FLOOD-FREQUENCY RELATIONS FOR URBAN STREAMS IN GEORGIA—1994 UPDATE

U.S. GEOLOGICAL SURVEY

Prepared in cooperation with the

GEORGIA DEPARTMENT OF TRANSPORTATION
DEKALB COUNTY
CONSOLIDATED GOVERNMENT OF COLUMBUS
CITY OF ALBANY
CITY OF MOULTRIE
CITY OF THOMASVILLE
CITY OF VALDOSTA

Water-Resources Investigations Report 95-4017

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Atlanta, Georgia

1995

U.S. DEPARTMENT OF THE INTERIOR
BRUCE BABBITT, Secretary

U.S. GEOLOGICAL SURVEY
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CONVERSION FACTORS

Multiply	by	To obtain
	Length	
inch (in.)	25.4	millimeter
foot (ft)	0.3048	meter
mile (mi)	1.609	kilometer
	Area	
square mile (mi^2)	2.590	square kilometer
	Flow	
cubic foot per second (ft^3/s)	0.02832	cubic meter per second

VERTICAL DATUM

Sea Level: In this report, “sea level” refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929) - a geodetic datum derived from a general adjustment of the first-order level nets of both the United States and Canada, formerly called “Sea Level Datum of 1929.”

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ABSTRACT

A statewide study of flood magnitude and frequency in urban areas of Georgia was made to develop methods of estimating flood characteristics at ungaged urban sites. A knowledge of the magnitude and frequency of floods is needed for the design of highway drainage structures, establishing flood-insurance rates, and other uses by urban planners and engineers.

A U.S. Geological Survey rainfall-runoff model was calibrated for 65 urban drainage basins ranging in size from 0.04 to 19.1 square miles in 10 urban areas of Georgia. Rainfall-runoff data were collected for a period of 5 to 7 years at each station beginning in 1973 in Metropolitan Atlanta and ending in 1993 in Thomasville, Ga. Calibrated models were used to synthesize long-term annual flood peak discharges for these basins from existing long-term rainfall records. The 2- to 500-year flood-frequency estimates were developed for each basin by fitting a Pearson Type III frequency distribution curve to the logarithms of these annual peak discharges.

Multiple-regression analyses were used to define relations between the station flood-frequency data and several physical basin characteristics, of which drainage area and total impervious area were the most statistically significant. Using these regression equations and basin characteristics, the magnitude and frequency of floods at ungaged urban basins can be estimated throughout Georgia.

INTRODUCTION

A knowledge of flood characteristics of streams is essential for the design of roadway drainage structures, establishing flood-insurance rates, and for other uses by urban planners and engineers. Because urbanization can produce significant changes in the flood-frequency characteristics of streams, natural (rural) basin flood-frequency relations are not applicable to urban streams.

Beginning in 1973 in Metropolitan Atlanta (Inman, 1983) and ending in 1986 in Athens, Augusta, Columbus, Rome, and Savannah (Inman, 1988), rainfall-runoff data were collected at 45 stations. These data were used to calibrate a model to produce a report for statewide use.

Recognizing the need for additional reliable urban peak-flood data and improved equations for estimating floods, the U.S. Geological Survey (USGS), in cooperation with the city governments of Albany, Moultrie, Thomasville, and Valdosta, Ga., initiated flood-frequency studies in 1986 to supplement data from earlier studies (Inman, 1983) and to update flood-frequency relations (Inman, 1988). The earlier studies (Inman, 1983, 1988) were conducted under cooperative agreements with between the USGS and the Georgia Department of Transportation, the Consolidated Government of Columbus, Ga., and DeKalb County, Ga.

Purpose and Scope

This report describes the results of a study to develop regression equations for estimating the magnitude and frequency of floods for urban streams Statewide. Recognizing the need for additional observed urban data in south Georgia, 20 basins were selected in four urban areas in south Georgia to supplement data from 45 basins used in earlier studies (table 1, in back of this report). Two basins were selected in Albany, four in Moultrie, six in Thomasville, and eight in Valdosta. Data from at least 40 floods per basin were used to calibrate a USGS rainfall-runoff model (RRM), as described by Bergmann, Inman, and Lumb (U.S. Geological Survey, written commun., 1990).

After the RRM was successfully calibrated for each drainage basin, long-term rainfall data from a nearby National Weather Service (NWS) station were used to synthesize about 60 to 90 years of annual peaks depending on the length of the long-term rainfall. These synthesized peaks were used to develop flood-frequency relations at each basin. Forty-five additional flood-frequency relations from the earlier urban studies were also used in the Statewide analysis. The final step in analyzing these data was to develop regression equations that can be used to estimate the magnitude and frequency of floods at ungaged urban sites throughout the State.

Previous Studies

Many urban flood-frequency studies have been undertaken in the State of Georgia and Metropolitan Atlanta, however, none have been based on the number of stations and the amount of observed data contained in this report. A few of the more prominent ones are listed in this section.

Lumb (1975), in his report, “UROS4: Urban Flood Simulation Model, Part 1, Documentation and Users Manual”, explained how the UROS4 model was used to simulate an annual series of flood peaks and perform a flood-frequency analysis at a selected point. James and Lumb (1975) applied the UROS4 model to eight watersheds in DeKalb County, Ga., with limited observed data for verification.

Golden (1977) presented flood-frequency relations for urban streams in Metropolitan Atlanta based on the technique used by Sauer (1974) for Oklahoma, which used the natural flood-frequency and rainfall-frequency characteristics of the local area. Sauer (1974) adjusted natural flood-frequency relations to urban conditions by using local rainfall-frequency characteristics, the percentage of impervious area in the basin, and the percentage of the basin served by storm sewers. Price (1979) used the same technique on a Statewide basis. Jones (1978) presented simplified equations that can be used on small watersheds (less than 200 acres) to estimate peak discharges in DeKalb County.

Lichty and Liscum (1978) described a procedure for computing estimates of 2- through 100-year floods that incorporates a rainfall information-transfer mechanism in the form of three maps, and a generalized definition of synthetic T-year flood potential as a function of fitted rainfall-runoff model parameters. Impervious area was incorporated in the T-year flood equations to account for urban development. This procedure is applicable for most of the Eastern United States.

An updated method for estimating the magnitude and frequency of floods on small streams in the Atlanta Metropolitan area was presented by Inman (1983). This method was based on observed peak-discharge data from 19 stations, which were used to calibrate a USGS rainfall-runoff model (Dawdy and others, 1972). The model was then used to synthesize long-term annual peak discharges for these 19 basins. The 2- to 100-year flood estimates were developed for the 19 basins from these synthetic, long-term peak discharges by fitting a Pearson Type III frequency distribution curve to the logarithms of the annual peak discharges. Multiple-regression analyses were used to define relations between the flood-frequency station data and certain physical characteristics of the basin, of which drainage area, main-channel slope, and measured total impervious area were found to be statistically significant. These relations were used to estimate the magnitude and frequency of floods at ungaged basins in the Atlanta Metropolitan area.

Regression equations and several other methods of estimating flood-frequency for urban watersheds on a nationwide basis were presented by Sauer and others (1983). Five basins from the Atlanta area were used in that analysis.

A method for estimating the magnitude and frequency of floods for urban streams on a statewide basis for Georgia was presented by Inman (1988). This method was based on observed data from 45 stations, which were used to calibrate a USGS rainfall-runoff model (Dawdy and others, 1972). The model was then used to synthesize long-term peak discharges for these basins. The 2- to 100-year peak discharge estimates were developed for each basin from these synthetic, long-term annual peak discharge records and by fitting a Pearson Type III frequency distribution curve to the logarithms of the annual peak discharges. Multiple-regression analyses were used to define relations between the flood-frequency station data and certain physical characteristics of the basin, of which drainage area, equivalent rural discharge, and measured total impervious area were found to be statistically significant. These relations were used to estimate the magnitude and frequency of floods at ungaged basins in urban areas on a statewide basis for Georgia.

Acknowledgments

The author wishes to acknowledge Thomas N. Debo, Georgia Institute of Technology, City Planning Department, Atlanta, Ga., for his assistance in the selection of sites and in the determination of impervious area for basins in this study. Long-term rainfall and daily pan-evaporation records were obtained from the U.S. Department of Commerce, National Weather Service (NWS), Asheville, N.C.

SITE SELECTION

Extensive field reconnaissance was conducted at about 300 sites and 65 basins were selected for this study. A broad range in drainage area, main-channel slope, and main-channel length was considered. Suitability for rain-gage location, hydraulic characteristics at the gaging site, absence of significant permanent surface storage, and land use also were factors involved in the selection process. One of the most important factors considered was land-use stability. Thomas N. Debo, Georgia Institute of Technology, City Planning Department, Atlanta, Ga., consulted with all city and county planners in the metropolitan areas involved in this study, and based on their data and general knowledge of the areas, determined the stability of developed areas. This information was presented on color-coded city and county maps as being either stable, fairly stable, or unstable.

The next step in this study was a field reconnaissance of selected basins in areas designated as stable and fairly stable. Many of these basins were excluded because their hydraulic characteristics were not suitable for indirect computations of peak discharge or because they contained no suitable location for a rain gage. The remaining basins were roughly delineated on USGS 7 1/2-minute topographic maps, and approximate drainage areas, main-channel slopes, and lengths were determined. From this information, about 100 sites were selected to provide broad ranges in drainage area, main-channel slope, and main-channel length.

Sixty-five urban basins were selected for study from about 100 deemed suitable. These generally were the basins with the best hydraulic characteristics for indirect computations of peak discharge and the most suitable rain-gage locations. The selected basins provide suitable distributions of drainage area, main-channel slope, and main-channel length. The locations of cities with gages and the number of gages in each city are shown in figure 1. Further information on the rural hydrologic regions shown in figure 1 can be obtained from Stamey and Hess (1993).

DATA COLLECTION AND PROCESSING

Digital recorders were used to collect stage and rainfall at 5-minute intervals in each basin. The recording stage gage for most basins was housed on top of an 18-in. vertical corrugated metal-pipe stilling well in the upstream approach section. Each stilling well had two 2-in. intakes near the base and 1/2-in. diameter holes drilled about every 6 in. above ground level to flood stage. Several of the stage gages also were housed on top of 3-in. galvanized pipe attached to the end of an upstream wing wall. All stilling wells were flushed after every flood event and intakes were cleaned during every inspection trip.

Each site had at least one rain gage, generally located near the stage gage. Rain gage recorders were housed on top of 8-ft collector wells made from 3-in. galvanized pipe. Collector wells of this size will hold about 11 in. of rainfall. A drain plug near the bottom of the collector well was used to drain the well during each inspection trip.

Crest-stage gages also were installed at each site, with at least one in the upstream approach section and one at the downstream end of the culvert. The fall in water surface elevation through the culverts obtained from these crest-stage gage relations and the culvert geometry were used to compute a theoretical stage-discharge relation described by Bodhaine (1968). All theoretical stage-discharge relations were verified by current-meter measurements.

The crest-stage relations also served other purposes. A plot of upstream crest-gage stage and downstream crest-gage stage was established for each site. These relations should remain fairly site-consistent or the reason for the inconsistency must be determined. These plots were used primarily on culverts having backwater control. For example, an accumulation of debris at a culvert entrance which could produce excessive fall, or a blockage downstream that would greatly reduce normal fall, could be detected from these crest-stage relations. For culverts with inlet or outlet control, the crest-stage relations are not consistent, but for large blockages, some indication of the problem might be evident. Many times city and county highway maintenance crews would remove debris from culverts between gage servicing trips. When this occurred, outliers from the crest-stage relations were the only evidence of blockage. Records of storm events that were influenced by blockages were not used in model calibration. At most sites, the stage at the recording gage was lower than the stage at the upstream crest-stage gage. This probably was caused by drawdown of the intakes rather than by intake lag, as can be demonstrated by the equation in Buchanan and Somers (1968, p. 13).

A relation between upstream crest-gage stage and recorder stage was established to enable plotting of the theoretical discharge computations, as described above, and the recorder stage. Thus, digital tapes could be processed without having to make a shift correction for each tape. The upstream crest-gage stage and the recorder stage relation also would indicate any problem with the stage hydrograph, such as a hanging float, a float tape that jumped the splines, or intakes clogged with sediment.

