific features--for example, roads and bridges--would be inundated. For ungaged streams, this information is difficult to estimate; therefore a method is needed to estimate the flood hydrograph associated with a design discharge. The objective of this study was to define techniques for simulating flood hydrographs for specific design discharges at ungaged sites in Georgia. The scope of this study was statewide for rural basins, and was restricted to the Atlanta metropolitan area for urban basins of up to 25 mi^2.

Acknowledgments

This study was carried out by the U.S. Geological Survey in cooperation with the Georgia Department of Transportation. Hourly rainfall records were obtained from monthly publications of the National Climatic Data Center.

The guidance and technical assistance of hydrologists in the U.S. Geological Survey and particularly Vernon B. Sauer are recognized and greatly appreciated. Also, the computer programming contributions of S. E. Ryan, Hydrologist, U.S. Geological Survey, have been invaluable to this study.

DATA BASE

Basins Less Than 20 Square Miles

The data base used in this phase of the study consisted of 80 stations throughout Georgia. Over 500 floods were selected from these 80 stations by reviewing the hydrographs obtained during model calibrations from earlier studies. The selection criteria were: (1) uniform, relatively short-duration rainfall, and (2) a simple (or noncompound) discharge hydrograph. Both rainfall and discharge at 5-, 10-, 15-, or 30-minute intervals were available for these floods in the files of the U.S. Geological Survey computer in Reston, Va. These data were downloaded to the computer in the Georgia District and made ready for further analysis.

Basins of 20 to 500 Square Miles

The data base for this part of the study consisted of 37 selected stations throughout the State. Over 200 floods were selected, coded, and entered on the Georgia District computer. The selection process was similar to the criteria used for the small basins in that: (1) rainfall must be relatively uniform, and (2) the discharge hydrographs must be simple (noncompound). Rainfall uniformity was more difficult to determine, because the basins were larger and the distribution of gages within and near the basins was more or less random. The uniformity of rainfall was determined by plotting the gaging stations on a State map, along with the hourly rainfall stations both in and near the basin, and by using the two, three, or four applicable rainfall stations to determine the uniformity of rainfall for the corresponding runoff. Once uniformity was determined, the rainfall gage nearest the center of the basin was used, along with the discharge data, for the analysis.

Discharge was obtained by applying the proper rating table (stage discharge relation) to the gage heights taken from computer output sheets. Next, the rainfall and discharge data were coded and entered on local computer files. Preparation of the data for the 20- to 500-mi^2 basins was the most time consuming step of the entire project.

Basins Greater Than 500 Square Miles

The data base for this section of the study consisted of the basins larger than 500 mi^2 in Georgia. The daily values for these basins are stored on the U.S. Geological Survey computer in Reston, Va., and are transfered to the Georgia District computer as the need arises.

HYDROGRAPH-SIMULATION PROCEDURE

Several traditional methods for simulating a hydrograph for a flood of selected recurrence interval at an ungaged watershed were considered for this study. However, a new procedure based on observed streamflow data was developed for this study and is presented in this section.

Basins Less Than 500 Square Miles

A dimensionless hydrograph was developed for use in basins up to 500 mi^2. Peak discharge of a selected recurrence interval and lagtime are necessary parameters to convert the dimensionless hydrograph to a simulated hydrograph for a given basin. Price (1979) presents a technique for estimating the peak discharge of a selected recurrence interval for rural streams in Georgia. Inman (1983) presents a technique for estimating the peak discharge of a selected recurrence interval for basins less than 25 mi^2 in the Atlanta urban area. Lagtime estimating equations were developed for Georgia streams as part of the present study and will be presented in a later section.

The dimensionless hydrograph was developed from observed flood hydrographs. Using the data base described earlier for basins less than 20 mi^2, the method is as follows:

(1) Compute a unit hydrograph and lagtime for three to five storms for each of the 80 gaging stations (figs. 1, 2). All unit hydrographs should be for the same time interval (duration) at a station. Lagtime is computed as the time at the centroid of the unit hydrograph minus one-half the time of the computation interval (duration). The unit hydrograph computation method is by O'Donnell (1960).

(2) Eliminate the unit hydrographs with inconsistent shapes and compute additional unit hydrographs if needed.

(3) Compute an average unit hydrograph for each station by aligning the peaks and averaging each ordinate of discharge for the final selection of unit hydrographs. Table 1 illustrates this step. 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

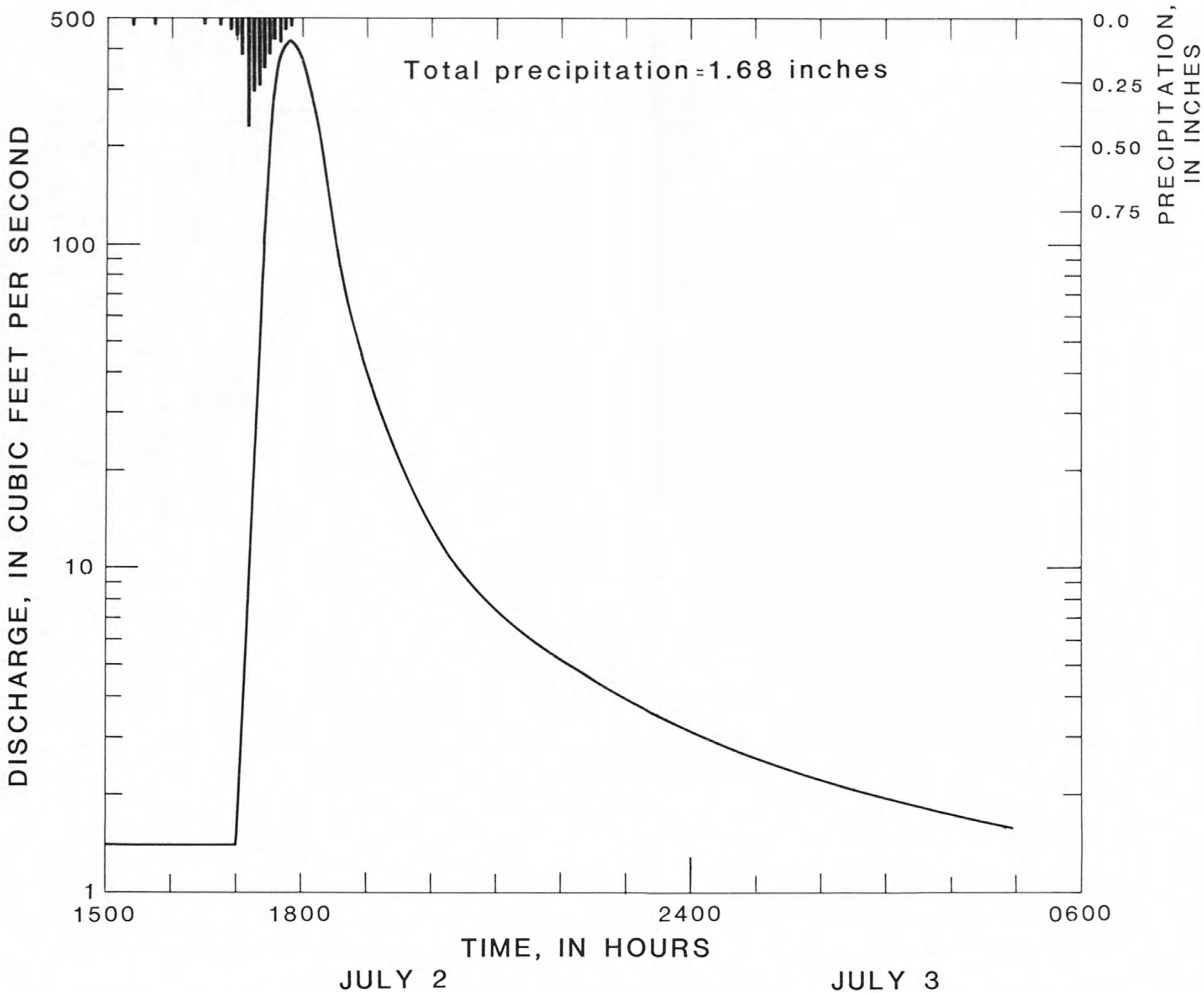

Figure 1.—Plot of observed flood hydrograph and unit precipitation from Conley Creek near Forest Park, July 2-3, 1974.

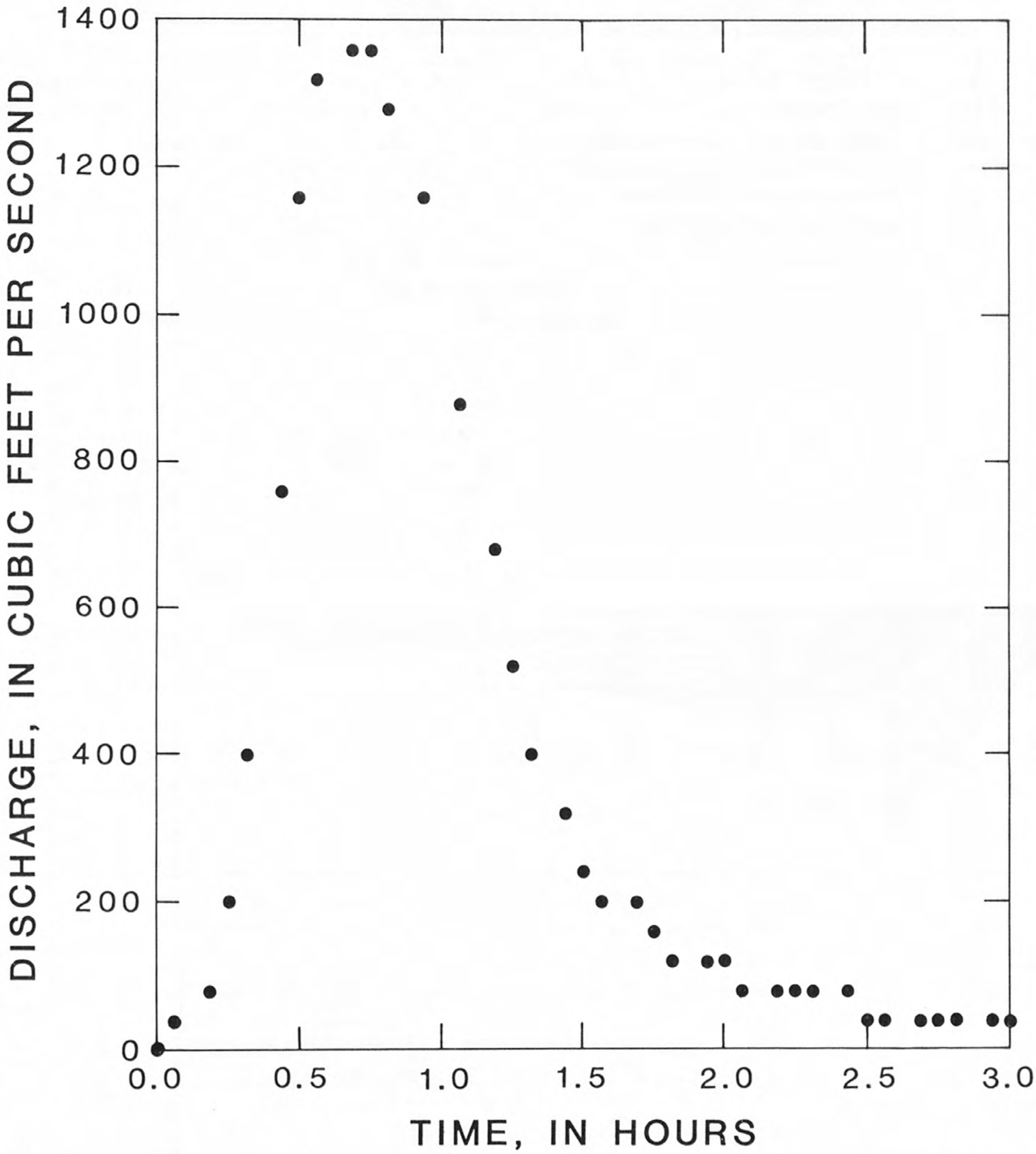

Figure 2.—Plot of unit hydrograph computed from observed data in figure 1, with runoff of 1.00 inch and lagtime of 1.03 hours.

Table 1.--Listing of discharges at 5-minute intervals with peaks aligned for seven unit hydrographs with dates of occurrence and the average hydrograph computed for Conley Creek near Forest Park

[Discharge in cubic feet per second]

Hydrographs							Average unit hydrograph
(09-09-73)	(07-02-74)	(01-10-75)	(03-24-75)	(06-10-75)	(06-19-75)	(11-05-77)	
0	0	0	0	0	0	0	0
0	0	0	0	0	0	50	7
147	0	0	0	98	0	101	49
295	0	0	0	197	0	151	92
359	0	86	0	295	0	208	135
423	0	173	0	444	0	266	187
487	0	259	0	592	0	324	237
550	29	340	30	667	176	381	310
614	83	420	80	743	351	438	390
678	190	501	130	818	629	496	492
742	384	582	180	893	907	542	604
803	744	621	464	969	1,185	611	771
859	1,115	712	819	1,044	1,296	641	927
903	1,270	777	911	1,083	1,408	671	1,003
909	1,298	800	1,004	1,111	1,519	701	1,049
889	1,292	790	959	1,018	1,444	696	1,013
851	1,225	773	914	925	1,255	681	946
779	1,103	732	869	691	996	662	833
731	964	690	824	499	739	630	725
650	826	648	779	378	550	599	633
510	662	607	734	293	431	563	543
371	493	566	578	213	348	551	446
256	367	524	459	190	285	513	370
195	297	424	390	167	191	476	306
156	248	373	346	144	98	438	258
123	204	322	301	121	78	396	221
111	176	270	260	97	58	359	190
97	151	219	226	81	38	313	161
82	127	197	193	72	36	258	138
71	113	160	168	63	34	208	117
61	102	134	159	59	32	159	101
55	92	124	151	52	30	105	87
52	84	110	144	44	30	100	81
46	74	101	136	42	29	95	75
40	66	89	128	38	29	90	69
36	61	80	119	35	28	85	63
34	56	76	113	34	28	80	60
33	52	68	108	29	27	74	56
32	46	61	101	26	27	68	52
30	40	54	93	24	26	62	47
26	38	51	86	21	26	57	44
25	35	46	82	17	25	51	40
21	32	42	79	17	25	45	37
20	29	40	75	16	24	40	35
19	27	38	69	16	24	35	33
17	26	37	65	12	23	30	30
15	24	35	62	10	23	25	28
15	22	30	59	11	22	20	26
14	22	32	56	13	22	16	25
13	21	31	52	11	21	16	24
12	20	27	49	9	21	17	22
12	19	26	47	9	20	19	22
11	18	25	45	8	20	21	21
10	17	22	43	7	19	22	20
10	16	21	40	6	19	22	19
9	15	22	38	6	18	20	18
9	14	19	36	4	17	18	17

time of the center of mass of the discharge hydrograph is obtained by adding one-half the unit hydrograph computation interval (duration) to that hydrograph's lagtime. Figure 3 illustrates the average unit hydrograph computed above with the correct timing of average center of mass.

