EFFECTS OF URBAN FLOOD-DETENTION RESERVOIRS ON PEAK DISCHARGES IN GWINNETT COUNTY, GEORGIA

By Glen W. Hess and Ernest J. Inman

U.S. GEOLOGICAL SURVEY

Water-Resources Investigations Report 94-4004

Prepared in cooperation with

GWINNETT COUNTY, GEORGIA

Atlanta, Georgia

1994

U.S. DEPARTMENT OF THE INTERIOR

BRUCE BABBITT, Secretary

U.S. GEOLOGICAL SURVEY

Robert M. Hirsch, Acting Director

For additional information
write to:
District Chief
U.S. Geological Survey
Water Resources Division
3039 Amwiler Road, Suite 130
Atlanta, GA 30360-2824

Copies of this report can be purchased from:
U.S. Geological Survey
Earth Science Information Center
Open-File Reports Section
Box 25286, MS 517
Denver Federal Center
Denver, CO 80225

CONTENTS

ILLUSTRATIONS

TABLES

CONVERSION FACTORS AND VERTICAL DATUM

CONVERSION FACTORS

Multiply	By	To obtain
	Length	
inch (in.)	25.4	millimeter
foot (ft)	0.3048	meter
	Area	
square mile (mi^2)	2.590	square kilometer
	Volume	
cubic feet (ft^3)	0.02832	cubic meters
	Flow	
cubic feet 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 - 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.

EFFECTS OF URBAN FLOOD-DETENTION RESERVOIRS ON PEAK DISCHARGES IN GWINNETT COUNTY, GEORGIA

By Glen W. Hess and Ernest J. Inman

ABSTRACT

The effects of flood-detention reservoirs on peak discharges along downstream reaches in six urban drainage basins in Gwinnett County, Georgia, were studied during 1986-93 using the U.S. Geological Survey's Distributed Routing Rainfall-Runoff Model (DR3M). Short-term rainfall-runoff data were collected at selected stations in six urban drainage basins in Gwinnett County. The basins range in size from 0.10 to 0.37 square miles and contain from 15- to 35- percent impervious areas. Each basin contains from two to six flood-detention reservoirs. The DR3M was calibrated using short-term rainfall-runoff data collected (1986-92) at each station. The model then was used to simulate long-term (1898-1980) peak discharges for these stations for conditions representing various amounts of detention ranging from the existing condition with all flood-detention reservoirs in place to the natural condition with no reservoirs. Flood-frequency relations were developed from the simulated annual peak discharges for each of these conditions by fitting the logarithms of the annual peak discharge data to a Pearson type III distribution curve. The effect of each flood-detention reservoir on peak discharges downstream was determined by comparison of peak discharges simulated with and without the flood-detention reservoirs. The cumulative effect of all flood-detention reservoirs in a basin on peak discharges downstream was determined by comparison of peak discharges for a flood with a given recurrence interval simulated with and without the reservoirs. Results of these comparisons indicate that removal of an individual flood-detention reservoir during simulations changes peak discharges from -1 to 24 percent for the 2-year recurrence interval, from -1 to 27 percent for the 10-year recurrence interval, and from -2 to 31 percent for the 100-year recurrence interval. The cumulative effect of removing all of the reservoirs from each of the six basins during simulation increases peak discharges from 1 to 38 percent for the 2-year recurrence interval, from 1 to 37 percent for the 10-year recurrence interval, and from 3 to 31 percent for the 100-year recurrence interval.

In this study of six basins, several factors influenced the effect of flood-detention reservoirs on peak discharges downstream. The contributing drainage area, the maximum storage capacity, the outflow-structure capacity, and the elevation-to-storage relation of the flood-detention reservoir affected peak discharges in several basins. The location in the drainage basin and number of flood-detention reservoirs affected peak discharges in some basins.

