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Simulation of Flood Hydrographs for Georgia Streams

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ABSTRACT

Flood hydrographs are needed for the design of many highway drainage structures and embankments. A method for simulating these flood hydrographs at urban and rural ungauged sites in Georgia is presented. The O'Donnell method was used to compute unit hydrographs from 355 flood events from 80 stations. An average unit hydrograph and an average lag time 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 lag time and then reduced to dimensionless terms by dividing the time by lag time and the discharge by peak discharge. Hydrographs were simulated for these 355 flood events 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 lag-time duration provided the best fit of the observed data. Multiple-regression analysis was used to define relations between lag time and certain physical basin characteristics, of which drainage area and slope were significant for the rural equations, with impervious area being added for the Atlanta urban equation. A hydrograph can be simulated from the dimensionless hydrograph, peak discharge of a specific recurrence interval, and lag time obtained from regression equations for any site of less than 500 mi² in Georgia. For simulating hydrographs at sites larger than 500 mi², the U.S. Geological Survey computer model CONROUT can be used. CONROUT produces a simulated outflow discharge hydrograph with a peak discharge of a specific recurrence interval. The diffusion analogy routing method with single linearization was used in this study.

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 alternative designs are necessary to determine the design with the least total expected cost. In order 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 of inundation of specific features, for example, roads and bridges. For ungauged streams, this information is difficult to obtain; therefore, there is a need for a method based on Georgia hydrologic data 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 ungauged sites in Georgia. The scope of this study was statewide for rural basins and the Atlanta metropolitan area for urban basins up to 25 mi².

HYDROGRAPH SIMULATION PROCEDURE

Several traditional methods for simulating a hydrograph for a flood of selected recurrence interval at an ungauged 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 mi²

A dimensionless hydrograph was developed for use in basins up to 500 mi². Peak discharge of a selected

recurrence interval and lag time are necessary parameters to convert the dimensionless hydrograph to a simulated hydrograph for a given basin. Price (1) presents a technique for estimating the peak discharge of a selected recurrence interval for rural streams in Georgia. Inman (2) presents a technique for estimating the peak discharge of a selected recurrence interval for basins less than 25 mi² in the Atlanta urban area. Lag-time estimating equations were developed for Georgia streams as part of the current study and will be presented in a later section.

The dimensionless hydrograph was developed from observed flood hydrographs. Using data from 80 basins having drainage areas less than 20 mi², the method is as follows:

1. Compute a unit hydrograph and lag time for three to five storms for each of the 80 gauging stations. All unit hydrographs should be for the same time interval (duration) at a station. Lag time 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 (3, pp. 546-557).

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. 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 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 lag time.

4. Transform the average unit hydrographs computed in step 3 to hydrographs having durations of one-fourth, one-third, one-half, and three-fourths lag time. These durations must be to the nearest multiple of the original duration (computation interval). These transformed unit hydrographs will have durations of two times, three times, four times, and six 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-min to a 20-min duration) can be accomplished through the use of the following equations:

$$TUHD(t) = 1/2[TUH(t) + TUH(t - 1)] \quad (D/\Delta t = 2) \quad (1)$$

$$TUHD(t) = 1/3[TUH(t) + TUH(t - 1) + TUH(t - 2)] \quad (D/\Delta t = 3) \quad (2)$$

$$TUHD(t) = 1/4[TUH(t) + TUH(t - 1) + TUH(t - 2) + TUH(t - 3)] \quad (D/\Delta t = 4) \quad (3)$$

$$TUHD(t) = 1/n[TUH(t) + TUH(t - 1) + \dots + TUH(t - n + 1)] \quad (D/\Delta t = n) \quad (4)$$

where

Δt = computation interval (the original unit hydrograph has an actual 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 that of 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 paper, design duration is expressed as a fractional part of lag time, such as one-fourth, one-third, one-half, and three-fourths. It is later shown that the design duration of one-half lag time provides the best fit of observed data.

5. Reduce the one-fourth, one-third, one-half, and three-fourths lag-time hydrographs to dimensionless terms by dividing the time by lag time and the discharge by peak discharge.