(4) Transform the average unit hydrographs computed in step 3 to hydrographs having durations of one-fourth, one-third, one-half, and three-fourths lagtime. These durations must be to the nearest multiple of the original duration (computation interval). For instance, if the original duration is 5 minutes and the average lagtime is 0.70 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 which would be rounded to 30 minutes. These 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	$TUHD(t)=1/n[TUH(t)+TUH(t-1) \ldots TUH(t-n+1)]$,

where Δt = computation interval, (the original unit hydrograph has a duration equal to Δt),

D = design duration of the unit hydrograph, (this must be a multiple of Δt),

TUHD(t) = ordinates of the desired unit hydrograph at time t, and

TUH(t), TUH(t-1), etc. = ordinates of the original unit hydrograph at times t, t-1, t-2, etc.

Duration may be thought of as actual duration or design duration, so a distinction must be made between the two. Actual duration which is highly variable may be defined as the time during which precipitation falls at a rate greater than the existing infiltration capacity. It is the actual time during which rainfall excess is occurring. Design duration is that duration which is most convenient for use on any particular basin. The design duration is that for which the unit hydrograph is computed. For this report, design duration is expressed as a fractional part of lagtime, such as one-fourth, one-third, one-half, and three-fourths lagtime. It is later shown that the design duration of one-half lagtime provides the best fit of observed data.

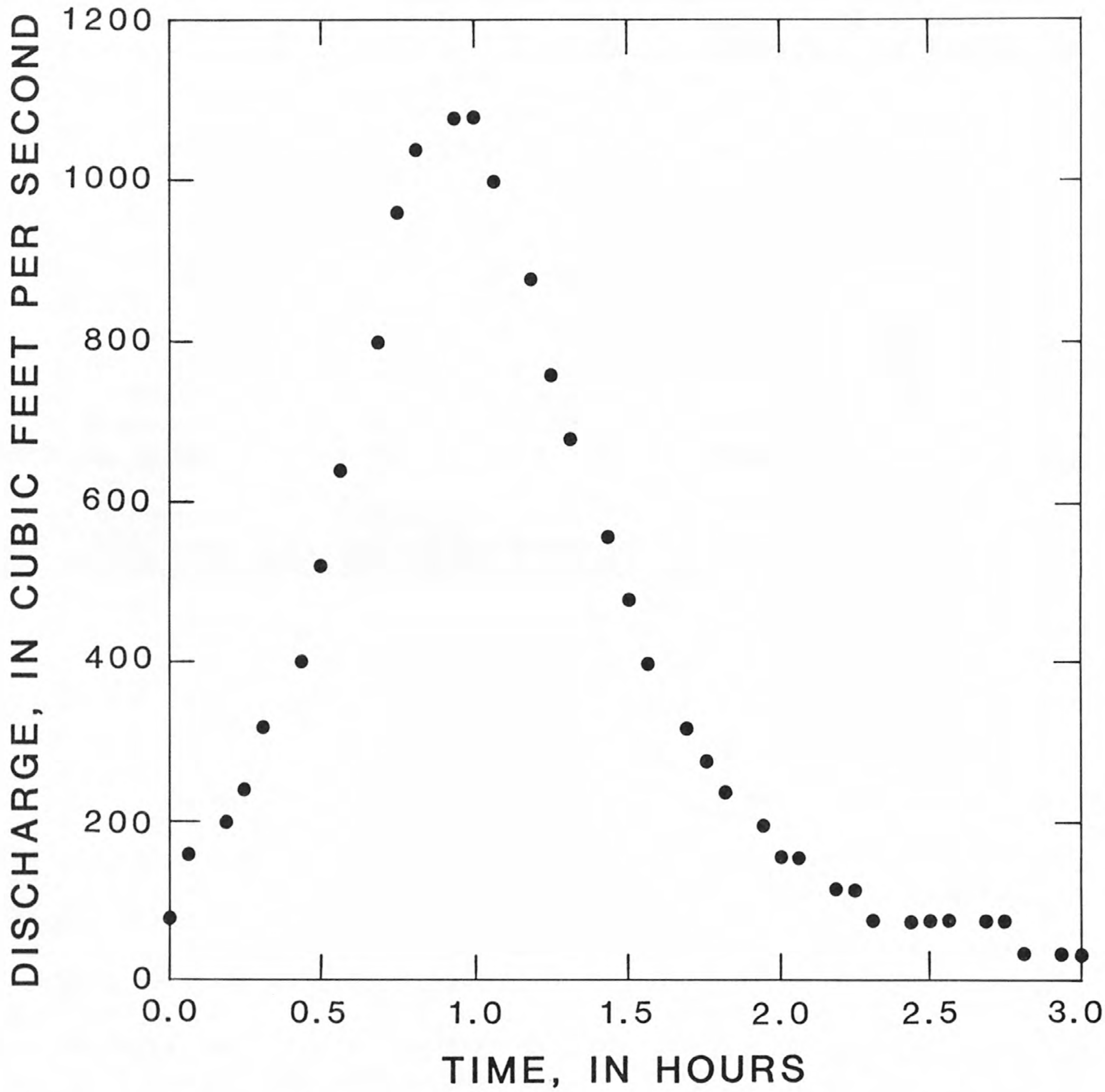

Figure 3.—Plot of average unit hydrograph from Conley Creek near Forest Park, with correct timing of average center of mass.

(5) Reduce the one-fourth, one-third, one-half, and three-fourths lagtime hydrographs to dimensionless terms by dividing the time by lagtime and the discharge by peak discharge. Figure 4 illustrates the results of this step for one basin.

(6) For Hydrologic Regions 1, 2, and 3 as defined by Price (1979) and the Atlanta urban area as reported by Inman (1983), compute an average dimensionless hydrograph by using the dimensionless hydrographs at the stations within that area or region. The average hydrographs were computed by aligning the peaks and averaging each ordinate of the discharge ratio, Q/Q_p. Figure 5 illustrates the average one-half lagtime duration dimensionless hydrograph in Region 1 and the range of the data from the 16 stations from which it was computed.

Steps 1 through 5 were done for all stations having data in the U.S. Geological Survey WATSTØRE unit-values file, which had hydrographs plotted from earlier studies. A total of 355 unit hydrographs from 80 stations, including 19 Atlanta urban sites, were used to develop the one-fourth, one-third, one-half, and three-fourths lagtime duration dimensionless hydrographs. A statistical analysis to select the best fitting design duration was done by comparing the widths of hydrographs estimated (or computed) from the one-fourth, one-third, one-half, and three-fourths lagtime duration dimensionless hydrographs from each region or area with the observed hydrograph widths from their respective regions or area. The one-half-lagtime duration was the best fit of width at 50 percent of peak flow and at 75 percent of peak flow. Plots of the one-half-lagtime duration dimensionless hydrograph as shown in figure 6 were made for Regions 1, 2, and 3, and for the Atlanta urban area. Based on these plots, one dimensionless hydrograph was computed and selected for both rural and urban conditions for the entire State. Figure 7 and table 2 illustrate and list this statewide dimensionless hydrograph.

Another statistical analysis to test the accuracy of the dimensionless hydrograph application technique was done by comparing the predicted hydrograph widths at 50 and 75 percent of peak flow from computed hydrographs using the statewide one-half lagtime duration dimensionless hydrograph with the 355 observed hydrographs. Figure 8 is an example of this comparison. The results were: the 50 percent of peak flow width comparison had a standard error of estimate of $\pm$ 31.8 percent and the 75 percent comparison had a standard error of estimate of $\pm$ 35.9 percent. The standard error of estimate of the width comparisons is based on mean-square difference between observed and estimated widths. Based on verification and bias testing, which are presented in a later section, this dimensionless hydrograph can be used for flood-hydrograph estimation for ungaged basins up to 500 mi^2. Steps 3 through 6 of the dimensionless hydrograph development and the statistical analyses were programed for computer use by S. E. Ryan (U.S. Geological Survey, written commun., 1985).

A comparison of the dimensionless hydrograph developed in this study, the Soil Conservation Service dimensionless hydrograph, and the Stricker-Sauer dimensionless hydrograph is illustrated in figure 9. Details on the development of the SCS dimensionless hydrograph can be obtained from the SCS National Engineering Handbook, Section 4, "Hydrology" (1972) and details of the Stricker-Sauer (1982) dimensionless hydrograph can be obtained from "Techniques for estimating flood hydrographs for ungaged urban watersheds".

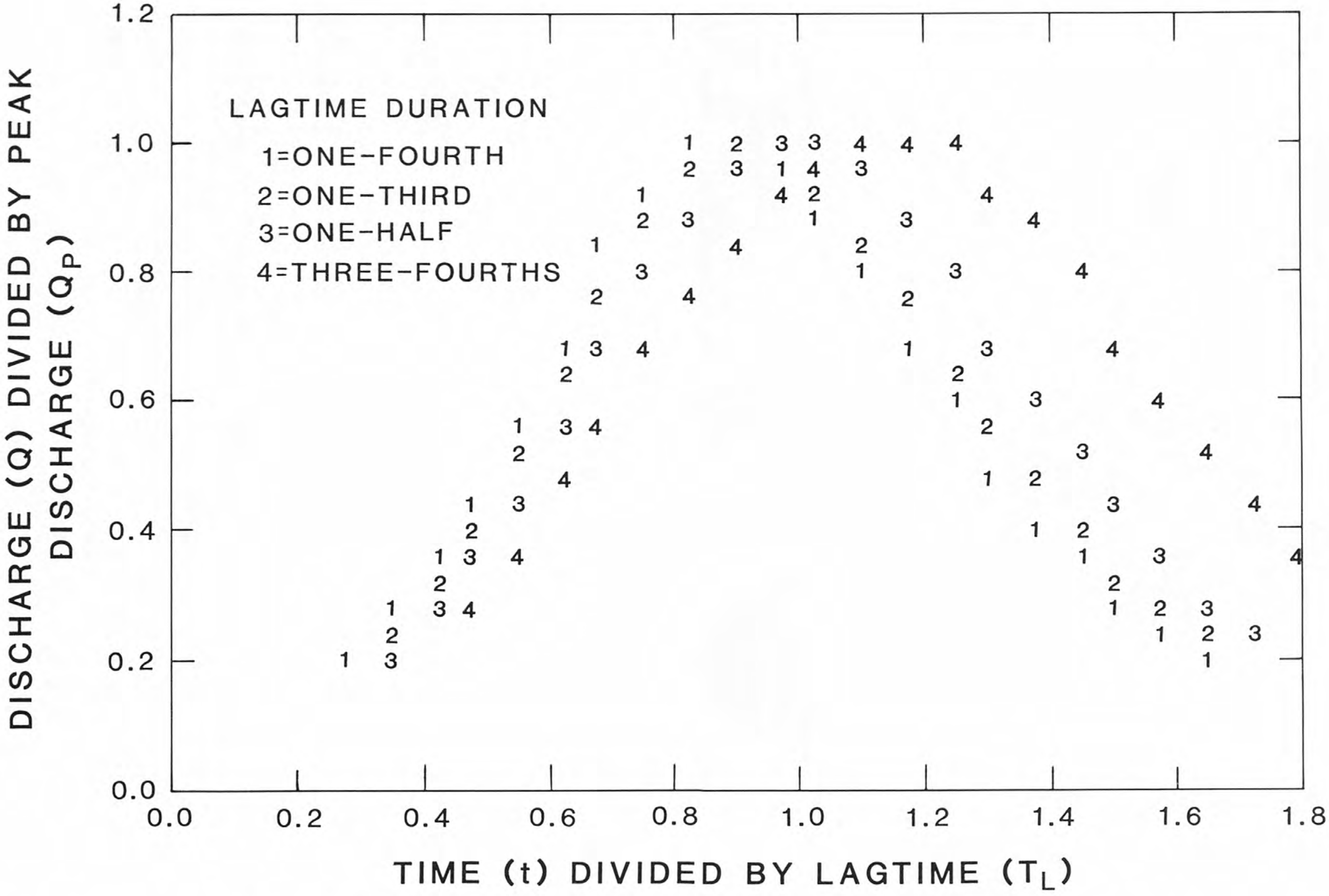

Figure 4.—Plot of one-fourth, one-third, one-half, and three-fourths lagtime duration dimensionless hydrographs for Conley Creek near Forest Park.