INTRODUCTION

Peak-discharge characteristics of many urban streams in Gwinnett County, Georgia, are affected by flood-detention reservoirs specifically constructed to minimize the effect of storm runoff from developed areas on the streams. Gwinnett County ordinances require that developers provide flood-detention reservoirs so that post-development peak discharges do not exceed peak discharges for natural (pre-development) conditions. However, little is known about the effect of flood-detention reservoir outflows on the discharge characteristics of receiving streams. To quantify the effects of these detention reservoirs on peak discharges of urban streams, the U.S. Geological Survey (USGS), in cooperation with Gwinnett County, conducted an investigation during 1986-93 of six representative urban stream basins containing flood-detention reservoirs.

Purpose and Scope

This report describes the results of a study to quantify the effects of flood-detention reservoirs on peak discharges along downstream reaches in six urban drainage basins in Gwinnett County, Georgia. The flood-detention reservoirs evaluated as part of this study included only those structures specifically constructed for detention of stormwater runoff. The effects of incidental temporary detention of stormwater runoff upstream from road embankments and other constrictions of the stream were not evaluated.

The study was based on an analysis of peak discharges and rainfall data for a relatively short period of record -- about 6 years. A 6-year flood record is a small sample and may poorly represent the long-term distribution of floods at a station. Thus, a frequency analysis of discharge based on that record would be a weak prediction of the 50- and 100-year peak discharges at the stations. To provide a more reliable prediction, a mathematical routing rainfall-runoff model was calibrated using short-term observed rainfall-runoff data; and then, the model was used to synthesize long-term peak discharges from rainfall records.

Previous Studies

In his report, "UROS4: Urban Flood Simulation Model, Part 1, Documentation and User's Manual", Lumb (1975) explained the use of the UROS4 model to simulate an annual series of flood peaks for use in a frequency analysis at a selected point. James and Lumb (1975) applied this model to eight watersheds in DeKalb County, Georgia, using limited observed data for verification.

Preliminary Flood-Frequency Relations for Urban Streams in Metropolitan Atlanta were presented by Golden (1977) based on a technique developed by Sauer (1974) that used the natural flood-frequency and rainfall-frequency characteristics of local areas in Oklahoma. Urban flood-frequency relations from natural flood-frequency relations were developed by Sauer (1974) by adding factors to account for local rainfall-frequency characteristics, the percentage of impervious area in a basin, and the percentage of a basin served by storm sewers. Sauer's (1974) technique to determine flood-frequency relations for urban streams on a statewide basis in Georgia was used by Price (1979). Simplified equations that can be used on small watersheds (200 acres or less) in DeKalb County, Ga., to estimate the magnitude and frequency of floods were presented by Jones (1978).

An updated technique for estimating the magnitude and frequency of floods on small streams in the Metropolitan Atlanta area was presented by Inman (1983). This technique involved the use of two models, the U.S. Geological Survey (USGS) rainfall-runoff model (Dawdy and others, 1972) and Distributed Routing Rainfall-Runoff Model (DR3M) (Alley and Smith, 1982) calibrated with observed data from 19 stations. These two models were used to synthesize the long-term annual peak discharges for the 19 stations. The 2- to 100-year flood discharges were calculated for each basin from these synthetic, long-term annual-peak discharges using the Pearson Type III distribution curve. Multiple-regression analyses defined relations between flood-frequency station data and certain physical characteristics of a basin. Drainage area, channel slope, and measured total impervious area were determined to be statistically significant. Inman (1983) indicated that these relations can be used to estimate the magnitude and frequency of floods at ungaged basins in the Metropolitan Atlanta area.

Equations and several other techniques for estimating flood-frequency relations for urban watersheds on a nationwide basis were presented by Sauer and others (1983); these techniques were based on an analysis of data from many urban basins, including five basins from the Atlanta area. Inman (1986) provided a technique for simulating flood hydrographs and estimating lag-time at ungaged stations in the Atlanta area using previously developed equations for estimating peak-flow, Inman (1983). Multiple-regression analysis defined relations between lag-time and certain physical characteristics of the basin; relations between lag-time and drainage area, slope, and impervious area were statistically significant in the Metropolitan Atlanta urban area.

