FLOOD-FREQUENCY RELATIONS FOR URBAN STREAMS IN METROPOLITAN ATLANTA, GEORGIA

By Ernest J. Inman

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METRIC CONVERSION FACTORS

The inch-pound units used in this report can be converted to equivalent SI (metric) units as follows:

Multiply inch-pound unit	By	To obtain metric units
cubic foot per second (ft^3/s)	0.02832	cubic meters per second (m^3/s)
foot (ft)	0.3048	meter (m)
square foot (ft^2)	0.0929	square meter (m^2)
inch (in.)	25.4	millimeter (mm)
square mile (mi^2)	2.59	square kilometer (km^2)
acre	0.4047	hectare (ha)

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ABSTRACT

A knowledge of the magnitude and frequency of floods is needed for the design of highway drainage structures, for establishing flood insurance rates, and for many other uses by urban planners and engineers. This report presents a method for estimating the magnitude and frequency of floods on small streams in the Atlanta, Georgia, metropolitan area.

The Distributed Routing Rainfall-Runoff Model Version II and the U.S. Geological Survey rainfall-runoff model were calibrated for 19 drainage basins in the Atlanta area ranging in size from 0.21 to 19.1 square miles and in impervious area from 19 to 42 percent. The models were used to synthesize long-term flood records for these basins. The 2- to 100-year floods were developed for each basin from these long-term flood peak records using the log-Pearson Type III distribution.

Multiple-regression analysis was then used to define relations between the flood-frequency station data and certain physical basin characteristics, of which drainage area, channel slope, and measured total impervious area were found to be significant. By using these relations, with these basin characteristics, the magnitude and frequency of floods at ungaged basins can be estimated.

INTRODUCTION

A knowledge of flood characteristics is essential for the design of highway drainage structures, for establishing flood insurance rates, and for many other uses by urban planners and engineers. Because urbanization can produce substantial changes in flood-runoff characteristics of streams, natural (rural) basin flood-frequency relations are not applicable to urban streams. This study was undertaken to: (1) collect rainfall-runoff data for selected stream basins in the metropolitan Atlanta area, and (2) analyze these data to provide a method for estimating the magnitude and frequency of floods for streams in this urban area.

Nineteen basins were selected in the metropolitan Atlanta area (fig. 1, table 1). The data from these basins, an average of 40 floods per basin, were used to calibrate the Distributed Routing Rainfall-Runoff Model Version II, as described by Alley and Smith (1982), and the U.S. Geological Survey rainfall-runoff model, as described by Dawdy and others (1972).

After the models were successfully calibrated, long-term rainfall data from the National Weather Service (NWS) Atlanta station were used to synthesize about 75 years of annual peaks. These synthesized peaks were used to develop flood-frequency relations at each site.

The next step in analyzing these data was to develop relations which can be used to estimate the magnitude and frequency of floods at ungaged sites. This was done by the multiple-regression method and will be described in a later section of this report.

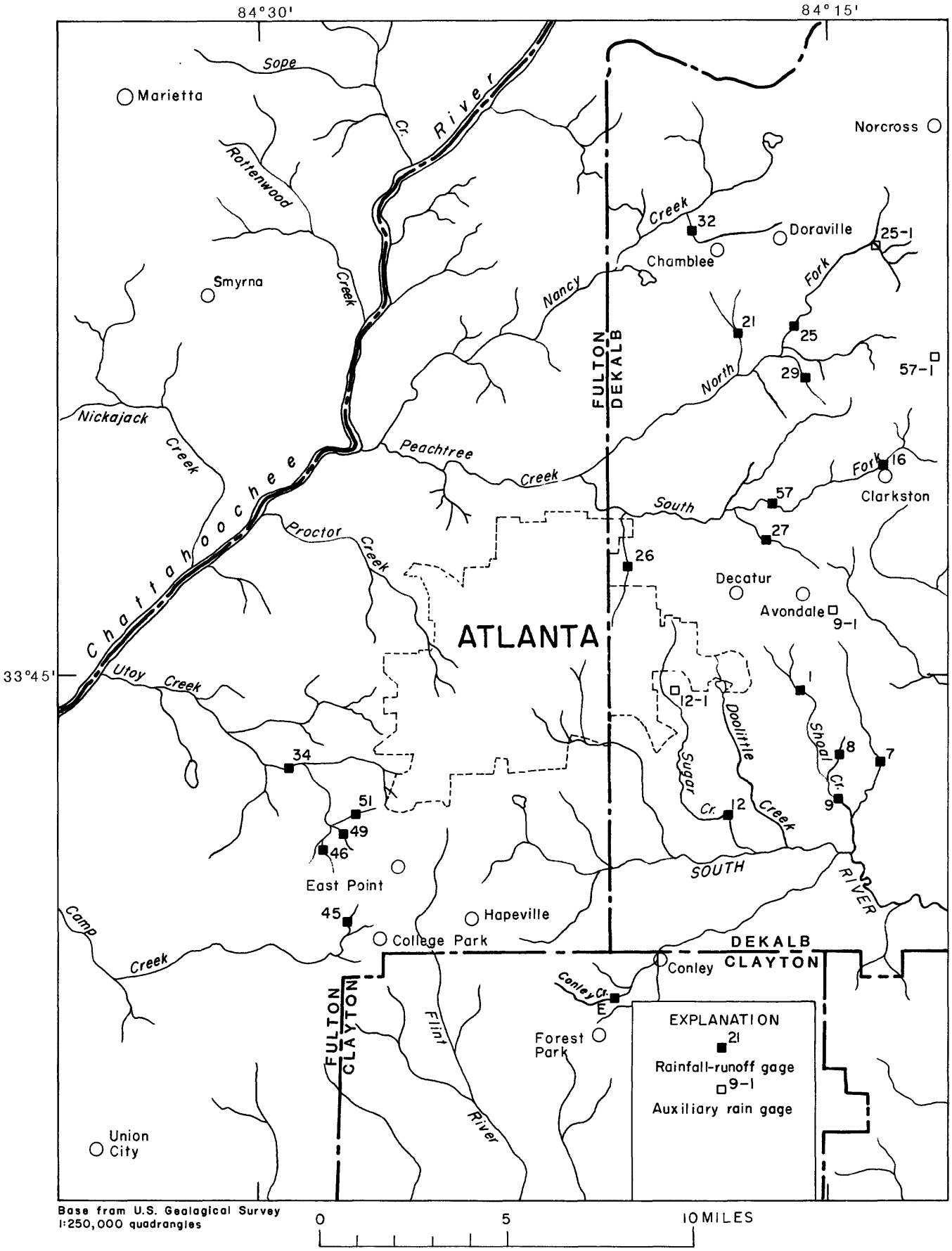

Figure 1.— Gage locations in the metropolitan Atlanta area.

Table 1.--Gages in the metropolitan Atlanta area

Local station number	National station number	Station name	Location
12	02203820	Sugar Creek near Atlanta, Ga.	Lat 33°41'41", long 84°18'15", DeKalb County, at culvert on Clifton Church Road near Atlanta.
1	02203835	Shoal Creek near Atlanta, Ga.	Lat 33°44'48", long 84°16'50", DeKalb County, at culvert on Line Street near Atlanta.
8	02203845	Shoal Creek tributary near Atlanta, Ga.	Lat 33°43'05", long 84°15'45", DeKalb County, at culvert on Glendale Drive near Atlanta.
9	02203850	Shoal Creek near Atlanta, Ga.	Lat 33°42'36", long 84°15'57", DeKalb County, at culvert on Rainbow Drive near Atlanta.
7	02203870	Cobbs Creek near Atlanta, Ga.	Lat 33°43'44", long 84°14'17", DeKalb County, at culvert on Snapfinger Road near Atlanta.
E	02203884	Conley Creek near Forest Park, Ga.	Lat 33°38'08", long 84°20'37", Clayton County, at culvert on Rock Cut Road near Forest Park.
25	02336080	North Fork Peachtree Creek near Chamblee, Ga.	Lat 33°51'43", long 84°17'13", DeKalb County, at culvert on Shallowford Road near Chamblee.
29	02336090	North Fork Peachtree Creek tributary near Chamblee, Ga.	Lat 33°50'53", long 84°17'57", DeKalb County, at culvert on Meadowcliff Drive near Chamblee.
21	02336102	North Fork Peachtree Creek tributary near Atlanta, Ga.	Lat 33°51'20", long 84°19'19", DeKalb County, at culvert on Drew Valley Road near Atlanta.
16	02336150	South Fork Peachtree Creek at Clarkston, Ga.	Lat 33°48'51", long 84°14'38", DeKalb County, at culvert on Montreal Road at Clarkston.
57	02336180	South Fork Peachtree Creek near Decatur, Ga.	Lat 33°48'20", long 84°17'52", DeKalb County, at bridge on Willivee Drive near Decatur.
27	02336200	South Fork Peachtree Creek tributary at Decatur, Ga.	Lat 33°47'21", long 84°17'50", DeKalb County, at culvert on Scott Boulevard at Decatur.

Table 1.--Gages in the metropolitan Atlanta area--Continued

Local station number	National station number	Station name	Location
26	02336238	South Fork Peachtree Creek tributary near Atlanta, Ga.	Lat 33°47'11', long 84°20'29", DeKalb County, at culvert on East Rock Springs Road near Atlanta.
32	02336325	Nancy Creek tributary near Chamblee, Ga.	Lat 33°54'22", long 84°18'21", DeKalb County, at culvert on Plantation Lane near Chamblee.
51	02336690	South Utoy Creek tributary No. 2 at East Point, Ga.	Lat 33°42'09", long 84°26'57", Fulton County, at culvert on Fort Valley Drive at East Point.
49	02336697	South Utoy Creek tributary No. 1 at East Point, Ga.	Lat 33°41'51", long 84°27'33", Fulton County, at culvert on Woodberry Avenue at East Point.
46	02336700	South Utoy Creek tributary at East Point, Ga.	Lat 33°41'25", long 84°28'05", Fulton County, at culvert on Headland Drive at East Point.
34	02336705	South Utoy Creek at Atlanta, Ga.	Lat 33°42'57", long 84°28'41", Fulton County, at culvert on Adams Drive at Atlanta.
45	02337081	Camp Creek at College Park, Ga.	Lat 33°39'39", long 84°27'44", Fulton County, at culvert on Park Terrace at College Park.

Acknowledgments

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The guidance and technical assistance of Harold G. Golden, U.S. Geological Survey, in the planning and early activities of the project are recognized and greatly appreciated. Also, the efforts of David W. Carlysle, Thomas G. Carlysle, Dennis L. Brown, Melody A. Wallrich, Elizabeth M. Brown, and many others in collecting and processing data are gratefully appreciated.