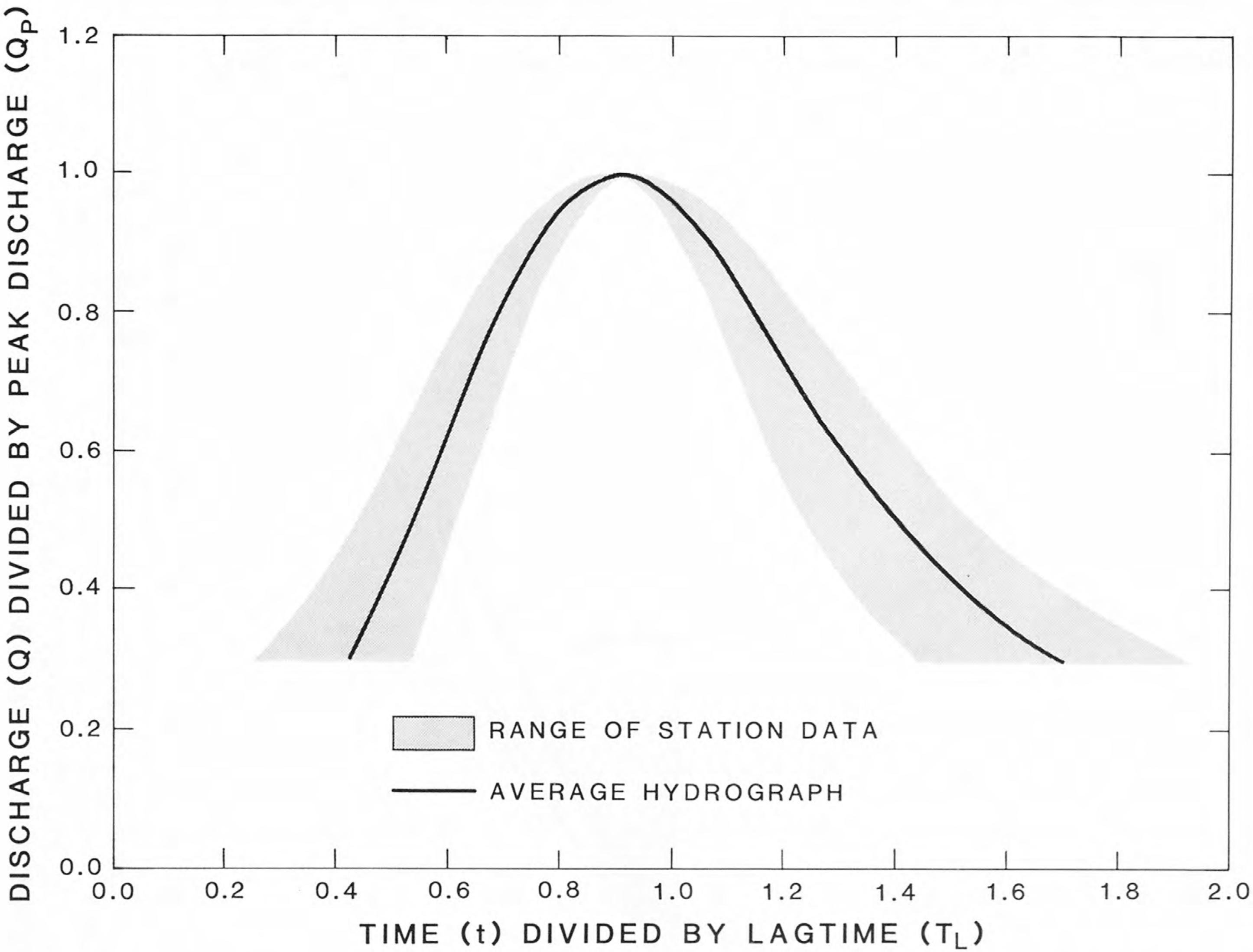

Figure 5.— Plot of the average, one-half-lagtime duration, dimensionless hydrograph in Region 1 and the range of the data from the 16 stations from which it was computed.

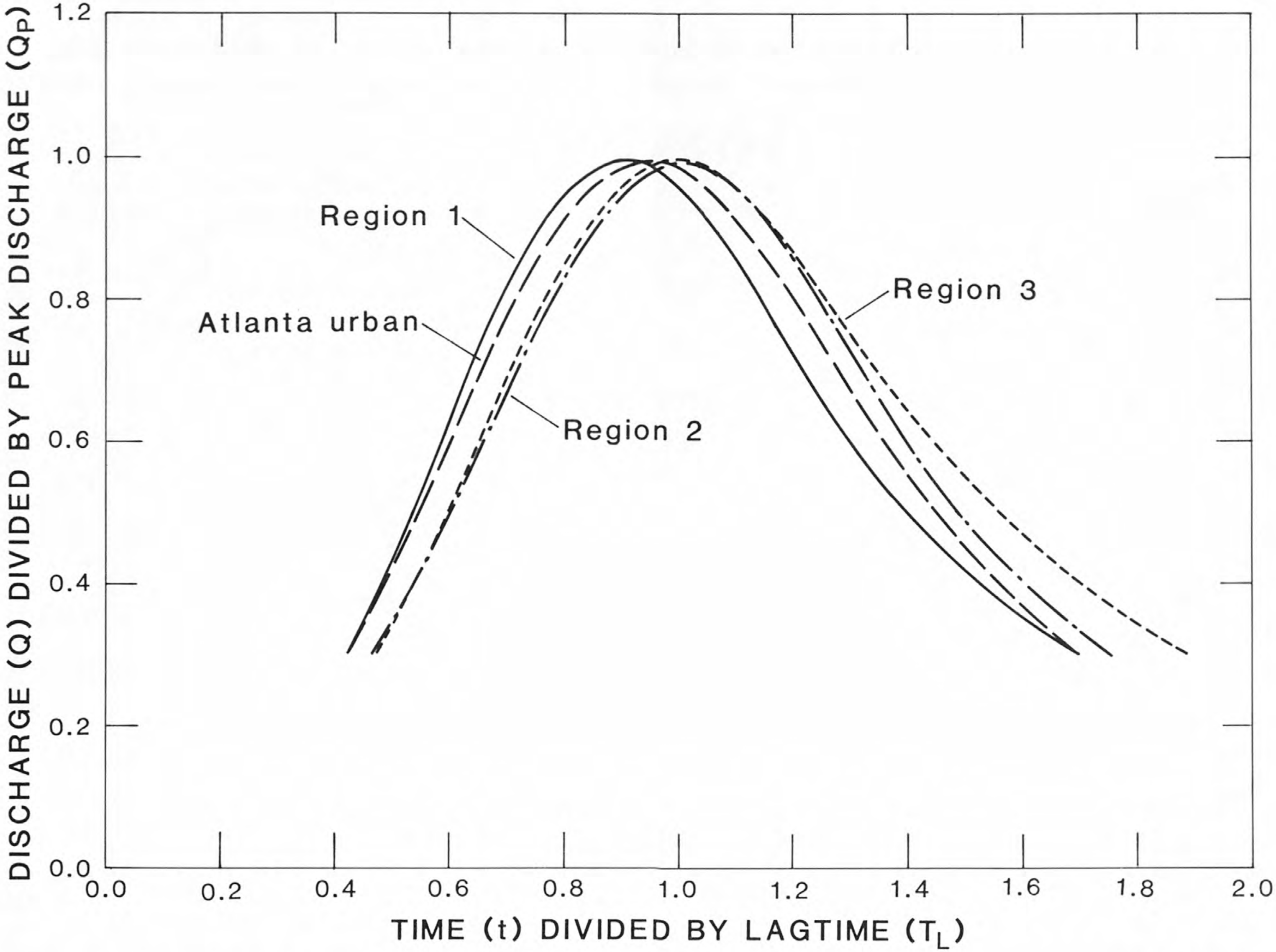

Figure 6.—Plot of the average, one-half-lagtime duration, dimensionless hydrographs for Regions 1, 2, 3, and the Atlanta urban area.

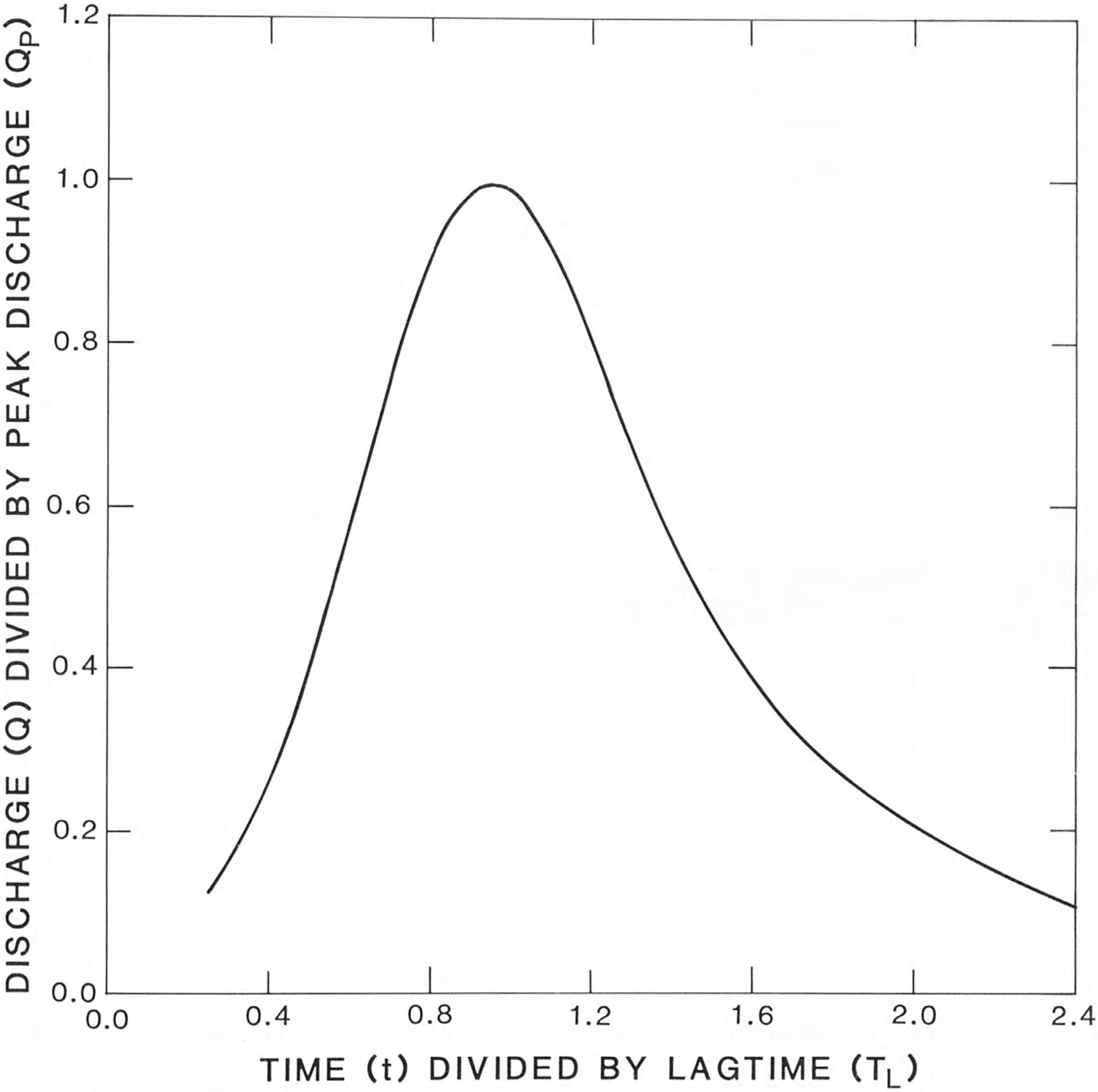

Figure 7.—Statewide dimensionless hydrograph.

Table 2.--Time and discharge ratios of the statewide dimensionless hydrograph

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

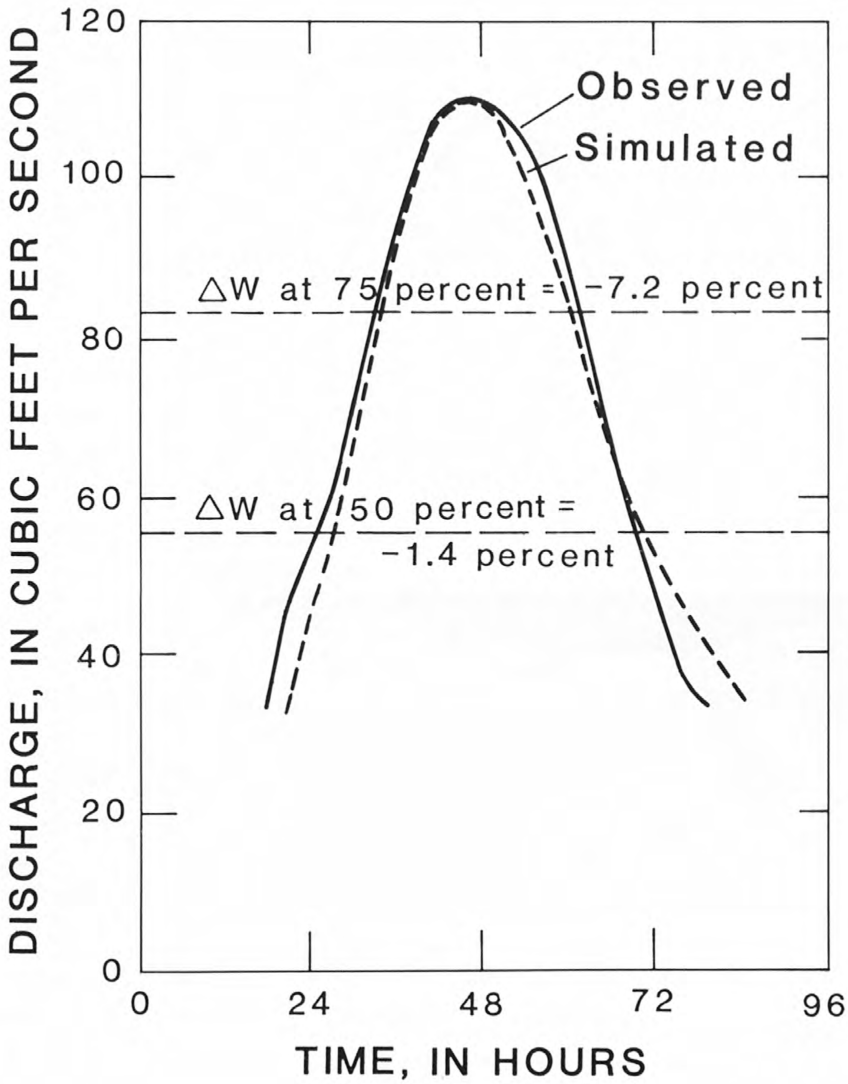

Figure 8.—Plot of observed and simulated hydrographs showing width comparisons at 50 and 75 percent of peak flow for an Atlanta urban station.