A technique for estimating the magnitude and frequency of floods on ungaged stations in urban areas throughout the State of Georgia was presented by Inman (1988). He used the USGS rainfall-runoff model (Dawdy and others, 1972) calibrated with data from 45 urban drainage basins in six urban areas in Georgia to synthesize long-term annual-peak discharges. Flood-frequency relations were developed from these long-term annual-peak discharges by fitting the logarithms of the annual-peak discharge data to a Pearson type III distribution curve. Multiple-regression analysis then was used to define relations between the flood-frequency station data and certain physical characteristics of the basin, of which drainage area, measured total impervious area and equivalent rural discharge were determined to be significant. These relations can be used to estimate the magnitude and frequency of floods at ungaged urban streams throughout Georgia.

Techniques that can be used to estimate urban peak-discharge-frequency relations, flood hydrographs, and flood volumes for ungaged urban streams in the Piedmont and Coastal Plain Provinces of South Carolina were described by Bohman (1992). Bohman used data from stations in South Carolina, Georgia, and North Carolina.

Acknowledgments

The authors thank the staff of the U.S. Department of Commerce, National Weather Service, Asheville, N.C., for providing long-term rainfall and evaporation data. Thanks also are extended to Abrams Aerial Survey Corporation for providing topographic maps and aerial photography of the area.

DATA COLLECTION AND PROCESSING

Rainfall-runoff data were collected from 1986-92 at six stations in Gwinnett County, Georgia, and processed for use in the DR3M. Physical characteristics of the basin were determined from field surveys, topographic maps, and aerial photographs.

Basin Selection

Extensive field reconnaissance of about 60 basins was required to select the six basins used in this study. The following factors were considered in the selection of the six study basins:

- drainage area;
- channel length;
- number of flood-detention reservoirs;
- rain-gage location;
- hydraulic characteristics at the station and flood-detention reservoir; and
- land-use stability.

Many stations were excluded because of their hydraulic characteristics or because they contained no suitable rain-gage location. The remaining basins were delineated on USGS 7-1/2 minute topographic maps, and approximate drainage areas were determined. Six stations then were selected for study from those basins deemed suitable. The selected basins generally had the best hydraulic characteristics for theoretical computations of peak discharges at the station and flood-detention reservoirs, and also the most suitable rain-gage locations. Final drainage areas were delineated from 5-ft contour topographic maps obtained from Abrams Aerial Survey Corporation[1] (AASC). The drainage basins selected for study range in size from 0.10 to 0.37 square miles and contain 15- to 35-percent impervious area. Each of the basins contain from two to six stormwater detention reservoirs. Location of the selected stations are listed in table 1 and shown in figure 1. Detailed maps of the six basins are shown in figures 2-7.

---

[1] The use of trade names in this report is for descriptive purposes only and does not constitute endorsement of products by the U.S. Geological Survey.

Table 1. -- Gaging stations in Gwinnett County, Georgia, used in this study

Site number (fig. 1)	USGS station number	Station name	Location
1	02205230	Wolf Creek near Suwannee	Lat 34°00'04", long 84°02'57" Gwinnett County, at culvert on Dean Road near Suwannee
2	02206105	Jackson Creek near Lilburn	Lat 33°53'12", long 84°12'42", Gwinnett County, at culvert on Angels Lane near Lilburn
3	02206136	Jackson Creek tributary near Lilburn	Lat 33°53'19", long 84°10'59", Gwinnett County, at culvert on Williams Road near Lilburn
4	02206165	Jackson Creek tributary 2 near Lilburn	Lat 33°54'09", long 84°10'10", Gwinnett County, at culvert on Worcester Place near Lilburn
5	02206465	Watson Creek tributary 2 at Snellville	Lat 33°51'46", long 84°02'07", Gwinnett County, at culvert on Tanglewood Drive at Snellville
6	02335347	Crooked Creek tributary 2 near Norcross	Lat 33°57'24", long 84°14'43", Gwinnett County, at culvert on Holcomb Bridge Road near Norcross