Current Data

All flood events with complete rain and stage data and without culvert blockages were processed and loaded into USGS computer storage on a near-current basis. Generally, five to eight storm events were processed annually for each site. Unit-rainfall, unit-discharge, and daily rainfall data were then retrieved and the unit data were plotted against time. The unit-data hydrographs were used to (1) visually edit data, allowing a bad punch by the recorder or a misread punch by the electronic-tape transmitter to be detected easily; (2) detect partially clogged rain-gage intakes or hanging floats; (3) serve as the basis for estimating the rising limb of a storm hydrograph if the stilling well intakes were out of the water at the beginning of a rise; (4) estimate the falling limb in the event that the intakes became partially clogged with sediment on the recession; and (5) estimate the routing parameters in the RRM. After editing and estimations were completed, the data were reloaded into USGS computer storage.

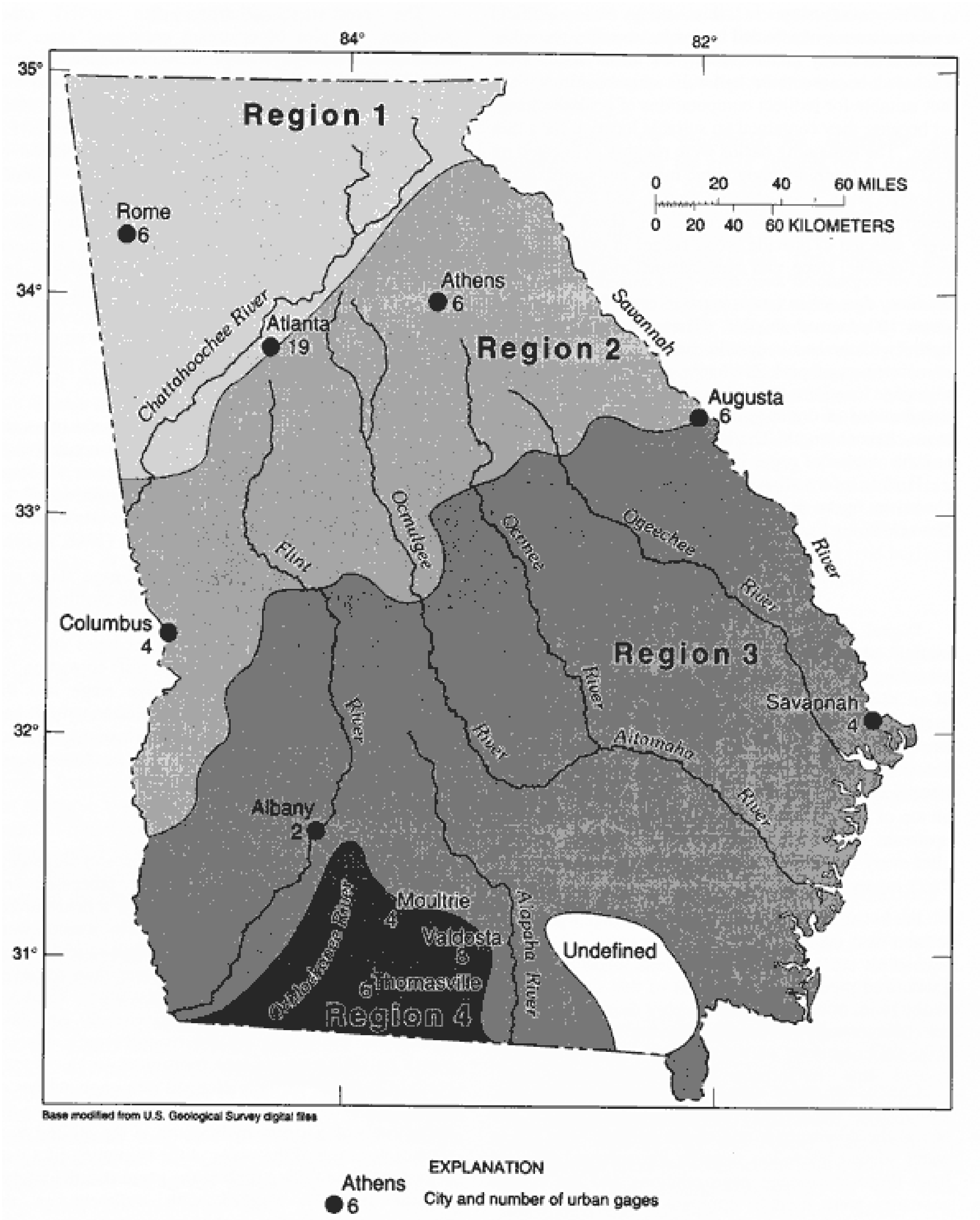

Figure 1. Rural flood-frequency regions in Georgia, cities where gaging stations are used in this study, and number of gages in each city.

Daily pan-evaporation data are needed to calibrate the RRM. Such data were available for Athens and Savannah from nearby NWS stations. However, there were no NWS evaporation stations available near Albany, Atlanta, Augusta, Columbus, Moultrie, Thomasville and Valdosta. Evaporation maps presented by Kohler and others (1959) were used as a guide to select the appropriate NWS evaporation station.

The University of Georgia Plant Science farm evaporation station data at Watkinsville were used to calibrate sites in Atlanta, Augusta, and Athens. Data from the Calhoun Experiment station were used for Rome, the Byron Experiment station for Columbus, and the Savannah Airport station for Savannah. Data from a NWS station near Tifton was used to calibrate sites in Albany, Moultrie, Thomasville, and Valdosta.

Long-Term Rainfall and Daily Pan-Evaporation Data

Long-term rainfall and daily pan-evaporation data are required for flood-peak simulation, as described later in this report. Daily rainfall records from six NWS stations were obtained from NWS (1948-87) publications and loaded into USGS computer storage (table 2). About four to eight rainfall events per year were selected based on hydrologic judgement and by scanning the daily rainfall totals. The dates of significant rainstorms since 1948 were obtained from hourly data in NWS publications (1948-87). For periods before 1948, the daily charts were obtained from the NWS for all daily rainfall events of 1-in. or more per day. The selected storm-rainfall data were coded at 5-minute intervals and loaded into USGS computer storage.

Table 2.—National Weather Service long-term rainfall stations used in Statewide urban study, 1994
[station number is based on latitude and longitude]

Station name	Period of record (in water years)[1/]	Station number
Atlanta	1898-1981	333900084260050
Augusta	1902-1973	332200081580050
Chattanooga	1901-1973	350200085120001
Macon	1900-1973	324200083390050
Savannah	1898-1987	320800081120050
Thomasville-Coolidge	1906-1933 1941-1973	304800083540050

[1/]Water year is the 12-month period beginning October 1 and ending September 30, and is designated in the calendar year in which it ends.

Daily pan-evaporation data were obtained from NWS publications for the five stations either at or near the cities where the rainfall-runoff data were collected (table 3). The observed record from each evaporation station was used to synthesize harmonic average evaporation data for periods prior to the observed record by use of USGS computer program H266 (Carrigan and others, 1977).

Table 3.—National Weather Service daily pan-evaporation stations used in Statewide urban study, 1994
[station number is based on latitude and longitude]

Location (near)	Synthetic period of record (in water years)[1/]	Observed period of record (in water years)[1/]	Station number
Ailey	1898-1946	1947-1981	321100082340050
Athens	1898-1939	1940-1992	335500083210050
Experiment	1897-1935	1936-1981	331600084170050
Rome	1898-1944	1945-1986	342100085100050
Tifton	1898-1936	1937-1993	312800083310050

[1/]Water year is the 12-month period beginning October 1 and ending September 30, and is designated in the calendar year in which it ends.

FLOOD-FREQUENCY RELATIONS

Several phases of data analysis are required to develop and test equations used to estimate peak discharges for various recurrence intervals. The first step is to calibrate and verify the RRM. The second step is to analyze the frequency characteristics of peak-discharge simulations from the RRM. Next, regression analyses are performed to relate flood-frequency estimates to basin characteristics. The final phase of the data analysis is statistical testing of the derived regression equations.

Description of Rainfall-Runoff Model

Program RRM, a lumped-parameter rainfall-runoff model, is described in detail by Bergmann, Inman, and Lumb (USGS, written commun., 1990). The original version of the rainfall-runoff model was described in detail by Dawdy and others (1972). Revisions to the original computer code were presented by Carrigan (1973). The model has three basic components: soil-moisture accounting, infiltration, and surface-runoff routing. Provisions for accounting for nonpervious areas were included in the code. Eleven parameters are used in the three basic components, and are listed and defined in table 4.

Table 4.— Infiltration, soil-moisture accounting, and surface-runoff routing parameters for the U.S. Geological Survey rainfall-runoff model (RRM)
[--, dimensionless parameter; RRM from Bergman, Inman, and Lumb (U.S. Geological Survey, written commun., 1990)]

Parameter identifier code	Units	Infiltration, soil-moisture accounting, and surface-runoff routing parameters
PSP	inches	combined effects of soil-moisture content and suction at the wetting front for soil moisture at field capacity
RGF	--	ratio of PSP for soil moisture at wilting point to that at field capacity
KSAT	inches per hour	minimum saturated value of hydraulic conductivity used to determine soil-infiltration rates
TIA	--	ratio of total impervious area to total basin area
BMSM	inches	soil moisture-storage volume at field capacity
EVC	--	coefficient to convert pan evaporation to potential evapotranspiration values
DRN	inches per hour	constant drainage rate for redistribution of soil moisture
RR	--	proportion of daily rainfall that infiltrates the soil
KSW	hours	time characteristic for linear reservoir storage
TC	minutes	time base of the triangular translation hydrograph
TP/TC	--	ratio of time to peak to base length of the triangular translation hydrograph

The soil-moisture accounting component determines the effect of antecedent conditions on infiltration, and is based on daily rainfall and evaporation. Four model parameters (BMSM, EVC, DRN, and RR), as described in table 4, are used in simulating continuous antecedent soil moisture.

The infiltration component of the model uses unit rainfall data, and the output from the soil-moisture accounting component that indicates the soil moisture content at the beginning of the storm rainfall, to compute infiltration losses. Four parameters (PSP, RGF, KSAT, and TIA), as described in table 4, are used with the modified Philip (1954) infiltration equation.

The surface runoff or routing component (parameters KSW, TC, and TP/TC (table 4)) is based on a modification of the Clark (1945) form of the instantaneous unit-hydrograph procedure. The routing component was modified, as described by Carrigan (1973), to incorporate a triangularly shaped translation hydrograph as an internal feature of the computer program rather than as an externally developed time-area histogram. This modification simplified the calibration procedure and allows separation of compound peaks, a feature that provides the model user with more events to use in calibration. Mitchell (1972) described the triangular representation of the translation hydrograph as a sufficiently accurate assumption for most drainage areas.

Calibration

An average of about 45 flood events per station was initially available for model calibration. The data to be used to fit model parameters were reviewed before beginning the calibration process. The review was made to identify gross violations of assumptions implicit in the RRM. The most evident assumption violation is that rainfall is uniform over the basin during periods of runoff simulation. The uniform rainfall assumption is almost never met by nature; therefore, an “averaging effect” is assumed to apply to the parameter fitting process. The error from the averaging consideration is likely to be within the range of model and data errors if there are a sufficient number (more than 30) of flood-events in the data set. Events that indicate a gross discrepancy between observed rainfall and runoff (such as total rainfall less than total runoff) were discarded because these events can influence the fitting of parameters, at best, and did not help in the fitting process. A secondary rain gage near or in the basin being modeled was used to verify the uniformity of rainfalls. Ideally, discarding events based on nonrepresentative rainfall should be done one time, prior to any parameter fitting, to avoid a tendency of merely trying to reduce errors that are inherent in the modeling process.