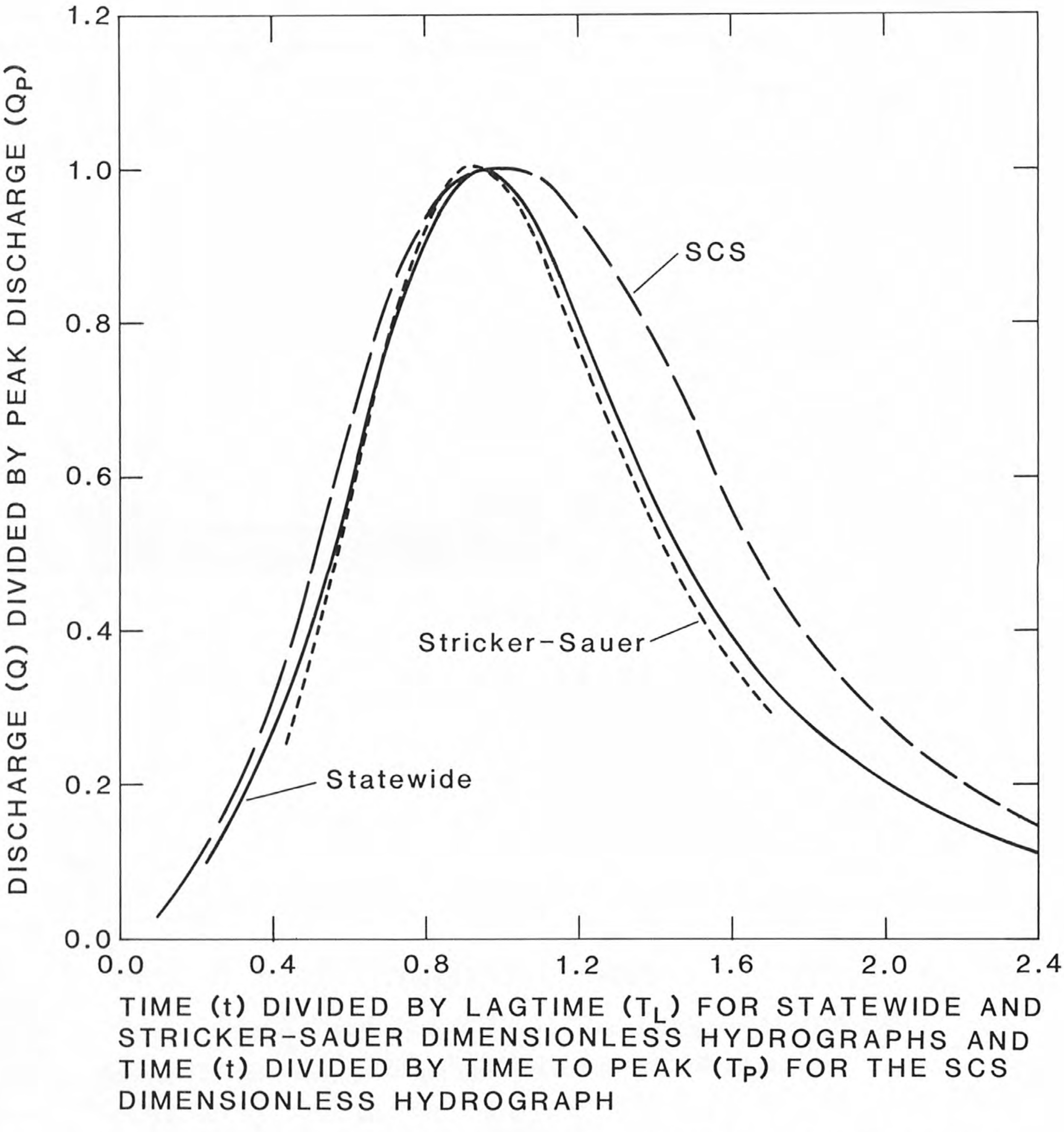

Figure 9.—Plot of statewide, Soil Conservation Service, and Stricker-Sauer dimensionless hydrographs.

Basins Greater Than 500 Square Miles

The method for simulating a hydrograph at basins greater than 500 mi^2 uses the U.S. Geological Survey computer model, CONROUT. The model routes streamflow from an upstream channel location to a user-defined location downstream. The product of CONROUT is a simulated outflow discharge hydrograph with a peak of a specific recurrence interval at the end of a reach. CONROUT is described in detail by Doyle and others (1983).

CONROUT provides the user with two methods of routing: diffusion analogy and storage-continuity. The diffusion analogy method with single linearization was used in this study (Keefer, 1976).

In the diffusion analogy method, two routing parameters, K and C, are necessary for use in the model

$$K = Q/2SW,$$

where

Q = the reference discharge in ft^3/s,

S = average bed slope in ft/ft, and

W = average channel width for a particular study reach in ft, and

$$C = (1/W)(dQ/dY),$$

where

dQ/dY = the slope of the rating curve (stage-discharge relation) at Q in ft^2/s, and

W = average channel width for a particular reach in ft.

K (in units of ft^2/s) is the wave dispersion or damping coefficient that controls the spreading of the wave, and C (in units of ft/s) is the flood wave celerity or the flood wave travel time. Physically a high C value means the flood wave will arrive sooner than one at a lower C value, and a high K value results in a hydrograph being flatter and more spread out than that resulting from a low K value.

Application of the model generally requires three steps: (1) model calibration, (2) model verification, and (3) simulation. For a typical flood-hydrograph simulation at least one corresponding set of carefully selected observed floods must be available at both ends of the reach for steps one and two. These observed floods should be the annual peak with corresponding dates at each station, or at least the same ranking at each station. This criterion lends some evidence that the rainfall was uniform over the entire reach. Also, simple (noncompound) peaks were generally selected. These observed flood hydrographs are generally available only from the daily values file in the U.S. Geological Survey computer in Reston, Va. The selected data were downloaded to the Georgia District computer and transformed into a more efficient format for use in CONROUT. By using these observed flood hydrographs during the calibration phase the routing parameters K and C are optimized.

For verification, data other than those used for calibration, and the optimized parameters from the calibration step, are used to synthesize an output data set. These output data are compared to observed data at the downstream site and if this comparison is satisfactory, then the calibration was successful.

After successful calibration and verification and accounting for ungaged flow, the model may be used to simulate system output for any input condition of interest. Ungaged flow can be accounted for by multiplying by the ratio of ungaged to gaged drainage areas, or by the ratio of mean annual flows of ungaged to gaged areas obtained from runoff maps developed by Carter (1983).

A simulated flood hydrograph from an ungaged site can also be routed to a downstream gaged location, as a later example will illustrate.

HYDROGRAPH-WIDTH RELATION FOR BASINS LESS THAN 500 SQUARE MILES

In some instances it is necessary only to know the period of time that a specific discharge will be exceeded, therefore the complete hydrograph is not needed. For these, a hydrograph-width relation was defined from the dimensionless hydrograph in table 2. Hydrograph width is denoted as W, in hours, and the width ratio, W/T_L, was determined by subtracting the value of t/T_L on the rising limb of the dimensionless hydrograph from the value of t/T_L on the falling limb of the hydrograph at the same discharge ratio, Q/Q_p. This relation is shown in table 3 and figure 10. The hydrograph width, W, can be estimated for a specified discharge, Q, by first computing the ratio Q/Q_p and then multiplying the corresponding W/T_L ratio by the estimated lagtime, T_L.

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

Discharge ratios Q/Q_p	Width ratios W/T_L
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

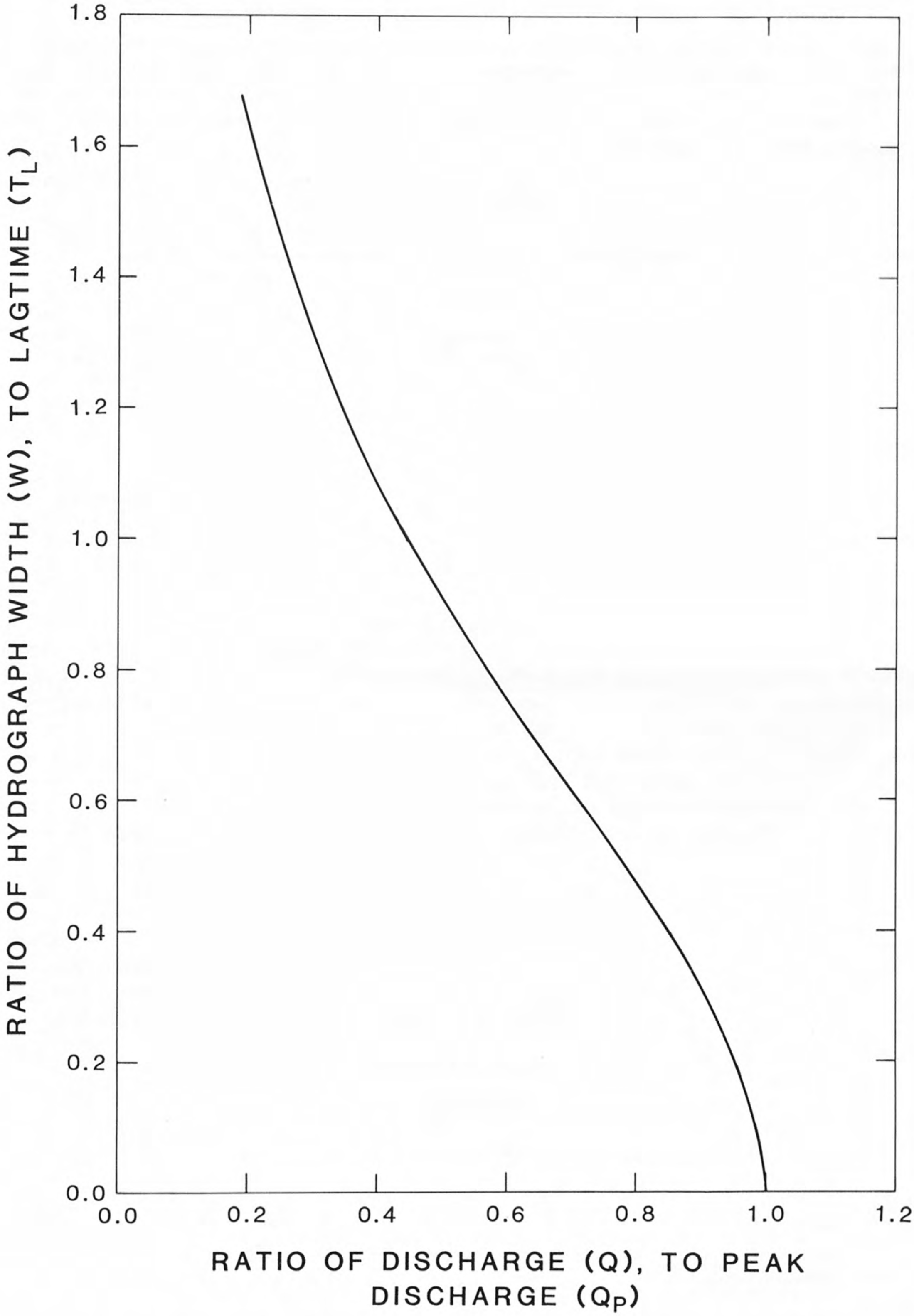

Figure 10.—Hydrograph-width relation for dimensionless hydrograph.

TESTING OF DIMENSIONLESS HYDROGRAPH

Four tests are generally required to establish the soundness of models. The first test is the standard error of estimate which has been explained and presented in prior sections of this report. The other tests are for verification, bias, and sensitivity.

Verification

For verification, the dimensionless hydrograph was applied to other hydrographs not used in its development. This test included the use of 138 floods from 37 stations having drainage areas of 20 to 500 mi^2 located throughout the State. The average station lagtime and peak discharge for each flood were used to simulate a theoretical flood hydrograph, which was compared to the observed hydrograph as figure 11 illustrates. At the 50 and 75 percent of peak flow widths the standard errors of estimate were $\pm$ 39.5 percent and $\pm$ 43.6 percent, respectively.