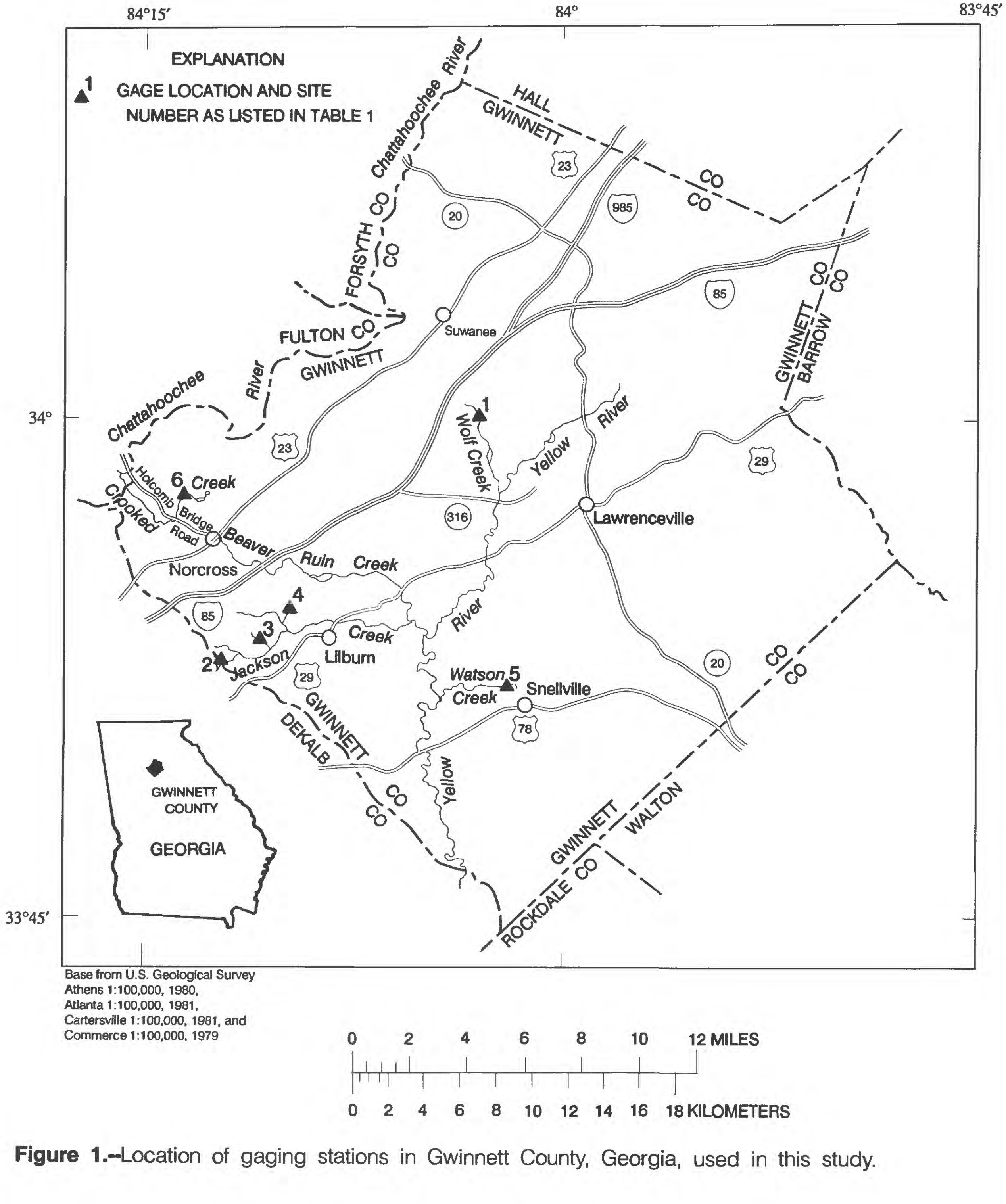