PROJECT BACKGROUND

The U.S. Geological Survey and the Savannah District, U.S. Army Corps of Engineers, initiated this flood-frequency study for the Atlanta metropolitan area in June 1972. DeKalb County began participation in the study in January 1973.

Previous Studies

A report by Golden (1977) presents preliminary flood-frequency relations for urban streams in Metropolitan Atlanta. His method is based on the technique used by Sauer (1974) for other parts of the United States, and on the natural flood-frequency and rainfall-frequency characteristics of the local area. The Sauer method adjusts the natural flood-frequency relations to that of urbanized conditions 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) presents the same technique for use on a statewide basis.

Sauer and others (1981) present estimating equations and several other methods of estimating flood-frequency for urban watersheds on a nationwide basis. Five basins from the Atlanta area were used in their nationwide analysis.

Lumb (1975) in his, "UROS4: Urban Flood Simulation Model, Part 1, Documentation and Users Manual," explains 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 this model to eight DeKalb County watersheds in varying degrees of detail, with limited observed data for verification.

Jones (1978) developed simplified equations that can be used on small watersheds (up to 200 acres) to estimate the expected peak-flood flows. Results from this study were compared with results using the Golden (1977) method, and several other methods (Jones, 1978). These methods were not generally compatible with the equations presented in Jones' study.

Lichty and Liscum (1978) developed a procedure for computing estimates of T-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. Imperviousness has been incorporated in the estimating equations to account for urban development.

DATA COLLECTION AND PROCESSING

Site Selection

Extensive field reconnaissance was required to select the 19 basins used in this study. More than 200 sites were inspected for possible use. A range in drainage area (0.2 to 20 mi^2), main channel slope, and channel length was considered. Suitability for rain gage locations, hydraulic characteristics at the gaged site, 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 with the City Planning Department of the Georgia Institute of Technology assisted in this phase of the study. His initial involvement was to consult with all city and county planners in the Atlanta metropolitan area, and based on their data and general knowledge of the areas, determine the stability of developed areas. This information was presented on color-coded city and county maps as being either stable, fairly stable, fairly unstable, or unstable.

The next step in the study was a field reconnaissance of the more than 200 stream basins in areas designated as stable and fairly stable. Of these basins, all but 62 were excluded because of their hydraulic characteristics, or because they contained no suitable locations for a rain gage (or gages). The 62 selected basins were roughly delineated on topographic maps, and approximate drainage areas and main channel slopes and lengths were determined. The topographic maps outlining the 62 basins were then given to Thomas Debo, who obtained more site specific information from the city and county planners. The information included the planning agency providing the data, the percentage of each basin developed, the existing land use, and a better, more specific estimate of expected development or stability.

The existing land use was divided into six categories: (1) single-family residences, (2) multi-family residences, (3) commercial, (4) industrial, (5) parks, and (6) undeveloped. This process eliminated 19 basins that were unstable or fairly unstable and five sites where land owners would not allow gages on their property.

Nineteen basins finally were selected for study. These generally were the sites that had the best hydraulic characteristics for theoretical computations of peak discharge and the most suitable rain-gage locations. Eighteen of the 19 sites were at culverts. The basins also were selected to provide a suitable distribution of drainage area, main channel slope, and main channel length. A certain degree of compromise was required in the selection process to obtain drainage areas on the larger end of the scale, even though land use in some parts of the basin was not stable. How this instability was accounted for during data analysis will be discussed in a later section of this report. Gage locations are shown in figure 1.

Instrumentation and Data Collection

Digital recorders having 5-minute punch intervals were used throughout the study area. The recording stage gage for each basin was housed on top of an 18-inch vertical corrugated metal pipe stilling well in the upstream approach section. Each stilling well had two 2-inch intakes near the base and 1/2-inch diameter holes drilled about every 6 inches above ground level to flood stage. The stilling wells were flushed after every flood and intakes were rodded out during every inspection trip.

Each station also had at least one rain gage, generally located near the stage gage. All stations on drainage areas larger than 2.5 mi^2 had one or two additional rain gages located in the headwaters of the basin. Rain gage recorders were housed on top of 8-foot collector wells made from 3-inch galvanized pipe. Collector wells of this size will hold about 11 inches of rainfall. A drain plug near the bottom of the collector well was used to drain the well on each inspection trip.

Crest-stage gages were also installed at each site, at least one in the upstream approach section and one at the downstream end of the culvert. At most sites, the stage at the recording gage was lower than the stage at the upstream crest-stage gages. This probably was due to drawdown of the intakes rather than to intake lag, as can be demonstrated by the equation in Buchanan and Somers (1968, p. 13). A plot of upstream crest-gage stage versus downstream crest-gage stage was established for each site. The fall through the culverts obtained from these crest-stage gage relations and the culvert geometry were used to compute a theoretical stage-discharge relation as described by Bodhaine (1968).

A relation between upstream crest-gage stage versus recorder stage was established. This relation was used to enable plotting the theoretical discharge computations, as described above, versus the recorder stage, thus enabling digital tapes to be processed without having to make a shift correction for each tape. The upstream crest-gage stage versus 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.

The upstream crest-gage stage versus the downstream crest-gage stage relations also served other purposes. These relations should remain fairly site-consistent or the reason for the inconsistency must be determined. 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 relations. Many times city and county highway maintenance crews had removed debris from culverts between gage servicing trips. When this occurred, plotting of the upstream crest-gage stage versus the downstream crest-gage stage relation was the only evidence of blockage. Plots of upstream crest-gage stage versus downstream crest-gage stage relations were used primarily on culverts having backwater control.

All theoretical stage-discharge relations were verified by current-meter measurements. Three of the larger basins were rated entirely by current-meter measurements.

Data Processing and Storage

Current Data

All usable flood events were processed and loaded into computer storage on a near-current basis. Generally five to eight storms per year were processed for each site. Unit rainfall, unit discharge, and daily rainfall were then retrieved and the unit data were plotted against time. The purposes of these plots were many. First, they served as a visual data editing tool allowing a bad punch by the recorder or a misread punch by the tape transmitter to be easily detected. A partially clogged rain-gage intake or a hanging float also would be evident. Second, if the stilling well intakes were out of the water at the beginning of a rise, the lower part of the rising limb of the hydrograph was easily estimated from base flow. Also, if the intakes became partially clogged with sediment on the recession, the falling limb of the hydrograph was easily estimated. Third, the plots served as a very useful tool for estimating TC and KSW (table 5), which are routing parameters in the USGS model. After needed editing and estimations, the data were reloaded into computer storage.

Daily evaporation data were also needed to calibrate both models. Because Atlanta does not have a National Weather Service (NWS) station that records evaporation data, these data were obtained from the Athens NWS. Evaporation maps by the NWS (Kohler and others, 1959) were used as a guide in selecting the appropriate evaporation station. Although Athens is about 65 miles east of Atlanta, the evaporation characteristics of the areas are similar.

Long-Term Rainfall and Evaporation

Long-term rainfall and evaporation are required for flood-peak simulation, as will be described later. Daily rainfall data for Atlanta were obtained from NWS publications and loaded into computer storage. The dates of rainstorms since 1948 were obtained from hourly data in NWS publications. Storms prior to 1948 were selected from the unpublished hourly rainfall log at the Atlanta weather station. For hourly rainfalls that exceeded 1/2 inch, the daily charts were obtained from the National Climatic Center, Asheville, N.C., and selections were made for unit rainfall coding based on hydrologic judgment. The storm-rainfall data on these charts were coded at 5-minute intervals. A total of 332 storms were processed for use. As a check on the procedure for selecting unit rainfall, the U.S. Geological Survey computer program E436 was used to select the five major rainfall periods for each water year. No significant difference was found between the storms selected by these two methods.

Daily evaporation data were obtained from the Athens NWS station for the period 1940-73. Prior to this period, U.S. Geological Survey computer program H266 was used to generate harmonic average evaporation data from the later period of observed continuous daily evaporation data. These observed and synthesized data were then loaded into computer storage.

DATA ANALYSIS

The first step in analyzing the rainfall-runoff data was to calibrate two U.S. Geological Survey models. The Distributed Routing Rainfall-Runoff Model Version II and the U.S. Geological Survey rainfall-runoff model for natural basins, with impervious area included, were used.

Description of Models

The Distributed Routing Rainfall-Runoff Model Version II

The Distributed Routing Rainfall-Runoff Model Version II, hereafter referred to as the DR3M model, is described in detail by Alley and Smith (1982). The model computes and routes rainfall excess through a branched system of pipes or natural channels using rainfall as input. It combines the rainfall-excess components developed by Dawdy and others (1972) with the kinematic-wave routing method presented by LeClerc and Schaake (1973). The rainfall-excess components include soil-moisture accounting, previous-area rainfall excess, and impervious-area rainfall excess. Model parameters are adjusted using optimization procedures discussed later.

The soil-moisture-accounting component (table 2) determines the effect of antecedent conditions on infiltration. The rainfall excess is routed over pervious surfaces and two types of impervious surfaces: (1) effective impervious areas--impervious areas draining directly into the channel drainage system, and (2) noneffective impervious areas--impervious areas that drain into pervious areas.

Table 2.--Parameters for soil-moisture-accounting and infiltration for the DR3M model

Soil-moisture-accounting
Parameters:
EVC—A pan coefficient for converting measured pan evaporation to potential evapotranspiration
RR—The proportion of daily rainfall that infiltrates into the soil for the period of simulation, excluding unit days
BMSM--Soil-moisture storage at field capacity, in inches
Infiltration
Parameters:
KSAT—The effective saturated value of hydraulic conductivity, in inches per hour
RGF—Ratio of suction at the wetting front for soil moisture at wilting point to that at field capacity
PSP—Suction at wetting front for soil moisture at field capacity, in inches

The only abstraction from rainfall on effective impervious area is impervious retention, which is a fixed amount, usually about 0.02 inch to 0.05 inch. This retention, which is user-specified, must be filled before runoff from effective impervious areas can occur.

Rain falling on noneffective impervious areas is assumed to run off onto the surrounding pervious area. The model assumes that this occurs instantaneously and that the volume of runoff is uniformly distributed over the pervious area. This volume, expressed in inches over the pervious area, is added to the rain falling on the pervious areas prior to computation of pervious-area rainfall excess. This computation is performed in the model by multiplying rainfall on pervious areas by the model parameter RAT:

$$\frac{DA2 + DA3}{DA3},$$

where DA2 = the area of the basin covered by noneffective impervious surfaces

and DA3 = the area of the basin covered by pervious surfaces.

