ESTIMATING FLOOD HYDROGRAPHS AND VOLUMES FOR ALABAMA STREAMS

By D.A. Olin and J.B. Atkins

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CONVERSION FACTORS

For use of readers who prefer to use metric (International System) units, conversion factors for inch-pound units used in this report are listed below:

Multiply inch-pound unit	by	To obtain
inch (in.)	25.4	millimeter (mm)
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	259.0	hectare (ha)
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)

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Abstract

The hydraulic design of highway drainage structures involves an evaluation of the effect of the proposed highway structures on lives, property, and stream stability. Flood hydrographs and associated flood volumes are useful tools in evaluating these effects. For design purposes, the Alabama Highway Department needs information on flood hydrographs and volumes associated with flood peaks of specific recurrence intervals (design floods) at proposed or existing bridge crossings. This report will provide the engineer with a method to estimate flood hydrographs, volumes, and lagtimes for rural and urban streams in Alabama with drainage areas less than 500 square miles.

Computer programs and methods to estimate flood hydrographs and volumes for ungaged streams have been developed in Georgia (Inman, 1986) and used in similar studies in Tennessee (Robbins, 1986). These computer programs and methods were applied to streams in Alabama for this study. Data for 120 storms from 37 stations were selected to test the method. Results showed applicability of the method to Alabama.

The report gives detailed instructions on how to estimate flood hydrographs for ungaged rural or urban streams in Alabama with drainage areas less than 500 square miles, without significant in-channel storage or regulations. Equations are also given for estimating basin lagtime and flood volume for both rural and urban ungaged streams in Alabama.

INTRODUCTION

Flood hydrographs and volumes associated with peak discharges of specific recurrence intervals (design floods) are needed for the economic design of bridges for highways, railroads, and other structures at or near streams. Hydrographs may also be useful in estimating the length of time that roads and bridges would be inundated. For ungaged streams, this information was difficult to obtain; therefore, a method was needed to estimate a flood hydrograph and flood volume for these streams. The objective of this study was to define a simplified method of simulating flood hydrographs for peak discharges of specific recurrence interval at ungaged streams in Alabama.

An average statewide dimensionless hydrograph (definitions can be found in glossary) for Georgia computed by Inman (1986) was tested in Alabama. To use Georgia's dimensionless hydrograph in Alabama, estimates of both the peak discharge for a specific recurrence interval flood and the basin lagtime are needed. The peak discharge can be estimated by using flood magnitudes taken from Olin (1985). Regional equations for estimating basin lagtime are provided in this report. The scope of this study was statewide for both rural and urban streams. The simulating method for flood hydrographs was developed and tested for large and small rural and urban drainage basins in Georgia (Inman, 1986), and was successfully tested and used in Tennessee (Robbins, 1986). On this basis, the method was tested to see if applicable to streams in Alabama.

This report was prepared by the U.S. Geological Survey in cooperation with the Alabama Highway Department and is based on flood data collected through 1985 as a part of cooperative programs with the Alabama Highway Department and other State and Federal agencies.

This is the first report that gives a method for estimating flood hydrographs and volumes for selected recurrence interval floods for ungaged streams in Alabama. Previous reports have been published giving methods of estimating the magnitude and frequency of floods in Alabama (Olin, 1985) and flood depths (Olin, 1986).

A FLOOD HYDROGRAPH SIMULATION METHOD

A statewide average dimensionless hydrograph can be used to estimate a hydrograph at an ungaged site for a specific recurrence interval. A method used by Inman (1986) in Georgia has been adopted for application to Alabama streams. Stations were identified that had both rainfall and runoff data. Data for events with simple (single peak) hydrographs and short-duration storms were chosen and stored in the computer. For each storm at each station an observed unit hydrograph, lagtime, and volume were computed. Lagtime is needed to compute the time coordinates of the dimensionless hydrograph.

Inman (1986) computed unit hydrographs and lagtimes using the O'Donnell method (1960) for 355 floods from 80 gaging stations in Georgia; 19 of which were located in urban areas. Using three to five storms, the average observed lagtime was computed for each station. The observed unit hydrographs were also averaged for each station. The average unit hydrograph was then transformed into four unit hydrographs having generalized rainfall-excess durations of one-fourth, one-third, one-half, and three-fourths lagtime. The transformation process resulted in hydrographs having lower peak discharges and wider time bases as the duration of rainfall excess increased. These four unit hydrographs were then each made dimensionless by dividing the time scale by the average observed lagtime and dividing the discharge scale by the peak discharge of each hydrograph.

The average dimensionless hydrographs for all stations were combined to generate one typical (average) dimensionless hydrograph for each of the four generalized durations. From these four generalized duration dimensionless hydrographs, along with the average basin lagtime and the actual peak discharge for each observed storm, simulated hydrographs were computed. The widths of these simulated hydrographs were compared to the widths of the observed hydrographs at 50 and 75 percent of peak flow. Inman (1986) found that Georgia's dimensionless hydrograph based on a rainfall-excess duration of one-half lagtime compared best with the observed data. The standard error of estimates was ±31.8 percent at 50 percent of peak flow width, and ±35.9 percent at 75 percent of peak flow width. The standard error of estimate of the width comparisons is based on mean-square differences between observed and estimated widths.

The one-half lagtime duration dimensionless hydrograph was applied to other hydrographs from Georgia not used in its development for verification of the model. Hydrographs for 138 floods from 37 gaging stations in Georgia having drainage areas of 20 to 500 square miles were used for this test. A theoretical flood hydrograph was simulated using the average station lagtime and peak discharge for each flood and was compared to the observed hydrograph. The standard error of estimates were ±39.5 percent for 50 percent of peak flow width, and ±43.6 percent for 75 percent of peak flow width.

A second verification was conducted to determine the total or accumulative prediction error of the entire simulating procedure on the largest flood hydrographs at 31 gaging stations in Georgia with drainage areas from 20 to 500 square miles. Simulated hydrographs were computed by using the dimensionless hydrograph, an estimated lagtime from a regional lagtime equation, and a peak discharge estimated from regional flood-frequency equations. The standard errors of estimate were ±51.7 and ±57.1 percent at the 50 and 75 percent of peak flow widths, respectively. These errors are representative of what might be expected if the procedure is applied to ungaged basins. The time and discharge ratios of the Georgia dimensionless hydrograph are listed in table 1.

TESTING GEORGIA'S DIMENSIONLESS HYDROGRAPH ON ALABAMA STREAMS

Georgia's dimensionless hydrograph was tested in Alabama using flood hydrographs from 37 gaging stations. The stations were selected from the six rural hydrologic areas and urban streams (Olin, 1985) on the basis of having both discharge and associated rainfall. The location of the six rural hydrologic areas and location of the stations selected are shown in figure 1 and

Table 1.--Time and discharge ratios of the dimensionless hydrograph from Georgia (Inman, 1986)

Time ratio (t/LT)	Discharge ratio (Qt/Qp)	Time ratio (t/LT)	Discharge ratio (Qt/Qp)
0.25	0.12	1.35	0.62
.30	.16	1.40	.56
.35	.21	1.45	.51
.40	.26	1.50	.47
.45	.33	1.55	.43
.50	.40	1.60	.39
.55	.49	1.65	.36
.60	.58	1.70	.33
.65	.67	1.75	.30
.70	.76	1.80	.28
.75	.84	1.85	.26
.80	.90	1.90	.24
.85	.95	1.95	.22
.90	.98	2.00	.20
.95	1.00	2.05	.19
1.00	.99	2.10	.17
1.05	.96	2.15	.16
1.10	.92	2.20	.15
1.15	.86	2.25	.14
1.20	.80	2.30	.13
1.25	.74	2.35	.12
1.30	.68	2.40	.11