6. For Hydrologic Regions 1, 2, and 3 as defined by Price (1) and the Atlanta urban area as reported by Inman (2), compute an average dimensionless hydrograph by using the dimensionless hydrographs at the stations within that area or region. The hydrographs were computed by aligning the peaks and averaging each ordinate of the discharge ratio (Q/Q_p).

Steps 1 through 5 were done for all stations having data in the U.S. Geological Survey (USGS) WATSTORE unit-values file, which had hydrographs plotted from earlier studies. A total of 355 unit hydrographs from 80 stations, including 19 Atlanta urban sites, was used to develop the one-fourth, one-third, one-half, and three-fourths lag-time 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 lag-time dimensionless hydrographs from each region or area with the observed hydrograph widths from their respective region or area. The one-half lag time was the best fit of width at 50 percent and 75 percent of peak flow. In Figure 1 plots of the one-half lag-time dimensionless hydrograph for Regions 1, 2, and 3 and for the Atlanta urban area are shown. On the basis of these plots, one dimensionless hydrograph was selected for both rural and urban conditions for the entire state as shown in Figure 2 and Table 1.

Another statistical analysis to test the accuracy of the dimensionless hydrograph application technique was done by comparing the simulated hydrograph widths at 50 and 75 percent of peak flow from simulated hydrographs using the statewide one-half lag-time dimensionless hydrograph with the 355 observed hydrographs. One example of this comparison is shown in Figure 3. The resulting standard error of estimate for the 50 percent of peak flow width comparison was ± 31.8 percent and that for the 75 percent comparison was ± 35.9 percent. The standard error of estimate of the width comparisons is based on mean-square difference between observed and simulated widths. Based on verification and bias testing, which are presented in a later section, this dimensionless hydrograph can be used for flood-hydrograph simulation for ungauged basins up to 500 mi². Steps 3 through 6 of the dimensionless hydrograph development and the statistical analyses were programmed for computer use by S.E. Ryan of the USGS.

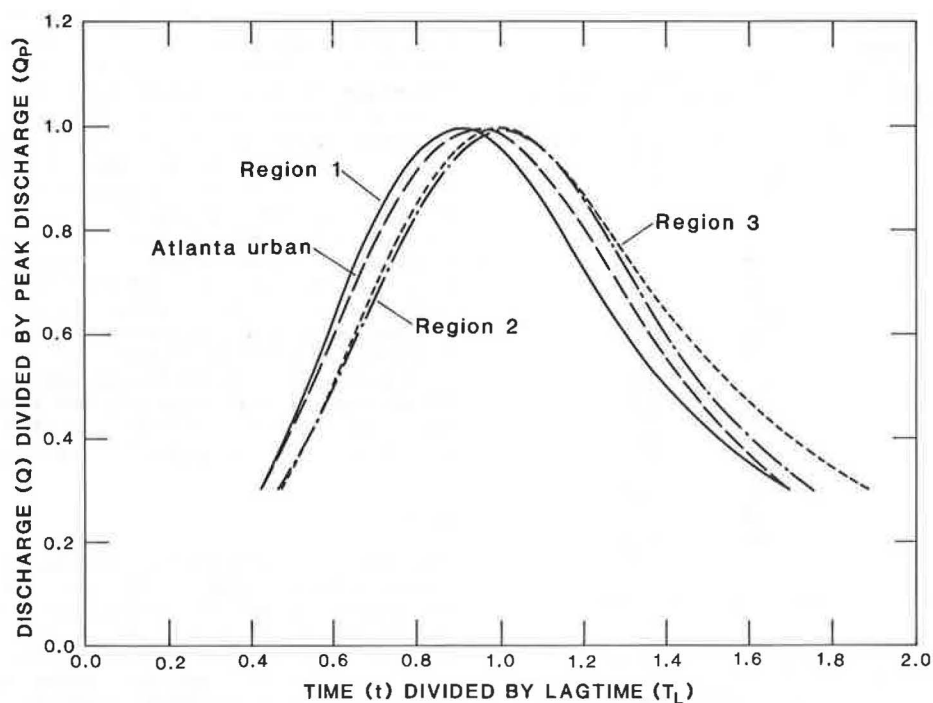


FIGURE 1 Average one-half lag-time dimensionless hydrographs for Regions 1, 2, and 3 and the Atlanta urban area.