The six soil-moisture-accounting and infiltration parameters used by the DR3M model presented in table 1 are taken from Alley and Smith (1982).

The parameter optimization component in the model is based on an optimization technique devised by Rosenbrock (1960). The technique is a trial-and-error, "hill-climbing" procedure that changes a parameter value and recomputes an objective function using the revised set of parameter values. The objective function is the sum of the squared deviations of the logarithms of the synthesized flood peaks or storm-runoff volumes from the observed flood peaks or storm-runoff volumes. If the results at the end of an iteration show a reduction in the value of the objective function, an improvement in model calibration has been achieved and the revised set of parameters is accepted; if not, the previous set is retained. Thus, the optimization procedure produces a nonlinear least-squares solution.

The routing component of the DR3M model uses the kinematic wave theory for routing flows over a given drainage basin. A basin is represented as a set of segments which jointly describe all subbasins in the total basin. The purpose for dividing the basin into segments is to reduce the rainfall-excess routing problem to the hydraulic problem of unsteady flow over uniform planes and channels. The model will accept as many as 99 total segments, which can be made up of four types:

(1) overland-flow segments,
(2) channel or pipe segments,
(3) reservoir segments, and
(4) nodal segments.

Overland-flow segments.--receive uniformly distributed lateral inflow from rainfall excess. They represent a rectangular plane of a given length, slope, roughness, and percent imperviousness.

Channel segments.--used to represent natural or manmade conveyances. Channel segments may receive upstream inflow from as many as three other segments, including combinations of other channel segments, reservoir segments, and nodal segments. They can also receive lateral inflow from overland-flow segments.

Reservoir segments.--can be used to describe an on-channel detention reservoir. They also can be used to simulate culverts that detain water due to limited capacity.

Nodal segments.--are used when more than three segments contribute inflow to the upstream end of a channel or reservoir segment or as input points where the user may specify an input hydrograph or constant discharge for each storm.

The assumptions behind limitations of the kinematic wave equations for channel and overland-flow routing should be recognized by any potential user of the model. The kinematic wave solution is based on the assumption that disturbances are allowed to propagate only in the downstream direction. Therefore, the model does not account for backwater effects or flow reversal. In addition, the capacity of circular-pipe segments is limited to nonpressurized-flow capacity. In addition to the assumptions behind the kinematic wave routing, other major assumptions are listed below.

Rainfall excess is assumed to be uniformly distributed over an over-land-flow segment.

Pervious and impervious parts of a segment are assumed to be uniformly distributed over the segment.

The complex uneven topography of the natural catchment can be approximated by planes.

Rainfall excess does not infiltrate as it moves overland (once rainfall excess is computed, it must end up in a channel).

When rainfall ceases, infiltration ceases.

Lateral inflows to channels are assumed to be uniformly distributed (in an urban environment lateral inflows may enter through a gutter rather than uniformly).

Changes in flow from laminar to turbulent or vice versa will not occur.

Rainfall on noneffective impervious areas is assumed to be instantaneously and uniformly distributed over the previous area of the watershed.

U.S. Geological Survey Rainfall-Runoff Model

The U.S. Geological Survey rainfall-runoff model, a lumped-parameter model, hereafter referred to as the USGS model, was described in detail by Dawdy and others (1972). A translation and extension of this model was written by Carrigan (1973). The soil-moisture-accounting and infiltration components developed for the USGS model also are used by the DR3M model except for the parameter DRN. DRN proved to be very insensitive when sensitivity analyses were run using the rural version of the USGS model applied to the Piedmont region of Georgia. It is, however, a parameter included in the USGS model. Seven soil-moisture-accounting and infiltration parameters and three routing parameters used in the USGS model are listed in table 3.

Table 3.--Soil-moisture-accounting, infiltration, and surface-runoff routing parameters for the USGS model

Parameter identifier code	Units	Application
PSP	Inches	Represents the 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	The minimum saturated value of hydraulic conductivity used to determine infiltration soil rates.
BMSM	Inches	Soil moisture-storage volume at field capacity.
EVC		Coefficient to convert pan evaporation to potential evapotranspiration values.
DRN	Inches per hour	A 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 surface-runoff routing component is based on a modification of the Clark (1945) form of the instantaneous unit hydrograph. 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. This modification simplified the calibration procedure and added a feature to allow separation of compound peaks that provided 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.

The primary difference between the DR3M and the USGS rainfall-runoff models is in the routing components. The DR3M model allows for detention storage, whereas, the USGS model does not. The differences between the two models will be discussed in more detail in a later section.

Calibration and Verification of Models

Initially an average of slightly more than 40 floods per station were available for model calibration. About 90 percent of these floods were used in the final calibrations. Some outliers were deleted because of nonrepresentative rainfall. These outliers could be fairly easily determined on stations having multiple rain gages, but on stations having only one rain gage they were not always easy to determine, particularly for summer thunderstorms. For example, several floods were deleted at some stations because runoff exceeded rainfall, or because rainfall greatly exceeded runoff. In another instance, several peaks were deleted because upstream crossings could have been clogged with debris, making the peak discharge at the gaging site artificially low.

The DR3M Model

The first step in calibrating the DR3M model was to optimize on effective impervious area (EAC) with all other parameters being held constant. This was accomplished by using only small storms for which runoff was largely contributed from the effective impervious area of the watershed. The starting value of EAC was set at 1.0, with a lower limit of 0.85 and an upper limit of 1.15. The model assumes that any adjustment to effective impervious area using EAC is offset by an adjustment in the noneffective impervious area in order to maintain a constant total drainage area. If the optimized value of EAC exceeds 1.0 and insufficient noneffective impervious area exists to compensate for the increased effective impervious area, then an appropriate amount of pervious area is converted to effective impervious area to maintain a constant total drainage area. The final optimized value of EAC was multiplied by the effective impervious area values of each subbasin, and this product was then subtracted from 1.0 to obtain pervious area and noneffective impervious area. An adjustment to RAT is necessary after EAC has been optimized.

After optimizing EAC, it was then necessary to optimize on the soil-moisture-accounting and infiltration parameters by using the large storms and holding EAC constant. EVC was fixed at 0.70, as determined from a combination of the average values of EVC obtained from optimization of the parameter for the rural stations in the Piedmont region of Georgia, and from NWS Technical Paper 37 (Kohler and others, 1959). Because the model parameters EVC and RR are highly interactive, only RR was optimized.

A range for KSAT of 0.05 to 0.40 was obtained from Chow (1964). Most of the soils of the 19 basins in this study were type B soils, with a small amount of type C soils. A starting value of 0.15 was used for this parameter. The range and starting values of the other soil-moisture-accounting and infiltration parameters RR, BMSM, RGF, and PSP were obtained from Golden and Price (1976).

The parameters PSP and KSAT are also very interrelated. Because the range of KSAT can be more accurately estimated, PSP was optimized first and KSAT held constant. KSAT was then optimized and PSP held constant. Both parameters were never optimized during the same run.

Routing is the final step of calibration. An explicit finite-difference method of routing was used for all calibrations for two reasons. First, it was the only routing option available until a later model version came out. The model had already been calibrated for about half of the basins in this study when the later version became available. Second, after the later model version was released, a method of characteristics routing and an implicit finite-difference method of routing were used on several basins and they made no significant difference in calibration results.

At least one cross section was field-surveyed, or obtained from other sources, for all channel segments. Some segments had many cross sections available from other sources. If visual inspection or Manning's equation using cross-section geometry, channel slope, and roughness values indicated that the main channel capacity would not be exceeded, then a triangular cross section was approximated from the surveyed field cross section. The approximated channel side slopes, actual bed slope, and roughness values were input to the model. From the cross-section input information, the model computes discharge using the kinematic wave equations $Q = \alpha A^m$, where α and m are constants that are determined from the channel and cross-section information listed above. In instances of discharge exceeding the main-channel capacity, there is an option in the model to specify two sets of α and m; one for discharges smaller than the main-channel capacity and one for discharges greater than the main-channel capacity. The discharge at channel capacity is referred to as "breakpoint" discharge. When two sets of α and m and a breakpoint discharge are required, these values must be computed and entered as input to the model. This was accomplished using Manning's equation to determine area and discharge for at least three stages below channel capacity and three stages above channel capacity. A plot of discharge versus area was made of these six points. Two relations were always evident. Each of the relations can be described by a simpler linear equation with α being the y intercept and m the slope of one line and a different α and m for the second line. The point at which the break in linearity occurs, or the second equation begins, is channel capacity, or "breakpoint" discharge.

Routing is the most expensive and computer-time consuming step of calibration. It would be expensive to adjust the routing parameters using an automatic method such as the Rosenbrock (1960) scheme, thus the routing part of DR3M was calibrated manually. Roughness (Manning's "n" value) and NDX are the two parameters that were adjusted. NDX is a model parameter that defines the number of length intervals for finite-difference routing. Only small changes were made to NDX. An increase in NDX increases discharge, and a decrease in NDX would decrease discharge. ALPADJ is a factor used to

adjust the combined effects of roughness, bed slope, and cross-sectional geometry which make up α. It was believed that slope, both channel and overland flow, was determined with sufficient accuracy from the topographic maps available in the Atlanta area, so that no adjustment was made to this parameter.

At upstream crossings that were determined from field inspection or from topographic maps to have storage potential, an elevation-outflow-storage table was prepared. The outflow was obtained from a theoretical rating at the outlet of the reservoir, generally a culvert. The storage of the reservoir, created by the road embankment, was computed from field surveys or obtained by planimetry on 5-foot contour maps. The outflow and storage were input to the model at corresponding elevations. This option made it possible to more accurately simulate higher observed flood events.

Only one of the 19 basins did not have significant detention storage. That basin, however, did have overbank flow on two channel segments. One basin had nine detention storage areas. The average number of detention storage areas for the 19 basins was about three and a half.

The optimized parameter values from the DR3M model and the number of segments from the final calibration are listed in table 4.

Graphs of simulated peak discharge versus observed peak discharge in figure 2 illustrate the results of the DR3M calibrations at three sites. The three stations were selected to give a range in standard error of estimate.