An additional verification, or test, of the entire simulating procedure was conducted on the largest flood hydrographs (simple or compound) at gaging stations where unit values were available in the Georgia District and where a station flood-frequency curve was available. Thirty-one stations having drainage areas of 20 to 500 mi^2 were tested as follows. The recurrence interval of the observed peak discharge (Q), was determined from the station-frequency curve. The appropriate regional flood-frequency-regression equation from Price (1979) was used to compute the corresponding peak discharge for this recurrence interval. The lagtime (T_L) for this station was computed from the appropriate regional lagtime regression equation. The regression Q and regression T_L were then used to simulate a flood hydrograph. A comparison of the simulated and observed hydrograph widths at 50 and 75 percent of peak flow yielded standard errors of estimate of $\pm$ 51.7 percent and $\pm$ 57.1 percent, respectively. Figure 12 illustrates an example of this comparison.

Bias

Two tests for bias were conducted, one for the simulated versus observed hydrograph width, and the other for geographical bias. The width bias test was performed on the widths at 50 percent and 75 percent of peak flow at the 31 stations used in the additional verification step. As explained earlier, these were the highest available floods at these stations. The average recurrence interval was about 30 years. The mean error, x, indicated that there was a positive error (simulated greater than observed) in the hydrograph widths at 50 percent of peak flow and a negative error (observed greater than simulated) in the hydrograph widths at 75 percent of peak flow. Also, there was a negative error (estimated less than observed) in the comparison of peak Q from regional regression equations and observed peak Q (table 4). However, the student's t-test indicated that these errors are not statistically significant at the 0.01 level of significance, and therefore, the simulated hydrograph widths and the estimated peak discharges are not considered biased.

The test for geographical bias was done by comparing the widths at 50 percent and 75 percent of the ratio, Q/Q_p, of the dimensionless hydrographs

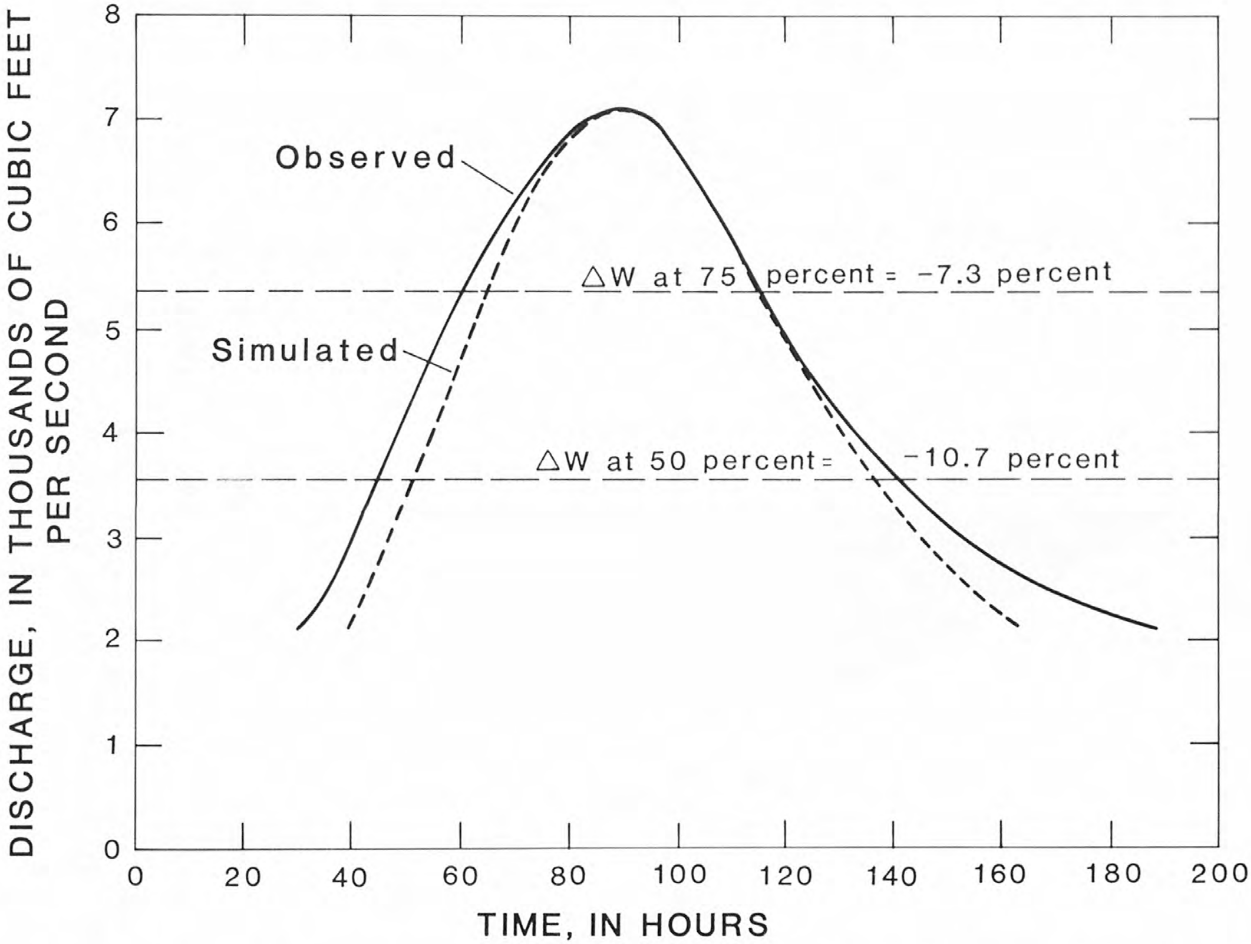

Figure 11.—Plot of observed and simulated hydrographs for width comparisons at 50 and 75 percent of peak flow for Spring Creek near Iron City.

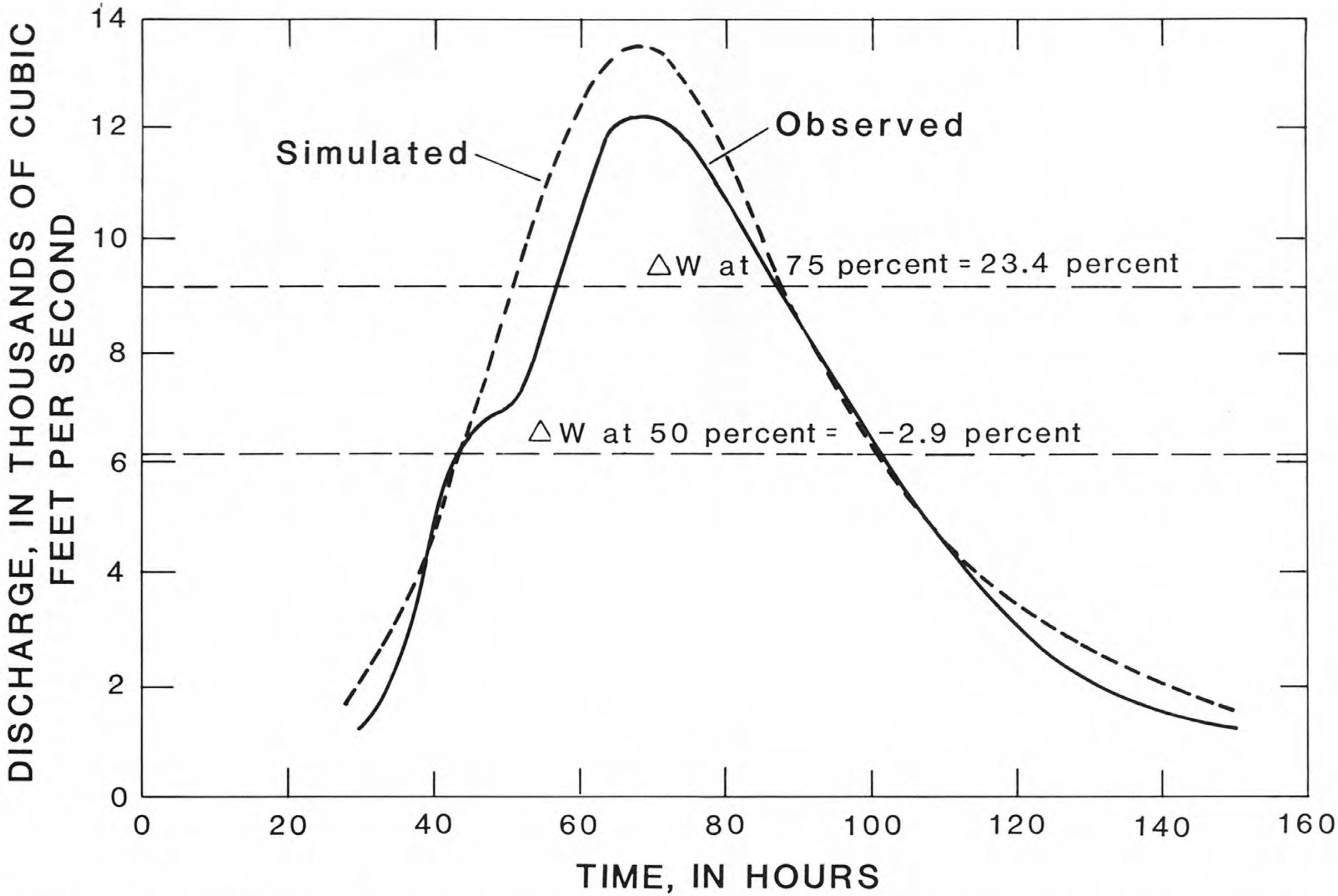

Figure 12.—Plot of observed and simulated hydrographs for width comparisons at 50 and 75 percent of peak flow for Flint River near Griffin.

Table 4.--Differences of hydrograph widths of estimated and observed hydrographs at 50 and 75 percent of observed peak flow, and differences of peak discharge computed from regional regression equations and observed peak discharge, both discharges being for the same recurrence interval, and the means ($\bar{x}$) of these three differences

Station number	Estimated hydrograph width at 50 percent of peak flow (hours)	Observed hydrograph width at 50 percent of peak flow (hours)	Estimated width minus observed width (hours)	Estimated hydrograph width at 75 percent of peak flow (hours)	Observed hydrograph width at 75 percent of peak flow (hours)	Estimated width minus observed width (hours)	Estimated peak discharge from regression equations (ft^3/s)	Observed peak discharge (ft^3/s)	Estimated peak discharge minus observed peak discharge (ft^3/s)
02177000	22.23	9.85	12.38	9.31	5.59	3.72	21,700	26,100	-4,400
02178400	11.33	7.11	4.22	6.03	3.98	2.05	7,690	8,430	-740
02188500	15.60	19.91	-4.31	9.90	10.27	-0.37	3,500	3,280	220
02191200	18.84	14.68	4.16	11.66	7.93	3.73	5,830	5,420	410
02197600	24.91	29.39	-4.48	16.18	15.53	.65	588	532	56
02197830	56.43	49.24	7.19	0	31.31	-31.31	6,670	10,700	-4,030
02203559	64.43	73.35	-8.92	49.78	42.73	7.05	1,540	795	745
02207500	60.38	56.44	3.94	41.14	35.93	5.21	11,200	9,250	1,950
02212600	18.15	15.15	3.00	8.49	8.39	.10	4,480	5,210	-730
02213050	7.40	7.67	-0.27	0	4.84	-4.84	5,050	7,410	-2,360
02217500	47.82	54.34	-6.52	24.37	25.33	-.96	11,500	12,900	-1,400
02219000	31.93	46.62	-14.69	16.14	8.42	7.72	6,190	6,980	-790
02223300	24.27	44.88	-20.61	15.19	30.44	-15.25	529	505	24
02224000	16.16	12.63	3.53	0	7.49	-7.49	5,470	9,100	-3,630
02226100	135.46	169.67	-34.21	92.48	95.63	-3.15	5,360	4,420	940
02227000	93.41	101.98	-8.57	63.37	59.20	4.17	2,740	2,290	450
02328000	26.74	12.38	14.36	10.38	6.68	3.70	2,960	3,620	-660
02331600	35.87	13.68	22.19	21.91	7.49	14.42	21,500	21,100	400
02333500	16.99	11.46	5.53	0	7.15	-7.15	15,300	21,100	-5,800
02337500	10.63	7.70	2.93	4.11	3.95	.16	5,240	6,420	-1,180
02343200	8.51	14.46	-5.95	0	9.16	-9.16	4,440	8,250	-3,810
02344500	57.54	59.24	-1.70	37.37	30.28	7.09	13,500	12,200	1,300
02344700	27.71	17.70	10.01	17.91	10.26	7.65	10,500	9,580	920
02349000	51.56	36.06	15.50	38.52	15.05	23.47	1,970	1,220	750
02349900	23.67	15.78	7.89	0	8.91	-8.91	3,600	4,820	-1,220
02357000	82.72	53.92	28.80	0	34.63	-34.63	9,940	13,400	-3,460
02379500	26.75	9.94	16.81	17.76	6.74	11.02	10,500	9,160	1,340
02380500	31.75	11.96	19.79	19.43	6.91	12.52	17,200	16,900	300
02382200	18.48	10.31	8.17	6.35	7.46	-1.11	11,300	14,000	-2,700
02398000	28.46	20.48	7.98	7.56	12.50	-4.94	15,900	20,400	-4,500
03558000	31.98	15.70	16.28	22.44	12.80	9.64	19,000	14,600	4,400
			$\bar{x}$ = 3.37			$\bar{x}$ = -.165			$\bar{x}$ = -882

simulated for Regions 1, 2, and 3, and for the Atlanta metropolitan area with the widths of the statewide dimensionless hydrograph. Figure 6 illustrates these four dimensionless hydrographs. There was no significant bias. In fact, the mean error, $\overline{x}$, was very small in both the 50 percent and the 75 percent test, which further confirmed the decision to use one dimensionless hydrograph statewide for basins up to 500 mi^2.