Defining events and sub-events are part of the initial data-review effort. The beginning and ending time as well as the initial base flow must be determined for each event and sub-event. Usually, there is little question as to where to begin an event or its value of base flow when no sub-event is involved; however, determining base flows when sub-events are involved is far more difficult. It was decided not to sub-divide an event with regard to runoff volume when the base flow is highly questionable although the sub-event could be defined for peak-flow simulation without great error from a questionable base-flow value. The ending time of an event or sub-event is much more subjective. RRM does not include any secondary flow component and base flow is assumed constant; therefore, attempts are made to balance the influence of increased base flow against some remaining surface flow when selecting the time to end an event. After determining rainfall uniformity and defining events and sub-events, the next step in calibration is to determine starting values and limits on the parameters listed in table 4.

A range for KSAT (table 4) of 0.05 to 0.40 was obtained from Chow (1964). A starting value of 0.15 was used for KSAT. The range and starting values of the other soil-moisture accounting and infiltration parameters RR, BMSM, RGF, and PSP (table 4) are obtained from Inman (1988). EVC (table 4) is obtained from NWS Technical Paper 37 (Kohler and others, 1959). TIA (table 4) is determined by a grid-overlay method from aerial photography (U.S. Department of Agriculture, Agricultural Stabilization and Conservation Service, 1992). (See TIA discussion in "Regression Analysis" section in this report). A lower limit of -25 percent of the computed value and an upper limit of +15 percent of the computed value are applied as limits to this parameter.

A sensitivity analysis of all parameters in the RRM was done for three basins in region 2 of Georgia for the small streams rural study (Golden and Price, 1976). Although DRN (table 4) is included in RRM as a parameter, model results are insensitive even to large changes in DRN. For all cities in this Statewide study, a value of 1.00 is used and held constant for DRN, as used by Alley and Smith (1982).

The starting values for the routing parameters KSW and TC (table 4) are obtained from plots of the discharge hydrographs and the rainfall hyetographs. Initial estimates of KSW and TC can be made completely from rainfall-runoff plots. KSW is the difference in the time of the discharge at the inflection point divided by three, and the time of the discharge at the inflection point. TC is estimated from flood events with intense short-duration rainfall and is defined as the time, in minutes, from the end of rainfall excess to the inflection point. The ratio TP/TC (table 4), was fixed at 0.50, as suggested by Mitchell (1972).

After determining starting values and limits for parameters, RRM calibration can be started. Model calibration is the process of determining a set of parameter values that can produce RRM simulations which best duplicate observed events. Further information on RRM calibration may be obtained from Bergmann, Inman, and Lumb (USGS, written commun., 1990). Because there is no provision in the RRM to account for storage, the routing parameters must be optimized and adjusted manually to reproduce the observed peak discharges. The higher peak discharges are given much more emphasis than lower peaks on the final runs of this phase of optimization because the calibrated RRM models are used to simulate relatively large events (annual peak discharges). The final optimized parameter values for the models are listed in table 5. The 45 stations from the earlier Statewide study by Inman (1988), are recalibrated using RRM and the new parameter values are also listed in table 5. Observed versus simulated plots of the final optimized runs of volumes and peak discharges are plotted and the slopes of the best-fit line are between 0.95 and 1.05; thereby assuring that no bias exists.

The RRM computer program uses input from only one rain gage in each basin. Eight of the larger basins (greater than 3 square miles) in the Atlanta area, that were included in this study, had two or more rain gages. The daily rainfall from the additional gages is combined into one daily record by applying coefficients, as suggested by Thiessen (1911), to each rain-gage record. Unit rainfall was combined into one record by a method described by Inman (1988). Thiessen (1911) coefficients are determined for each rain-gage record, and a total Thiessen weighted rainfall for the resulting flood is computed. A ratio of the Thiessen weighted total rainfall to the total rainfall at the gage having the largest Thiessen coefficient, is multiplied by each 5-minute increment of rainfall at the gage having the largest Thiessen weight to provide one record of weighted-unit rainfall. This method of combining unit rainfall is used to maintain the integrity of the individual increments. Weighting unit rainfall in the same manner as daily rainfall tends to have a smoothing effect on the incremental rainfall; and therefore, is not used.

Verification

Verification is the procedure where estimates of the dependent variables computed by the calibrated RRM are compared to observed data different than the observed data used for calibration. The RRM parameters are considered acceptable (verified) if the mean square error obtained during the verification process falls within preselected acceptable values. The use of part of the data from a basin for calibration, and a different part for verification, is referred to as split-sample testing and is the primary basis to assess the accuracy of the RRM for purposes of prediction.

Table 5.--Optimized rainfall-runoff model parameter values for each study site, by city

[RRM, rainfall-runoff model; parameters are defined in table 4; parameters DRN and TP/TC are assigned fixed values of 1.00 and 0.50, respectively, for all stations and not optimized; parameter EVC is assigned a fixed value of 0.77 for the Savannah area, and 0.75 for all other areas and not optimized; SE, standard error of estimate of calibration results, based on the mean-square difference of logs of observed and synthesized peaks]

Station number	RRM infiltration, soil-moisture accounting, and surface-runoff routing parameters								SE, in percent
	PSP	KSAT	RGF	BMSM	RR	KSW	TC	TIA	
					Albany				
02352605	2.75	0.175	29.6	5.85	0.880	1.80	82.0	28.8	28.4
02352964	2.99	.241	13.0	11.7	.700	1.50	100	11.0	35.5
					Athens				
02217505	1.53	.150	28.2	2.29	.947	.678	63	40.2	20.6
02217506	2.51	.146	37.8	3.27	.950	.330	22	31.0	29.7
02217730	2.45	.125	26.2	2.20	.950	.780	26	35.6	22.4
02217750	.71	.1435	10.2	3.28	.948	.500	35	38.1	22.5
02217905	2.99	.173	22.8	3.94	.90	.22	15	61.6	23.2
02217990	.70	.091	37.7	3.01	.950	.600	36	33.7	22.0
					Atlanta				
02203820	1.01	.143	23.1	7.70	.918	2.00	210	30.5	29.7
02203835	1.95	.121	12.8	3.20	.831	1.00	90	25.6	26.7
02203845	1.47	.139	10.0	2.83	.948	.45	70	30.6	26.1
02203850	1.32	.115	10.0	6.85	.900	1.44	193	28.2	19.3
02203870	2.49	.108	7.9	2.41	.950	1.35	120	25.8	26.0
02203884	1.13	.165	10.0	5.00	.900	.74	116	26.7	29.3
02336080	.65	.114	32.5	2.57	.942	3.50	475	31.4	29.1
02336090	.63	.180	28.0	6.30	.903	.45	45	19.0	36.9
02336102	.96	.103	47.3	2.24	.958	.64	156	27.2	24.4
02336150	.90	.100	14.4	2.06	.950	1.85	200	24.1	26.2
02336180	1.02	.081	18.4	5.69	.940	2.92	540	25.9	25.9
02336200	1.26	.137	7.3	6.75	.922	0.64	48	32.3	23.5
02336238	1.83	.117	31.4	11.30	.930	0.45	35	33.6	26.0
02336325	.81	.112	25.0	3.56	.903	1.10	56	42.0	21.2
02336690	.93	.104	21.2	4.11	.949	.78	42	20.3	26.3
02336697	.97	.086	36.3	4.46	.947	.50	30	19.1	26.2
02336700	2.23	.114	10.3	6.43	.949	.80	40	28.3	28.1
02336705	.97	.098	23.6	5.92	.926	1.15	130	29.5	28.3
02337081	1.16	.210	15.3	5.74	.915	.60	35	28.6	23.7
					Augusta				
02196570	1.52	.097	40.0	2.09	.944	.92	51	19.9	35.5
02196605	1.69	.198	24.6	2.26	.736	.40	24.5	28.1	22.9
02196725	1.18	.195	34.2	4.39	.851	2.30	110.0	42.4	28.5
02196730	1.78	.245	22.3	2.27	.925	3.10	190.0	33.4	25.9
02196760	1.35	.250	39.9	3.60	.806	.65	74.2	23.0	27.9
02196850	1.17	.080	11.2	4.96	.752	.32	10.0	28.5	27.6

Table 5.--Optimized rainfall-runoff model parameter values for each study site, by city

[RRM, rainfall-runoff model; parameters are defined in table 4; parameters DRN and TP/TC are assigned fixed values of 1.00 and 0.50, respectively, for all stations and not optimized; parameter EVC is assigned a fixed value of 0.77 for the Savannah area, and 0.75 for all other areas and not optimized; SE, standard error of estimate of calibration results, based on the mean-square difference of logs of observed and synthesized peaks]

Station number	RRM infiltration, soil-moisture accounting, and surface-runoff routing parameters								SE, in percent
	PSP	KSAT	RGF	BMSM	RR	KSW	TC	TIA	
					Columbus				
02341542	2.00	0.150	8.3	3.33	0.800	4.20	150	0.98	37.8
02341544	1.18	.140	38.9	6.95	.950	.83	30	17.6	24.2
02341546	1.20	.200	10.0	5.90	.900	1.00	75	16.0	30.0
02341548	1.57	.214	10.0	5.02	.950	.95	92	16.8	28.8
					Moultrie				
02318565	2.34	.310	32.0	14.3	.833	1.03	54	23.2	20.3
02327202	1.02	.066	21.3	3.28	.882	.91	68	33.9	25.4
02327203	2.01	.193	12.7	5.20	.892	.78	49	21.0	28.1
02327204	.71	.089	33.8	3.97	.784	1.33	79	22.3	20.9
					Rome				
02395990	.981	.167	26.7	5.31	.950	.90	50	16.6	27.5
02396290	2.514	.206	39.6	4.52	.825	1.20	80	6.6	30.4
02396510	.23	.071	36.0	15.0	.911	.55	45	17.4	32.5
02396515	1.16	.125	38.6	4.60	.946	.85	125	18.3	26.6
02396550	1.44	.071	26.6	4.60	.911	.40	11	18.8	27.2
02396680	.58	.074	23.4	5.74	.947	1.50	100	22.1	27.6
					Savannah				
02203541	2.19	.266	18.0	7.18	.702	.90	69	59.5	23.6
02203542	.80	.070	27.0	2.90	.913	4.00	250	19.5	30.0
02203543	2.87	.202	25.9	8.83	.936	2.25	150	29.7	24.4
02203544	1.04	.155	24.2	7.80	.945	.85	65	25.9	28.3
					Thomasville				
02326182	1.15	.356	9.39	3.01	.834	.14	13	16.4	22.1
02327467	.87	.119	39.5	3.87	.816	1.50	110	20.9	29.3
02327468	1.28	.107	30.6	8.23	.83	1.84	113	25.6	27.4
02327471	3.00	.444	39.8	14.9	.862	.37	22	42.4	22.3
02327473	1.53	.165	31.0	5.16	.890	.70	35	26.6	33.1
02327474	1.76	.112	32.5	7.03	.744	.43	33	6.1	33.4
					Valdosta				
02317564	2.17	.114	37.9	4.79	.818	2.85	155	22.3	21.9
02317566	.84	.121	40.0	3.98	.943	5.51	337	20.4	28.7
023177551	1.18	.070	39.5	4.72	.827	1.00	75	20.7	33.7
023177553	.91	.066	23.7	5.20	.916	1.40	73	28.7	25.8
023177554	1.50	.168	39.8	4.13	.793	1.25	65	29.8	28.2
023177556	.90	.110	25.6	3.90	.936	.42	35	11.8	20.4
023177557	1.35	.162	16.0	6.82	.949	1.24	75	27.1	21.2
023177558	2.87	.225	22.3	9.73	.743	1.00	40	34.4	26.9

The RRM was verified at six of the Atlanta area sites by split-sample testing. Flood events at each site were divided into two samples. The flood events were arranged in descending order according to peak magnitude. The odd-numbered events made up the first sample and the even-numbered events the second sample. RRM was recalibrated using only the flood events in the first sample. The computed peak discharges for the second sample were compared with the observed data, and the standard error of estimate was computed. The results (table 6) were considered to be acceptable (within about 30 percent) and additional split-sample testing was not necessary.