The USGS Model

Calibration of the USGS model was similar to calibration of the DR3M model, in that effective impervious area and the soil-moisture-accounting and infiltration parameters were optimized in the same way with the same starting values and limits. In the effective impervious area optimization, there was no RAT to compute or adjust. In the soil-moisture-accounting procedure, the major difference between the models was that the USGS model uses the DRN parameter but the DR3M model does not. A sensitivity analysis of all parameters in the USGS model was conducted for three basins in the Piedmont area of Georgia for the rural study. Although DRN was included as a parameter in the USGS model, model results proved to be very insensitive to large changes in DRN. It was, therefore, held constant at 0.20, which was about the average value for the basins in the Piedmont area of the rural study.

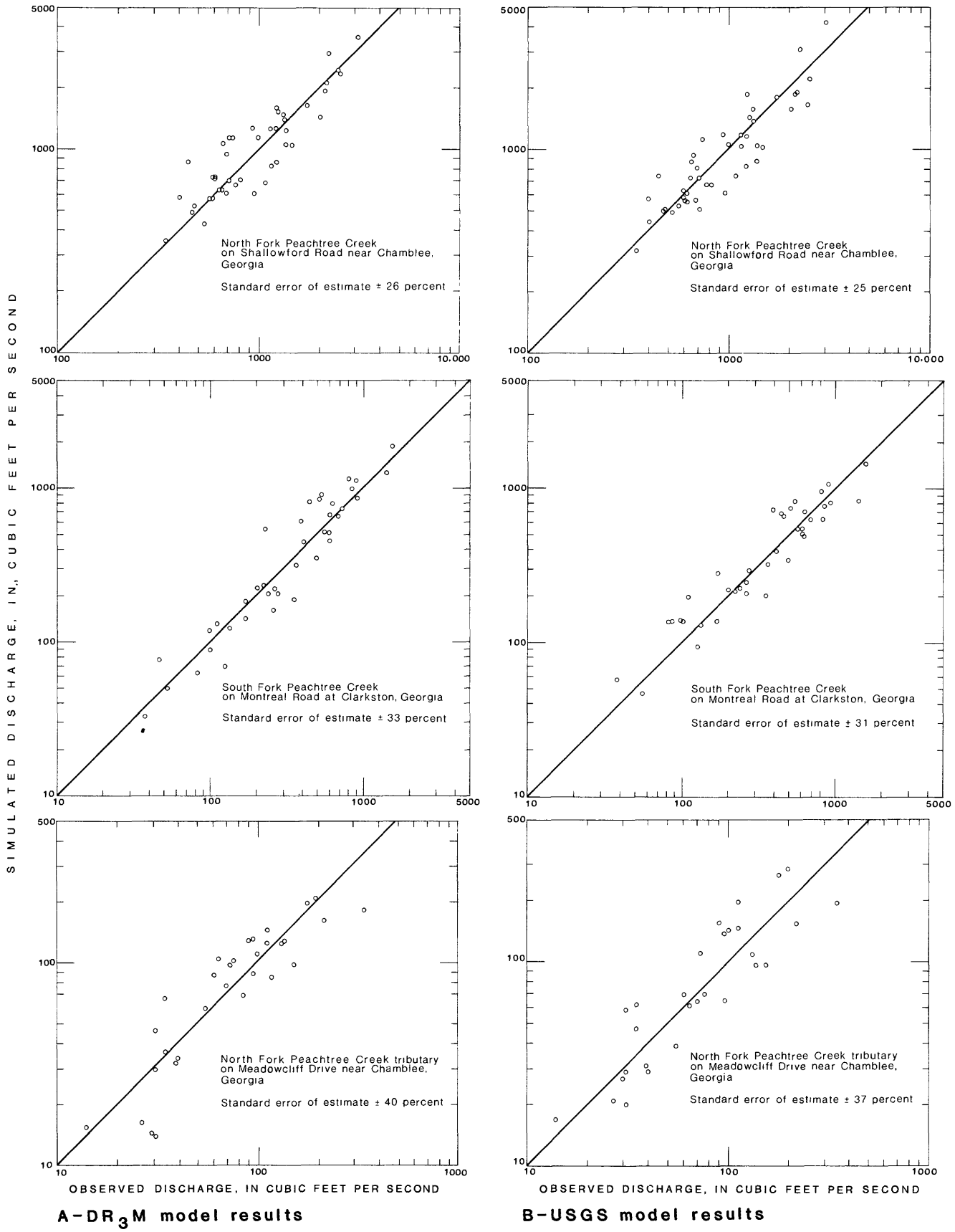
The major difference in the two models is in the routing phase. With the USGS model, the routing parameters can be optimized. The type of routing is also different, as explained earlier. The starting values for the routing parameters KSW and TC were obtained from analog plots of the discharge hydrographs and the rainfall hyetographs. The first model run was made as an additional data editing step and to obtain line-printer hydrograph plots of simulated and observed data. The parameter TC was adjusted and fixed on the basis of the comparison of the simulated and observed hydrographs of the higher peaks of the first run. TP/TC was fixed at 0.50, as suggested by Mitchell (1972).

Table 4.--Optimized DR3M model parameters*

Station number	PSP	KSAT	RGF	BMSM	RR	EAC	RAT	Number of segments	S.E.[1]
02203820	4.33	0.150	21.7	3.24	0.90	0.88	1.16	55	± 24
02203835	2.79	.168	22.6	2.66	.71	1.12	1.07	27	± 35
02203845	2.54	.150	10.8	2.56	.94	.86	1.15	10	± 34
02203850	1.83	.121	24.2	4.17	.78	1.18	1.05	49	± 29
02203870	2.78	.141	10.0	2.55	.82	.98	1.08	46	± 34
02203884	2.42	.150	14.4	3.51	.78	1.15	1.00	48	± 35
02336080	2.25	.103	24.6	2.87	.90	1.13	1.02	18	± 26
02336090	1.80	.151	7.8	3.67	.95	1.07	1.08	24	± 40
02336102	1.69	.150	24.4	7.79	.90	1.14	1.03	30	± 40
02336150	1.61	.125	14.3	3.14	.88	1.08	1.06	41	± 33
02336180	1.94	.118	24.8	5.55	.93	1.01	1.06	70	± 28
02336200	3.10	.121	5.2	5.67	.93	.92	1.12	17	± 31
02336238	2.40	.145	13.2	6.39	.93	.86	1.18	10	± 34
02336325	2.62	.121	15.0	5.74	.84	1.10	1.00	12	± 22
02336690	1.80	.095	16.4	6.21	.94	1.14	1.05	6	± 30
02336697	1.93	.121	19.1	4.27	.94	.91	1.11	11	± 34
02336700	2.40	.121	13.2	6.67	.94	.90	1.16	6	± 26
02336705	1.61	.141	10.9	5.59	.92	.91	1.11	29	± 30
02337081	2.78	.158	13.7	5.41	.83	.80	1.18	18	± 29

* Parameter EVC was assigned a fixed value of 0.70 and not optimized.

1 Standard error of estimate of calibration results, in percent, based on mean-square difference between observed and synthesized peaks.



A-DR_3M model results **B-USGS model results**

Figure 2.— Graphs of observed peak discharges versus simulated peak discharges of the results of the DR_3M and USGS model calibrations at three sites.

The optimized parameter values from the U.S. Geological Survey model are listed in table 5.

Graphs of simulated peak discharge versus observed peak discharge in figure 2 illustrate the results of the USGS calibrations at the same three sites as illustrated in figure 2 for the DR3M model.

The USGS model, or more specifically program A634, uses input from only one rain gage in each basin. Eight of the 19 basins in this study have two or more rain gages. The daily rainfall from the multiple gages was combined into one daily record by applying coefficients, as suggested by Thiessen (1911), to each rain record. The unit rainfall was combined into one record by a method suggested by R. W. Lichty (U.S. Geological Survey, oral commun., 1981). Thiessen coefficients were determined for each rain record, and a total Thiessen weighted rainfall for the resulting flood was computed. A ratio of the Thiessen weighted total rainfall to the total rainfall at the gage having the largest Thiessen coefficient was 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 was used to maintain the intensity of the individual increments. Weighting unit rainfall in the same manner as daily rainfall would tend to have a smoothing effect on the incremental rainfall and, therefore, was not used.

A more complex version of the USGS model, program G824, will accept precipitation from 1 to 5 rainfall gages. This version takes about five times longer to run than program A634 and requires user definition of effective impervious area for each of 20 travel bands used in the routing phase of the model. Program G824 was not used because of time and financial constraints.

Verification

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

Split-sample testing was used on both the DR3M and the USGS models. On the DR3M model, split-sample testing was done for seven basins. Because time and financial constraints would not allow the model to be split-sample tested at all sites, these seven basins were selected to give a range in drainage-area size and percent of effective impervious area. The floods at each site were divided into two samples. The events were arrayed in descending order according to peak magnitude. The odd-numbered events made up the first sample and the even-numbered events the second sample. The DR3M model was recalibrated using only the events in one of the samples. The computed peak discharges for the events not used in the calibration were compared with the observed data and the standard error of estimate was computed. The results were all acceptable, as shown in table 6.

Table 5.--Optimized USGS model parameters*

Station number	PSP	KSAT	RGF	BMSM	RR	KSW	TC	EIA	S.E.[1]
02203820	2.57	0.185	23.4	2.76	0.74	1.90	200	24.7	± 26
02203835	2.96	.129	16.9	3.83	.79	.85	75	18.9	± 38
02203845	2.60	.150	10.4	2.70	.90	.34	58	23.4	± 30
02203850	2.26	.119	15.5	5.94	.93	1.40	140	21.6	± 21
02203870	2.66	.177	8.6	1.16	.75	1.00	100	19.5	± 23
02203884	3.38	.150	10.3	4.23	.94	.53	83	23.1	± 31
02336080	2.04	.138	16.8	3.03	.95	3.5	450	29.0	± 25
02336090	2.34	.145	13.6	7.16	.95	.40	45	17.0	± 37
02336102	3.11	.150	16.9	7.64	.95	.47	100	19.6	± 37
02336150	1.34	.218	10.1	3.39	.85	1.75	185	20.3	± 31
02336180	1.92	.191	23.8	5.72	.93	2.00	475	18.4	± 28
02236200	1.96	.150	13.8	5.06	.81	.44	39	26.2	± 25
02336238	2.31	.131	13.1	6.54	.76	.40	30	27.6	± 26
02336325	2.60	.123	15.7	5.62	.92	.80	55	42.3	± 26
02336690	1.98	.115	15.4	6.26	.94	.59	40	17.0	± 23
02336697	1.73	.166	16.0	9.94	.95	.58	39	20.0	± 33
02336700	2.21	.127	12.6	5.93	.90	.55	38	16.8	± 34
02336705	1.62	.130	15.9	9.98	.94	1.00	123	21.8	± 22
02337081	2.38	.141	9.3	6.55	.95	.52	35	16.0	± 26

* Parameters DRN, EVC, and TP/TC were assigned fixed values of 0.20, 0.70, and 0.50, respectively, and not optimized.