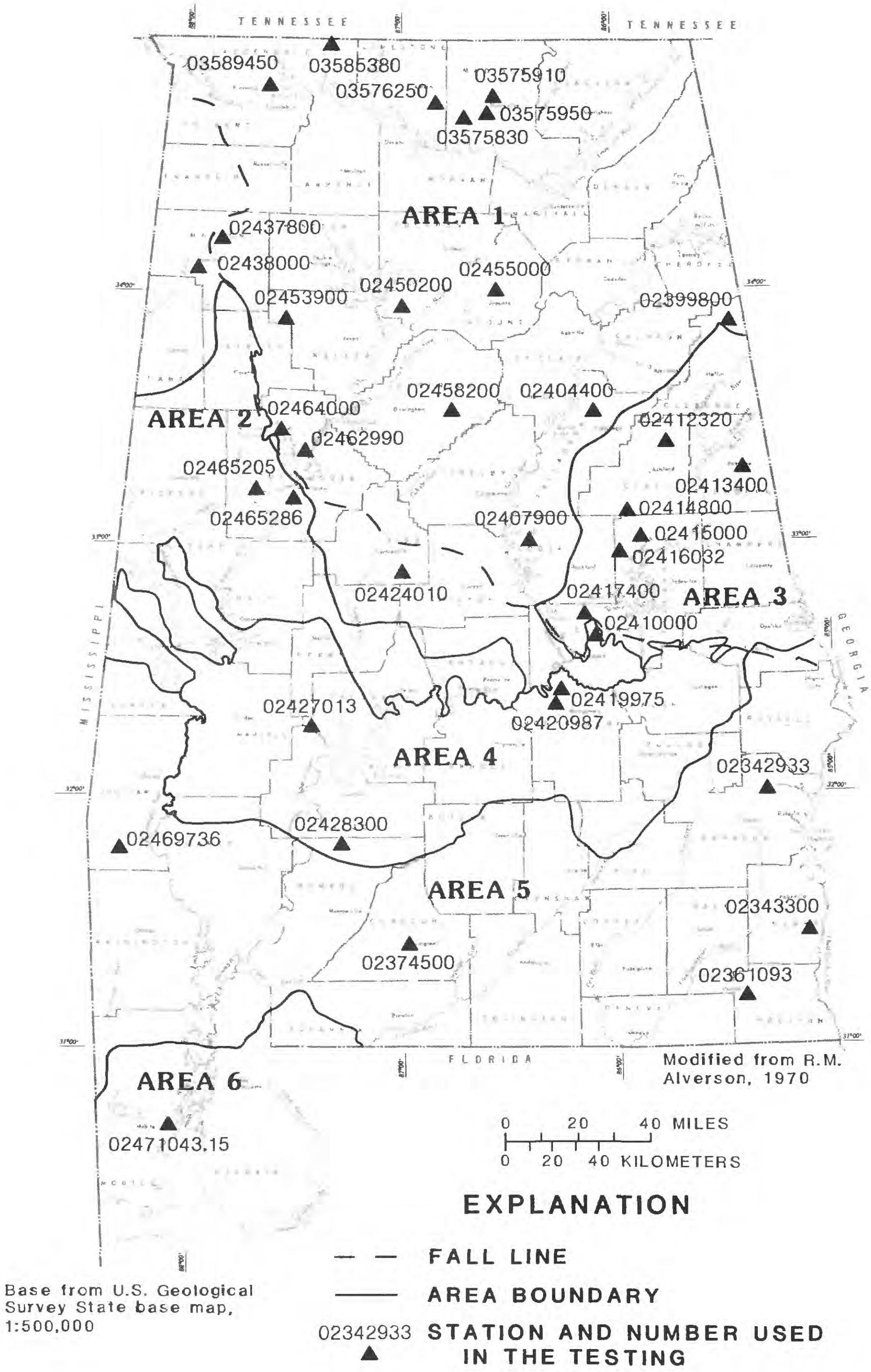

Figure 1.--Hydrologic areas and Fall Line, and location of gaging stations used for flood-hydrograph testing.

in table 2. Fifteen stations were selected from Hydrologic Areas 1 and 6, one from Area 2, five from Area 3, two from Area 4, four from Area 5, and ten urban stations statewide. A total of 76 events were selected from the 27 rural stations. The drainage areas of these basins range from 0.59 to 481 square miles. Forty-four urban events were chosen from ten stations which have drainage areas ranging from 0.16 to 41.8 square miles and impervious areas ranging from 20.0 to 42.9 percent.

Standard Error of Estimate

Georgia's dimensionless hydrograph was tested in Alabama using the same procedures that were used in the development of the model in Georgia (Inman, 1986). An average unit hydrograph and average basin lagtime were determined for each of the 37 stations using three to five observed storms. This was accomplished using the computer programs (S.E. Ryan, U.S. Geological Survey, written commun., 1986) used by Inman (1986). Average dimensionless unit hydrographs with rainfall-excess durations of one-forth, one-third, one-half, and three-quarters lagtime were computed for all stations and compared to the observed hydrographs. This comparison, based on the delta percent difference in the widths (in hours) of the hydrographs at 50 and 75 percent of the peak flow, showed that the one-half lagtime duration gave the best results. Comparison of the average dimensionless unit hydrographs for a rainfall-excess duration of one-half lagtime to the Georgia dimensionless hydrograph indicated that the Georgia hydrograph would be appropriate for use in Alabama. Using the average basin lagtime and observed peak discharges for 76 hydrographs from 27 rural stations, hydrographs were simulated using Georgia's dimensionless hydrograph and compared to the observed hydrographs. The standard error, based on the one-half lagtime duration dimensionless hydrograph, was ±26.7 and ±26.2 percent at 50 and 75 percent of peak flow widths, respectively. Similarly, 44 urban hydrographs were compared using Georgia's dimensionless hydrograph and the standard error of estimate at the 50 and 75 percent peak flow widths was ±18.4 and ±20.2 percent, respectively. The final check used both the rural and the urban stations combined, and the standard error of estimate at 50 and 75 percent peak flow widths was ±23.9 and ±24.1 percent, respectively.

Verification

The largest discharge events from selected stations were used for verification of the method. Ten rural stations (five north and five south of the Fall Line) and nine urban stations statewide were chosen. In this test, the observed lagtime and peak discharge were not used. Lagtime was instead estimated from regional equations (discussed later) and estimated peak discharge was computed from regional equations (Olin, 1985 and given in table 3) for the observed recurrence interval flood. The standard error of prediction for 50 and 75 percent flow widths was ±31.7 and ±34.0 percent, respectively. These standard errors are less than the ones Inman reported for Georgia in 1986. One possible reason for this is the smaller data base used in Alabama.