Table 6.—U.S. Geological Survey rainfall-runoff model (RRM) split-sample test results for peak discharges for six selected sites
[RRM from Bergman, Inman, and Lumb, U.S. Geological Survey, written commun., 1990]

Station number	Standard error of estimate of calibration and verification results for peak discharges (in percent)		
	Calibration (all events)	Calibration (odd-numbered events)	Verification (even numbered events)
02336080	± 25	± 27	± 27
02336238	± 26	± 25	± 27
02336325	± 26	± 30	± 26
02336690	± 23	± 25	± 22
02336697	± 33	± 21	± 34
02336705	± 22	± 22	± 30

Flood-Frequency Analysis

The calibrated RRM is run with NWS long-term precipitation (table 2) and evaporation data (table 3) to simulate annual peaks for each of the 65 stations used in the study. Because Atlanta, Augusta, Savannah, and Thomasville have long-term rainfall stations located in or near each city, the long-term rainfall data are used directly with the nearby rainfall-runoff sites. Macon is the only NWS station close to Columbus with a long enough period of record to be used to simulate annual peaks. Thomasville-Coolidge is the only NWS station close to Albany, Moultrie, and Valdosta with a long enough period of record to be used to simulate annual peaks. The sites in Rome and Athens have two long-term stations, each for use in peak simulation. Rainfall-frequency isopluvial maps prepared by the NWS (1961) are used as a guide in selecting weighting values for each of the long-term stations. For Rome, a weight of 0.60 is given to frequency curves generated from Atlanta long-term rainfall record, and a weight of 0.40 is given to frequency curves generated from Chattanooga long-term rainfall record. For Athens, a weight of 0.50 was given to frequency curves generated from Atlanta long-term rainfall record, and a weight of 0.50 to frequency curves generated from Augusta long-term rainfall record.

The Pearson Type III frequency distribution is fit to the logarithms of the annual peak discharges at each site in accordance with “Guidelines for Determining Flood Flow Frequency”, Bulletin 17B (Interagency Advisory Committee on Water Data (IACWD), 1982) recommendations. These recommendations include the proper handling of low and high outliers. Frequency curves for flood peaks simulated by the RRM represent an “as is” storage condition that may be present at upstream roadway embankments with culverts of limited capacity, or minor flood plain storage.

Skew coefficients are computed directly from the simulated data. No attempt was made to adjust the skew coefficients of the frequency curves, because the data did not meet the criteria specified in the IACWD (1982). The generalized skew-coefficient map in IACWD (1982), used in the adjustment computations, is for rural watersheds; and therefore, is not applicable to the simulated urban flood peaks.

Twenty-one of the 65 sites had 10 or more years of observed record. However, no attempt was made to combine the observed flood-frequency data with the simulated flood-frequency data because:

- the 100-year flood from 19 of the 21 stations is less than the 100-year flood from the simulated data;
- the 100-year flood from 10 of the 21 stations is less than the equivalent rural 100-year flood—estimated using equations from Stamey and Hess (1993); and
- at 64 of the 65 stations, the two highest simulated peaks occurred well before the observed record began.

It was therefore concluded that the period of observed record for the 65 stations in this study was a relatively dry period. Flood-frequency data from the log-Pearson Type III frequency analyses for selected recurrence intervals are shown in table 7.

Table 7.—Flood-frequency data from long-term synthesis for Albany, Athens, Atlanta, Augusta, Columbus, Moultrie, Rome, Savannah, Thomasville, and Valdosta stations

Station number	Drainage area (in square miles)	Peak-discharge data, in cubic feet per second, for indicated recurrence interval, in years							
		2-year	5-year	10-year	25-year	50-year	100-year	200-year	500-year
					Albany				
02352605	0.16	38.5	58.1	71.4	88.5	101	114	127	144
02352964	.05	7.2	12.7	16.9	22.8	27.7	32.8	38.4	46.3
					Athens				
02217505	1.44	488	770	973	1,250	1,460	1,690	1,920	2,250
02217506	.19	79.2	130	168	223	268	317	370	446
02217730	.30	116	176	220	278	322	368	416	484
02217750	.35	180	280	349	440	509	580	653	752
02217905	.42	335	500	616	770	889	1,010	1,140	1,320
02217990	.30	152	228	283	355	410	467	527	610
					Atlanta				
02203820	8.67	1,310	2,050	2,560	3,210	3,700	4,190	4,670	5,320
02203835	3.43	996	1,610	2,020	2,540	2,920	3,300	3,670	4,150
02203845	.84	411	646	801	995	1,130	1,270	1,400	1,580
02203850	7.50	1,660	2,500	3,040	3,710	4,190	4,660	5,120	5,720
02203870	3.68	900	1,380	1,710	2,140	2,450	2,770	3,080	3,510
02203884	1.88	652	1,020	1,270	1,580	1,800	2,020	2,240	2,510
02336080	19.10	2,060	3,010	3,680	4,550	5,230	5,930	6,650	7,640
02336090	.32	149	243	304	376	426	473	517	572
02336102	2.19	592	932	1,170	1,460	1,680	1,900	2,120	2,420
02336150	5.29	1,120	1,640	1,990	2,410	2,720	3,030	3,330	3,730
02336180	11.00	1,210	1,800	2,200	2,720	3,110	3,510	3,910	4,460
02336200	.98	502	768	942	1,160	1,310	1,460	1,610	1,800
02336238	.92	416	675	859	1,100	1,290	1,480	1,670	1,940
02336325	1.35	532	789	957	1,170	1,320	1,480	1,630	1,830
02336690	.52	220	335	415	521	602	684	769	885
02336697	.21	106	165	207	262	305	349	395	458
02336700	.79	316	494	611	758	864	967	1,069	1,200
02336705	8.80	2,350	3,600	4,410	5,390	6,090	6,750	7,390	8,210
02337081	.88	360	583	729	909	1,040	1,160	1,280	1,420
					Augusta				
02196570	0.66	177	280	359	474	570	676	792	964
02196605	1.67	640	1,050	1,390	1,900	2,340	2,850	3,420	4,300
02196725	1.44	141	230	301	407	499	602	718	893
02196730	4.06	353	575	759	1,040	1,280	1,550	1,870	2,350
02196760	1.56	256	456	629	902	1,150	1,440	1,770	2290
02196850	.30	264	386	470	581	666	753	842	965

Table 7.—Flood-frequency data from long-term synthesis for Albany, Athens, Atlanta, Augusta, Columbus, Moultrie, Rome, Savannah, Thomasville, and Valdosta stations

Station number	Drainage area (in square miles)	Peak-discharge data, in cubic feet per second, for indicated recurrence interval, in years							
		2-year	5-year	10-year	25-year	50-year	100-year	200-year	500-year
					Columbus				
02341542	6.54	624	1,050	1,380	1,850	2,230	2,630	3,070	3,700
02341544	1.58	483	824	1,080	1,420	1,680	1,960	2,240	2,640
02341546	.26	78.6	126	162	212	254	298	346	416
02341548	1.42	358	599	784	1,040	1,260	1,480	1,720	2,070
					Moultrie				
02318565	.27	79.4	120	148	184	210	237	264	300
02327202	.48	280	377	440	520	579	638	697	776
02327203	.39	180	262	318	389	444	499	555	632
02327204	1.65	668	944	1,110	1,310	1,450	1,580	1,700	1,860
					Rome				
02395990	.37	107	174	216	268	305	340	373	415
02396290	.62	62.5	123	172	242	301	364	432	528
02396510	.04	25.6	37.5	45.4	55.4	62.7	69.8	76.8	86.2
02396515	.29	66.6	107	135	170	197	222	249	283
02396550	.19	120	184	228	282	324	365	408	463
02396680	1.31	394	563	675	814	917	1,020	1,120	1,260
					Savannah				
02203541	.24	108	158	193	240	276	313	351	404
02203542	1.27	237	365	456	577	672	770	872	1,010
02203543	.95	170	284	373	498	601	711	830	1,000
02203544	.18	85.1	135	168	207	235	261	286	318
					Thomasville				
02326182	.12	147	210	252	308	350	392	436	495
02327467	1.07	336	489	586	702	784	863	939	1,040
02327468	2.90	790	1,190	1,440	1,750	1,980	2,190	2,400	2,670
02327471	.21	117	168	202	246	278	311	345	390
02327473	1.04	531	780	951	1,170	1,340	1,510	1,680	1,920
02327474	.12	62.6	100	125	156	178	199	219	245
					Valdosta				
02317564	1.27	204	318	397	498	575	653	731	836
02317566	3.81	422	644	800	1,000	1,160	1,320	1,490	1,710
023177551	.16	70.1	100	120	144	162	179	196	219
023177553	.99	456	616	720	848	942	1,030	1,130	1,250
023177554	2.66	790	1,220	1,510	1,890	2,170	2,460	2,740	3,130
023177556	.16	128	180	213	254	283	312	341	378
023177557	.55	221	314	375	451	506	560	614	686
023177558	1.18	383	575	706	875	1,000	1,130	1,260	1,440

Regional Regression Analysis

So that flood magnitude and frequency could be estimated for ungaged sites, the 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year floods, obtained from the 65 urban basins in the study, are related to the basin characteristics of their origin. This is done by the generalized least-squares (GLS) regression method (Johnston, 1972). Further information on GLS applications can be obtained from Stedinger and Tasker (1985), Tasker and others (1986), Livingston and Minges (1987), and Tasker and Stedinger (1989).

GLS is used in this study because simulated rainfall-runoff records generated from the same historical rainfall and evaporation data are highly correlated and because GLS reduces the weight given to sites having high correlation. GLS should also be used (1) if the site-to-site variances of the streamflow characteristics are not similar, and (2) for stations with different record lengths. In this study, all flood-frequency estimates are based on about the same length of rainfall record, so little was gained by the GLS analysis from this aspect.

Regression equations provide a mathematical relation between response variables (2- to 500-year flood peaks) and explanatory variables (basin characteristics). All variables are transformed into logarithms before analysis to insure a linear-regression model and to achieve equal variance about the regression line throughout the range of explanatory variables (Riggs, 1968). In the analyses performed, 95-percent confidence limits are used to evaluate the statistical significance of independent variables.

Basin characteristics used in this analysis are defined below and individual station data are shown in table 8.

Drainage area (A).—Area of the basin, in square miles, planimetered from USGS 7 1/2-minute topographic maps. All basin boundaries were field checked.

Channel slope (S).—The main-channel slope, in feet per mile (ft/mi), 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, with L and S defined above.

Total impervious area (TIA).--The percentage of drainage area that is impervious to infiltration of rainfall. This parameter is determined from aerial photography (U.S. Department of Agriculture, Agricultural Stabilization and Conservation Service) by use of a grid-overlay method. According to Cochran (1963), a minimum of 200 points, or grid intersections, per area or subbasin can 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 total impervious area. On several of the larger basins in the Atlanta area, where some development occurred during the period of data collection, this parameter was determined from aerial photographs made near the beginning of data collection, and then averaged with the values obtained from aerial photographs made near the end of data collection.

Rural regression discharge (RQ_T).—The peak discharge, in cubic feet per second (ft^3/s), for an equivalent rural drainage basin in the same hydrologic region as the urban basin, and for recurrence interval T. The equivalent rural discharges were computed from regression equations by Stamey and Hess (1993). The equations for computing RQ_t are given in table 9.

Table 9.—Regional flood-frequency relations for rural streams in Georgia

Flood discharge, Q_T, for T-year recurrence interval	Flood-frequency relations for indicated regions (fig. 1) in the form $Q_T=aA^b$, where A is the drainage area, in square miles, and a and b are as presented below			
	1	2	3	4
RQ_2	$207A^{0.654}$	$182A^{0.622}$	$76A^{0.620}$	$142A^{0.591}$
RQ_5	$357A^{0.632}$	$311A^{0.616}$	$133A^{0.620}$	$288A^{0.589}$
RQ_{10}	$482A^{0.619}$	$411A^{0.613}$	$176A^{0.621}$	$410A^{0.591}$
RQ_{25}	$666A^{0.605}$	$552A^{0.610}$	$237A^{0.623}$	$591A^{0.595}$
RQ_{50}	$827A^{0.595}$	$669A^{0.607}$	$287A^{0.625}$	$748A^{0.599}$
RQ_{100}	$1,010A^{0.584}$	$794A^{0.605}$	$340A^{0.627}$	$926A^{0.602}$
RQ_{200}	$1.220A^{0.575}$	$931A^{0.603}$	$396A^{0.629}$	$1,120^{0.606}$
RQ_{500}	$1,530A^{0.563}$	$1,130A^{0.601}$	$474A^{0.632}$	$1,420^{0.611}$

Preliminary regression analyses were performed using procedures defined by SAS Institute, Inc., (1989). The two specific SAS analyses performed were (1) REG-estimate parameters within confidence limits, and (2) GLM-plots of predicted and observed peak discharges, and plots of residuals as a function of the significant variables. Additional information on the methods is available from the SAS Institute, Inc., (1989). The preliminary regression results indicates that the most significant variables are drainage area, total impervious area, and rural regression discharge.