1 Standard error of estimate of calibration results, in percent, based on mean-square difference between observed and synthesized peaks.

Table 6.—DR3M model split-sample test results

DR3M split-sample tests			
	Standard error of estimate of calibration and verification results (in percent)		
Station number	Calibration, all events	Calibration based on odd-numbered events	Verification based on even-numbered events
02203820	± 24	± 23	± 32
02203835	± 35	± 39	± 34
02203870	± 34	± 31	± 36
02336080	± 26	± 29	± 27
02336238	± 34	± 28	± 36
02336325	± 22	± 21	± 25
02336705	± 30	± 24	± 30

The USGS model was verified at six sites using the same procedure that was used with the DR3M model. The results were acceptable, as shown in table 7.

Table 7.—USGS model split-sample test results

USGS split-sample tests			
	Standard error of estimate of calibration and verification results (in percent)		
Station number	Calibration, all events	Calibration based on odd-numbered events	Verification based on 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

Annual peaks were simulated by both models for each of the 19 stations. The USGS model simulation was a relatively simple operation, as compared to the DR3M model simulation. The USGS model was run using the final calibrated model parameters along with unit and daily long-term rainfall and long-term daily evaporation to simulate annual peaks for each station for the period 1898-1973. Because there is no provision in the model to account for storage, the routing parameters must be optimized and adjusted manually to reproduce the observed flood peaks. The higher floods are given more emphasis in this phase of optimization. These routing parameters which are affected by storage, along with the soil-moisture-accounting and infiltration parameters, are then used for the long-term simulation. A log-Pearson Type III frequency curve was fitted to each series of flood peaks in accordance with U.S. Water Resources Council Bulletin 17B (1981) recommendations. Frequency curves for flood peaks simulated by the USGS model represent an "as is" storage condition for each site. Bias in synthetic frequency curves caused by the calibration regression effect (loss of variance) was corrected, using a technique suggested by Kirby (1975). The technique involves dividing the standard deviation of the distribution of flood peaks computed for the synthetic annual peaks by the magnitude of the correlation coefficient determined in the final calibration run. A new frequency curve was then computed using the adjusted standard deviation and the original skew coefficient. Adjusting the frequency curves in this manner tends to increase discharges at the higher recurrence intervals.

No attempt was made to adjust the skew coefficients of the USGS frequency curves, because the data did not meet the criteria specified in the U.S. Water Resources Council Bulletin 17B (1981). The generalized skew coefficient map in Bulletin 17B, used in the adjustment computations, is for rural watersheds and, therefore, is not applicable to the simulated urban flood peaks.

The DR3M model was run using the final calibrated model parameters along with unit and daily long-term rainfall and long-term daily evaporation to simulate annual peaks for each station for the period 1898-1973. The reservoirs in the DR3M model, as described in an earlier section, were removed before the long-term peak synthesis step. By removing the reservoirs, a storage-free frequency curve was computed. Simulated annual peaks were analyzed using a log-Pearson Type III discharge-frequency analysis in accordance with the U.S. Water Resources Council Bulletin 17B (1981). No bias corrections were made to the frequency curves computed from the DR3M annual peaks. No skew coefficient adjustments were made to the DR3M frequency curves for the same reason that none were made to the USGS model frequency curves.

Near the end of the project, frequency curves were computed for three basins using both models and data for a longer period, 1898-1981. Model results showed less than a 1-percent change in the 50- and 100-year floods when compared with similar data for the period 1898-1973. It was concluded, therefore, that using the 1898-1973 time period of rainfall data in the frequency computations for the other 16 basins would be adequate.

Flood-frequency data from the log-Pearson Type III frequency curves for selected recurrence intervals from both models are shown in table 8.

Table 8.--Flood-frequency data from long-term synthesis

Station number	Drainage area (mi2)	Model	Q_2	Q_5	Q_{10}	Q_{25}	Q_{50}	Q_{100}
			Cubic feet per second					
02203820	8.67	USGS	1,090	1,830	2,440	3,360	4,140	5,030
		DR3M	1,510	2,470	3,200	4,240	5,090	6,010
02203835	3.43	USGS	924	1,550	2,000	2,590	3,060	3,530
		DR3M	975	1,610	2,090	2,730	3,250	3,780
02203845	.84	USGS	411	660	836	1,070	1,240	1,420
		DR3M	456	795	1,050	1,410	1,700	2,000
02203850	7.50	USGS	1,670	2,610	3,270	4,130	4,780	5,440
		DR3M	1,890	3,020	3,830	4,890	5,700	6,530
02203870	3.68	USGS	937	1,490	1,880	2,390	2,780	3,170
		DR3M	1,100	1,850	2,430	3,230	3,890	4,580
02203884	1.88	USGS	667	1,090	1,390	1,780	2,090	2,400
		DR3M	868	1,420	1,840	2,410	2,870	3,350
02336080	19.1	USGS	1,970	2,970	3,710	4,730	5,540	6,410
		DR3M	2,220	3,390	4,300	5,640	6,770	8,020
02336090	.32	USGS	152	260	339	446	530	616
		DR3M	266	473	619	806	944	1,080
02336102	2.19	USGS	657	1,120	1,470	1,950	2,340	2,740
		DR3M	663	1,190	1,610	2,230	2,760	3,340
02336150	5.29	USGS	950	1,490	1,880	2,410	2,820	3,250
		DR3M	1,100	1,780	2,270	2,960	3,500	4,070
02336180	11.0	USGS	1,010	1,610	2,050	2,660	3,140	3,660
		DR3M	1,480	2,370	3,020	3,920	4,630	5,370
02336200	.98	USGS	502	812	1,030	1,320	1,540	1,760
		DR3M	535	918	1,210	1,610	1,940	2,280
		DR3M*	501	836	1,080	1,410	1,670	1,940
02336238	.92	USGS	511	834	1,060	1,370	1,600	1,840
		DR3M	454	821	1,120	1,550	1,920	2,320
02336325	1.35	USGS	562	859	1,070	1,340	1,540	1,750
		DR3M	788	1,260	1,610	2,070	2,430	2,810
02336690	.52	USGS	237	377	472	592	680	766
		DR3M	247	439	587	794	960	1,140
02336697	.21	USGS	90.4	152	197	256	302	348
		DR3M	108	209	287	394	479	566
02336700	.79	USGS	367	592	748	947	1,100	1,240
		DR3M	438	773	1,030	1,380	1,660	1,950
02336705	8.80	USGS	2,460	4,060	5,210	6,740	7,920	9,130
		DR3M	3,180	5,590	7,390	9,810	11,700	13,700
02337081	.88	USGS	430	691	871	1,100	1,270	1,450
		DR3M	496	934	1,290	1,800	2,220	2,680

* DR3M with storage.

Table 8 also includes flood-frequency data from the log-Pearson Type III frequency analysis from the DR3M model with storage for station 02336200. Station 02336200 was selected to run the long-term synthesis using the DR3M model with storage and to perform the flood-frequency analysis, because this station had only two reservoirs caused by embankment storage and there was no flow over the road at those two embankments. No other stations were tested in this manner because the model comparisons were acceptable and because of time and financial constraints. Flood-frequency data with storage from the DR3M model are lower than flood-frequency data without storage from the DR3M model in all storms, as expected, with the percentage increasing as the recurrence interval increases. Flow does not get over the road at the two roadway embankments which act as detention reservoirs and, therefore, the greater the discharge, the greater the effect of storage on the model with storage.

Another comparison of flood-frequency data from table 8 for this station is between the USGS model with storage and the DR3M model with storage. At lower recurrence intervals, the computed peaks by the two models are very close and at the 100-year recurrence interval, the USGS peak is within 9 percent of the DR3M peak. Thus the two models produce comparable flood-frequency data for at least this data set.

A comparison of flood-frequency data from long-term synthesis from the DR3M model results with and without storage, and the USGS model results with storage for South Fork Peachtree Creek Tributary on Scott Boulevard, at Decatur, Ga., station number 02336200, is presented in figure 3.

Regression Analysis

So that flood magnitude and frequency could be estimated for ungaged sites, the 2-, 5-, 10-, 25-, 50-, and 100-year floods obtained from the 19 basins in the study area were related to their basin characteristics. This was done by the linear, multiple-regression method described by Riggs (1968).

The regression equations provide a mathematical relation between dependent variables (2- to 100-year flood peaks) and independent variables (the basin characteristics found to be statistically significant). All variables were transformed into logarithms before analysis: (1) to obtain a linear regression model, and (2) to achieve equal variance about the regression line throughout the range (Riggs, 1968, p. 10). In the analyses performed, a 95-percent confidence limit was specified to select the significant independent variables.