Table 2.--Rural and urban stations selected to test Georgia's dimensionless hydrograph and selected basin characteristics

Rural stations

Station No.	Name	Drainage area, A (square mile)	Slope S (feet per mile)	Hydrologic Area	Average basin lagtime LT (hour)
02342933	South Fork Cowikee Creek near Batesville	112	13.9	5	15.5
02343300	Abbie Creek near Haleburg	146	8.20	5	16.4
02374500	Murder Creek near Evergreen	176	9.00	5	40.5
02399800	Little Terrapin Creek near Borden Springs	15.9	32.4	1	9.77
02407900	Paint Creek near Marble Valley	12.7	33.0	1	8.75
02404400	Choccolocco Creek at Jackson Shoal near Lincoln	481	6.70	1	35.4
02410000	Paterson Creek near Central	4.91	69.0	1	3.00
02412320	Elder Creek near Dempsey	1.79	99.7	3	3.04
02413400	Wedowee Creek above Wedowee	6.87	41.8	3	3.76
02414800	Harbuck Creek near Hackneyville	7.97	67.2	3	3.88
02415000	Hillabee Creek near Hackneyville	190	22.2	3	13.2
02417400	Stearns Creek near Seman	1.27	74.8	3	1.74
02424010	Sandy Creek near Centreville	.59	72.9	1	2.02
02427013	Caine Creek near Safford	2.69	83.3	4	2.07
02428300	Tallatchee Creek near Vredenburgh	13.2	24.5	4	9.00
02437800	Barn Creek near Hackleburg	13.1	35.2	1	9.76
02438000	Buttahatchee River below Hamilton	277	6.20	1	23.5
02450200	Dorsey Creek near Arkadelphia	13.0	26.8	1	9.47
02453900	Cheatham Creek near Carbon Hill	4.77	49.0	1	9.20
02455000	Locust Fork near Cleveland	303	6.40	1	23.1
02462990	Yellow Creek at Northport	8.38	28.0	1	8.75
02464000	North River near Samantha	223	5.20	1	30.6
02465205	Jay Creek near Coker	3.65	47.5	2	4.38
02469735	Souwilpa Creek at Bolinger	7.25	25.8	5	5.58
03575830	Indian Creek near Madison	49.0	15.2	1	10.8
03576250	Limestone Creek near Athens	119	10.6	1	19.2
03585380	West Fork Anderson Creek near Lexington	5.92	23.0	1	4.41

Urban stations

Station No.	Name	Drainage area, A (square mile)	Slope S (feet per mile)	Impervious area, IA (percent)	Average basin lagtime LT (hour)
02361093	Tributary to Beaver Creek Ross Clark Circle in Dothan	1.81	33.8	30.5	1.39
02416032	Sugar Creek at Alexander City	1.67	38.1	20.2	1.68
02419975	Three Mile Branch at Biltmore Avenue in Montgomery	7.26	16.0	25.0	2.46
02420987	Hannon Slough at Montgomery	1.32	39.3	42.9	1.16
02458200	Village Creek (Apalachee Street) in Birmingham	15.6	19.9	33.3	1.91
02465286	Cribbs Mill Creek (Second Avenue East) in Tuscaloosa	2.75	60.2	28.9	1.59
02471043.15[a]	Woodcock Creek (Airport Blvd) in Mobile	1.85	10.6	25.0	1.64
03575910	Pinehaven Ditch at Huntsville	.16	231.3	20.0	.335
03575950	Huntsville Spring Branch (Johnson Road) in Huntsville	41.8	21.3	21.4	3.26
03589450	Sweetwater Creek at Florence	4.92	55.8	24.1	1.49

[a] Station number used only for this report.

Table 3.--Summary of peak discharge regression equations for rural and urban streams in Alabama

Rural Equations

[Regression equations for indicated hydrologic areas, where A = drainage area in square miles, ST = storage in percent of basin] (Olin, 1985)

Recurrence Interval (Years)	Hydrologic Areas 1 & 6	2	3	4	5
2	$182A^{0.706}$	$149A^{0.689}(ST+1.0)^{-.219}$	$270A^{0.569}$	$292A^{0.631}$	$226A^{0.567}$
5	$291A^{0.711}$	$310A^{0.642}(ST+1.0)^{-.172}$	$419A^{0.566}$	$480A^{0.647}$	$376A^{0.577}$
10	$372A^{0.714}$	$459A^{0.616}(ST+1.0)^{-.142}$	$524A^{0.564}$	$630A^{0.653}$	$495A^{0.582}$
25	$483A^{0.717}$	$696A^{0.590}(ST+1.0)^{-.109}$	$675A^{0.559}$	$845A^{0.660}$	$668A^{0.587}$
50	$571A^{0.720}$	$904A^{0.574}(ST+1.0)^{-.084}$	$807A^{0.554}$	$1024A^{0.665}$	$813A^{0.591}$
100	$664A^{0.722}$	$1144A^{0.560}(ST+1.0)^{-.060}$	$937A^{0.550}$	$1215A^{0.669}$	$972A^{0.595}$

All regression coefficients are statistically significant at the 5 percent level.

Flood magnitude and frequency for urban streams

The following equations can be used to estimate flood magnitudes for streams draining urban areas with more than 5 percent impervious cover (Olin, 1985).

Equation	Standard error of regression (percent)
$Q(u)_2 = 150\ A^{0.70}\ IA^{0.36}$	26
$Q(u)_5 = 210\ A^{0.70}\ IA^{0.39}$	24
$Q(u)_{10} = 266\ A^{0.69}\ IA^{0.39}$	24
$Q(u)_{25} = 337\ A^{0.69}\ IA^{0.39}$	24
$Q(u)_{50} = 396\ A^{0.69}\ IA^{0.38}$	25
$Q(u)_{100} = 444\ A^{0.69}\ IA^{0.39}$	25

where Q(u) = the estimated urban discharge, in cubic feet per second, for the indicated recurrence interval,

A = the contributing drainage area, in square miles, and

IA = impervious area, in percent.

All regression coefficients are statistically significant at the 5 percent level.

Limitations of Equations

The peak discharge equations should be used only for streams that have drainage areas, and, in urban areas, percent impervious area within the ranges given below.

Rural Stations

Hydrologic area	Variable	Minimum	Maximum	Units
1, 2, 5	A	1.0	1,500	square miles
3	A	1.0	800	square miles
4, 6	A	1.0	500	square miles

Urban Stations

Variable	Minimum	Maximum	Units
A	0.16	83.5	square miles
IA	5.0	42.9	percent

Note: Rural equations should be used in hydrologic area 4. Both rural and urban equations should not be used for streams where temporary in-channel or overbank storage significantly affects the magnitude of peak flow.

Bias

The first test for bias was performed using the simulated versus observed hydrograph widths. This test used residuals in percent of mean width (in hours) at the 50 and 75 percent of peak flow for the 19 stations used in the previous verification test. The mean errors are -1.04 percent at the 50 percent peak flow width and +2.80 percent at the 75 percent peak flow width.

A students t-test (Dixon and Massey, 1957) for a significance level of 1 percent was made on this mean of the differences (in hours) to see if the mean was biased.

The following steps were used to compute the t-values for the hydrograph width analysis:

a. The difference between the observed value and the computed value at both 50 and 75 percent of the peak discharges was computed retaining the positive and negative signs.

b. The mean of all the differences was computed.

c. The standard deviation of all the differences (2.462 [in hours] for 50 percent and 2.964 [in hours] for 75 percent of the peak) was computed.

d. The mean was multiplied by the number of observations (40) raised to the one-half power and then divided by the standard deviation to compute t.

The t-values (-1.448 for 50 percent and +0.002 for 75 percent) were compared to the standard table and had to fall between $\pm$2.423 to be unbiased. The test determined that these errors were not statistically different from zero at the 1 percent level of significance and, therefore, the simulated hydrograph widths are not biased.

A test for geographical bias was made by plotting the residual differences in the simulated and observed hydrograph widths at 50 and 75 percent of the peak flow on a map. Although the residuals varied considerably between some stations, no specific geographic trends could be detected. These two tests indicate no bias in the use of Inman's (1986) method to simulate hydrographs in Alabama.