Sensitivity

The fourth test was to analyze the sensitivity of the simulated hydrograph widths to errors in the two independent variables (Q and T_L) that are used to simulate the hydrograph. This test was done by holding one variable constant and varying the other by $\pm$ 10 percent, and $\pm$ 20 percent at the hydrograph widths corresponding to 50 percent and 75 percent of peak flow. When peak Q was varied, the test results indicated that the hydrograph width did not change at 50 percent or 75 percent of that varied peak Q. When lagtime was varied, the test results indicated that the hydrograph widths would vary by the same percentage.

REGRESSION ANALYSIS OF LAGTIME

So that lagtime could be estimated for ungaged sites, the average station lagtimes obtained from the stations used in the dimensionless hydrograph development were related to their basin characteristics. This was done by the linear, multiple-regression method described by Riggs (1968). Lagtimes were computed for each flood event with the same program that computed the unit hydrographs. These storm-event lagtimes were then averaged to compute an average station lagtime, which was in turn used in the regression analyses. Lagtime is generally considered to be constant for a basin and is defined as the time from the centroid of rainfall excess to the centroid of the runoff hydrograph (Stricker and Sauer, 1982). Lagtime for the 19 Atlanta urban stations was analyzed separately, because of the effect of urbanization on lagtime.

The regression equations provide a mathematical relation between the dependent variable (lagtime) and the independent variables (the basin characteristics found to be statistically significant). 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). In the analyses performed, a 95-percent confidence limit was specified to select the significant independent variables.

The regression analyses were performed using "Statistical Analysis System" (SAS) (SAS Institute, Inc., 1982). Six specific SAS analyses performed were: (1) backward-backward elimination, (2) stepwise-stepwise regression, forward and backward, (3) MAXR-forward selection with pair switching, (4) MINR-forward selection with pair searching, (5) forward-forward searching, and (6) GLM-plots predicted versus observed lagtimes and residuals versus significant parameters. Additional information on the models is available in the SAS Institute, Inc., User's Guide (1982): Statistics.

The independent variables, or physical basin characteristics, are defined in the following paragraphs and the selected basin characteristics for stations above the Fall Line are shown in table 5, and the selected basin characteristics for stations below the Fall Line are shown in table 6. Table 7 shows the selected basin characteristics for the Atlanta urban stations.

Lagtime (T_L).--The elapsed time, in hours, from the centroid of rainfall excess to the centroid of the resultant runoff hydrograph. Lagtime is computed from the unit hydrograph.

Drainage area (A).--Area of the basin, in square miles, planimetered from U.S. Geological Survey 7 1/2-minute topographic maps. Basin boundaries were all field checked.

Channel slope (S).--The main channel slope, in feet per mile, as determined from topographic maps. The main channel slope was computed as the difference in elevation, in feet, at the 10- and 85-percent points divided by the length, in miles, between the two points.

Channel length (L).--The length of the main channel, in miles, as measured from the gaging station upstream along the channel to the basin divide.

$L/S^{0.5}$.--A ratio, where L and S have been previously defined.

Measured total impervious area (IA).--The percentage of drainage area that is impervious to infiltration of rainfall. This parameter was determined by a grid-overlay method using aerial photography. According to Cochran (1963) a minimum of 200 points, or grid intersections, per area or subbasin will provide a confidence level of 0.10. Three counts of at least 200 points per subbasin were obtained and the results averaged for the final value of measured total impervious area. On several of the larger basins where some development occurred during the period of data collection, this parameter was determined from aerial photographs made in 1972 (near the beginning of data collection), and then averaged with the values obtained from aerial photographs made in 1978 (near the end of data collection).

Measured effective impervious area (MEIA).--The percentage of impervious area which is directly connected to the channel drainage system. Noneffective impervious area, such as house rooftops that drain onto a lawn, are subtracted from this total. This parameter was obtained in conjunction with measured total impervious area. When the minimum of 200 points were counted, three totals per subbasin were obtained. The first total was pervious points, the second definite impervious points such as streets and parking lots, and the third rooftops. One building out of three was field checked to determine the percentage of effective impervious area of its roof and gutter system. An average percentage of effective impervious area was determined for the buildings field checked in the subbasin, and this factor was multiplied by the total number of building points. The resulting product was added to the definite impervious points, and this total of effective impervious area points was divided by the total number of points counted in the subbasin to determine the MEIA percentage.

Table 5.--Selected physical characteristics of basins north of the Fall Line

Station No.	T_L	A	S	L	$\frac{L}{S}$
02177000	27.0	207	37.2	45.8	7.51
02178400	18.2	56.5	72.8	13.9	1.63
02188500	23.5	35.8	10.9	20.0	6.06
02189020	5.33	7.63	22.2	6.14	1.30
02191200	21.7	61.1	13.4	19.9	5.44
02191270	8.23	8.75	21.1	4.59	.999
02191280	1.12	.32	175	.59	.045
02191600	3.71	4.77	44.0	4.04	.609
02191750	9.68	16.0	25.8	10.7	2.11
02191930	6.81	5.80	45	4.05	.604
02192400	5.67	5.49	28.5	5.04	.944
02193300	3.50	6.30	42	3.18	.491
02193600	1.37	1.14	50	1.89	.267
02207500	40.5	378	6.68	51.9	20.1
02208200	1.96	1.03	70	1.59	.190
02211459	2.17	2.36	51.4	2.80	.391
02212600	22.8	72.2	9.94	12.9	4.09
02213050	17.5	29.0	18.5	9.42	2.19
02217250	1.12	.39	146	.72	.060
02217400	2.91	2.68	71	2.72	.323
02217500	37.5	398	6.32	42.9	17.1
02217660	1.49	.87	62	1.37	.174
02218100	2	1.95	43	2.35	.358
02218450	10.7	11.9	19	8.84	2.03
02219000	38.9	176	6.32	50.6	20.1
02331600	22.1	315	26.6	32.4	6.28
02333500	22.6	153	28.7	25.6	4.78
02337500	15.6	37	20.1	13.2	2.94
02344500	74.8	272	2.80	44.9	26.8
02344700	46.7	101	8.70	23.5	7.97
02346210	5.05	6.62	32.8	4.60	.803
02346217	2.37	2.82	51.3	2.73	.381
02379500	31.2	134	23.5	30.1	6.21
02380500	33.7	236	24.5	33.4	6.75
02381100	1.70	2.41	105	3.09	.302
02381600	4.25	9.99	111	6.19	.588
02381900	2.59	3.50	110	4.75	.453
02382200	22.3	119	19.9	30.8	6.90
02382600	5.27	7.30	231	6.04	.397
02382800	5.26	3.06	145	2.84	.236
02383000	6.45	6.17	44.9	3.00	.448
02384600	7.68	4.28	27.5	3.63	.692
02387560	3.02	3.56	65.5	2.96	.366
02387800	5.01	3.82	72	3.10	.365
02388200	6.93	6.02	65	3.22	.399
02388400	2.11	3	93	3.28	.340
02398000	32.6	192	6.60	33.3	13.0
03558000	39.7	177	30.4	29.6	5.37
03566660	6.07	4.44	19.6	3.44	.777

Table 6.--Selected physical characteristics of basins south of the Fall Line

Station No.	T_L	A	S	L	$\frac{L}{\sqrt{S}}$
02197600	24.2	28.0	14.3	13.8	3.65
02197830	105	473	4.82	61.1	27.8
02201000	62.3	109	8.29	23.8	8.27
02201110	11.3	8.36	19.6	5.38	1.22
02201160	9.11	7.05	23.3	4.18	.866
02202810	14.5	5.05	26.2	4.42	.864
02202910	4.90	1.14	22.5	1.54	.325
02202950	6.40	1.39	26.5	2.48	.482
02203559	69.4	33.0	2.89	11.1	6.52
02215230	16.3	7.80	19.0	5.23	1.20
02215245	7.56	1.44	46.0	1.75	.258
02215280	6.30	2.45	43.1	2.35	.358
02216610	5.70	2.71	19.8	2.37	.533
02223300	31.3	31.0	15.6	11.1	2.81
02223700	4.93	2.13	36.7	2.32	.383
02224000	35.5	62.9	12.1	15.3	4.40
02224200	15.8	16.1	15.5	6.52	1.66
02225210	9.76	3.53	26.8	3.84	.742
02225330	16.9	9.58	19.0	5.48	1.26
02226100	105	210	1.30	25.7	22.5
02226190	13.2	6.38	16.7	6.08	1.49
02227000	112	150	2.60	29.0	18.0
02315650	1.41	.14	55.0	.54	.073
02315670	17.5	3.95	14.6	4.30	1.13
02315980	5.25	1.21	33.6	1.47	.254
02316260	13.8	4.16	5.40	3.73	1.61
02317710	2.39	.86	30.8	1.38	.249
02317765	6.66	.98	26.0	2.05	.402
02317770	14.6	6.48	18.0	5.65	1.33
02317795	13.4	6.21	19.1	3.77	.863
02317905	7.62	4.22	21.6	3.69	.794
02318015	5.25	1.36	25.2	1.74	.347
02318700	104	269	6.32	54.8	21.8
02327350	5.50	1.81	28.5	2.38	.446
02327400	9.10	3.70	12.7	3.08	.864
02328000	36.9	60.0	12.8	10.6	2.96
02343200	31.3	70.0	22.2	14.0	2.97
02349000	46.6	93.4	17.8	15.7	3.72
02349900	32.3	45.0	8.70	11.0	3.73
02350600	57.6	197	7.50	25.6	9.35
02351890	95.1	362	4.21	60.6	29.5
02357000	93.9	485	4.20	42.3	20.6

Table 7.--Selected physical characteristics of Atlanta urban basins

Station No.	T_L	A	S	L	$\frac{L}{\sqrt{S}}$	EIA	IA
02203820	3.81	8.67	28.0	7.58	1.43	21.7	30.5
02203835	1.41	3.43	61.0	2.66	.340	18.9	25.6
02203845	.83	.84	67.6	1.93	.235	23.4	30.6
02203850	2.06	7.50	34.8	5.91	1.00	21.0	28.2
02203870	2.18	3.68	37.5	3.95	.645	19.9	25.8
02203884	1.21	1.88	74.1	2.22	.258	23.4	26.7
02336080	6.41	19.1	16.0	7.43	1.86	26.4	31.4
02336090	.71	.32	129	1.12	.099	11.4	19.0
02336102	1.27	2.19	62.8	2.50	.316	19.6	27.2
02336150	2.56	5.29	25.8	5.06	.996	18.0	24.1
02336180	4.57	11.0	19.0	9.03	2.07	21.5	25.9
02336200	1.01	.98	94.5	1.47	.151	26.2	32.3
02336238	.68	.92	106	1.60	.155	24.8	33.6
02336325	.96	1.35	53.8	2.14	.292	39.6	42.0
02336690	.81	.52	90.7	1.22	.128	14.1	20.3
02336697	.86	.21	136	1.09	.094	11.1	19.0
02336700	.76	.79	75.8	1.46	.168	18.2	28.3
02336705	2.48	8.80	33.7	4.95	.853	23.5	29.5
02337081	.78	.88	86.9	1.43	.153	19.9	28.6

Regionalization

The initial regression run utilized data from 91 rural stations of less than 500 mi^2 located throughout the State. A geographical bias was detected. The area north of the Fall Line, consisting of Regions 1 and 2 as defined by Price (1979), tended to overpredict lagtime, whereas, the area south of the Fall Line, consisting of Regions 3, 4, and 5 as defined by Price (1979), tended to underpredict lagtime.

The next step was to make separate regression runs for each of the five regions. Region 1 had only one independent variable significant at the 95-percent confidence limit. The standard error of estimate of the regression using only one variable ranged from 43 to 51 percent. Such large standard errors are not desirable. Region 2, also, had only one independent variable significant at the 95-percent confidence limit. The standard error of estimate of the regression ranged from 34 to 37 percent, with a tendency to overpredict on the lower end of the curve and underpredict on the upper end.

Regions 1 and 2 were combined and analyzed as one region. Two equations with two parameters each were significant at the 95-percent confidence limit. Based on the verification step, as explained in a later section, the equation selected was lagtime (T_L) = $4.64A^{0.49}\ S^{-0.21}$. Region 4 had only five stations, and Region 5 only three. Therefore, neither region could be analyzed separately. Regions 3, 4, and 5 were combined and analyzed as one region. Only one equation had two variables significant at the 95-percent confidence limit. The equation was $T_L = 13.6A^{0.43}\ S^{-0.31}$.

The Atlanta urban area was analyzed separately owing to the effects of urbanization on lagtime. IA and MEIA were added as independent variables in the analysis. The equation that was selected, $T_L = 161A^{0.22}\ S^{-0.66}\ IA^{-0.67}$, is similar to the rural equations, in that both rural and urban equations have area and slope as independent variables. Impervious area accounts for the urbanization effect. Drainage area, (A), had a significance level of 6.8 percent, but was retained in order to provide continuity with the rural equations. The Atlanta urban equation should be considered preliminary, and subject to revision after more urban data are analyzed in the Rome, Athens, Augusta, and Columbus metropolitan areas. If these additional data show the same regionalization pattern as the rural data north of the Fall Line, then these data will be analyzed with the Atlanta data, which could possibly change the Atlanta urban equation.

The accuracy of regression equations can be expressed by two standard statistical measures: the coefficient of determination, R-square (the correlation coefficient squared), and the standard error of regression. R-square measures how much variation in the dependent variable can be accounted for by the independent variables. For example, an R-square of 0.94 would indicate that 94 percent of the variation is accounted for by the independent variables, and that 6 percent is due to other factors. The standard error of regression (or estimate) is, by definition, one standard deviation on each side of the regression line and contains about two-thirds of the data within this range. A summary of the lagtime equations and their related statistics are given in table 8.