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

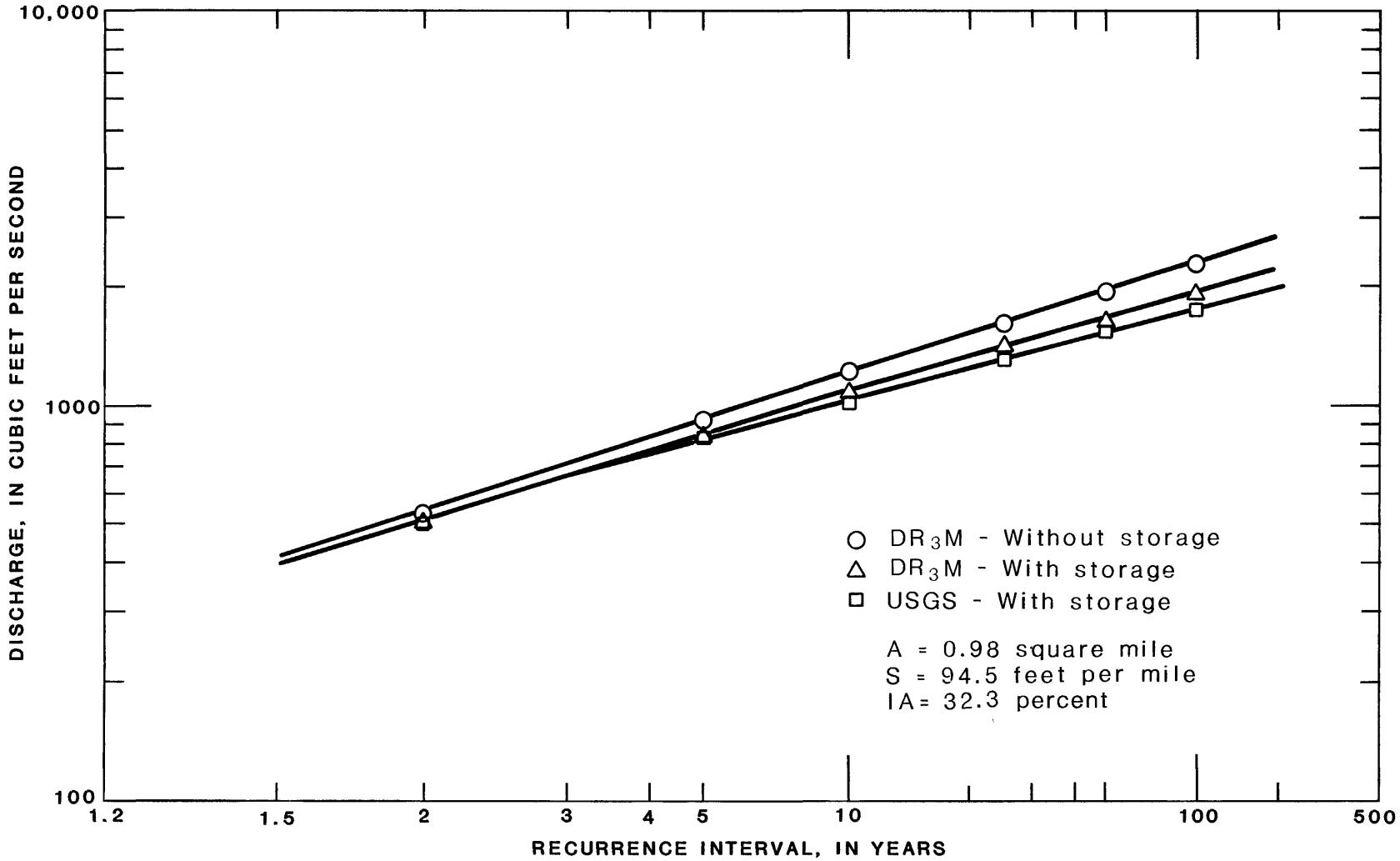


Figure 3.— Comparison of flood-frequency data from long-term synthesis from DR_3M model results with and without storage, and USGS model results with storage for South Fork Peachtree Creek tributary on Scott Boulevard at Decatur, Georgia, station 02336200.

The independent variables, or physical basin characteristics, are defined in the following paragraphs and the individual station data are shown in table 9.

Table 9.--Selected physical characteristics of urban hydrology gaging stations in Atlanta

Station number	A	S	L	IA	MEIA	USGS IA	DR3M IA	BDF
02203820	8.67	28.0	7.58	30.5	21.7	24.7	19.0	7
02203835	3.43	61.0	2.66	25.6	18.9	18.9	20.5	7
02203845	.84	67.6	1.93	30.6	23.4	23.4	20.2	7
02203850	7.50	34.8	5.91	28.2	21.0	21.6	24.8	7
02203870	3.68	37.5	3.95	25.8	19.9	19.7	19.5	7
02203884	1.88	74.1	2.22	26.7	23.4	23.1	26.8	5
02336080	19.1	16.0	7.43	31.4	26.4	29.0	29.9	4
02336090	.32	129	1.12	19.0	11.4	17.0	12.1	7
02336102	2.19	62.8	2.50	27.2	19.6	19.6	22.3	7
02336150	5.29	25.8	5.06	24.1	18.0	20.3	19.5	6
02336180	11.0	19.0	9.03	25.9	21.5	18.4	21.6	7
02336200	.98	94.5	1.47	32.3	26.2	26.2	24.1	5
02336238	.92	106	1.60	33.6	24.8	27.6	21.3	7
02336325	1.35	53.8	2.14	42.0	39.6	42.3	43.6	2
02336690	.52	90.7	1.22	20.3	14.1	17.0	16.2	7
02336697	.21	136	1.09	19.0	11.1	20.0	10.1	7
02336700	.79	75.8	1.46	28.3	18.2	16.8	16.5	7
02336705	8.80	33.7	4.95	29.5	23.5	21.8	21.4	6
02337081	.88	86.9	1.43	28.6	19.9	16.0	16.0	7

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

Channel slope (S-12).—The main channel slope, in feet per mile, as determined from topographic maps. The main channel slope was computed as the difference in elevation, in feet, at the 10 and 85 percent points divided by the length, in miles, between the two points. Values of measured slope were reduced by 12 to improve the linearity of the regression equations and improve the standard error of estimate of some of the equations by almost 3 percent. The value of the constant was determined by trial and error.

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.

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

Measured effective impervious area (MEIA).—The percent of impervious area which is directly connected to the channel drainage system. Noneffective impervious areas, such as house rooftops that drain onto a lawn, are subtracted from this total. This parameter was obtained in conjunction with measured total impervious area. When the minimum of 200 points were counted, three totals per subbasin were obtained. The first total was pervious points, the second definite impervious points such as streets and parking lots, and the third rooftops. One building out of three was field checked to determine the percent 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 subbasin to determine the MEIA percentage.

Optimized effective impervious area, USGS (USGSIA).—The optimized value of effective impervious area obtained from the USGS model.

Optimized effective impervious area, DR3M (DR3MIA).—The optimized value of effective impervious area obtained from program DR3M.

Basin development factor (BDF).—A basin development factor, which is computed by subdividing the basin into thirds (upper, middle, and lower). The prevalence of storm sewers, channel improvements, impervious channel linings, and curb and gutter streets was determined within each third of the basin. Each condition that was prevalent, was assigned a code of one (1). A total of all codes equals BDF. The range of BDF is 0 to 12 (Sauer and others, 1981).

Three-Parameter Estimating Equations

Peak discharges for the 2-, 5-, 10-, 25-, 50-, and 100-year floods were related to the eight independent variables listed in table 9, four of which are impervious area or derivatives of impervious area. Of the independent variables used, drainage area was the most significant, being statistically significant within the 1-percent level for all equations developed. Slope (S-12) was the next most significant variable, being within the 1-percent significance level for all USGS model regression equations, and within the 3-percent significance level for the DR3M regression equations. Measured total impervious area (IA) was within the 1-percent significance level for all USGS estimating equations, and from the 5- to 8-percent significance level for the DR3M equations. Even though IA was not as highly significant for the DR3M model equations, it was retained in order to provide continuity with the USGS model equations.

Length was found not to be significant at the 5-percent level for most equations, and it also had a correlation coefficient of 0.966 with drainage area. It was, therefore, not included in any of the equations. Measured effective impervious area was slightly less significant than measured total impervious area for most equations. It could be used with only a small increase in the standard error of regression. However, it is much more difficult to obtain than measured total impervious area, so it was not used in any of the equations. Optimized impervious areas from both the USGS and DR3M models would be very difficult or impossible for most users to obtain, so they were not used in any final equations. Optimized impervious area was much less significant than either of the other two impervious-area variables for both models.

The parameter BDF also proved to be insignificant, because most of the 19 basins in this study were made up of residential type development, which resulted in very little range in this parameter.

Detention storage, caused by roadway embankments, was not included as an independent variable in the regression analysis because the upper range of the elevation-outflow-storage tables, which were prepared for the DR3M model calibrations, was not high enough to handle the large floods produced by the long-term simulations. These tables could be extended at the points of known detention storage as determined in the calibration phase, but there could be many other areas of storage produced by the much higher floods of long-term simulation. Without the aid of the calibration phase and its observed flood events, these additional points of detention storage would be difficult to locate and quantify. In most instances, the largest observed flood peaks used in calibration were much smaller than the large peaks from long-term simulation. To prepare these tables for the complete range in peak discharges and at the additional points of detention storage would require computation of flow over the road in most instances and extending or creating the outflow ratings and storage tables in all instances. Detention storage is difficult to quantify, because its effect varies with the magnitude of the flood. When increasing discharge causes flow over the road, the increasing rate of storage will be reduced. In basins with multiple crossings, this effect is very complex and not subject to regionalization.

The primary reason detention storage was not included in the model is that the user would find it very difficult to compute this parameter at ungaged sites. Detailed field surveys would be needed based on preliminary runs with the DR3M model. Thus, for regression analyses the USGS model was used with its average storage condition, and the DR3M model with its storage-free condition.

It is left to the user to determine if there is storage in the basin being analyzed. If no storage is evident, use the DR3M equations. If there is storage in the basin, determined by field inspections or by carefully examining the best available topographic maps, then use the USGS equations or a combination of both equations.

A comparison of the DR3M, the USGS, and the rural Piedmont equations is presented in figure 4. The rural Piedmont equation uses only drainage area as an independent variable, and is of the form $Qn = aA^b$.

Table 10 gives the three-parameter estimating equations for both sets of model results, along with the standard error of regression and the standard error of prediction. The accuracy of each equation can be expressed by

Table 10.—Summary of three-parameter estimating equations

Recurrence interval (years)	Model	Estimating equation	Standard error of regression (percent)	Standard error of prediction (percent)
2	USGS	$6.68A^{.87}(S-12)^{.43}\ IA^{.70}$	± 17	± 31
2	DR3M	$20.0A^{.81}(S-12)^{.32}\ IA^{.56}$	± 20	± 35
5	USGS	$11.6A^{.90}(S-12)^{.48}\ IA^{.62}$	± 16	± 27
5	DR3M	$36.8A^{.80}(S-12)^{.36}\ IA^{.50}$	± 20	± 32
10	USGS	$15.5A^{.92}(S-12)^{.50}\ IA^{.58}$	± 15	± 25
10	DR3M	$46.2A^{.80}(S-12)^{.37}\ IA^{.50}$	± 20	± 31
25	USGS	$21.0A^{.93}(S-12)^{.51}\ IA^{.55}$	± 14	± 24
25	DR3M	$58.8A^{.79}(S-12)^{.38}\ IA^{.50}$	± 20	± 31
50	USGS	$25.8A^{.94}(S-12)^{.52}\ IA^{.52}$	± 13	± 24
50	DR3M	$65.9A^{.79}(S-12)^{.39}\ IA^{.52}$	± 20	± 31
100	USGS	$30.7A^{.95}(S-12)^{.53}\ IA^{.51}$	± 13	± 25
100	DR3M	$72.4A^{.79}(S-12)^{.39}\ IA^{.54}$	± 20	± 32