The preliminary results, also, indicated that the residuals for the Rome sites were consistently negative, meaning that the observed was less than the predicted. Therefore, a qualitative variable was created to account for the apparent bias in the Rome sites. This log-transformed qualitative variable (QV) is one (1) if the site is in Rome, and zero (0) otherwise. The preliminary and final equations were rewritten by adjusting the constant and producing a set of equations without the qualitative variable for Rome only.

Table 8.—Basin characteristics for Statewide urban study sites and estimated peak discharges for equivalent rural basins

[A, drainage area, in square miles; L, channel length, in miles; S, channel slope, in feet per mile; $L/S^{0.5}$, a ratio, where L and S have been previously defined; TIA, area that is impervious to infiltration of rainfall, in percent; R, flood-frequency region where the basin is located (Stamey and Hess, 1993); $RQ_{2\sim500}$, peak discharge, in cubic feet per second, for an equivalent rural drainage basin in the same hydrologic area as the urban basin, and for 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence intervals using the Stamey and Hess (1993) equation]

Station number	Basin characteristics						Estimated rural peak discharges							
	A	L	S	$L/(S^{0.5})$	TIA	R	RQ_2	RQ_5	RQ_{10}	RQ_{25}	RQ_{50}	RQ_{100}	RQ_{200}	RQ_{500}
							Albany							
02352605	0.16	0.58	33.3	0.10	28.8	3	24	43	56	76	91	108	125	149
02352964	.05	.36	22.5	.08	11.1	3	11	20	26	35	42	49	57	68
							Athens							
02217505	1.44	1.89	91.6	.20	40.2	2	228	389	514	690	835	909	1,160	1,410
02217506	.19	.75	214	.05	31.0	2	65	112	148	200	244	291	342	416
02217730	.30	.70	106	.80	35.6	2	86	148	196	265	322	383	450	548
02217750	.35	.90	122	.08	38.1	2	95	163	216	291	354	421	494	601
02217905	.42	.76	158	.07	61.6	2	106	182	241	325	395	470	552	671
02217990	.30	1.04	102	.10	33.7	2	86	148	196	265	322	383	450	548
							Atlanta							
02203820	8.67	7.58	28.0	1.43	30.5	2	697	1,180	1,540	2,060	2,480	2,930	3,420	4,140
02203835	3.43	2.66	61.0	.34	25.6	2	392	664	875	1,170	1,410	1,670	1,960	2,370
02203845	.84	1.93	67.6	.24	30.6	2	163	279	369	496	602	715	838	1,020
02203850	7.50	5.91	34.8	1.00	28.2	2	637	1,080	1,410	1,890	2,270	2,690	3,140	3,800
02203870	3.68	3.95	37.5	.64	25.8	2	409	669	914	1,220	1,480	1,750	2,040	2,470
02203884	1.88	2.22	74.1	.26	26.7	2	269	459	605	811	981	1,160	1,360	1,650
02336080	19.0	7.43	16.0	1.86	31.4	1	1,420	2,300	2,990	3,970	4,780	5,650	6,650	8,050
02336090	.32	1.12	129	.10	19.0	1	98	174	238	334	420	520	634	806
02336102	2.19	2.50	62.8	.32	27.2	1	346	586	783	1,070	1,320	1,600	1,910	2,380
02336150	5.29	5.06	25.8	1.00	24.1	1	615	1,020	1,350	1,830	2,230	2,670	3,180	3,910
02336180	11.0	9.03	19.0	2.07	25.9	1	993	1,620	2,130	2,840	3,440	4,100	4,840	5,900
02336200	.98	1.47	94.5	.15	32.3	1	204	352	476	658	817	998	1,210	1,510
02336238	.92	1.60	106	.16	33.6	1	196	339	458	633	787	962	1,163	1,460
02336325	1.35	2.14	53.8	.29	42.0	1	252	432	580	799	989	1,200	1,450	1,810
02336690	.52	1.22	90.7	.13	20.3	1	135	236	322	448	560	689	838	1,060
02336697	.21	1.09	136	.09	19.1	1	75	133	183	259	327	406	497	635
02336700	.79	1.46	75.8	.17	28.3	1	177	308	417	577	719	880	1,070	1,340
02336705	8.80	4.95	33.7	.85	29.5	1	858	1,410	1,850	2,480	3,020	3,600	4,260	5,200
02337081	.88	1.43	86.9	.15	28.6	1	190	329	445	616	766	937	1,130	1,420
							Augusta							
02196570	0.66	1.67	96.0	0.17	19.9	2	140	241	319	428	520	617	725	880
02196605	1.67	1.86	117	0.17	28.1	2	250	427	563	755	913	1,080	1,270	1,540
02196725	1.44	2.67	118	0.25	42.4	3	95.0	167	22t	297	360	427	498	597
02196730	4.06	3.97	80.6	0.42	33.4	3	181	317	420	567	689	819	956	1150
02196760	1.56	2.07	111	0.20	23.0	3	100	175	232	313	379	449	524	628
02196850	0.30	1.06	239	0.07	28.5	2	86	148	196	265	322	383	450	548

Table 8.—Basin characteristics for Statewide urban study sites and estimated peak discharges for equivalent rural basins

[A, drainage area, in square miles; L, channel length, in miles; S, channel slope, in feet per mile; $L/S^{0.5}$, a ratio, where L and S have been previously defined; TIA, area that is impervious to infiltration of rainfall, in percent; R, flood-frequency region where the basin is located (Stamey and Hess, 1993); $RQ_{2\sim500}$, peak discharge, in cubic feet per second, for an equivalent rural drainage basin in the same hydrologic area as the urban basin, and for 2-, 5-, 10-, 25-, 50-, 100-, 200-, and 500-year recurrence intervals using the Stamey and Hess (1993) equation]

Station number	Basin characteristics						Estimated rural peak discharges							
	A	L	S	$L/(S^{0.5})$	TIA	R	RQ_2	RQ_5	RQ_{10}	RQ_{25}	RQ_{50}	RQ_{100}	RQ_{200}	RQ_{500}
							Columbus							
02341542	6.54	4.96	36.8	0.82	.98	2	585	989	1,300	1,740	2,090	2,470	2,890	3,490
02341544	1.58	2.20	69.7	.26	17.6	2	242	412	544	730	883	1,050	1,230	1,490
02341546	.26	1.06	81.8	.12	16.0	2	79	136	180	243	295	352	413	503
02341548	1.42	2.23	60.5	.29	16.8	2	226	386	510	684	828	982	1,150	1,400
							Moultrie							
02318565	.27	.80	60.3	.10	23.2	4	66	133	189	271	341	421	507	638
02327202	.48	1.02	45.6	.15	33.9	4	92	187	266	382	482	595	718	907
02327203	.39	.74	48.7	.11	21.0	4	80	163	231	332	419	517	623	786
02327204	1.65	2.10	25.4	.42	22.3	4	191	387	551	796	1,010	1,250	1,520	1,930
							Rome							
02395990	.37	.82	76.4	.08	16.6	1	108	190	260	365	458	565	689	874
02396290	.62	.97	98.6	.10	6.6	1	151	264	359	499	622	764	927	1,170
02396510	.04	.34	772	.01	17.4	1	26	49	69	99	127	161	200	260
02396515	.29	.72	257	.06	18.3	1	92	163	224	315	396	491	599	762
02396550	.19	.70	345	.04	18.8	1	70	125	172	244	308	383	470	601
02396680	1.31	2.41	55.2	.32	22.1	1	247	423	570	784	971	1,180	1,420	1,780
							Savannah							
02203541	.24	.84	17.5	.20	59.5	3	31	55	73	97	118	139	161	192
02203542	1.27	2.02	13.6	.55	19.5	3	88	154	204	275	333	395	460	551
02203543	.95	1.78	13.0	.49	29.7	3	74	129	170	230	278	329	383	459
02203544	.18	.51	29.9	.09	25.9	3	26	46	61	81	98	116	135	160
							Thomasville							
02326182	.12	.46	89.7	.05	16.4	4	41	83	117	167	210	258	310	389
02327467	1.07	1.65	31.5	.29	20.9	4	145	295	420	605	766	948	1,150	1,450
02327468	2.90	3.05	24.9	.61	25.6	4	268	542	774	1,120	1,420	1,770	2,150	2,740
02327471	.21	.60	82.2	.07	42.4	4	56	115	163	234	294	361	435	547
02327473	1.04	1.21	60.6	.16	26.6	4	145	295	420	605	766	948	1,150	1,450
02327474	.12	.60	110	.06	6.1	4	41	83	117	167	210	258	310	389
							Valdosta							
02317564	1.27	1.70	11.8	.49	22.3	3	88	154	204	275	333	395	460	551
02317566	3.81	3.69	9.39	1.20	20.4	3	174	305	404	545	662	787	918	1,100
023177551	.16	.74	23.4	.15	20.7	4	48	98	139	199	250	307	369	463
023177553	.99	1.52	26.3	.30	28.7	4	141	286	408	587	744	920	1,110	1,410
023177554	2.66	3.03	19.4	.69	29.8	4	253	512	731	1,060	1,340	1,670	2,030	2,580
023177556	.16	.49	40.8	.08	11.8	4	48	98	139	199	250	307	369	463
023177557	.55	.89	47.9	.13	27.1	4	100	203	288	414	523	646	780	985
023177558	1.18	1.52	41.2	.24	34.4	4	157	317	452	652	826	1,020	1,240	1,570

Regional Flood-Frequency Estimating Equations

Because the synthetic annual peak series exhibit strong correlations between sites based on a common rainfall record, ordinary least-squares (OLS) regression analysis is not an efficient method of estimating regression coefficients and their standard errors. Stedinger and Tasker (1985) and Tasker and Stedinger (1989) described a generalized least-squares (GLS) regression method that accounts for correlation in the dependent variables.

Application of the GLS regression method to urban streams in Georgia requires regional estimates of the standard deviations of the synthetic series of annual peaks at each site; estimates of the cross correlation coefficients of the annual peaks at each pair of sites; and an estimate of the effective record length at each site. The standard deviation of annual peaks were estimated from a regional regression of sample standard deviations of annual peaks against RQ_T, DA, and TIA. Cross-correlations of annual peaks are based on average cross correlations for sites based on a common rainfall record. These correlations are estimated to be 0.9 for sites within the same city.

For sites in different cities, the correlation between sites was estimated to be 0.0 except for sites in Atlanta, Rome, and Athens. The correlations between sites in Atlanta and sites in Rome are estimated as 0.45 because the Rome peak discharges were, in part, based on the Atlanta rainfall record. The correlations between sites in Atlanta and Athens and in Athens and Augusta are also estimated to be 0.45 because the Athens peak discharges are based on the Atlanta and Augusta rainfall records. Estimates of the effective record length for the synthetic record were computed based on methods described by Lichty and Liscum (1978). These estimated effective record lengths vary with recurrence interval as shown in table 10.