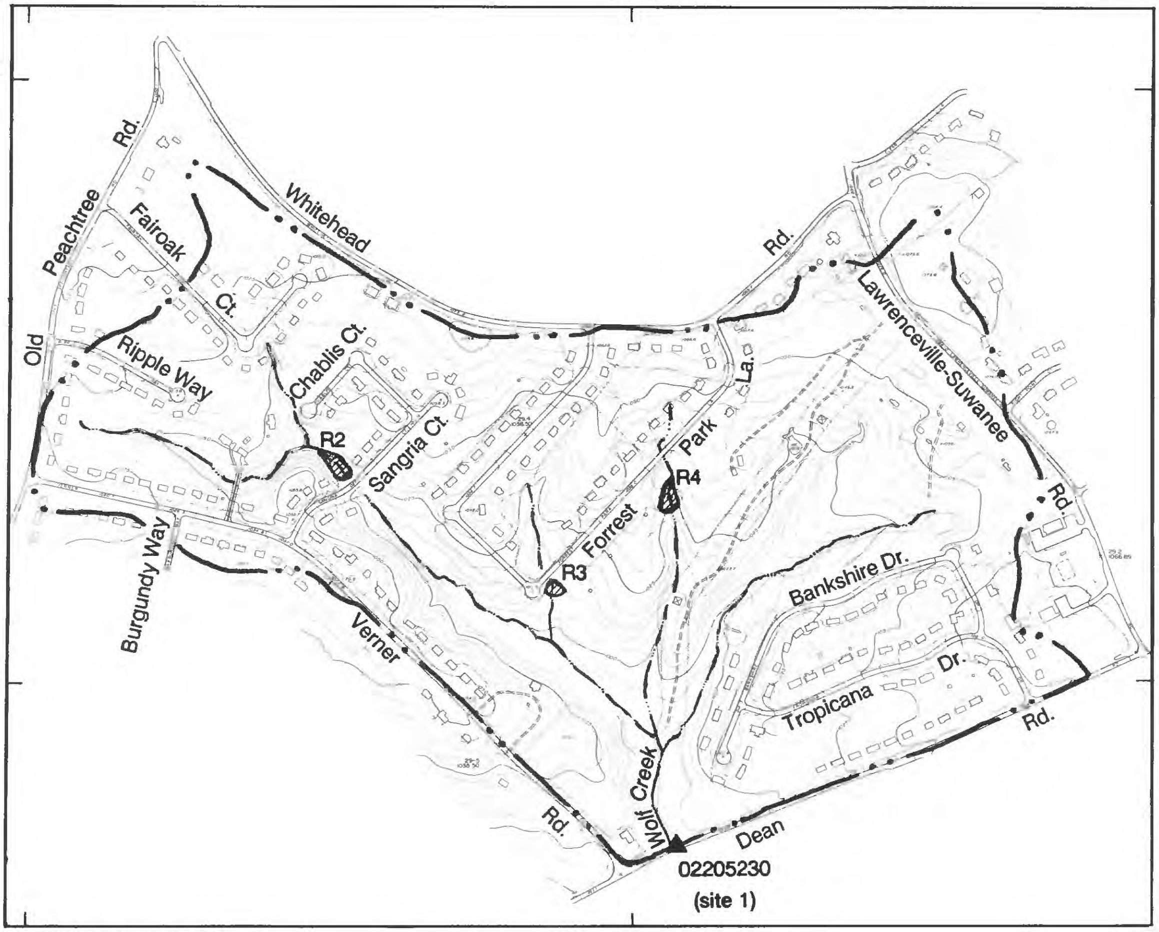