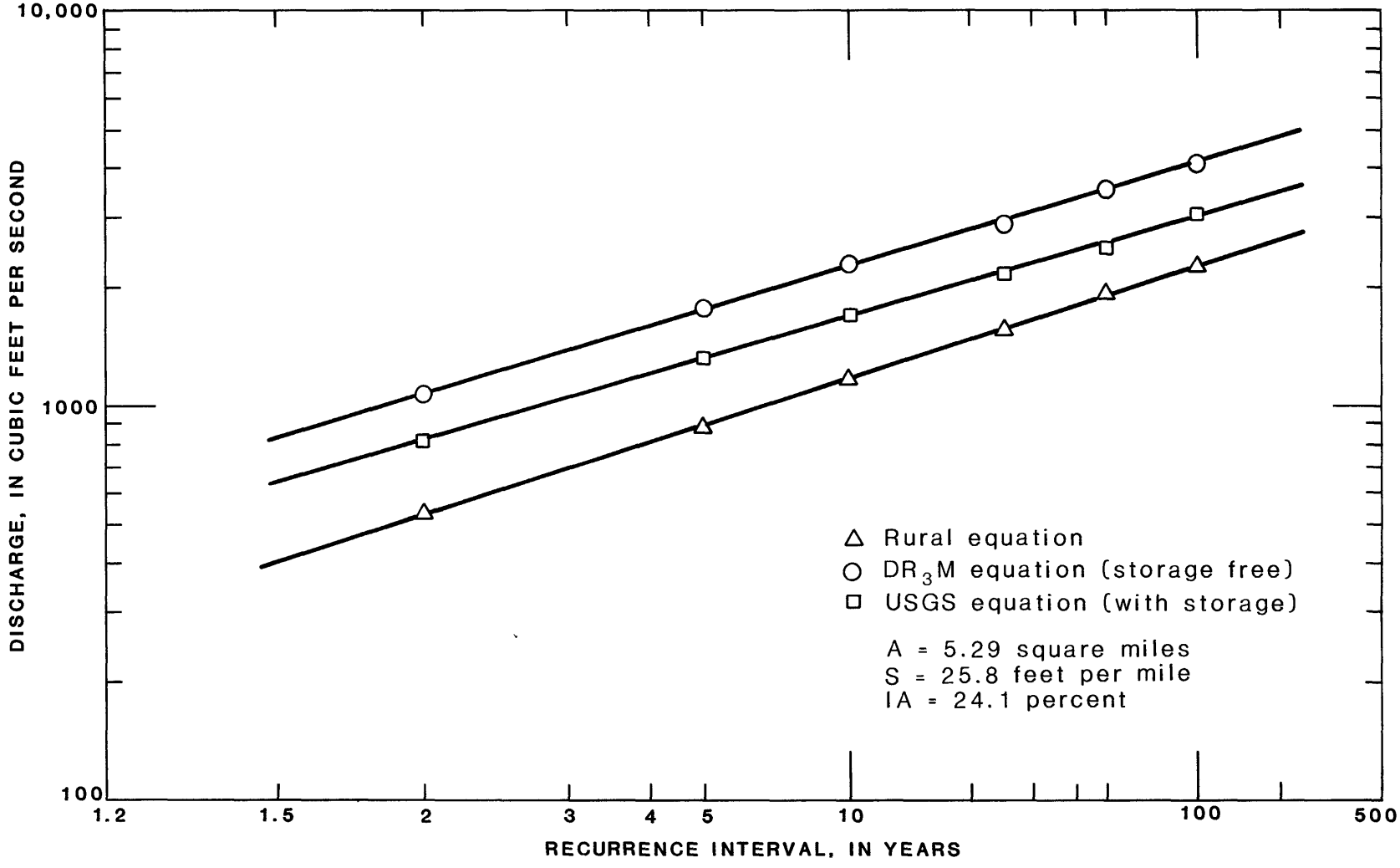


Figure 4.— Comparison of DR_3M, the USGS, and the rural Piedmont equations for station 02336150, South Fork Peachtree Creek at Montreal Road, at Clarkston, Georgia.

the standard error of regression (or estimate), which is, by definition, one standard deviation on each side of the regression line and contains about two-thirds of the data within this range. The standard error of regression is a measure of how well the equations estimate flood-frequency data at the 19 gaged sites used in this study, whereas the standard error of prediction is a measure of the estimating ability at all sites, gaged and ungaged. Because the DR3M model uses spacially distributed parameters it has greater variation (more degrees of freedom) than the lumped parameter USGS model. The estimating equations for the USGS models also have lumped effects in the independent variables. Therefore, the USGS estimating equations may not be giving a better answer even though they have lower standard errors of regression.

Computation of the standard error of prediction (SEP) is described by Hardison (1971). The interstation correlation is a factor in computation of the SEP and was assumed to be 0.9, based on information obtained from other similar studies (V. B. Sauer, U.S. Geological Survey, oral commun., 1981). Other factors necessary for the computation of SEP were the average standard deviation and the average skewness of the logarithms of annual peak discharges. These factors were computed from the synthesized data for each station and were 0.255 and -0.157, respectively, for the USGS model and 0.281 and -0.100, respectively, for the DR3M model. As described previously, the number of stations used in each regression was 19, and the number of independent variables in each regression was 3.

The equivalent length of synthesized record is also a factor in the computation of the standard error of prediction. Because 75 years of synthetic data are certainly not equivalent to 75 years of observed data, it was necessary to compute the equivalent length of the synthesized data for use in this computation. Table 11 gives the recurrence interval and its corresponding equivalent length of synthesized data. The results in table

Table 11.—Recurrence interval and corresponding equivalent length of record of synthesized data

Recurrence interval (years)	Equivalent length of record of synthesized data (years)
2	5
5	9
10	14
25	19
50	21
100	21

10 were computed from information given in a report by Lichty and Liscum (1978) in which they studied the synthesis of flood peak data at 98 gaging stations in the Eastern United States. They assumed, for purposes of computing the equivalent length of synthesized data, that model error, or space sampling error, is analogous to their average map-model error variance. This estimate of model error was converted to equivalent length of synthesized data by the methods described by Hardison (1971). These methods also require an estimate of the average skewness and average standard deviation of the logarithms of the peak discharge, which were assumed to be equal to the observed values given by Lichty and Liscum (1978). The results in table 11 are based on studies of the USGS model and are also assumed to apply to the DR3M model.

Testing of Regression Equations

Four tests or evaluations are generally required to establish the soundness of regression equations. The first test is the standard error of regression, which has been explained and presented in prior sections of this report. Another is the bias test which was performed on parameter bias only because no geographic bias would be evident in the relatively small area surrounding the 19 basins used in the regression. The test for parameter bias was made by plotting the residuals (difference between observed and predicted floods) versus each of the independent variables for all stations. 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. One station (02336705) did seem to exhibit a bias in that the DR3M 100-year flood was under-predicted by about 40 percent and the USGS 100-year flood was under-predicted by about 25 percent. After carefully examining the verified stage-discharge relation, rainfall, calibrations, and physical basin characteristics at this station, no cause for deletion was found and the station was left in the analysis.

A verification test was possible only at the 2-year (median) flood level, because of the short periods of record (5 to 8 years) and the small sample size (19 stations). The median flood computed from observed data at each station was plotted against the predicted 2-year floods for both the USGS and DR3M models. Figure 5 is a plot of median floods from observed data versus 2-year floods predicted by the USGS equation. Figure 6 is a plot of median floods from observed data versus 2-year floods predicted by plot of median floods from observed data versus 2-year floods predicted by the DR3M equation. Careful examination of figure 6 indicates that the estimated 2-year floods generally are higher than the median observed floods. This is as expected, because the DR3M predicted floods are storage free, as explained in an earlier section.

The fourth test was to analyze the sensitivity of the 2-, 25-, and 100-year computed peak discharges to errors in the three independent variables in both the USGS and the DR3M equations. The test results are shown in table 12 for the USGS model equations and in table 13 for the DR3M model equations. These tables were computed by assuming that all independent variables were constant, except the one being tested for sensitivity. Slope (S-12) was tested at different levels because of the wide range in values and the constant 12 subtracted from this variable make the small values of slope very sensitive. If the measured value of slope is small, an error of

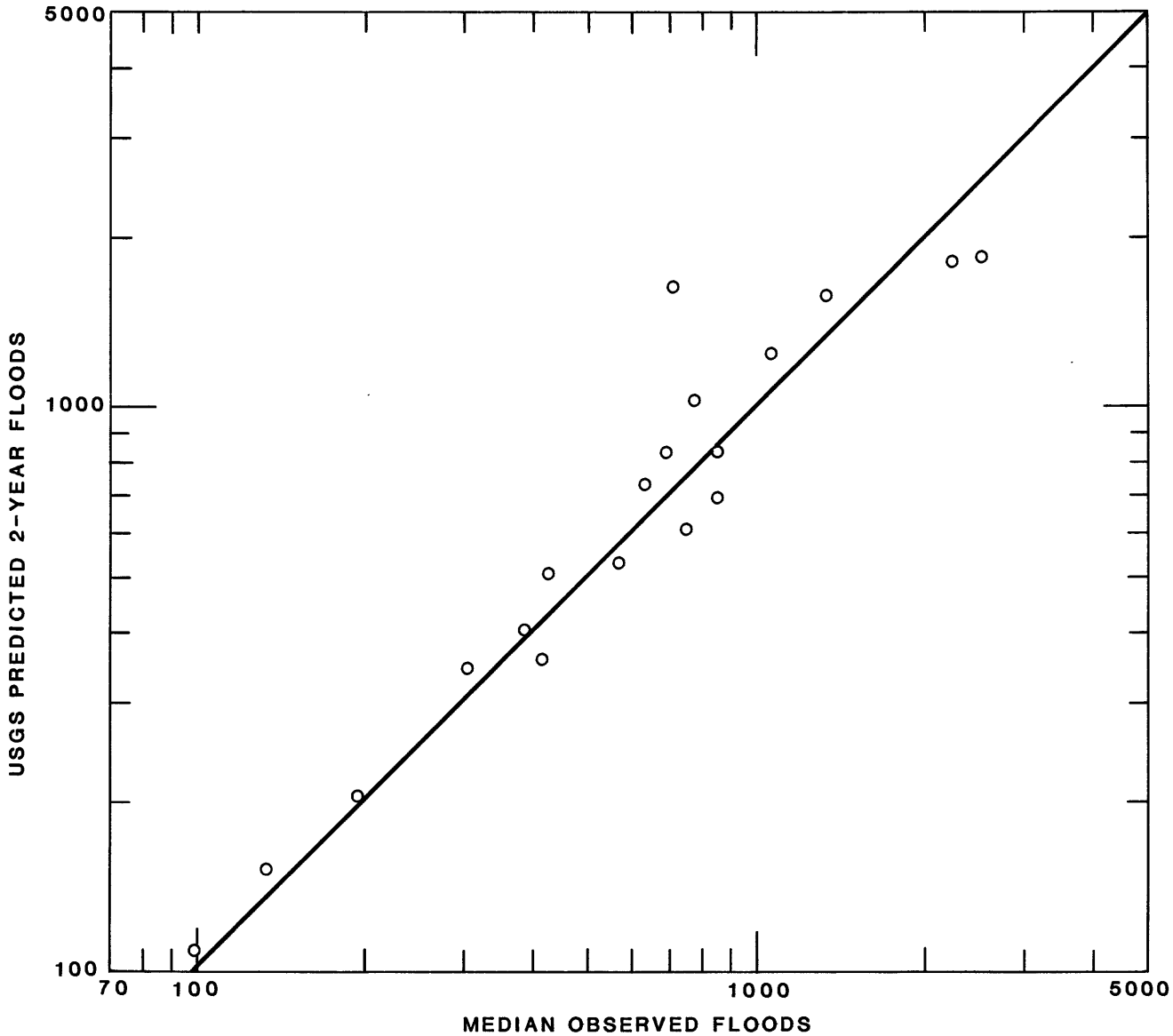


Figure 5. — Comparison of median observed floods with predicted 2-year floods from the USGS equation.