Table 10.—Estimated effective record lengths for 2- to 500-year recurrence intervals

Recurrence interval (in years)	Effective record lengths (in years)
2	5
5	9
10	14
25	19
50	21
100	21
200	21
500	21

The updated regional equations for the log-transformed model parameters for each recurrence interval are given in table 11. Because RQ_T is highly correlated with DA and is a function of only DA, the updated estimating equations can be expressed as functions of DA and TIA only, for each region. An example of the originally developed equation, with A, TIA, RQ_T, and QV for Rome is, $UQ_{10} = 1.59A^{0.15} TIA^{0.21} RQ_{10}{}^{0.90} QV^{-0.21}$. This equation can be expressed as functions of A and TIA only, by adjusting the constant and exponent of A to give an equation of the form $UQ_{10} = 249A^{0.70} TIA^{0.21}$. The Rome equations should be used only in the immediate Rome area, otherwise, in region 1, use region 1 equations. The equations in table 11 supersede the equations in the previous report "Flood-Frequency Relations for Urban Streams in Georgia" (Inman, 1988), and the equations in the report "Flood-Frequency Relations for Urban Streams in Metropolitan Atlanta, Georgia" (Inman, 1983).

Testing of Regression Equations

Two tests or evaluations generally are required to establish the soundness of regression equations. These tests are *Bias* and *Sensitivity*, as explained below.

Bias

Two tests for bias are performed, one for variable bias and the other for geographical bias. The variable-bias tests are made by plotting the residuals (difference between observed and predicted floods) against each of the independent variables for all stations. The plots are made during the preliminary OLS regression analysis. These plots were visually inspected to determine whether there was a consistent over-prediction or under-prediction within the range of any of the independent variables. These plots also verified the linearity assumptions of the equations. On the basis of visual inspection of the plots, the equations are free of variable bias throughout the range of independent variables.

Geographical bias is tested by determining the number of positive and negative residuals at sites in a city. Although some cities do have a majority of negative or positive residuals, the Wilcoxin Signed Ranks test, as described by Tasker (1982), when applied to the residuals in each of the 10 cities, indicates that the estimated peak discharges are not biased.

Sensitivity

The second test analyzes the sensitivity of 2-, 25-, and 100-year computed discharges to errors in the two independent variables in the estimating equations. The test results (table 12) are computed by using a constant value for all independent variables except the one being tested for sensitivity. The sensitivity of the region 1 equations is the only one tested because TIA has the same exponent in each region, and the exponent for A changes by only a small amount.

Table 11.—Regional flood-frequency equations for urban streams in Georgia

[UQ_T, peak discharge for an urban drainage basin, in cubic feet per second; A, drainage area, in square miles; TIA, area that is impervious to infiltration of rainfall, in percent; ±, plus-minus]

UQ_T recurrence (in years) interval	Flood-frequency estimating equations (region 1)	Average standard error of prediction (in percent)	Flood-frequency estimating equations (Rome)	Average error of standard prediction (in percent)	Flood-frequency estimating equations (region 2)	Average standard error of prediction (in percent)	Flood-frequency estimating equations (region 3)	Average error of standard prediction (in percent)	Flood-frequency estimating equations (region 4)	Average standard error of prediction (in percent)
2	$167A^{0.73}$ $T1A^{031}$	±34	$107A^{0.73}T1A^{0.31}$	±40	$145A^{0.70}$ $TIA^{0.31}$	±35	$54.6A^{0.69}$ $TIA^{0.31}$	±34	1 $10A^{0.66}$ $TIA^{0.31}$	34
5	$301A^{0.71}$ $TIA^{0.26}$	±31	$183A^{0.71}T1A^{0.26}$	±36	$258A^{0.69}$ $TIA^{0.26}$	±31	$99.7A^{0.69}$ $TIA^{0.26}$	±31	$237A^{0.66}$ $TIA^{0.26}$	31
10	$405A^{0.70}$ $TIA^{0.21}$	±31	$249A^{0.70}T1A^{021}$	±35	35 $lA^{0.70}$ $T1A^{0.21}$	±31	164 $A^{0.71}$ $T1A^{0.21}$	±32	$350A^{0.68}$ $TIA^{0.21}$	30
25	$527A^{0.70}$ $TIA^{0.20}$	±29	$316A^{0.70}T1A^{0.20}$	±33	$452A^{0.70}$ $T1A^{0.20}$	±29	226 $A^{0.71}$ $T1A^{0.20}$	±30	$478A^{0.69}$ $TIA^{0.20}$	29
50	$643A^{0.69}$ $TIA^{0.18}$	±28	$379A^{0.69}$ $TIA^{0.18}$	±33	$548A^{0.70}$ $TIA^{0.18}$	±29	288 $A^{0.72}$ $TIA^{0.18}$	±30	$596A^{0.70}$ $TIA^{0.18}$	28
100	$762A^{0.69}$ $TIA^{0.17}$	±28	$440A^{0.69}$ $TIA^{0.17}$	±33	$644A^{0.70}$ $TIA^{0.17}$	±29	355 $A^{0.72}$ $TIA^{0.17}$	±30	$717A^{0.70}$ $TIA^{0.17}$	28
200	$892A^{0.68}$ $TIA^{0.16}$	±28	$505A^{0.68}$ $TIA^{0.16}$	±34	$747A^{0.70}$ $TIA^{0.16}$	±28	428 $A^{0.72}$ $TIA^{0.16}$	±30	$843A^{0.70}$ $TIA^{0.16}$	28
500	$1063A^{0.68}$ $TIA^{0.14}$	±28	$589A^{0.68}$ $TIA^{0.14}$	±34	$888A^{0.70}$ $TIA^{0.14}$	±28	531 $A^{0.72}$ $TIA^{0.14}$	±30	$1017A^{0.71}$ $TIA^{0.14}$	28

Table 12.—Sensitivity of computed peak discharges to errors in independent variables in the 2-, 25-, and 100-year estimating equations

[A, drainage area, in square miles; TIA, area that is impervious to infiltration of rainfall, in percent]

Percent error in independent variable	Independent variables					
	Percent error in computed 2-yr flood		Percent error in computed 25-yr flood		Percent error in computed 100-yr flood	
	A	TIA	A	TIA	A	TIA
+50	+34.4	+13.4	+32.8	+8.5	+32.3	+7.1
+25	+17.7	+7.1	+16.9	+4.6	+16.7	+3.8
+10	+ 7.2	+3.1	+7.0	+2.0	+6.8	+1.6
-10	-7.3	-3.2	-7.1	-2.1	-7.0	-1.8
-25	-19.1	-8.3	-18.2	-5.6	-18.0	-4.8
-50	-39.8	-19.3	-38.5	12.9	-38.0	-11.1

Standard Error of Prediction

One measure of how good the GLS regression model is for prediction is the average standard error of prediction (table 12) which is the error expected two thirds of the time when averaged over watersheds similar to those used in the analysis. For further information on this statistic, refer to Stedinger and Tasker (1985).

Use of Flood-Frequency Relations

Flood-peak discharges at specific recurrence intervals can be estimated by locating the drainage basin in one of the hydrologic regions (figs. 2-5), determining drainage area, impervious area, and using the appropriate equation from table 11. The ranges of basin variables listed below should not be exceeded.

A comparison with the equivalent rural peak discharge also is helpful for small values of total impervious area. If the equivalent rural peak discharge exceeds the peak computed from the urban equations, then use the equivalent rural peak discharge. In the immediate Rome area, the urban equations may very well compute peak discharges that are less than the equivalent rural peak discharges. It is left to the discretion of the user, based on their hydrologic judgement and knowledge of the area, to decide which computed peak discharge to use. The user is also cautioned that the equations presented in this report are applicable only to basins having insignificant surface storage, and insignificant embankment storage.

The ranges of basin variables used in the estimating equations presented in this report are listed below.

Variable	Minimum	Maximum	Units
A	0.04	19.1	in square miles
TIA	1.00	62	in percent

SUMMARY

Rainfall-runoff data were collected at 65 urban basins in 10 urban areas of Georgia ranging in size from 0.04 to 19.1 square miles and in total impervious area from about 1 to 62 percent. Extensive field reconnaissance was required to select the 65 basins used in this study. Many sites were inspected for possible use. A range in drainage area, main channel slope, and channel length also were considered. Another very important factor was land-use stability. Each site has a stage gage and at least one rain gage equipped with digital recorders with 5-minute punch intervals. All flood events with complete rain and stage data and without culvert blockages were processed and loaded into U.S. Geological Survey (USGS) computer storage. The USGS rainfall-runoff model (RRM) is calibrated for the 65 basins, and verified by split-sample testing at six basins.

After the RRM is successfully calibrated, long-term rainfall and daily pan-evaporation data from the appropriate U.S. Department of Commerce, National Weather Service (NWS) stations are used to synthesize about 60 to 90 years of annual peak-discharge data. The synthesized peaks are used to develop flood-frequency relations at each site. GLS multiple-regression analysis is used to define relations between the flood-frequency data and selected basin characteristics, of which drainage area, measured total impervious area, and rural regression discharge are statistically significant. Tests indicate that the equations are not parametrically or geographically biased. Estimates of magnitude and frequency of urban peak discharges at ungaged sites throughout Georgia can be determined for the 2- to 500-year recurrence intervals by using these equations. Average standard errors of prediction of the five regional equations ranged from ± 40 percent at the 2-year recurrence interval to ± 28 percent at the 500-year recurrence interval. The use of the regional equations is limited to basins within the range of physical characteristics listed.

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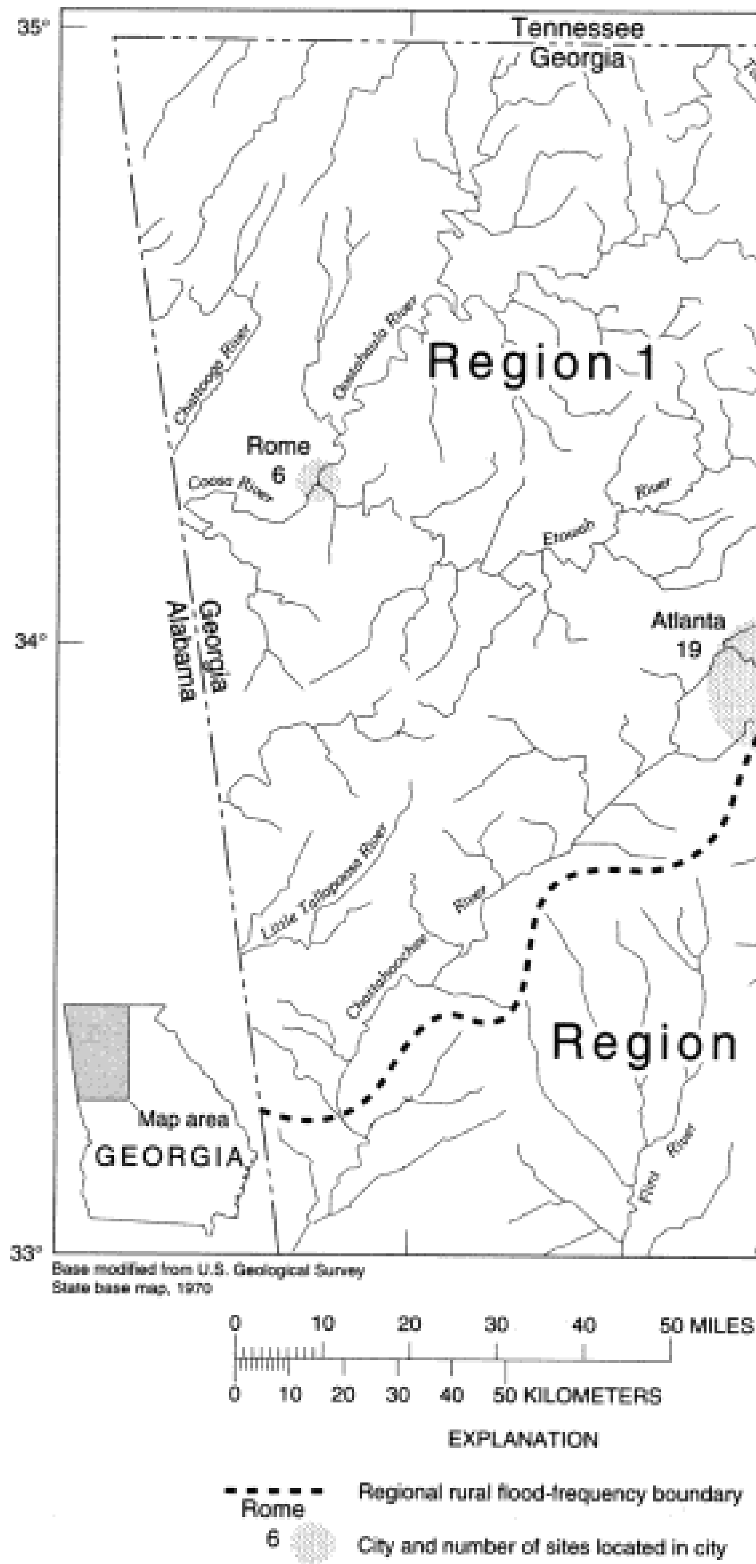

Figure 2. Regional rural flood-frequency boundaries and cities and number of urban sites in northwest Georgia.