REGIONALIZATION OF BASIN LAGTIME AND FLOOD VOLUME

Lagtime Regression Analysis

Seventy-six rural and twenty-one urban stations were available for use in the lagtime regression analyses. The rural lagtimes were from the 27 stations used in the testing of the dimensionless hydrograph and 29 other stations not used in the testing in Alabama. Also, nine stations from Georgia, seven from Mississippi, and four from Tennessee were available. The 10 urban stations used in the testing of the dimensionless hydrograph along with 11 other urban stations, where lagtime was computed in a rainfall-runoff modeling study by Olin and Bingham (1982), were available for use in the urban regression. Basin characteristics used as independent variables in the regressions were: Lagtime (LT), Drainage area (A), Main channel slope (S), Main channel length (L), $(L/S^{0.5})$, and Impervious area (IA). The stations and their basin characteristics are listed in table 4.

In the first analysis of rural basins only LT, A, and S were used, and after examining the residuals, a slight geographical bias was detected north and south of the Fall Line (fig. 1). It was determined that 63 percent of the stations north of the Fall Line overestimated lagtimes, while 64 percent of the stations south of the Fall Line underestimated lagtimes.

Regression analyses were then performed for the rural stations north of the Fall Line and for rural stations south of the Fall Line. Drainage area and main channel slope were the only independent variables significant at the 5 percent level. The final two regional equations (table 5), one for the rural basins north of the Fall Line and one for rural basins south of the Fall Line, used only 71 stations; 42 north and 29 south of the Fall Line. The 42 stations north of the Fall Line had a standard error of estimate of ± 31.6 percent and R-square of 0.91. The 29 stations south of the Fall Line had a standard error of estimate of ± 31.2 percent and R-square of 0.90.

Lagtimes for 21 stations statewide were used for a regression for urban basins. Drainage area (A), main channel slope (S), and impervious area (IA) were the only independent variables significant at the 5 percent level. The urban equation is shown in table 5.

Verification of Lagtime Equations

The verification of the lagtime equations used the split-sample procedure. The stations in the three data sets (north of the Fall Line, south of the Fall Line, and urban) were arranged in descending order of drainage area magnitude. The data set was then divided into odd and even numbered stations. Separate equations were regressed for both the even and odd stations. The equation developed from each subset of stations was used to predict the lagtime for the other subset. Lagtimes predicted by the odd and even equations were then compared to the observed lagtimes to estimate the magnitude of the average prediction error, and to determine whether the same variables were significant. Both the odd and even equations developed were not significantly different than the regression equation based on all of the stations. Table 6 shows these results.

Bias

Two tests for bias were performed; one for parameter bias and another for geographical bias. The parameter-bias tests were made by plotting the residuals (difference between the observed and predicted lagtimes) against each independent variable for each station. Inspection of the plots showed that the equations were free of parameter bias throughout the range of all independent variables. These plots also proved the linearity assumptions of the equations.

Table 4.--Stations and drainage basin characteristics used in lagtime regression analyses

Station No.	Station name	Drainage area, DA (square mile)	Channel length, L (mile)	Channel slope, S (feet per mile)	Average basin lagtime, LT (hour)	Estimated lagtime from equations, LT (hour)
	Stations north of the Fall Line					
02399800	Little Terrapin Creek near Borden Springs	15.9	9.53	32.4	9.77	7.19
02404400	Choccolocco Creek at Jackson Shoal near Lincoln	481	49.3	6.70	35.4	39.1
02400690	Jacks Creek near Fort Payne	6.76	5.91	33.0	8.26	4.84
02401500	Big Canoe Creek near Gadsden	253	38.0	7.50	35.0	28.9
02404000	Choccolocco Creek near Jenifer	277	37.5	6.80	31.0	30.3
02405500	Kelly Creek near Vincent	193	25.5	8.90	27.0	25.1
02406000	Talladega Creek near Talladega	101	23.1	22.2	13.0	17.3
02407900	Paint Creek near Marble Valley	12.7	7.35	33.0	8.75	6.47
02408340	Little Hatchet Creek near Goodwater	8.09	5.68	32.9	8.00	5.26
02408600	Hatchet Creek near Rockford	233	37.0	13.9	19.0	26.4
02410000	Paterson Creek near Central	4.91	3.60	69.0	3.00	3.94
02422500	Mulberry Creek at Jones	203	36.0	13.7	24.0	24.9
02423800	Little Cahaba River near Bierfield	147	19.8	10.0	21.0	22.0
02424010	Sandy Creek near Centreville	.59	1.40	72.9	2.02	1.48
02438000	Buttahatchee River below Hamilton	277	44.8	6.20	23.5	30.6
02450200	Dorsey Creek near Arkadelphia	13.0	6.84	26.8	9.47	6.65
02451550	Jaybird Creek near West Point	1.42	2.27	52.4	1.37	2.28
02451750	Vest Creek near Baldwin	1.64	2.05	109.7	1.88	2.29
02454200	Wolf Creek near Oakman	85.0	26.7	8.50	33.0	17.3
02455000	Locust Fork near Cleveland	303	58.4	6.40	23.1	31.8
02456000	Turkey Creek at Morris	80.9	19.3	24.7	10.3	15.5
02462600	Blue Creek near Oakman	5.32	4.30	65.0	4.90	4.11
02462800	Davis Creek below Abernant	45.3	9.40	14.8	8.00	12.4
02464000	North River near Samantha	223	34.5	5.20	30.6	28.0
03574405	Little Dry Creek near Garth	3.91	3.94	286.2	4.12	3.17
03575830	Indian Creek near Madison	49.0	16.7	15.2	10.8	12.8
03576250	Limestone Creek near Athens	119	26.5	10.6	19.2	19.8
03585380	West Fk Anderson Creek near Lexington	5.92	5.75	23.0	4.41	4.69
02412320	Elder Creek near Dempsey	1.79	2.88	99.7	3.04	2.41
02413400	Wedowee Creek above Wedowee	6.87	5.90	41.8	3.76	4.79
02414800	Harbuck Creek near Hackneyville	7.97	4.50	67.2	3.88	4.94
02415000	Hillabee Creek near Hackneyville	190	24.0	22.2	13.2	23.2
02417400	Stearns Creek near Seman	1.27	2.22	74.8	1.74	2.10
	Stations in Georgia					
02207500	Yellow River near Covington	378	51.9	6.68	40.5	35.0
02217500	Middle Oconee River near Athens	398	42.9	6.32	37.5	36.0
02337500	Snake Creek near Whitesburg	37.0	13.2	20.1	15.6	23.5
02398000	Chattooga River at Summerville	192	33.3	6.60	32.6	25.7
03558000	Toccoa River near Dial	177	29.6	30.4	39.7	21.9
	Stations in Tennessee					
03420400	Mud Creek near Summitville	7.30	4.05	30.6	5.15	5.05
03519640	Baker Creek near Greenback	16.0	8.79	17.4	6.71	7.58
03535160	Beaver Creek near Halls Crossroads	14.1	6.78	15.8	5.03	7.20
03597400	Wartrace Creek near Bell Buckle	9.59	6.08	31.7	4.44	5.71

Table 4.--Stations and drainage basin characteristics used in lagtime regression analyses--continued