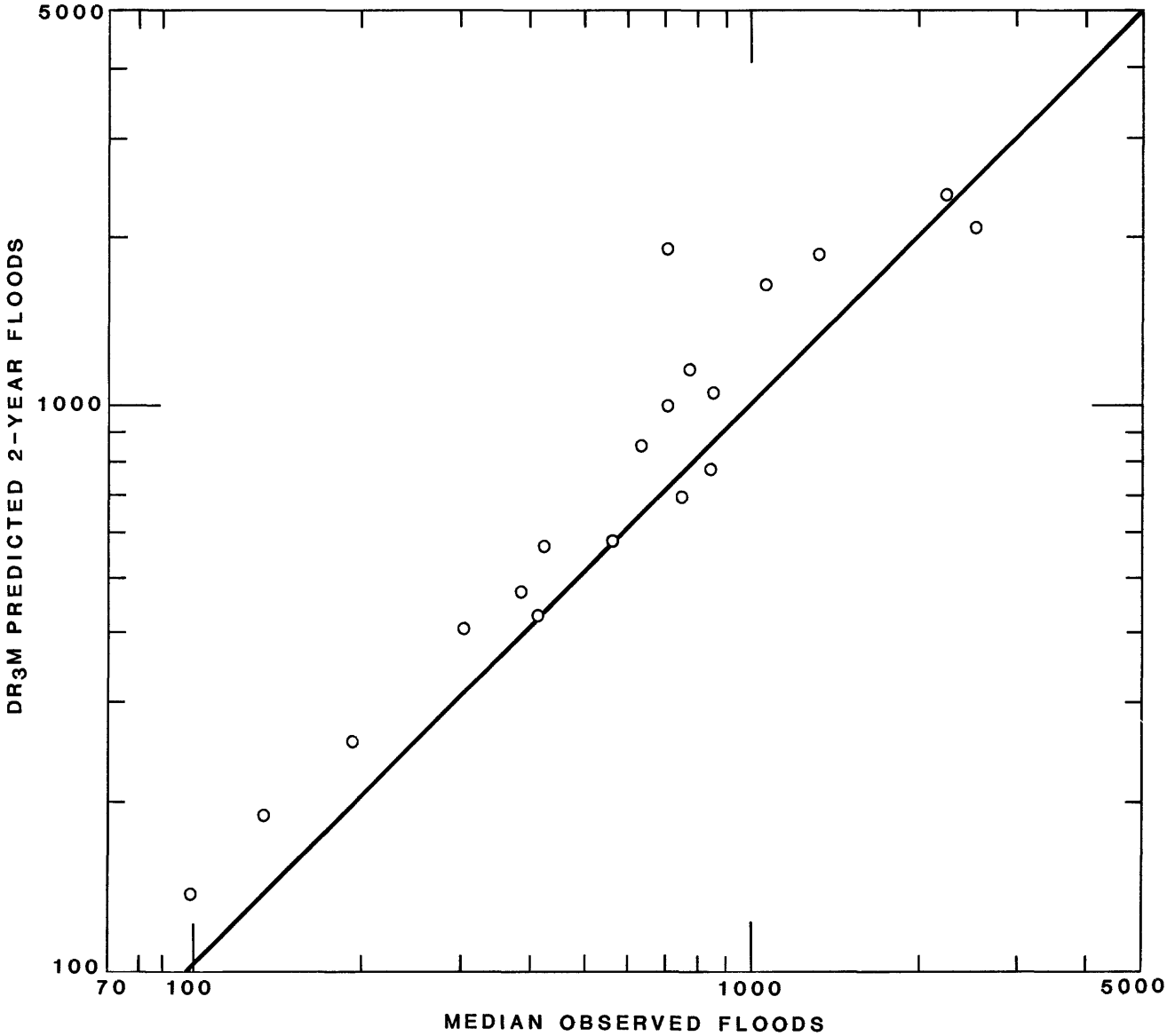


Figure 6.— Comparison of median observed floods with predicted 2-year floods from the DR_3M equation.

Table 12.—Sensitivity of computed peak discharges to errors in independent variables with the USGS equations

Percent error in independent variable	Independent variables											
	A	S-12	S-12	IA	A	S-12	S-12	IA	A	S-12	S-12	IA
		Small values	Large values			Small values	Large values			Small values	Large values	
	Percent error in computed 2-yr flood				Percent error in computed 25-yr flood				Percent error in computed 100-yr flood			
+50	+42.3	+44.6	+20.8	+32.8	+45.8	+54.8	+25.1	+25.0	+47.0	+57.5	+26.2	+23.0
+25	+21.4	+25.0	+11.0	+16.9	+23.1	+30.2	+13.2	+13.1	+23.6	+31.6	+13.8	+12.0
+10	+8.6	+10.9	+4.6	+6.9	+9.3	+13.0	+5.5	+5.4	+9.5	+13.6	+5.7	+5.0
-10	-8.8	-12.7	-4.9	-7.1	-9.3	-14.9	-5.8	-5.6	-9.5	-15.4	-6.0	-5.2
-25	-22.1	-38.6	-13.0	-18.2	-23.5	-44.0	-15.2	-14.6	-23.9	-45.2	-15.7	-13.6
-50	-45.3	-56.7*	-29.2	-38.4	-47.5	-62.9*	-33.6	-31.7	-48.2	-64.4*	-34.6	-29.8

* Minimum value of 13 ft/mi used for slope.

Table 13.--Sensitivity of computed peak discharges to errors in independent variables with the DR3M equations

Percent error in independent variable	Independent variables											
	A	S-12	S-12	IA	A	S-12	S-12	IA	A	S-12	S-12	IA
		Small values	Large values			Small values	Large values			Small values	Large values	
	Percent error in computed 2-yr flood				Percent error in computed 25-yr flood				Percent error in computed 100-yr flood			
+50	+38.9	+31.6	+15.1	+25.5	+37.8	+38.5	+18.2	+22.5	+37.8	+39.7	+18.7	+24.5
+25	+19.8	+18.0	+8.1	+13.3	+19.3	+21.8	+9.7	+11.8	+19.3	+22.4	+10.0	+12.8
+10	+8.0	+8.0	+3.4	+5.5	+7.8	+9.6	+4.0	+4.9	+7.8	+9.8	+4.2	+5.3
-10	-8.2	-9.6	-3.7	-5.7	-8.0	-11.3	-4.3	-5.1	-8.0	-11.6	-4.4	-5.5
-25	-20.8	-30.4	-9.8	-14.9	-20.3	-35.0	-11.5	-13.4	-20.3	-35.8	-11.8	-14.4
-50	-43.0	-46.4*	-22.6	-32.2	-42.2	-52.3*	-26.2	-29.3	-42.2	-53.2*	-26.8	-31.2

* Minimum value of 13 ft/mi used for slope.

a given percentage will have a greater effect than if the measured value were large. For example, if the measured value of slope is 19.0 ft/mi and the value used in the USGS 100-year flood equation is 50 percent greater, or 28.5 ft/mi, then the computed USGS 100-year flood would be in error by +57.5 percent. However, if the same +50-percent error occurs when the true value of slope is 128 ft/mi, then the computed USGS 100-year flood would be only +26.2 percent in error.

Impervious area, or IA, was more sensitive at the 2-year flood level than at the 25-year flood level, and more sensitive at the 25-year level than at the 100-year level in the USGS equations. However, this was not true with the DR3M equations. With the DR3M equations, IA was more sensitive at the 2-year flood level than at the 25-year flood level, as in the USGS equations, but IA was slightly less sensitive at the 25-year level than at the 100-year level. This is probably due to the fact that IA was within the 1-percent significance level for all USGS estimating equations, and from the 5- to 8-percent significance level for the DR3M equations. The trend that IA follows in the USGS equations agrees with the theory that IA has more effect on the 2-year flood than on the 100-year flood.

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

Variable	Minimum	Maximum	Units
A	0.2	25	square miles
S	13*	175	feet per mile
IA	19	50	percent

* Minimum value of slope for use in equations is 13 feet per mile. No basins, within the limits given above, having slopes less than 13 feet per mile have been observed in the Atlanta area. If so, use 13 feet per mile.

SUMMARY

Rainfall-runoff data were collected at 19 drainage basins in the Atlanta area ranging in size from 0.21 to 19.1 mi^2 and in impervious area from 19 to 42 percent. The DR3M and the USGS models were calibrated for the 19 basins. The DR3M model was verified at seven basins, and the USGS model was verified at six basins.

After the models were successfully calibrated, long-term rainfall data from the National Weather Service, Atlanta station, were used to synthesize about 75 years of annual peaks. These synthesized peaks were used to develop flood-frequency relations at each site.

Multiple-regression analysis was then used to define relations between the flood-frequency data and selected basin characteristics, of which drainage area, channel slope, and measured total impervious area were found to be significant. Using these relations, the magnitude and frequency of floods at ungaged basins can be estimated. The user must estimate the effect of storage in the ungaged basin to select the proper equation for use or weight the results of computations using the with storage (USGS) and storage free (DR3M) equations.

The user is cautioned not to extend the range of independent variables used in the estimating equations outside the limits of the variables listed in the section on testing of regression equations.

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