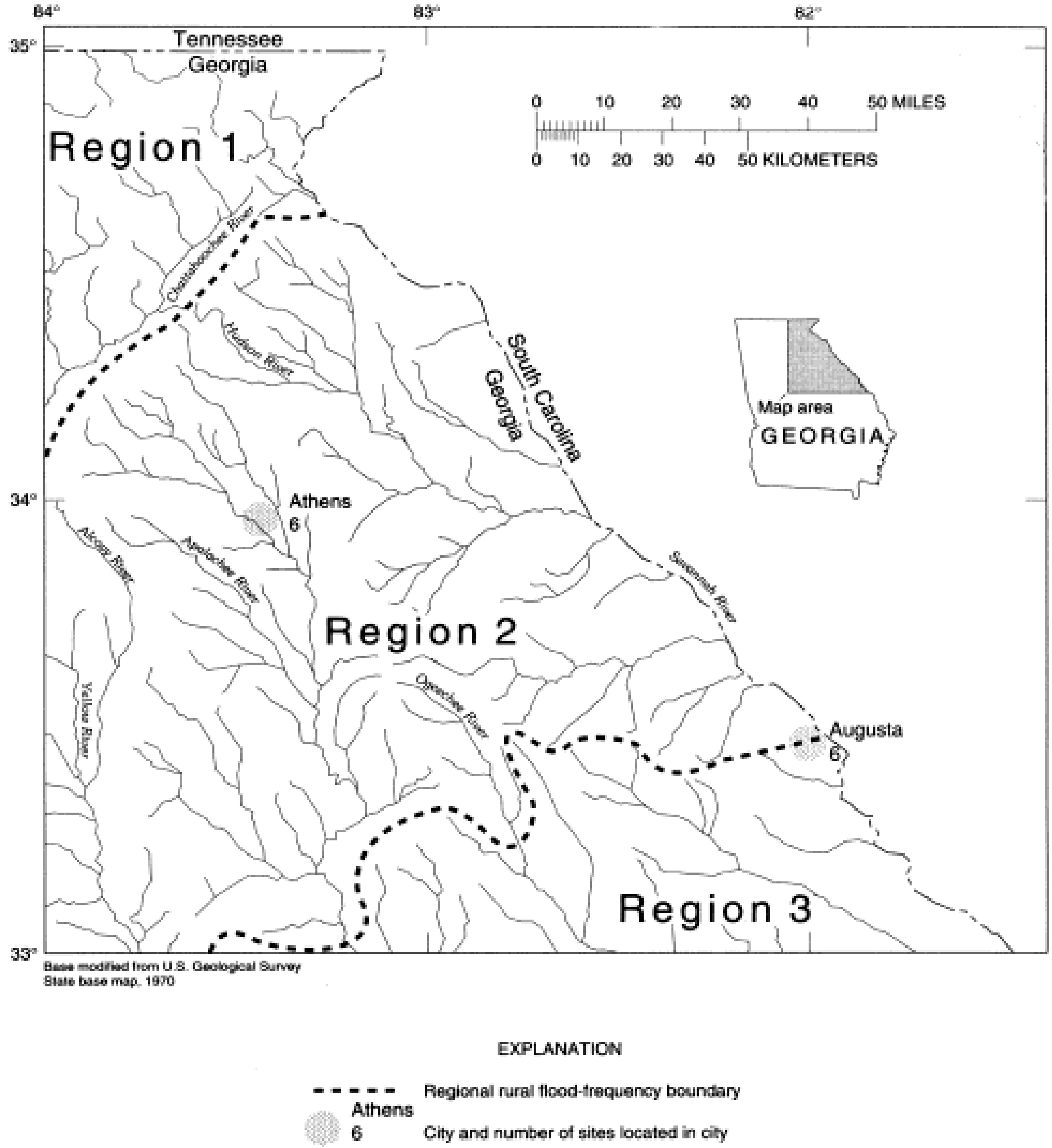

Figure 3. Regional rural flood-frequency boundaries and cities and number of urban sites in northeast Georgia.

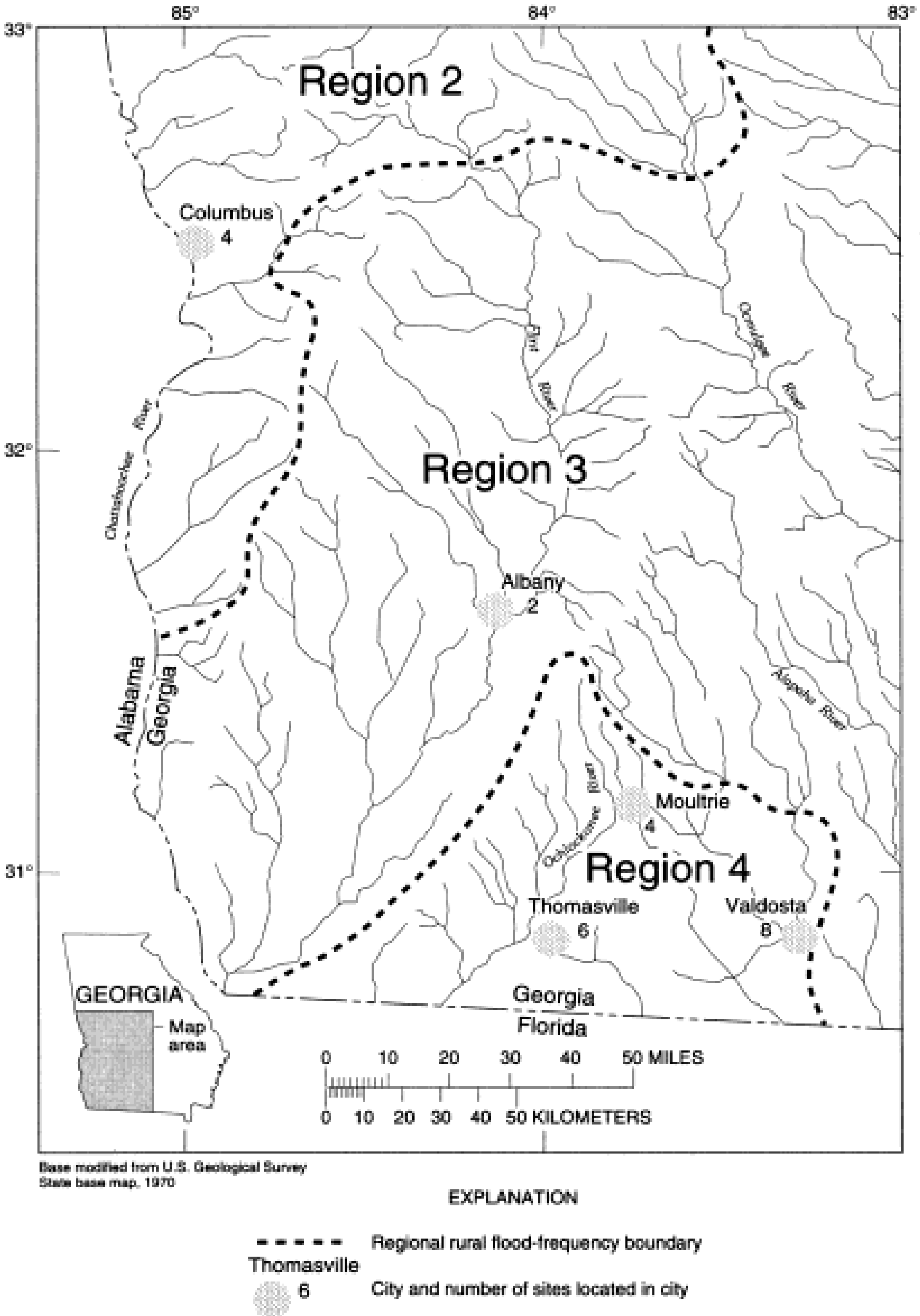

Figure 4. Regional rural flood-frequency boundaries and cities and number of urban sites in southwest Georgia.

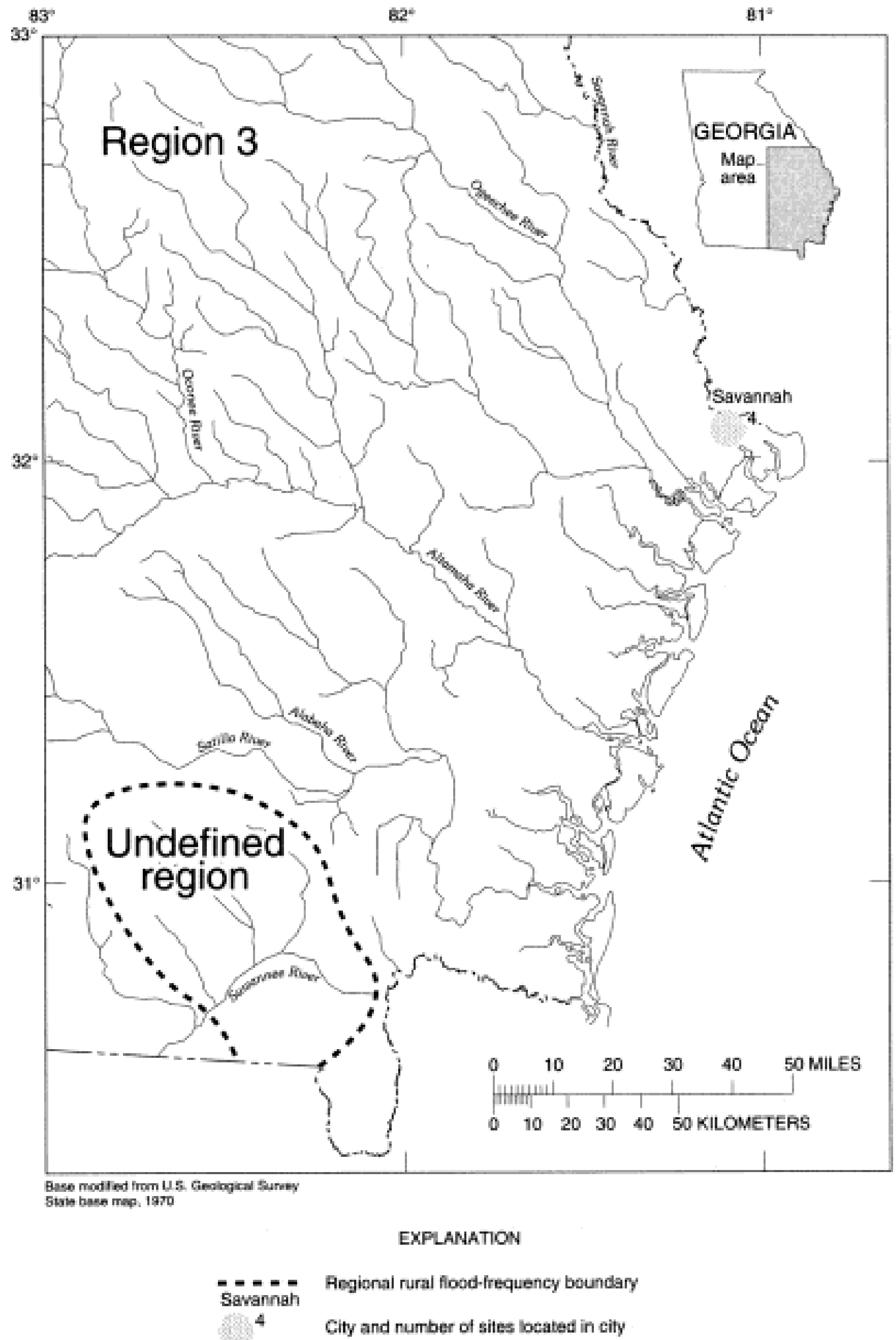

Figure 5. Regional rural flood-frequency boundaries and cities and number of urban sites in southeast Georgia.