Station No.	Station name	Drainage area, DA (square mile)	Channel length, L (mile)	Channel slope, S (feet per mile)	Average basin lagtime, LT (hour)	Estimated lagtime from equations, LT (hour)
	Stations south of the Fall Line					
02342200	Phelps Creek near Marble Valley	6.76	5.20	42.3	6.00	6.22
02343700	Stevenson Creek near Headland	14.0	7.40	33.3	5.35	9.39
02362745	Hurricane Creek near Clayton	4.40	3.20	34.2	5.74	5.24
02363055	Moores Branch near Victoria	2.17	2.39	59.2	4.42	3.30
02365310	Grants Branch tributary near Fadette	1.44	1.53	34.8	3.05	2.99
02371200	Indian Creek near Troy	8.87	4.52	27.7	10.0	7.76
02372510	Catoe Creek near Andalusia	2.46	2.46	31.5	7.22	3.98
02374500	Murder Creek near Evergreen	176	23.8	9.00	40.5	43.3
02421300	Ivy Creek at Mulberry	10.7	8.26	27.4	7.00	8.54
02427013	Caine Creek near Safford	2.69	2.60	83.3	2.13	3.43
02427300	Prairie Creek near Oak Hill	10.3	3.70	30.0	7.00	8.23
02428300	Tallatchee Creek near Vredenburgh	13.2	5.50	24.5	9.00	9.70
02437800	Barn Creek near Hackleburg	13.1	6.25	35.2	9.76	8.98
02437900	Woods Creek near Hamilton	14.3	7.58	31.5	8.75	9.60
02442000	Luxapallila Creek near Fayette	130	25.9	10.3	26.0	36.2
02465205	Jay Creek near Coker	3.65	2.94	47.5	4.38	4.47
02469735	Souwilpa Creek at Bolinger	7.25	4.40	25.8	5.58	7.11
02471026	Watson Creek near Stockon	2.25	2.13	43.8	2.23	3.56
02479583	Flat Creek near Wilmer	6.55	4.17	25.5	8.60	6.78
	Stations in Georgia					
02343200	Pataula Creek near Lumpkin	70.0	14.0	22.2	31.3	22.8
02349000	Whitewater Creek near Butler	93.4	15.7	17.8	46.6	27.5
02357000	Spring Creek near Iron City	485	42.3	4.20	93.9	83.6
	Stations in Mississippi					
02429980	Pollard Mill Branch near Paden	2.05	2.75	82.9	4.73	2.99
02447280	Lawson Branch near Betheden	1.11	2.00	32.0	2.17	2.67
02469672	Little Okatuppa Creek near Quitman	4.35	3.50	41.2	4.54	5.02
02473850	Tallahoma Creek tributary at Lake Como	3.21	3.40	31.5	4.10	4.55
02479094	Blown Pine Creek near Hattiesburg	1.92	3.30	31.9	3.63	3.51
02481505	Mill Creek tributary near Lizana	2.29	2.20	46.1	5.62	3.56
02488540	New Hebron Gulley at New Hebron	2.50	2.15	44.7	3.15	3.74

Table 4.--Stations and drainage basin characteristics used in lagtime regression analyses--continued

Station No.	Station name	Drainage area, DA (square mile)	Percentage of impervious area, IA (percent)	Channel length, L (mile)	Channel slope, S (feet per mile)	Average basin lagtime LT (hour)	Estimated lagtime from equations, LT (hour)
Urban areas							
02361093	Tributary to Beaver Creek at Ross Clark Circle in Dothan	1.81	30.5	2.23	31.8	1.39	1.23
02416032	Sugar Creek at Alexander City	1.67	20.2	1.76	38.1	1.68	1.22
02419975	Three Mile Branch at Biltmore Avenue in Montgomery	7.26	25.0	3.09	16.0	2.46	2.15
02420987	Hannon Slough at Montgomery	1.32	42.9	1.31	39.3	1.16	1.04
02423580	Shades Creek at Homewood	20.7	16.3	1.21	12.4	3.59	3.22
02457000	Fivemile Creek in Ketona	23.9	17.0	10.1	29.0	3.95	2.86
02458200	Village Creek (Apalachee Street) in Birmingham	15.6	33.3	7.88	19.9	1.91	2.50
02458300	Village Creek (24th St) in Birmingham	26.0	25.0	10.8	17.8	2.45	3.07
02458450	Village Creek (Avenue W) in Ensley	33.5	25.0	14.3	13.6	4.06	3.47
02461200	Valley Creek at Cleburne Ave near Powderly	20.1	36.8	6.63	15.2	2.51	2.80
02465286	Cribbs Mill Creek at Second Avenue East in Tuscaloosa	2.75	28.9	3.76	60.2	1.59	1.24
02471043.15[a]	Woodcock Creek at Airport Blvd in Mobile	1.85	25.0	1.35	10.6	1.64	1.55
02471065	Montlimar Creek in Mobile	7.28	30.4	4.72	31.8	1.64	1.85
03575686	Aldridge Creek (Dunsmore Street) in Huntsville	1.15	10.2	2.08	295.6	.924	.808
03575696	Aldridge Creek near Lily Flagg	13.9	8.40	7.67	26.1	2.52	2.69
03575880	Five Points Ditch in Huntsville	.62	20.0	1.42	27.8	.733	.963
03575890	Pinhook Creek in Huntsville	22.5	12.0	1.24	27.8	2.40	2.94
03575910	Pinehaven Ditch in Huntsville	.16	20.0	1.44	231.3	.335	.438
03575930	Broglan Branch at Homes Ave in Huntsville	8.87	19.3	5.88	35.6	1.75	2.03
03575950	Huntsville Spring Branch (Johnson Road) in Huntsville	41.8	21.4	9.70	21.3	3.26	3.48
03589450	Sweetwater Creek at Florence	4.92	24.1	3.15	55.8	1.49	1.53

[a] Station number used only for this report.

Table 5.--Lagtime Equations

Area	Equation	Standard Error of estimate (percent)	Coefficient of determination, R^2
North of the Fall Line (rural)	$LT = 2.66(A)^{0.46}(S)^{-0.08}$	31.6	0.91
South of the Line (rural)	$LT = 5.06(A)^{0.50}(S)^{-0.20}$	31.2	.90
Statewide (urban)	$LT = 2.85(A)^{0.295}(S)^{-0.183}(IA)^{-0.122}$	21.0	.89

Limitations

The lagtime equations should be used only for streams that have drainage areas, main channel slopes, and, in urban areas, percent impervious area within the ranges given below.

Rural (North of the Fall Line)

Variable	Minimum	Maximum	Units
A	0.59	481.0	square miles
S	5.20	296.2	feet per mile

Rural (South of the Fall Line)

Variable	Minimum	Maximum	Units
A	1.11	485.0	square miles
S	4.20	83.3	feet per mile

Urban (Statewide)

Variable	Minimum	Maximum	Units
A	0.16	41.8	square miles
S	10.6	295.6	feet per mile
IA	8.40	42.9	percent

Table 6.--Lagtime equations split-sample test results

Sample designation	Number of stations	Equation	Standard error of estimate (percent)	Average standard error of prediction (percent)	Coefficient of determination, R^2
North of Fall Line					
Odd	21	$LT = 2.92(A)^{0.44}(S)^{-0.11}$			
			± 29.0	---	0.92
Even	21	---			
			---	± 39.9	.91
Even	21	$LT = 2.30(A)^{0.48}(S)^{-0.035}$			
			± 36.2	---	.90
Odd	21	---			
			---	± 31.5	.88
South of Fall Line					
Odd	15	$LT = 4.31(A)^{0.50}(S)^{-0.16}$			
			± 29.3	---	.93
Even	14	---			
			---	± 37.9	.90
Even	14	$LT = 6.57(A)^{0.51}(S)^{-0.27}$			
			± 36.8	---	.87
Odd	15	---			
			---	± 31.6	.86
Urban					
Odd	11	$LT = 4.81(A)^{0.25}(S)^{-0.24}(IA)^{-0.20}$			
			± 19.3		.94
Even	10	---			
			---	± 31.7	.93
Even	10	$LT = 1.46(A)^{0.37}(S)^{-0.023}(IA)^{-0.092}$			
			± 23.6		.89
Odd	11	---			
			---	± 33.6	.89

Geographical bias was checked by plotting the residuals for each equation on a State map at the location of the stream gaging station. Although the residuals varied considerably between some stations, no specific geographic trends could be detected.