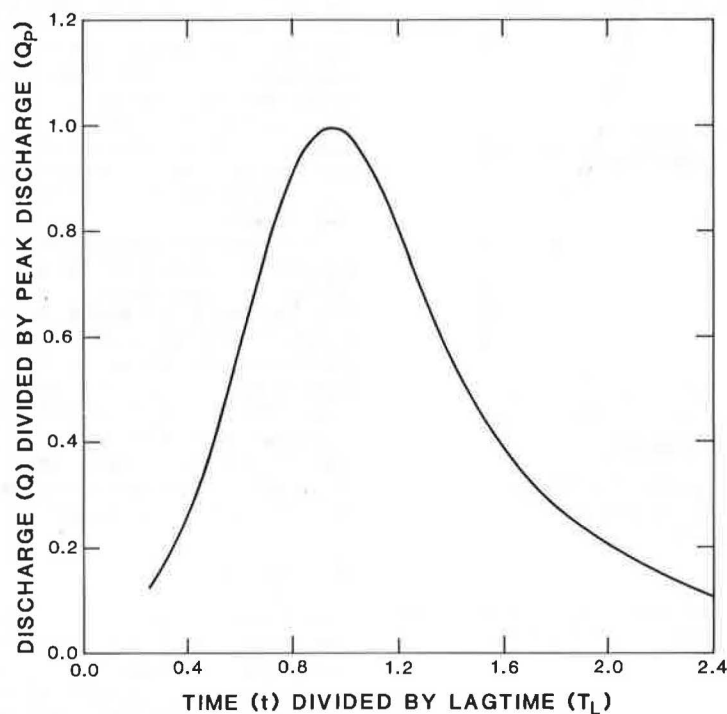


FIGURE 2 Statewide dimensionless hydrograph.

Basins Greater Than 500 mi²

The method for simulating a hydrograph at basins greater than 500 mi² uses the USGS computer model CONROUT. The model routes streamflow from an upstream channel location to a user-defined location downstream. CONROUT is described in detail by Doyle et al. (4).

CONROUT provides the user with two methods of routing: diffusion analogy and storage continuity.

The diffusion-analogy method with single linearization as recommended by Keefer (5) was used in this study.

TESTING OF DIMENSIONLESS HYDROGRAPHS

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 pre-

TABLE 1 Time and Discharge Ratios of Statewide Dimensionless Hydrograph

Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)	Time Ratio (t/T_L)	Discharge Ratio (Q/Q_p)
0.25	0.12	1.35	0.62
0.30	0.16	1.40	0.56
0.35	0.21	1.45	0.51
0.40	0.26	1.50	0.47
0.45	0.33	1.55	0.43
0.50	0.40	1.60	0.39
0.55	0.49	1.65	0.36
0.60	0.58	1.70	0.33
0.65	0.67	1.75	0.30
0.70	0.76	1.80	0.28
0.75	0.84	1.85	0.26
0.80	0.90	1.90	0.24
0.85	0.95	1.95	0.22
0.90	0.98	2.00	0.20
0.95	1.00	2.05	0.19
1.00	0.99	2.10	0.17
1.05	0.96	2.15	0.16
	0.92	2.20	0.15
1.15	0.86	2.25	0.14
1.20	0.80	2.30	0.13
1.25	0.74	2.35	0.12
1.30	0.68	2.40	0.11

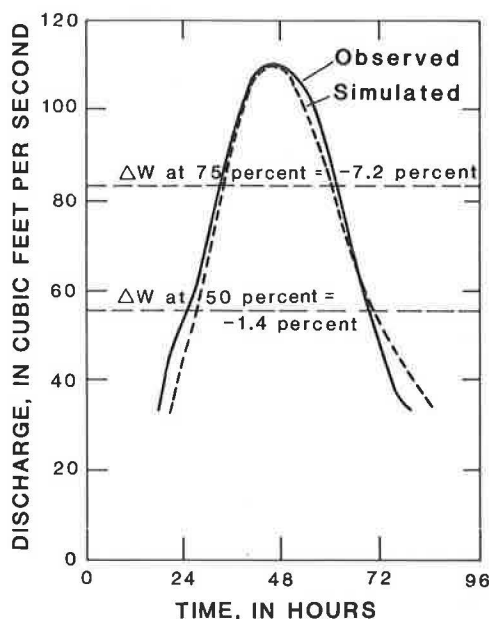


FIGURE 3 Observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow, Atlanta urban station.

sented in prior sections of this paper. 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 flood events from 37 stations having drainage areas of 20 to 500 mi^2 and located throughout the state. The average station lag time and peak discharge for each flood event were used to simulate a theoretical flood hydrograph, which was compared with the observed hydrograph. At 50 and 75 percent of peak flow widths the standard errors of estimate were ± 39.5 and ± 43.6 percent, respectively. Figure 4 gives an example of this comparison.