Figure 1.--Location of gaging stations in Gwinnett County, Georgia, used in this study.



Base printed with permission from
Abrams Aerial Survey Corporation
Dean Road Site 1:2,400, 1989

0 400 800 1,200 1,600 2,000 FEET

0 100 200 300 400 500 600 METERS

EXPLANATION

R2 DETENTION RESERVOIR AND IDENTIFICATION NUMBER

DRAINAGE BASIN BOUNDARY

STREAM

02205230 (site 1) GAGING STATION AND IDENTIFICATION NUMBER

Figure 2.--Drainage basin and flood-detention reservoirs for site 1, Wolf Creek at Dean Road near Suwanee, Georgia.

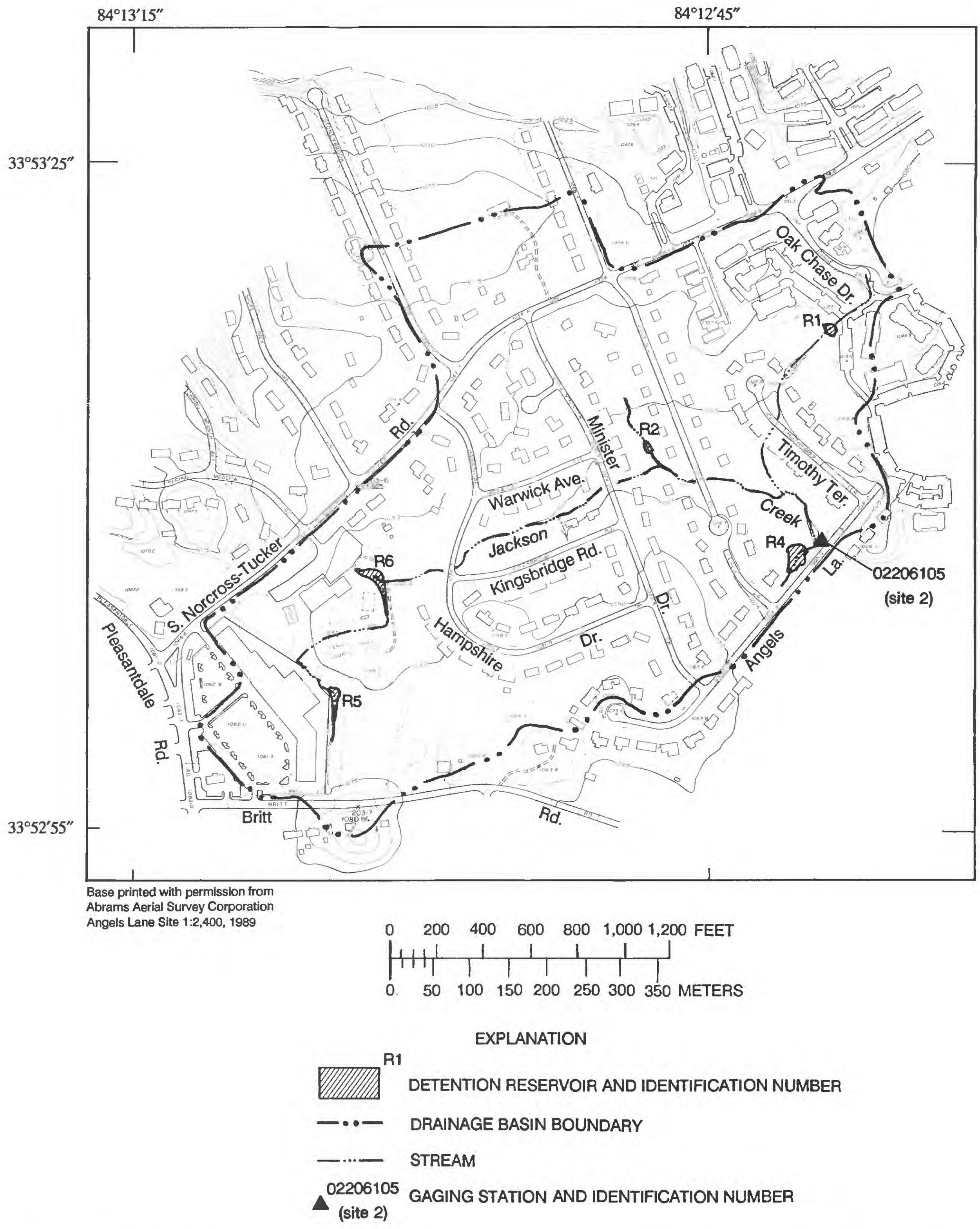


Figure 3.--Drainage basin and flood-detention reservoirs for site 2, Jackson Creek at Angels Lane near Lilburn, Georgia.