Sensitivity

The basin characteristics needed to use the lagtime equations must be measured from topographic maps or aerial photos. To determine how sensitive the equations are to errors made in the estimates of these characteristics, an analysis was made. For the rural equations the main channel slope was held constant and the drainage area was adjusted by ±10, ±25, and ±50 percent, and the percent change in lagtime was determined. Then the drainage area was held constant and the main channel slope was changed by the same percentages. For the urban equation, the same analysis was made, but also included the percent of the basin covered with impervious area. The results of these analyses are shown in figure 2.

Flood Volume

Estimates of flood volumes may be useful in the design of a structure that will store floodwater for flood protection. Therefore, a theoretical (direct) equation that estimates flood volumes from basin and flood characteristics has been developed by summing the ordinates of the average dimensionless hydrograph (table 2). This equation was statistically tested by comparing the observed volumes (from 76 storms at 21 stations) with the volumes computed from the theoretical equation. These stations had drainage areas that ranged from 0.16 to 481 square miles; peak discharges ranged from 12.4 to 30,100 ft^3/s (cubic feet per second); and lagtimes ranged from 0.335 to 44.3 hours. The user is cautioned not to use the equation for streams where the basin and flood characteristics are not within these ranges.

The volume equation and the standard error of estimate (taken from the comparison of the equation estimates to the observed volumes) are shown below.

$$V = \frac{0.00169\, Q_p LT}{A}$$

$$SEr = \pm 23.2 \text{ percent}$$

where A is the drainage area, in square miles
Qp is the peak discharge, in cubic feet per second
LT is the basin lagtime, in hours
V is the estimated volume, in inches
and SEr is the standard error of estimate.

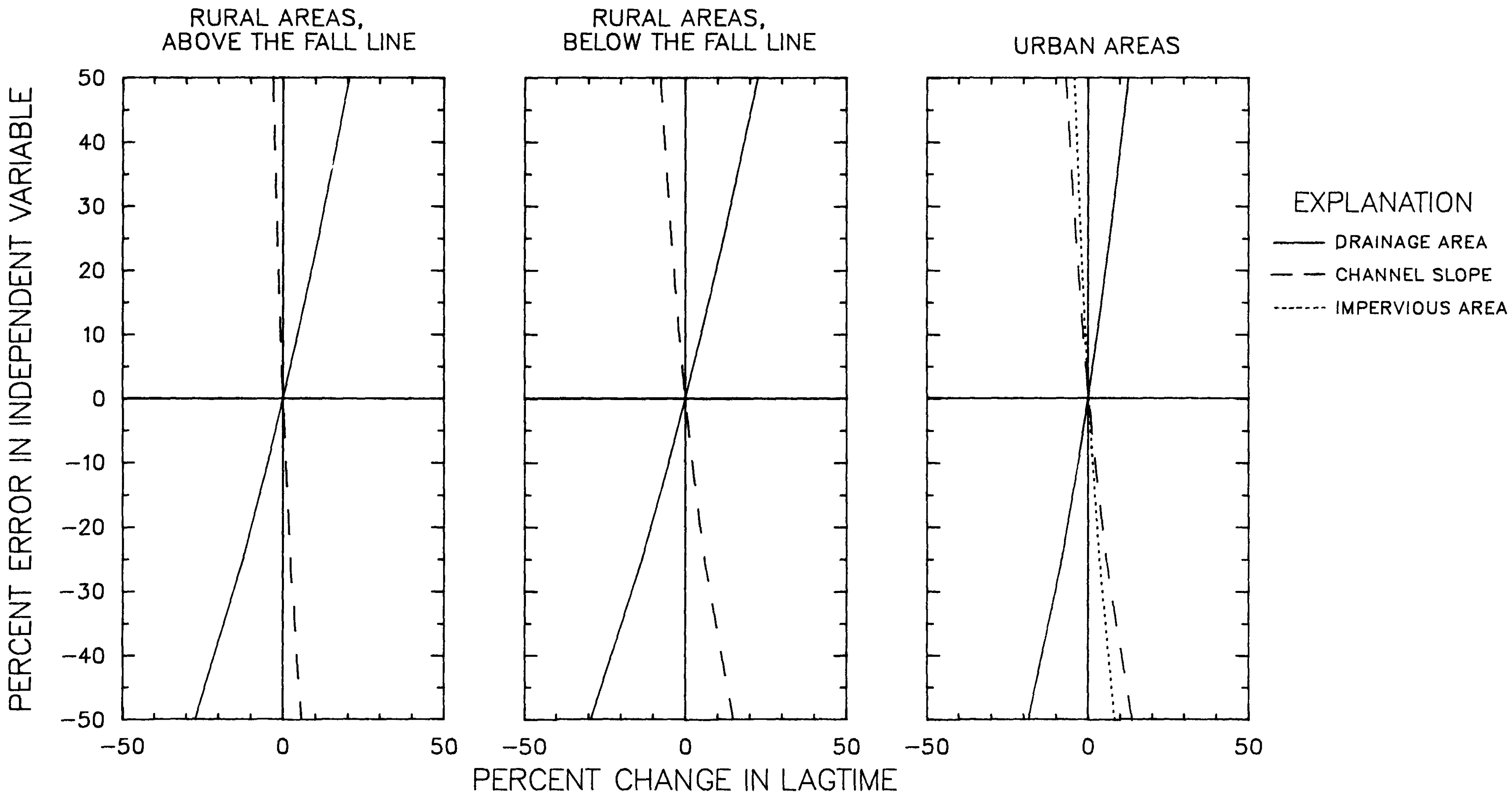

Figure 2.--Change (percent) in lagtime resulting from errors in measuring drainage area, main channel slope, and impervious area.

HYDROGRAPH-WIDTH RELATION

For some instances, it is only necessary to know the amount of time a certain flow will be exceeded and the complete hydrograph is not needed. For these situations, a hydrograph-width relation was defined by Inman (1986), and is shown in table 7 and figure 3. The hydrograph width, W, can be estimated for a specified discharge Q, by first computing the ratio Q/Qp (where Qp is the peak discharge computed from the flood-magnitude equations in table 3) and then multiplying the corresponding W/LT ratio (from table 7 and figure 3) by the estimated lagtime, LT. This is the period of time a specified discharge, Q, will be exceeded. Hydrograph width is denoted as W, in hours, and the width ratio, W/LT, was determined by subtracting the value of t/LT on the rising limb of the dimensionless hydrograph from the value of t/LT on the falling limb of the hydrograph at the same discharge ratio, Q/Qp.