Table 8.--Summary of lagtime estimating equations

Area	Equation	Standard error of regression (percent)	Coefficient of determination, R^2
Above Fall Line (rural)	$T_L = 4.64A^{.49}S^{-.21}$	$\pm$ 31	0.94
Below Fall Line (rural)	$T_L = 13.6A^{.43}S^{-.31}$	$\pm$ 25	.96
Metropolitan Atlanta (urban)	$T_L = 161A^{.22}S^{-.66}IA^{-.67}$	$\pm$ 19	.94

Limits of Independent Variables

The effective usable range of basin characteristics for the rural equations are as follows:

North of the Fall Line

Variable	Minimum	Maximum	Units
A	0.3	500	square miles
S	5.0	200	feet per mile

South of the Fall Line

Variable	Minimum	Maximum	Units
A	0.2	500	square miles
S	1.3	60	feet per mile

The effective usable range of basin characteristics for the Atlanta urban equation is as follows:

Variable	Minimum	Maximum	Units
A	0.2	25	square miles
S	13	175	feet per mile
IA	14	50	percent

TESTING OF LAGTIME-REGRESSION EQUATIONS

The lagtime regression equations were tested with the same four tests as the dimensionless hydrograph. The standard error of estimate has been explained and presented in a prior section of this report. Verification, bias, and sensitivity are the other tests.

Verification

Split-sample testing is the process by which part of a data set is used for calibration and the remaining part for verification or prediction. The standard error of estimate, obtained from the calibration phase, is a measure of how well the regression equations will estimate the dependent variable at the sites used to calibrate them. The standard error of prediction, on the other hand, is a measure of how well the regression equations will estimate the dependent variable at other than calibration sites (Sauer and others, 1983). Split-sample testing was used for verification of the regression equations, both north and south of the Fall Line. It was also used to estimate the magnitude of the average prediction error, and to determine whether the same variables were significant. The stations from each region were divided into two groups of about equal size. The sites were arrayed in ascending order according to drainage-area magnitude. The odd-numbered sites made up the first sample and the even-numbered sites the second sample. Multiple-regression analyses were performed on both regions using only the sites in one of the samples, then recalibrated using the sites in the other sample. The results were all acceptable, as shown in table 9. The split-sample-regression analyses yielded regression equations similar to the equations originally developed using all the sites in each region.

The first set of equations tentatively selected had area (A) and $L/S^{0.5}$ as the two independent variables. The standard errors of regression were about the same as for the equations with A and slope (S) as independent variables for both regions. However, when split-sample testing was performed, $L/S^{0.5}$ was not significant at the 95-percent confidence limit for either the odd or even sample north of the Fall Line. The equation with A and $L/S^{0.5}$ was split-sample tested for the area south of the Fall Line with A not being significant at the 95-percent confidence limit for either the odd or the even sample.

No attempt was made to analyze the Atlanta urban equation with split-sample testing because of the limited number of stations available.

Bias

Two tests for bias were performed, one for variable bias and the other for geographical bias. The variable-bias tests were made by plotting the residuals (difference between observed and predicted lagtime) versus each of the independent variables for all stations. These plots were visually inspected to determine whether there was a consistent overprediction or underprediction within the range of any of the independent variables. These plots also verified the linearity assumptions of the equations. The equations were found to be free of variable bias throughout the range of all independent variables.

Table 9.--Lagtime equations split-sample test results

Area	Sample designation	Number of stations	Equation	Standard error of regression (percent)	Standard error of prediction (percent)	Coefficient of determination, R^2
North of Fall Line	Odd	25	$T_L = 4.88A^{0.48}S^{-0.22}$	$\pm$ 32	--	0.94
	Even	24	--	--	$\pm$ 32	.93
	Even	24	$T_L = 4.51A^{0.50}S^{-0.21}$	$\pm$ 31	--	.94
	Odd	25	--	--	$\pm$ 32	.94
South of Fall Line	Odd	21	$T_L = 36.8A^{0.35}S^{-0.57}$	$\pm$ 18	--	.98
	Even	21	--	--	$\pm$ 41	.92
	Even	21	$T_L = 8.63A^{0.48}S^{-0.21}$	$\pm$ 26	--	.96
	Odd	21	--	--	$\pm$ 29	.96

Geographical bias was tested by plotting the residuals of observed lagtimes minus predicted lagtimes on a State map. The plot was visually inspected to determine if any area of the State was being consistently overestimated or underestimated. Because this test indicated no consistent overestimation or underestimation in any part of the State, it can be concluded that no geographical bias exists.

The same variable bias analysis was performed on the Atlanta urban equation. There was no variable bias.

Sensitivity

The fourth test was to analyze the sensitivity of lagtime to errors in the two independent variables in the regression equations. The computation of these independent variables is subject to errors in measurement and judgement. To illustrate the effect of such errors, the equations were tested to determine how much error was introduced into the computed lagtime from specified percentage errors in the independent variables. The test results are shown in tables 10 and 11. These tables were computed by assuming that all independent variables were constant, except the one being tested for sensitivity.

Table 10.--Sensitivity of computed lagtime to errors in independent variables with the north of Fall Line equation

Percent error in independent variable	INDEPENDENT VARIABLES (Percent error in computed lagtime)	
	Area	Slope
+50	+21.9	-8.2
+25	+11.5	-4.6
+10	+4.8	-2.0
-10	-5.0	+2.2
-25	-13.1	+6.2
-50	-28.5	+15.7

Table 11.--Sensitivity of computed lagtime to errors in independent variables with the south of Fall Line equation

Percent error in independent variable	INDEPENDENT VARIABLES (Percent error in computed lagtime)	
	Area	Slope
+50	+19.2	-11.8
+25	+10.1	-6.7
+10	+4.2	-2.9
-10	-4.5	+3.3
-25	-11.7	+9.4
-50	-25.9	+24.1

The Atlanta urban equation was tested for sensitivity of lagtime to errors in the three independent variables in the same manner as the two rural equations. The test results are shown in table 12.

Table 12.--Sensitivity of computed lagtime to errors in independent variables with the Atlanta urban equation

Percent error in independent variable	INDEPENDENT VARIABLES (Percent error in computed lagtime)		
	Area	Slope	Impervious area
+50	+9.9	-23.4	-23.9
+25	+5.4	-13.5	-14.0
+10	+2.7	-5.9	-6.3
-10	-2.2	+7.2	+7.2
-25	-5.9	+21.2	+21.2
-50	-14.0	+58.1	+59.0

APPLICATION OF TECHNIQUE

An application of hydrograph and lagtime estimation, and routing is illustrated in the following example. The problem is to simulate a hydrograph with a 50-year recurrence interval for peak discharge on the Ogeechee River at State Highway 24 in Jefferson County. This is an ungaged site for which the drainage area lies in two hydrologic regions. Then, simulate a hydrograph with a 50-year recurrence interval for peak discharge on the Ogeechee River at U.S. Highway 1, near Louisville in Jefferson County, by routing flow from the upstream site at Highway 24. The procedure is as follows:

(1) Locate the sites on the best available topographic maps and determine the drainage areas and slopes upstream from the highway crossings. At State Highway 24 the drainage area is 500 mi^2 and the slope is 5.58 ft/mi.

(2) Using figure 13, determine the hydrologic regions involved. For the example basins they are Regions 2 and 4 for determining peak discharge, and Regions north of the Fall Line and south of the Fall Line for computing lagtime. Compute the percentage of total drainage area in each region (48 percent in Region 2 and north of the Fall Line, and 52 percent in Region 4 and south of the Fall Line) for the site at State Highway 24.

(3) Using the equation for Region 2 (Price, 1979), the 50-year peak discharge for a 500-mi^2 basin is 26,700 ft^3/s, and using the equation for Region 4 (Price, 1979), the 50-year peak discharge for a 500-mi^2 basin is 7,490 ft^3/s.

(4) Prorate the discharges computed in step 3 by the percentage of drainage area computed in step 2, as follows:

<u>Region 2</u>: 26,700 ft^3/s x 48 percent = 12,800 ft^3/s

<u>Region 4</u>: 7,490 ft^3/s x 52 percent = 3,890 ft^3/s

SUM = 16,690 ft^3/s*

(*USE = 16,700 ft^3/s)

(5) Using the equation for north of the Fall Line, lagtime is determined to be 68.0 hours, and using the equation for south of the Fall Line, lagtime is determined to be 116 hours.

(6) Prorate the lagtimes computed in step 5 by the percentage of drainage area computed in step 2, as follows:

<u>North of the Fall Line</u>: 68.0 hours x 48 percent = 32.6 hours

<u>South of the Fall Line</u>: 116 hours x 52 percent = 60.3 hours

SUM = 92.9 hours

(7) Simulate a hydrograph using the statewide dimensionless hydrograph, the estimated 50-year peak discharge, and the estimated lagtime for this 500-mi^2 basin. Table 13 and figure 14 illustrate this simulated hydrograph.

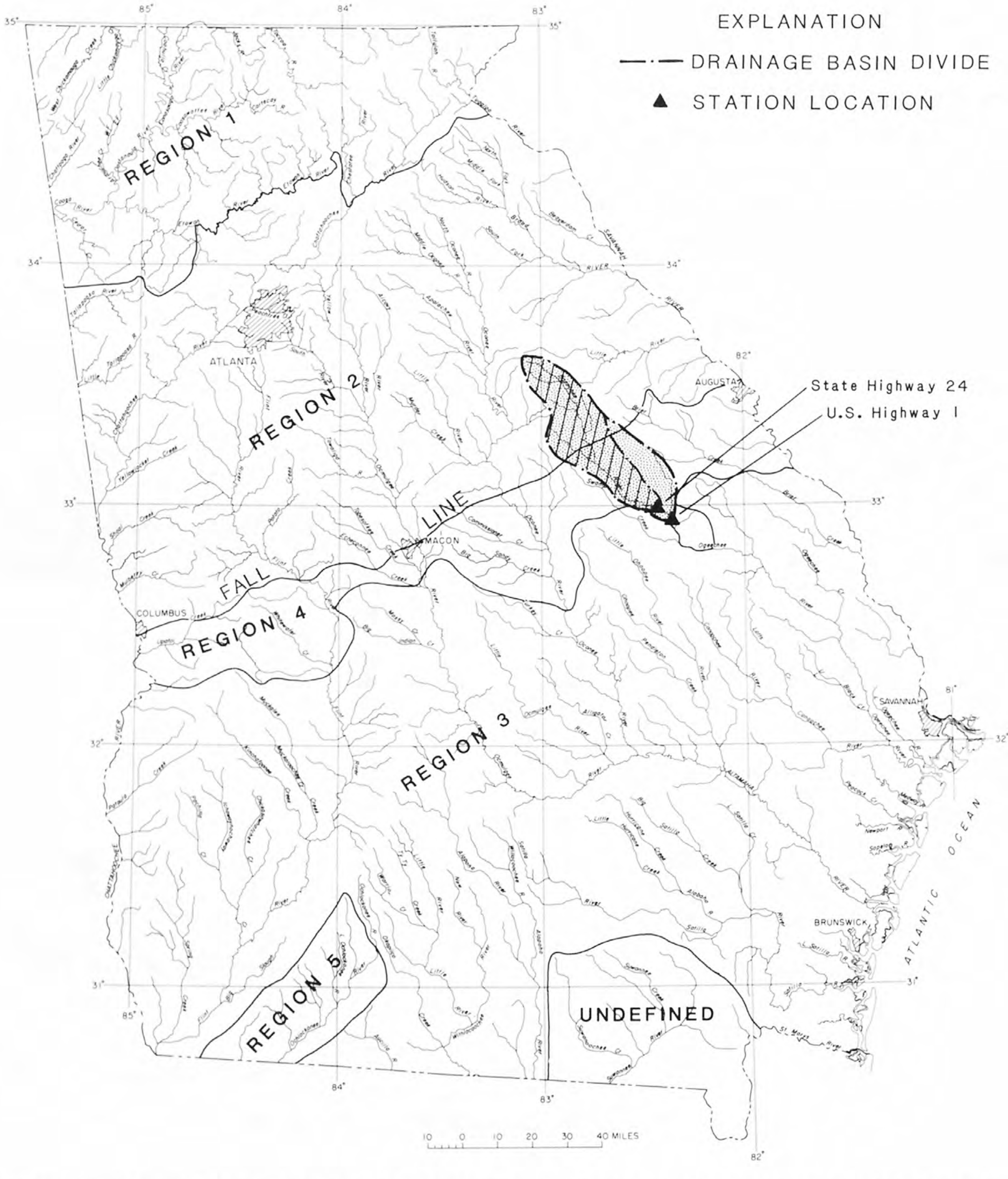

Figure 13.—Regional boundaries for flood-frequency and lagtime estimating equations.