An additional verification, or test, of the entire simulation procedure was conducted on the highest peaks (simple or compound) having unit values available in the Georgia District and a station flood-frequency curve. Thirty-one stations having drainage areas of 20 to 500 mi^2 were tested as follows. The recurrence interval of this observed peak discharge (Q) was determined from the station frequency curve. The appropriate regional frequency equation from Price (1) was used to compute the corresponding peak discharge for this recurrence interval. The lag time (T_L) for this station was computed from the appropriate regional lag-time 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 ± 51.7 and ± 57.1 percent, respectively. Figure 5 gives an example of this comparison.

Bias

Two tests for bias were conducted, one for simulated versus observed hydrograph width and the other for geographical bias. The width-bias test was performed on the widths at 50 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 (\bar{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 peak Q from station frequency curves. However, Student's t -test indicated that these errors are not statistically significant at the 0.01 level, and therefore the simulated hydrograph widths are not biased.

The test for geographical bias was done by comparing the widths at 50 and 75 percent of the ratio (Q/Q_p) of the dimensionless hydrographs simulated for Regions 1, 2, and 3 as defined by Price (1), and shown in Figure 6, and for the Atlanta metropolitan area with the widths of the statewide dimensionless hydrograph. Figure 1 shows these four dimensionless hydrographs. There was no significant bias. In fact, the mean error (\bar{x}) was very small in both the 50 and the 75 percent tests, 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 ± 10 and ± 20 percent at the hydrograph widths corresponding to 50 and 75 percent of peak flow, respectively. When peak Q was varied, the test results indicated that the hydrograph width did not change at 50 or 75 percent of that varied peak Q . When lag time was varied, the test results indicated that the hydrograph width varied by the same percentage.

REGRESSION ANALYSIS OF LAG TIME

So that lag time could be estimated for ungauged sites, the average station lag times obtained from

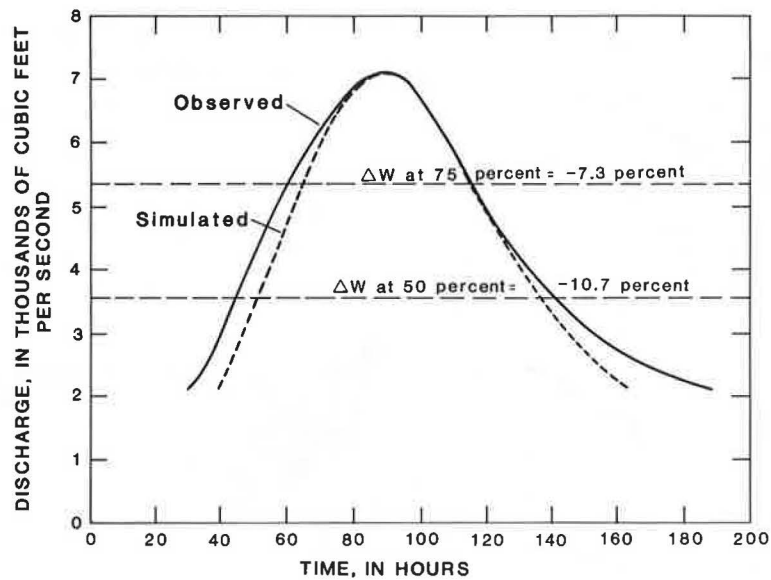


FIGURE 4 Observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow, Spring Creek near Iron City.