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Table 1.—Gaging stations in the Statewide urban study, by city, 1994

Station number[1/]	Station name	Location
	Albany	
02352605	Flint River tributary 1, at Albany	Lat 31°32'52", long 84°09'28", Dougherty County, at culvert on Emily Avenue, at Albany
02352964	Percosin Creek tributary, at Albany	Lat 31°35'47", long 84°14'03", Dougherty County, at culvert on Dean's Road, at Albany
	Athens	
02217505	Brooklyn Creek, at Athens	Lat 33°56'32", long 83°24'07", Clarke County, at culvert on Dudley Drive, at Athens
02217506	Brooklyn Creek tributary, at Athens	Lat 33°56'26", long 83°23'48", Clarke County, at culvert on McWhorter Road, at Athens
02217730	Tributary of North Oconee River tributary no. 2, at Athens	Lat 33°58'16", long 83°23'59", Clarke County, at culvert on U.S. Highway 29, at Athens
02217750	North Oconee River tributary, at Athens	Lat 33°58'11", long 83°23'14", Clarke County, at culvert on Barber Street, at Athens
02217905	Tanyard Creek, at Athens	Lat 33°57'05", long 83°22'42", Clarke County, at culvert on Baxter Street, at Athens
02217990	Cedar Creek tributary, near Whitehall	Lat 33°55'02", long 83°20'05", Clarke County, at culvert on Forest Road, near Whitehall
	Atlanta	
02203820	Sugar Creek, near Atlanta	Lat 33°41'41", long 84°18'15", DeKalb County, at culvert on Clifton Church Road, near Atlanta
02203835	Shoal Creek, near Atlanta	Lat 33°44'48", long 84°16'50", DeKalb County, at culvert on Line Street, near Atlanta
02203845	Shoal Creek tributary, near Atlanta	Lat 33°43'05", long 84°15'45", DeKalb County, at culvert on Glendale Drive near Atlanta
02203850	Shoal Creek, near Atlanta	Lat 33°42'36", long 84°15'57", DeKalb County, at culvert on Rainbow Drive, near Atlanta
02203870	Cobbs Creek, near Atlanta	Lat 33°43'44", long 84°14'17", DeKalb County, at culvert on Snapfinger Road, near Atlanta
02203884	Conley Creek, near Forest Park	Lat 33°38'08", long 84°20'38", Clayton County, at culvert on Rock Cut Road, near Forest Park
02336080	North Fork Peachtree Creek, near Chamblee	Lat 33°51'43", long 84°17'13", DeKalb County, at culvert on Shallowford Road, near Chamblee
02336090	North Fork Peachtree Creek tributary, near Chamblee	Lat 33°50'53", long 84°17'57", DeKalb County, at culvert on Meadowcliff Drive, near Chamblee
02336102	North Fork Peachtree Creek tributary, near Atlanta	Lat 33°51'20", long 84°19'19", DeKalb County, at culvert on Drew Valley Road, near Atlanta
02336150	South Fork Peachtree Creek, at Clarkston	Lat 33°48'51", long 84°14'38", DeKalb County, at culvert on Montreal Road, at Clarkston
02336180	South Fork Peachtree Creek, near Decatur	Lat 33°48'20", long 84°17'52", DeKalb County, at bridge on Wi11ivee Drive near Decatur

Table 1.—Gaging stations in the Statewide urban study, by city, 1994--Continued

Station number[1/]	Station name	Location
	Atlanta—Continued	
02336200	South Fork Peachtree Creek, tributary, at Decatur	Lat 33°47'21", long 84°17'50", DeKalb County, at culvert on Scott Boulevard, at Decatur
02336238	South Fork Peachtree Creek tributary, near Atlanta	Lat 33°47'11", long 84°20'29", DeKalb County, at culvert on East Rock Springs Road, near Atlanta
02336325	Nancy Creek tributary, near Chamblee	Lat 33°54'22", long 84°18'21", DeKalb County, at culvert on Plantation Lane, near Chamblee
02336690	South Utoy Creek tributary no. 2, at East Point	Lat 33°42'09", long 84°26'57", Fulton County, at culvert on Fort Valley Drive, at East Point
02336697	South Utoy Creek tributary no. 1, at East Point	Lat 33°41'51", long 84°27'33", Fulton County, at culvert on Woodberry Avenue, at East Point
02336700	South Utoy Creek tributary, at East Point	Lat 33°41'25", long 84°28'05", Fulton County, at culvert on Headland Drive, at East Point
02336705	South Utoy Creek, at Atlanta	Lat 33°42'57", long 84°28'4l", Fulton County, at culvert on Adams Drive, at Atlanta
02337081	Camp Creek, at College Park	Lat 33°39'39", long 84°27'44", Fulton County, at culvert on Park Terrace, at College Park
	Augusta	
02196570	Raes Creek tributary no. 2, at Augusta	Lat 33°32'19", long 82°02'34", Richmond County, at culvert on Skinner Mill Road at junction with Boy Scout Road, at Augusta
02196605	Raes Creek tributary no. 1, at Augusta	Lat 33°29'36", long 82°02'17", Richmond County, at culvert on Boy Scout Road, at Augusta
02196725	Oates Creek, at Augusta	Lat 33°27'19", long 82°02'23", Richmond County, at culvert on White Road, at Augusta
02196730	Oates Creek at Old Savannah Road, at Augusta	Lat 33°26'39", long 8l°59'39", Richmond County, at culvert on Old Savannah Road, at Augusta
02196760	Rocky Creek tributary, at Augusta	Lat 33°27'07", long 82°02'57", Richmond County, at culvert on U.S. Highways 78 and 278, at Augusta
02196850	Butler Creek tributary, at Augusta	Lat 33°25'00", long 82°04'41", Richmond County, at culvert on Meadowbrook Drive, at Augusta
	Columbus	
02341542	Flat Rock Creek, at Columbus	Lat 32°32'57", long 84°53'07", Muscogee County, at bridge on Warm Springs Road, at Columbus
0234l544	Mill Branch, at Columbus	Lat 32°28'19", long 84°53'58", Muscogee County, at culvert on Chalbena Road, at Columbus
02341546	Bull Creek tributary, at Columbus	Lat 32°28'38", long 84°55'36", Muscogee County, at culvert on Woodland Drive, at Columbus
02341548	Lindsey Creek tributary, at Columbus	Lat 32°31'33", long 84°56'21", Muscogee County, at culvert on Canberra Avenue, at Columbus

Table 1.—Gaging stations in the Statewide urban study, by city, 1994--Continued

Station number[1/]	Station name	Location
	Moultrie	
02318565	Okapilco Creek tributary, at Moultrie	Lat 31°10'12", long 83°46'40", Colquitt County, at culvert on Southeast 10th Street, at Moultrie
02327202	Ochlockonee River tributary, at Moultrie	Lat 31°10'25", long 83°48'03", Colquitt County, at culvert on Southwest 11th Street, at Moultrie
02327203	Tributary to Ochlockonee River tributary, at Moultrie	Lat 31°09'54", long 83°47'35", Colquitt County, at culvert on Southwest 4th Street, at Moultrie
02327204	Ochlockonee River tributary, at Moultrie	Lat 31°09'38", long 83°48'11", Colquitt County, at culvert on West Boulevard, at Moultrie
	Rome	
02395990	Etowah River tributary, near Rome	Lat 34°16'02", long 85°08'18", Floyd County, at culvert on Atteiram Road, near Rome
02396290	Silver Creek tributary no. 1, near Rome	Lat 34°10'24", long 85°09'21", Floyd County, at culvert on Silver Creek Road, near Rome
02396510	Silver Creek tributary no. 2 at Lindale Road, near Rome	Lat 34°12'56", long 85°10'09", Floyd County, at culvert on Lindale Road, near Rome
02396515	Silver Creek tributary no. 2 at U.S. Highways 27 and 411, near Rome	Lat 34°13'08", long 85°10'27", Floyd County, at culvert on U.S. Highways 27 and 411, at junction with Old Lindale Road, near Rome
02396550	Silver Creek tributary no. 3, at Rome	Lat 34°13'26", long 85°09'14", Floyd County, at culvert on U.S. Highway 27, 0.4 mile north of U.S. Highway 411 interchange, at Rome
02396680	Horseleg Creek, at Rome	Lat 34°16'03", long 85°13'29", Floyd County, at culvert on Castlewood Drive, at Rome
	Savannah	
02203541	Harmon Canal tributary, at Savannah	Lat 32°00'02", long 81°06'49", Chatham County, at culvert on Hodgson Memorial Drive, at Savannah
02203542	Harmon Canal, near Savannah	Lat 32°00'00", long 81°07'45", Chatham County, at culvert on Perimeter Road, within the limits of Hunter Army Airfield, 50 feet upstream from Montgomery Cross Road, near Savannah
02203543	Wilshire Canal, near Savannah	Lat 31°59'27", long 81°08'15", Chatham County, at culvert on Tibet Avenue, near Savannah
02203544	Wilshire Canal tributary, near Savannah	Lat 31°58'25", long 81°08'20", Chatham County, at culvert on Windsor Road, near Savannah
	Thomasville	
02326182	Olive Creek tributary, at Thomasville	Lat 30°49'51", long 83°57'51", Thomas County, at culvert on Baybrook Street, at Thomasville
02327467	Oquina Creek, at Thomasville	Lat 30°50'12", long 83°59'38", Thomas County, at culvert on Wolf Street, at Thomasville

Table 1.—Gaging stations in the Statewide urban study, by city, 1994--Continued

Station number[1/]	Station name	Location
	Thomasville--Continued	
02327468	Oquina Creek, at Cairo Road at Thomasville	Lat 30°51'02", long 84°00'10", Thomas County, at culvert on Cairo Road (Highway 84 West), at Thomasville
02327471	Bruces Branch, at Thomasville	Lat 30°50'39", long 83°58'36", Thomas County, at culvert on North Hansell Street, at Thomasville
02327473	Bruces Branch at Walcott Street, at Thomasville	Lat 30°51'07", long 83°58'42", Thomas County, at culvert on Walcott Street, at Thomasville
02327474	Bruces Branch tributary, at Thomasville	Lat 30°51'20", long 83°58'18", Thomas County, at culvert on Fontaine Drive, at Thomasville
	Valdosta	
02317564	Dukes Bay Canal, at Valdosta	Lat 30°49'13", long 83°16'20", Lowndes County, at culvert on South Patterson Street at intersection with State Route 94, at Valdosta
02317566	Dukes Bay Canal at Industrial Boulevard, at Valdosta	Lat 30°48'34", long 83°15'43", Lowndes County, at culvert on Industrial Boulevard, at Valdosta
023177551	Sugar Creek tributary, at Valdosta	Lat 30°49'51", long 83°18'27", Lowndes County, at culvert on Hyde Park Road, at Valdosta
023177553	Onemile Branch, at Valdosta	Lat 30°50'58", long 83°16'38", Lowndes County, at culvert on Vallotton Street, at Valdosta
023177554	Onemile Branch, at Wainwright Drive at Valdosta	Lat 30°50'34", long 83°18'04", Lowndes County, at culvert on Wainwright Drive, at Valdosta
023177556	Twomile Branch, at Valdosta	Lat 30°52'09", long 83°16'43", Lowndes County, at culvert on Bemiss Road, at Valdosta
023177557	Twomile Branch at University Drive, at Valdosta	Lat 30°52'05", long 83°17'04", Lowndes County, at culvert on University Drive, at Valdosta
023177558	Twomile Branch at Oak Street, at Valdosta	Lat 30°51'48", long 83°17'32", Lowndes County, at culvert on Oak Street, at Valdosta

[1/]U.S. Geological Survey downstream order number.