Table 13.--Simulated coordinates of the flood hydrograph for Ogeechee River at State Highway 24

T/T_L (from table 2)	xT_L	= time (hr)	Q/Q_P (from table 2)	xQ_P	= Discharge (ft^3/s)
0.25	92.9	23.2	0.12	16,700	2,000
.30	92.9	27.9	.16	16,700	2,670
.35	92.9	32.5	.21	16,700	3,510
.40	92.9	37.2	.26	16,700	4,340
.45	92.9	41.8	.33	16,700	5,510
.50	92.9	46.4	.40	16,700	6,680
.55	92.9	51.1	.49	16,700	8,180
.60	92.9	55.7	.58	16,700	9,690
.65	92.9	60.4	.67	16,700	11,200
.70	92.9	65.0	.76	16,700	12,700
.75	92.9	69.7	.84	16,700	14,000
.80	92.9	74.3	.90	16,700	15,000
.85	92.9	79.0	.95	16,700	15,900
.90	92.9	83.6	.98	16,700	16,400
.95	92.9	88.2	1.00	16,700	16,700
1.00	92.9	92.9	.99	16,700	16,500
1.05	92.9	97.5	.96	16,700	16,000
1.10	92.9	102.2	.92	16,700	15,400
1.15	92.9	106.8	.86	16,700	14,400
1.20	92.9	111.5	.80	16,700	13,400
1.25	92.9	116.1	.74	16,700	12,400
1.30	92.9	120.8	.68	16,700	11,400
1.35	92.9	125.4	.62	16,700	10,400
1.40	92.9	130.1	.56	16,700	9,350
1.45	92.9	134.7	.51	16,700	8,520
1.50	92.9	139.4	.47	16,700	7,850
1.55	92.9	144.0	.43	16,700	7,180
1.60	92.9	148.6	.39	16,700	6,510
1.65	92.9	153.3	.36	16,700	6,010
1.70	92.9	157.9	.33	16,700	5,510
1.75	92.9	162.6	.30	16,700	5,010
1.80	92.9	167.2	.28	16,700	4,680
1.85	92.9	171.9	.26	16,700	4,340
1.90	92.9	176.5	.24	16,700	4,010
1.95	92.9	181.2	.22	16,700	3,670
2.00	92.9	185.8	.20	16,700	3,340
2.05	92.9	190.4	.19	16,700	3,170
2.10	92.9	195.1	.17	16,700	2,840
2.15	92.9	199.7	.16	16,700	2,670
2.20	92.9	204.4	.15	16,700	2,500
2.25	92.9	209.0	.14	16,700	2,340
2.30	92.9	213.7	.13	16,700	2,170
2.35	92.9	218.3	.12	16,700	2,000
2.40	92.9	223.0	.11	16,700	1,840

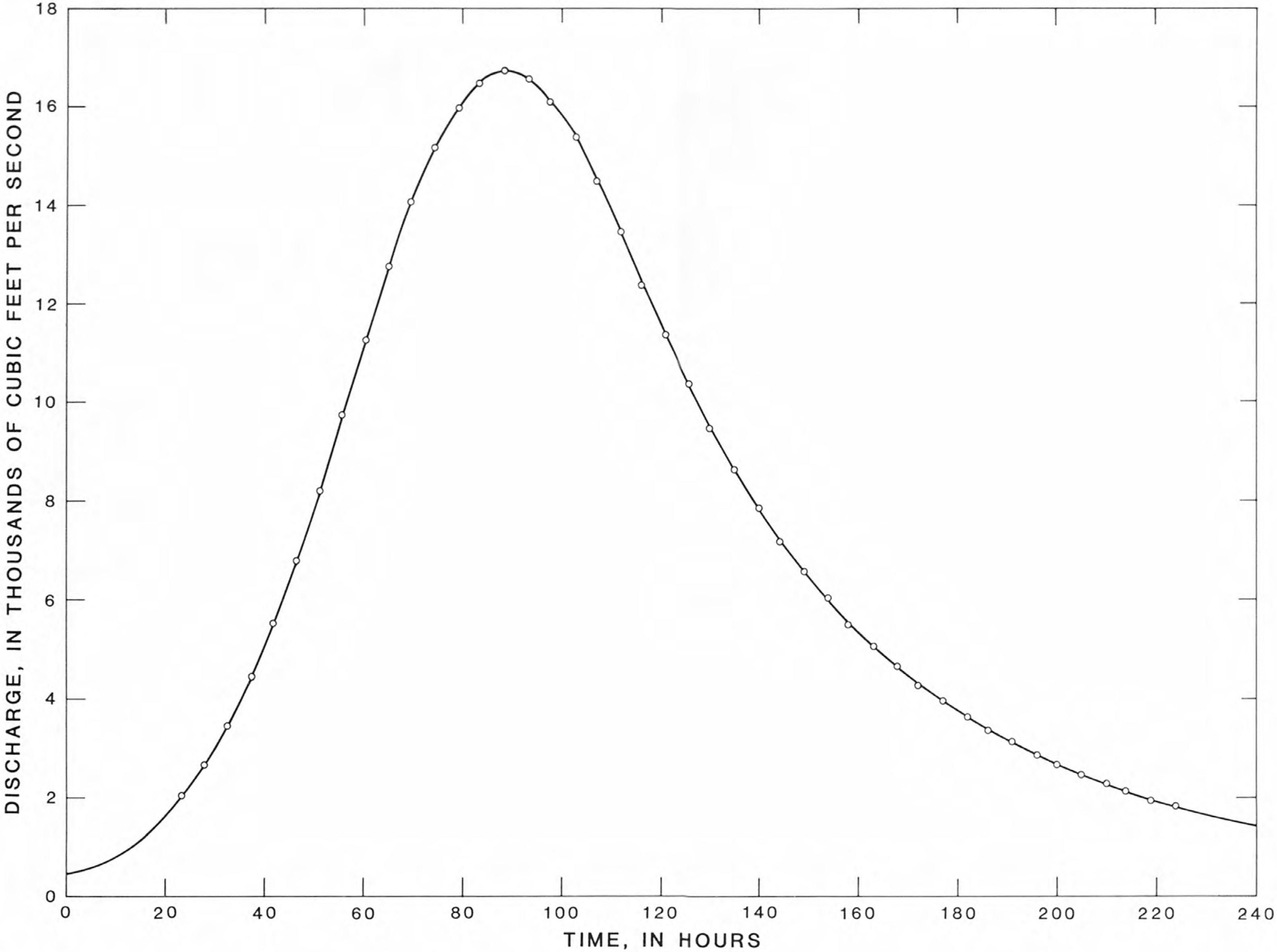

Figure 14.—Simulated flood hydrograph for Ogeechee River at State Highway 24.

(8) Route the simulated hydrograph from step 7 to the downstream site at U.S. Highway 1, near Louisville (or any site between). This downstream site has a drainage area of 800 mi^2 and about 29 years of gaged record. The slope for the reach between State Highway 24 and U.S. Highway 1 is 1.87 ft/mi. Compute the routing parameters C and K, and the ratio of mean annual flow at the downstream site to the mean annual flow at the upstream site. Then use the CONROUT model to route the simulated hydrograph to the downstream site. The routed hydrograph discharge ordinates at the downstream site are increased by the mean annual flow ratio. A hydrograph was simulated for the downstream site with a peak discharge of 26,400 ft^3/s that was verified by comparison with the 50-year flood of 26,300 ft^3/s based on observed record at the site. The hyrograph shape appears reasonable when compared with the highest observed flood hydrograph (about a 15-year recurrence interval) at the downstream site. Figure 15 illustrates this comparison.

SUMMARY

A dimensionless hydrograph was developed for Georgia streams having drainage areas of less than 500 mi^2. This dimensionless hydrograph can be used to simulate flood hydrographs at ungaged sites for rural streams statewide and urban streams in the Atlanta area. Over 350 observed flood hydrographs were used for its development. For verification, the dimensionless hydrograph was applied to 169 flood hydrographs not used in its development.

Multiple-regression analysis was used to define relations between lagtime and selected basin characteristics, of which drainage area and slope were significant for the rural basins, and drainage area, slope, and impervious area were significant for the Atlanta urban basins. Two rural-stream equations were developed--for areas north of and south of the Fall Line. Both rural equations were verified by split-sample testing. There was no variable or geographical bias in either rural equation, or in the Atlanta urban equation. Sensitivity tests indicated that drainage area is the most sensitive basin characteristic in the rural equations, and that impervious area is the most sensitive in the Atlanta urban equation.

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

For basins larger than 500 mi^2, the U.S. Geological Survey computer model CONROUT is used for simulating flood hydrographs. CONROUT routes streamflow from an upstream channel location to a user-defined location downstream. The product of CONROUT is a simulated discharge hydrograph for the downstream site which has a peak of a specific recurrence interval.

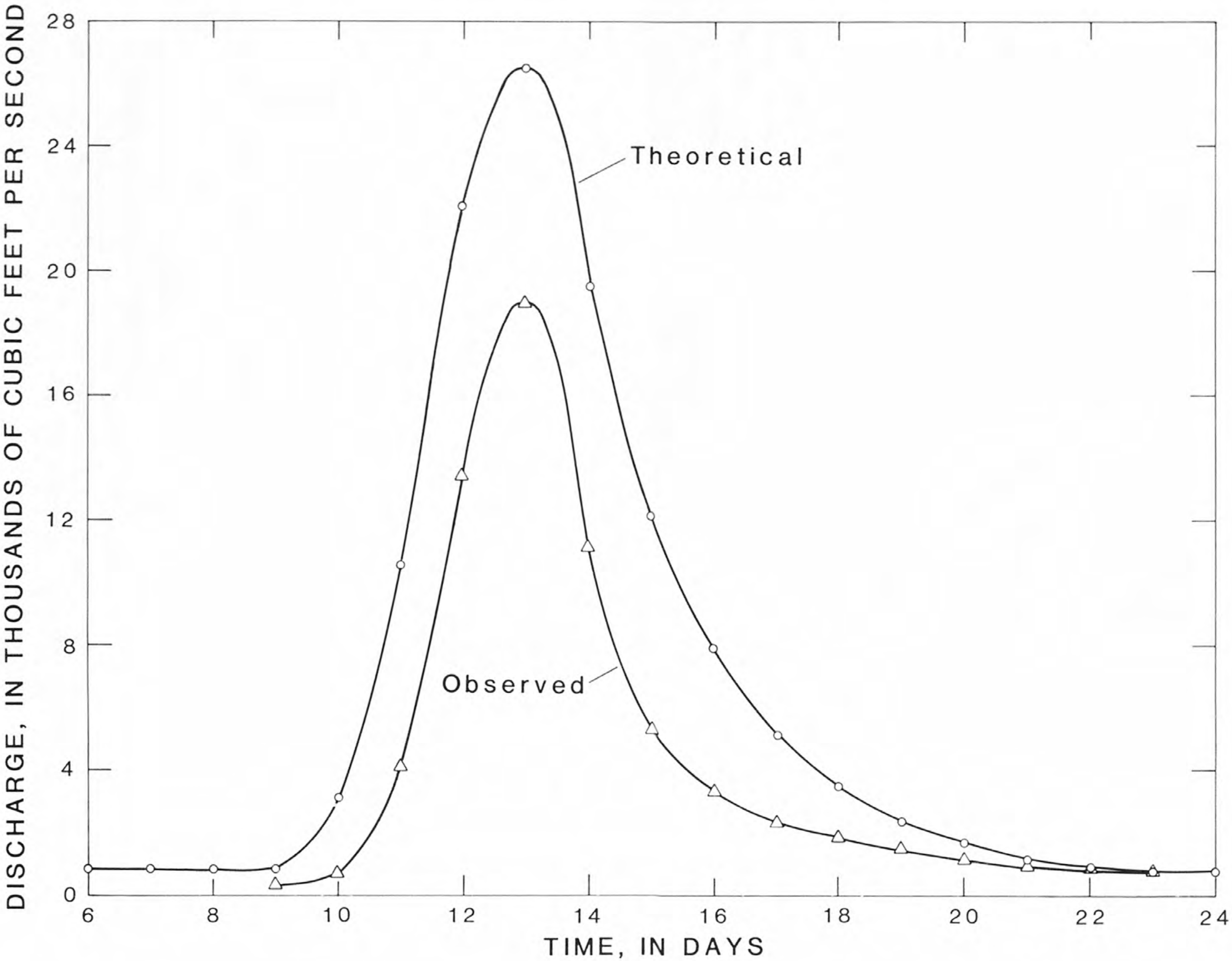

Figure 15.—Routed theoretical 50-year flood hydrograph and highest observed flood hydrograph at Ogeechee River at U.S. Highway 1 near Louisville.

REFERENCES

Carter, R. F., 1983, Storage requirements for Georgia streams: U.S. Geological Survey Water-Resources Investigations 82-557, 65 p.

Cochran, W. G., 1963, Sampling techniques: New York, John Wiley, p. 71-86.

Corry, M. L., Jones, J. S., and Thompson, P. L., 1980, The design of encroachments on flood plains using risk analysis: U.S. Department of Transportation, Federal Highway Administration, 84 p.

Doyle, H. W., Sherman, J. D., Stiltner, G. J., and Krug, W. R., 1983, A digital model for streamflow routing by convolution methods: U.S. Geological Survey Water-Resources Investigations 83-4160, 130 p.

Inman, E. J., 1983, Flood-frequency relations for urban streams in metropolitan Atlanta, Georgia: U.S. Geological Survey Water-Resources Investigations 83-4203, 38 p.

Keefer, W. R., 1976, Comparison of linear systems and finite difference flow routing techniques: Water Resources Research, v. 12, no. 5, p. 997-1006.

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

Price, McGlone, 1979, Floods in Georgia, magnitude and frequency: U.S. Geological Survey Water-Resources Investigations 78-137, 269 p.

Riggs, H. C., 1968, Some statistical tools in hydrology: U.S. Geological Survey Techniques of Water-Resources Investigations, book 4, chapter A1, 39 p.

SAS Institute, Inc., 1982, SAS User's Guide: Statistics, 583 p.

Sauer, V. B., Thomas, W. O., Stricker, V. A. and Wilson, K. V., 1983, Flood characteristics of urban watersheds in the United States: U.S. Geological Survey Water-Supply Paper 2207, 63 p.

Stricker, V. A., and Sauer, V. B., 1982, Techniques for estimating flood hydrographs for ungaged urban watersheds: U.S. Geological Survey Open-File Report 82-365, 24 p.

U.S. Department of Agriculture, Soil Conservation Service, 1972, Hydrographs: National Engineering Handbook, Section 4, p. 16.1-16.26.