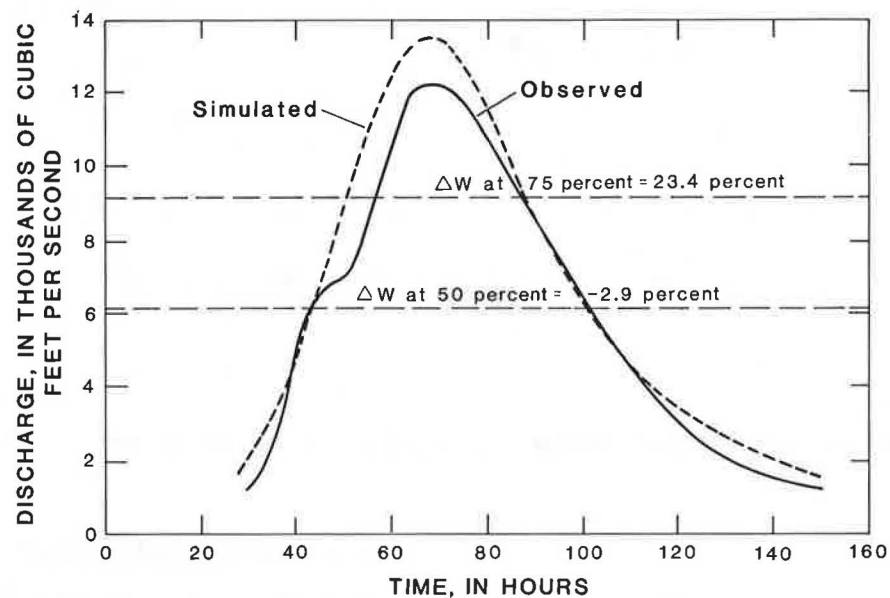


FIGURE 5 Observed and predicted hydrographs for width comparisons at 50 and 75 percent of peak flow, Flint River near Griffin.

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 (6). Lag times were computed for each flood event with the same program that computed the t -hour unit hydrographs. These storm-event lag times were then averaged to compute an average station lag time, which was in turn used in the regression analyses. Lag time is generally considered to be constant for a basin and is defined by Stricker and Sauer (7) as the time from the centroid of rainfall excess to the centroid of the runoff hydrograph. Lag time for the 19 Atlanta urban stations was analyzed separately because of the effect of urbanization.

The regression equations provide a mathematical relation between the dependent variable (lag time) and the independent variables (the basin character-

istics found to be statistically significant). All variables were transformed into logarithms before analysis to (a) obtain a linear regression model and (b) achieve equal variance about the regression line throughout the range. In the analyses performed, a 95 percent confidence limit was specified to select the significant independent variables.

The independent variables, or physical basin characteristics, are defined in the following paragraphs.

Lag Time (T_L)

T_L is the elapsed time in hours from the centroid of rainfall excess to the centroid of the resultant runoff hydrograph. Lag time is computed from the unit hydrograph.