Table 7.--Relation of discharge ratios to hydrograph width ratios for Georgia's dimensionless hydrograph

[Modified from Inman (1986)]

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

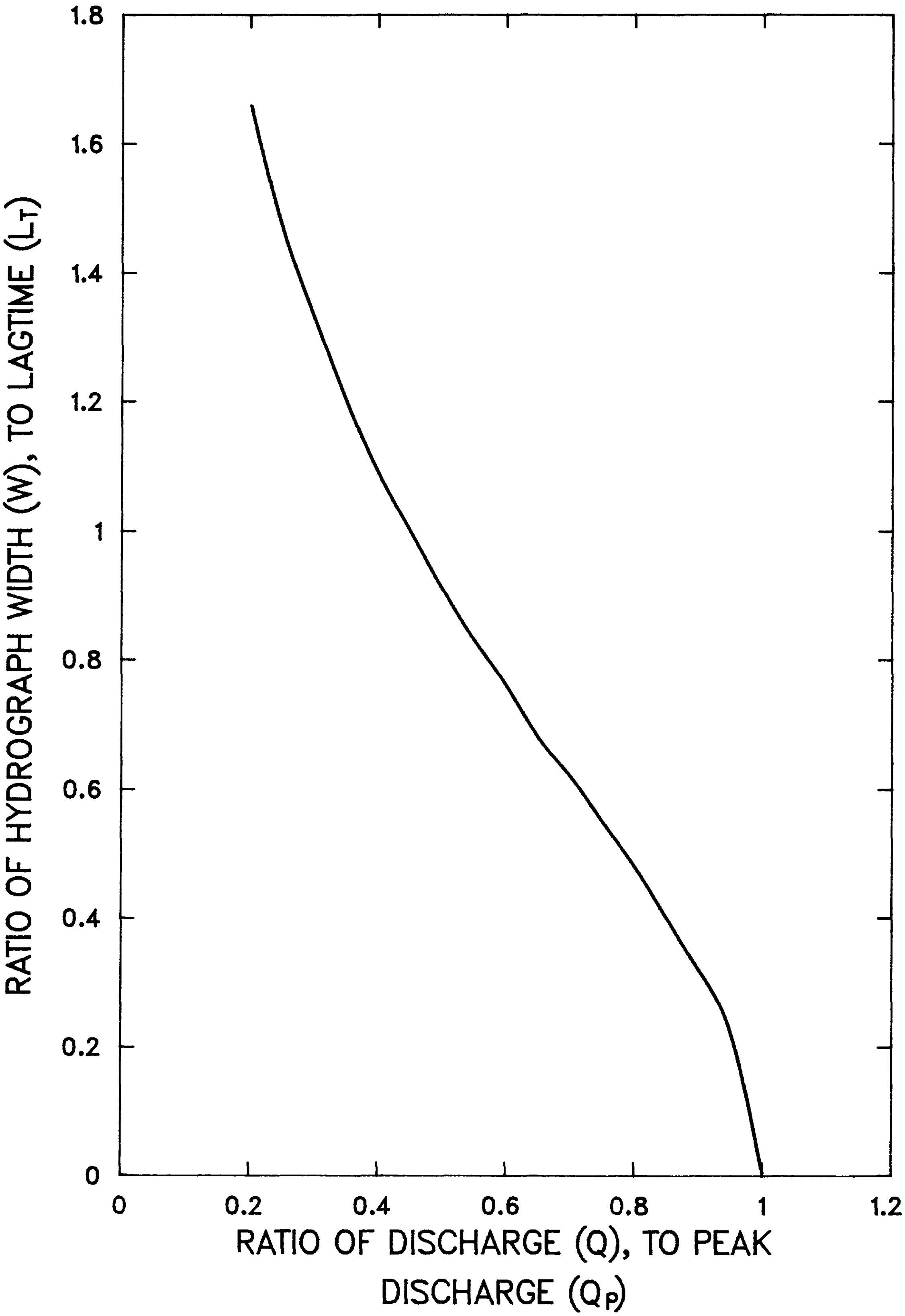

Figure 3.--Hydrograph-width relation for dimensionless hydrograph (modified from Inman, 1986)

APPLICATION OF TECHNIQUE

The procedure for estimating a flood hydrograph and volume, and hydrograph width is outlined in the steps given below.

1. Determine the drainage area and the main channel slope from the latest topographic map. Impervious area is required for urban watersheds, and can be computed from topographic maps or aerial photos.

2. Locate the site in figure 1 to determine which hydrologic area it is in and, for rural stations, if it is north or south of the Fall Line.

3. Estimate the peak discharge from the flood magnitude equation for the particular hydrologic area in the flood frequency report (table 3 or Olin, 1985).

4. Compute the basin lagtime from the appropriate equation in table 5.

5. Compute the coordinates of the flood hydrograph by multiplying the value of lagtime by the time ratios and the value of peak discharge by the discharge ratios (table 1).

6. Compute the volume for the selected flood using flood volume equation.

7. Compute the hydrograph width for discharge ratio using either table 7 or figure 3.

Example Problem

The following illustrates the procedure for computing a flood hydrograph and volume for a 50-year flood at a bridge crossing on an ungaged rural stream.

1. A site on a county road bridge crossing of a creek in Winston County is located on a topographic map. The drainage area is 26.0 square miles and the main channel slope is 35.0 feet per mile.

2. From figure 1 the site is located north of the Fall Line and in Hydrologic Area 1.

3. The peak discharge of 5,960 ft^3/s is computed for the 50-year recurrence interval flood from the equation listed in table 3 for Hydrologic Areas 1 and 6 and a recurrence interval of 50 years.

Table 8.-- Simulated coordinates of the flood hydrograph for sample creek in example problem

Time ratio (t/LT)	x LT	= time hr.	Q/Qp from table 1	x Qp	= Discharge (ft^3/s)
0.25	8.96	2.24	0.12	5960	715
.30	8.96	2.69	.16	5960	954
.35	8.96	3.14	.21	5960	1250
.40	8.96	3.58	.26	5960	1550
.45	8.96	4.03	.33	5960	1970
.50	8.96	4.48	.40	5960	2380
.55	8.96	4.93	.49	5960	2920
.60	8.96	5.38	.58	5960	3460
.65	8.96	5.82	.67	5960	3990
.70	8.96	6.27	.76	5960	4530
.75	8.96	6.72	.84	5960	5010
.80	8.96	7.17	.90	5960	5360
.85	8.96	7.62	.95	5960	5660
.90	8.96	8.06	.98	5960	5840
.95	8.96	8.51	1.00	5960	5960
1.00	8.96	8.96	.99	5960	5900
1.05	8.96	9.41	.96	5960	5720
1.10	8.96	9.86	.92	5960	5480
1.15	8.96	10.3	.86	5960	5130
1.20	8.96	10.8	.80	5960	4770
1.25	8.96	11.2	.74	5960	4410
1.30	8.96	11.6	.68	5960	4050
1.35	8.96	12.1	.62	5960	3700
1.40	8.96	12.5	.56	5960	3340
1.45	8.96	13.0	.51	5960	3040
1.50	8.96	13.4	.47	5960	2800
1.55	8.96	13.9	.43	5960	2560
1.60	8.96	14.3	.39	5960	2320
1.65	8.96	14.8	.36	5960	2150
1.70	8.96	15.2	.33	5960	1970
1.75	8.96	15.7	.30	5960	1790
1.80	8.96	16.1	.28	5960	1670
1.85	8.96	16.6	.26	5960	1550
1.90	8.96	17.0	.24	5960	1430
1.95	8.96	17.5	.22	5960	1310
2.00	8.96	17.9	.20	5960	1190
2.05	8.96	18.4	.19	5960	1130
2.10	8.96	18.8	.17	5960	1010
2.15	8.96	19.3	.16	5960	954
2.20	8.96	19.7	.15	5960	894
2.25	8.96	20.2	.14	5960	834
2.30	8.96	20.6	.13	5960	775
2.35	8.96	21.1	.12	5960	715
2.40	8.96	21.5	.11	5960	656