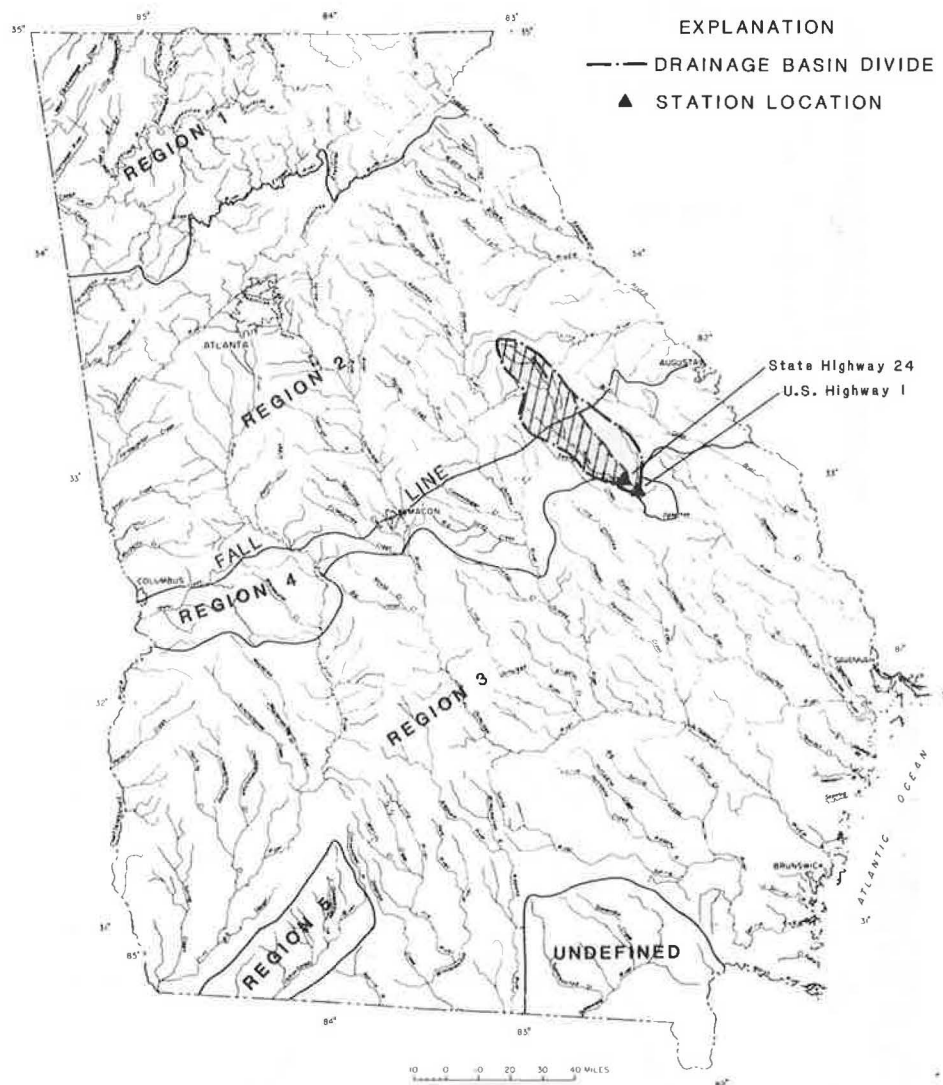


FIGURE 6 Regional boundaries for flood-frequency and lag-time estimating equations.

Drainage Area (A)

Area of the basin is in square miles and is planimeted from USGS 7.5-min topographic maps. Basin boundaries were all field checked.

Channel Slope (S)

The main channel slope is 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 is in miles, as measured from the gauging station upstream along the channel to the basin divide.

$$L/S^{0.5}$$

L and S have been previously defined.

Measured Total Impervious Area (IA)

The percentage of drainage area that is impervious to infiltration of rainfall is determined by a grid-overlay method using aerial photography. According to Cochran (8, pp. 71-86), 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, was obtained in conjunction with measured total impervious area. Noneffective impervious area, such as house rooftops that drain onto a lawn, is subtracted from

this total. When the minimum of 200 points was 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 percent 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 subbasins to determine the MEIA percentage.

Regionalization

The initial regression run utilized data from 91 rural stations of less than 500 mi² 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 (1) and shown in Figure 6, tended to overpredict lag time, whereas the area south of the fall line, consisting of Regions 3, 4, and 5 as defined by Price (1) and shown in Figure 6, tended to underpredict lag time.

The next step was to make separate regression runs for each of the five regions. Region 1 had no equations with two or more variables 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 no equations with two or more variables significant at the 95 percent confidence limit. The standard error of estimate of the regression using only one variable ranged from 34 to 37 percent, with a tendency to overpredict at the lower end of the curve and underpredict at the upper end.

Regions 1 and 2 were combined and analyzed as one region. Two equations each have two variables significant at the 95 percent confidence limit. The equation selected was

$$T_L = 4.64A^{0.49}S^{-0.21} \quad (5)$$

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

$$T_L = 13.6A^{0.43}S^{-0.31} \quad (6)$$

The Atlanta urban area was analyzed separately because of the effects of urbanization on lag time. IA and MEIA were added as independent variables in the analysis. The following equation was selected:

$$T_L = 161A^{0.22}S^{-0.66}IA^{-0.67} \quad (7)$$

It 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 (7) 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 do 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^2 (the correlation coefficient squared), and the standard error of regression. R^2 measures how much variation in the dependent variable can be accounted for by the independent variables. For example, an R^2 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 lag-time equations and their related statistics is given in Table 2.

Limits of Independent Variables

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

Variable	Minimum	Maximum	Unit
North of fall line			
A	0.3	500	Square miles
S	5.0	200	Feet per mile
South of fall line			
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	Unit
A	0.2	25	Square miles
S	13	175	Feet per mile
IA	14	50	Percent

TESTING OF LAG-TIME REGRESSION EQUATIONS

The lag-time regression equations were tested with the same four tests as those used for the dimensionless hydrograph. The standard error of estimate has been explained and presented in a prior section. Verification, bias, and sensitivity are the other tests.

TABLE 2 Summary of Lag-Time Estimating Equations

Area	Equation	Standard Error of Regression (%)	Coefficient of Determination (R^2)
North of the fall line (rural)	$T_L = 4.64A^{0.49}S^{-0.21}$	±31	0.94
South of the fall line (rural)	$T_L = 13.6A^{0.43}S^{-0.31}$	±25	0.96
Metropolitan Atlanta (urban)	$T_L = 161A^{0.22}S^{-0.66}IA^{-0.67}$	±19	0.94

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 according to Sauer et al. (9). 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 events made up the first sample and the even-numbered events the second sample. Multiple-regression analyses were performed on both regions using only the sites in one of the samples; then the equations were recalibrated using the sites in the other sample. The results were all acceptable, as shown in Table 3. The regression analyses yielded new regression equations similar to the equations originally developed by 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 the even sample above the fall line. The equations with A and $L/S^{0.5}$ was split-sample tested for the area south of the fall line, and A was not 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 lag time) 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.

Geographical bias was tested by plotting the residuals of observed lag times minus predicted lag times on a state map. The plot was visually inspected to determine whether any area of the state was 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 bias analyses were performed on the Atlanta urban equation. There was no geographical or variable bias.

Sensitivity

The fourth test was to analyze the sensitivity of lag time to errors in the two independent variables in the regression equations. The computation of these independent variables is subject to errors in measurement and judgment. To illustrate the effect of such errors, the equations were tested to determine how much error was introduced into the computed lag time from specified percentage errors in the independent variables. The test results are shown in Table 4, which was computed by assuming that all independent variables were constant except the one being tested for sensitivity.

The Atlanta urban equation was tested for sensitivity of lag time to errors in the three independent variables in the same manner as that for the two rural equations. The test results are shown in Table 4.

SUMMARY

A dimensionless hydrograph was developed for Georgia streams having drainage areas of less than 500 mi². This dimensionless hydrograph can be used to simulate flood hydrographs at ungauged sites for both rural and urban streams statewide. More than 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 lag time 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. The rural equation was regionalized into one equation for the area north of the fall line and one equation for the area south of the fall line. Both rural equations were verified by split-sample testing. There was neither variable nor

TABLE 3 Split-Sample Test Results for Lag-Time Equations

Area	No. of Stations	Equation	Standard Error of Regression (%)	Standard Error of Prediction (%)	Coefficient of Determination (R ²)
North of fall line					
Odd	25	$T_L = 4.88A^{0.48}S^{-0.22}$	±32	—	0.94
Even	24	—	—	±32	0.93
Even	24	$T_L = 4.51A^{0.50}S^{-0.21}$	±31	—	0.94
Odd	25	—	—	±32	0.94
South of fall line					
Odd	21	$T_L = 36.8A^{0.35}S^{-0.57}$	±18	—	0.98
Even	21	—	—	±41	0.92
Even	21	$T_L = 8.63A^{0.48}S^{-0.21}$	±26	—	0.96
Odd	21	—	—	±29	0.96

Note: Dashes indicate data not applicable.

TABLE 4 Sensitivity of Computed Lag Time to Errors in Independent Variables

Percentage Error in Independent Variable	Percentage Error in Computed Lag Time by Independent Variable		
	Area	Slope	Impervious Area
Equation for North of Fall Line			
+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	
Equation for South of Fall Line			
+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	
Atlanta Urban Equation			
+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

geographical bias in either the rural equation or the Atlanta urban equation. Sensitivity tests indicated drainage area as the most sensitive basin characteristic in the rural equation and impervious area as the most sensitive in the Atlanta urban equation.

A simulated flood hydrograph may be computed by applying lag time, 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 lag time by the time ratios and peak discharge by the discharge ratios in Table 1.

For basins larger than 500 mi² the USGS 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 outflow discharge hydrograph with a peak of a specific recurrence interval at the end of a reach.

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