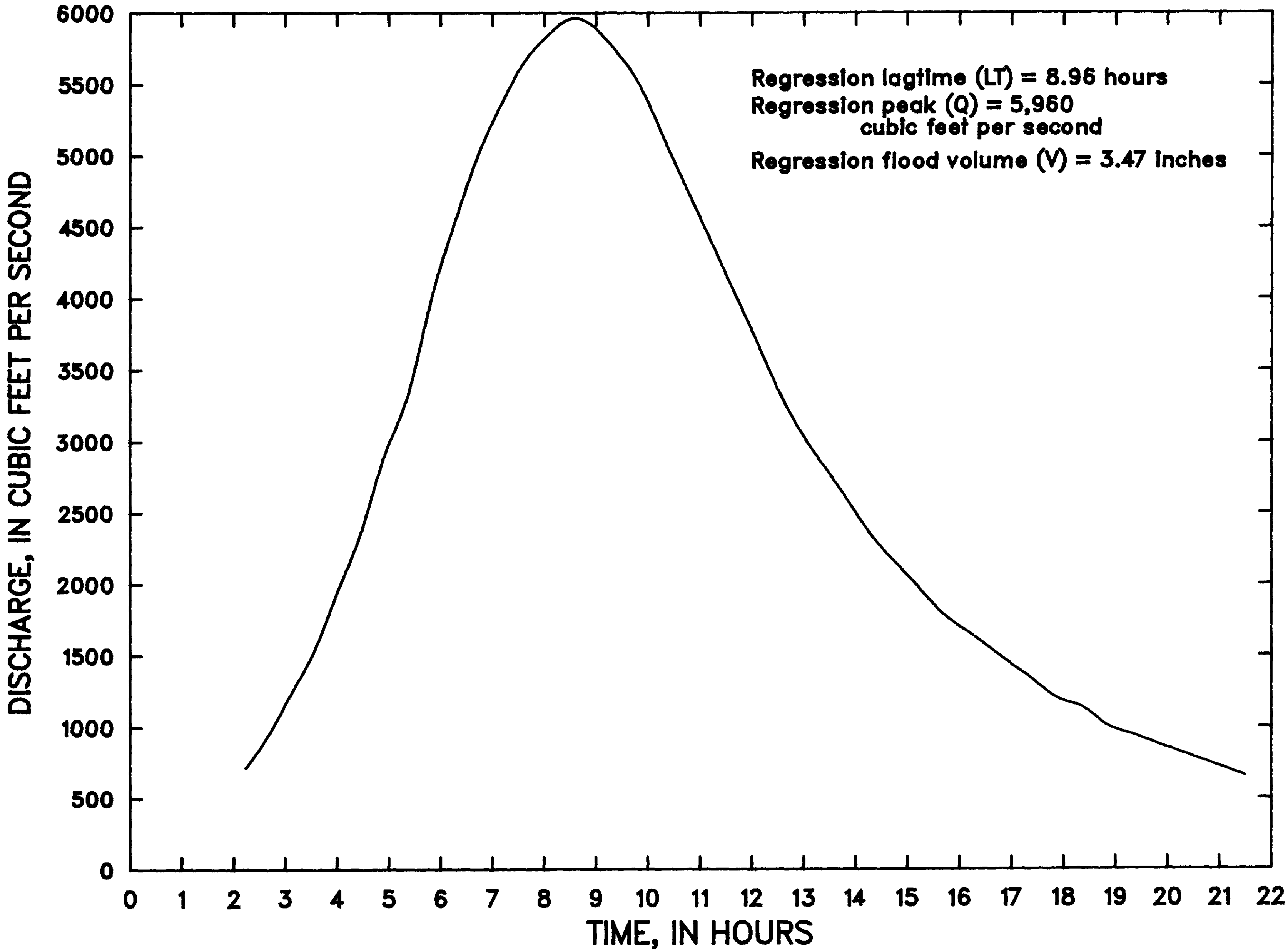

Figure 4.--Simulated hydrograph for sample creek in example problem.

4. A lagtime of 8.96 hours is computed from the lagtime equation for streams located above the Fall Line.

5. The coordinates of the flood hydrograph are computed by multiplying the estimated lagtime of 8.96 hours by the time ratios and the peak discharge (5,960 ft^3/s) by the discharge ratios from table 1. Table 8 and figure 4 show the resultant hydrograph.

6. The flood volume, 3.47 inches, is computed from the volume equation.

7. If the length of time the road is overtopped is needed, and the overtopping discharge is 3,000 ft^3/s, the road overflow time is computed as follows:

   a. Q/Qp = 3,000/5,960 = 0.50

   b. from figure 3 or table 7, W/LT = 0.91

   c. basin lagtime = 8.96 hours

   d. road overflow time = (W/LT) (LT)
      = (0.91) (8.96)
      = 8.15 hours

SUMMARY

The dimensionless hydrograph developed for Georgia (Inman, 1986) was tested to see if it could be applied to Alabama streams. The test was made by comparing observed hydrographs for Alabama streams to those simulated using Georgia's (Inman, 1986) dimensionless hydrograph procedure. Test results at 50 and 75 percent of peak flow widths showed that the Georgia dimensionless hydrograph could be used in Alabama. Therefore, the coordinates of the Georgia dimensionless hydrograph can be used to simulate flood hydrographs for both rural and urban ungaged streams in Alabama using equations developed to estimate peak discharge and lagtime in Alabama. A total of 120 observed hydrographs from 37 stations were used to test the Georgia dimensionless hydrograph. An additional 40 flood hydrographs were used for a verification test.

Multiple-regression analyses were used to develop relations between lagtime and selected basin characteristics. Drainage area and main channel slope were significant for the rural basins; drainage area, main channel slope, and percent impervious area were significant for the urban basins. Two equations were developed for the rural basins; one for areas north of the Fall Line and another for south of the Fall Line. There was no parameter or geographic bias in either the rural equation or the urban equation. All equations were verified by split-sample testing.

A theoretical analysis was used to also develop a relation between flood volumes (the dependent variable), and drainage area, peak discharge, and basin lagtime (the significant independent variables). One equation applies statewide for both rural and urban basins.

An estimated hydrograph can be computed by applying lagtime and the specified peak discharge to the dimensionless hydrograph time and discharge ratios. The coordinates of the estimated hydrograph are computed by multiplying the lagtime by the time ratios and the peak discharge by the discharge ratios. The volume of the flood hydrograph can be computed from the regionalized volume equation.

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GLOSSARY

Dimensionless hydrograph is a unit hydrograph where the discharge scale is divided by the peak discharge and the time scale is divided by the basin lagtime.

Drainage area (A)--in square miles, the contributing area determined by outlining the basin on the latest topographic map available and planimetering the area within the outline.

Hydrograph width (W)--in units of time, is the width of a hydrograph at a specified discharge.

Impervious area (IA)--in percent, the contributing drainage basin occupied by impervious surfaces. This parameter was measured using the grid method and the latest topographic maps and aerial photos.

($L/_S$ 0.5)--a ratio, where L and S have been previously defined.

Lagtime (LT)--in hours, the time from the centroid of the rainfall excess to the centroid of the resultant runoff hydrograph.

Main channel length (L)--in miles, the distance along the longest channel from the gaging station to the basin divide.

Main channel slope (S)--in feet per mile, the average slope between points 10 and 85 percent of the total channel length from the gaging station to the basin divide.

R-squared (R^2), the coefficient of determination (square of the multiple correlation coefficient), measures how much variation in the dependent variable can be accounted for by the model (dependent variables).

Standard error of estimate (SEr)--in percent, is the standard deviation of the distribution of the residuals about the regression line.

Standard error of prediction (SEp)--in percent, is a measure of the estimating ability at all sites, gaged and ungaged.

Unit hydrograph is the hydrograph of direct runoff resulting from a storm having 1 inch of rainfall excess (runoff) uniformly distributed over the drainage basin during a specified duration of time.

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