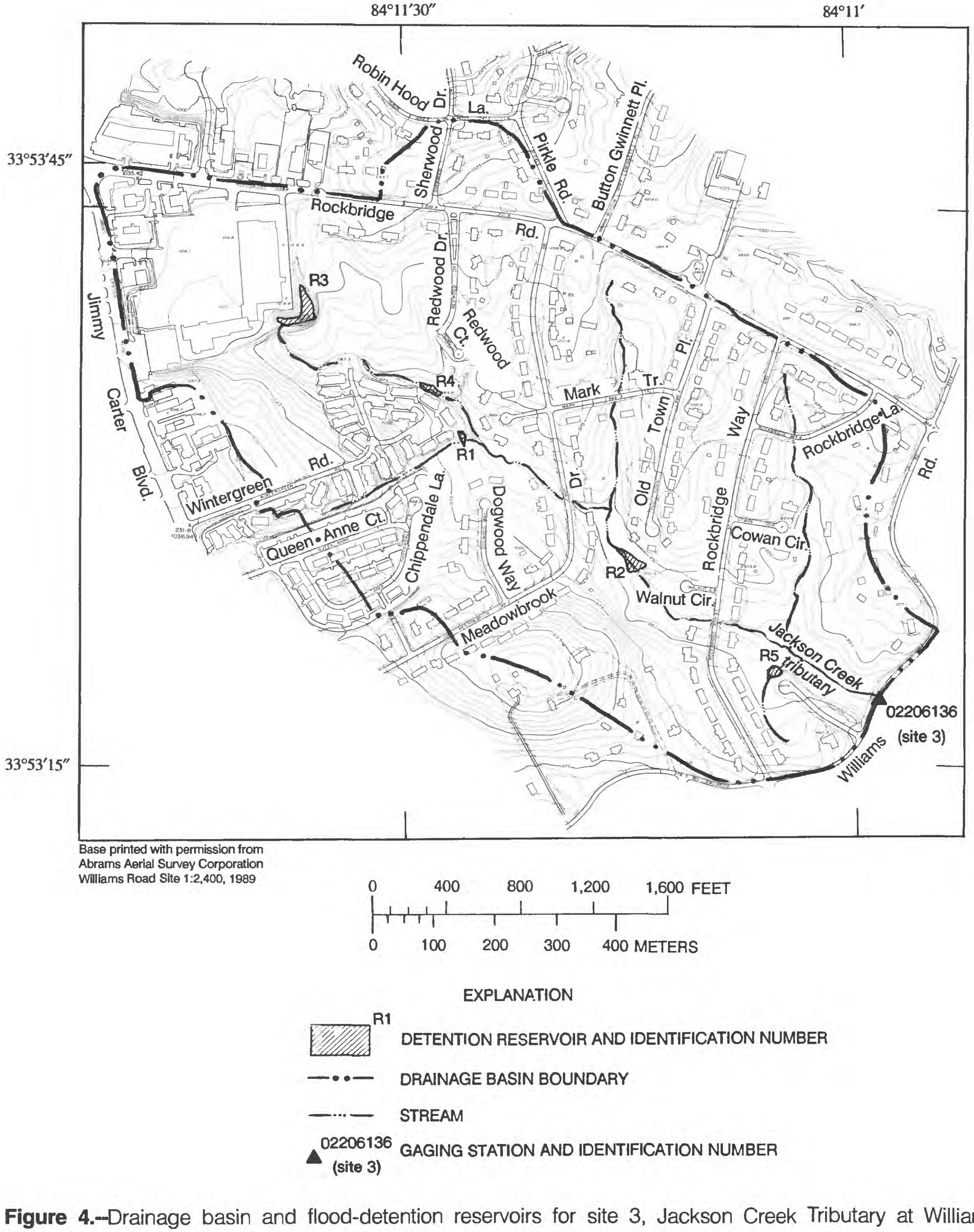
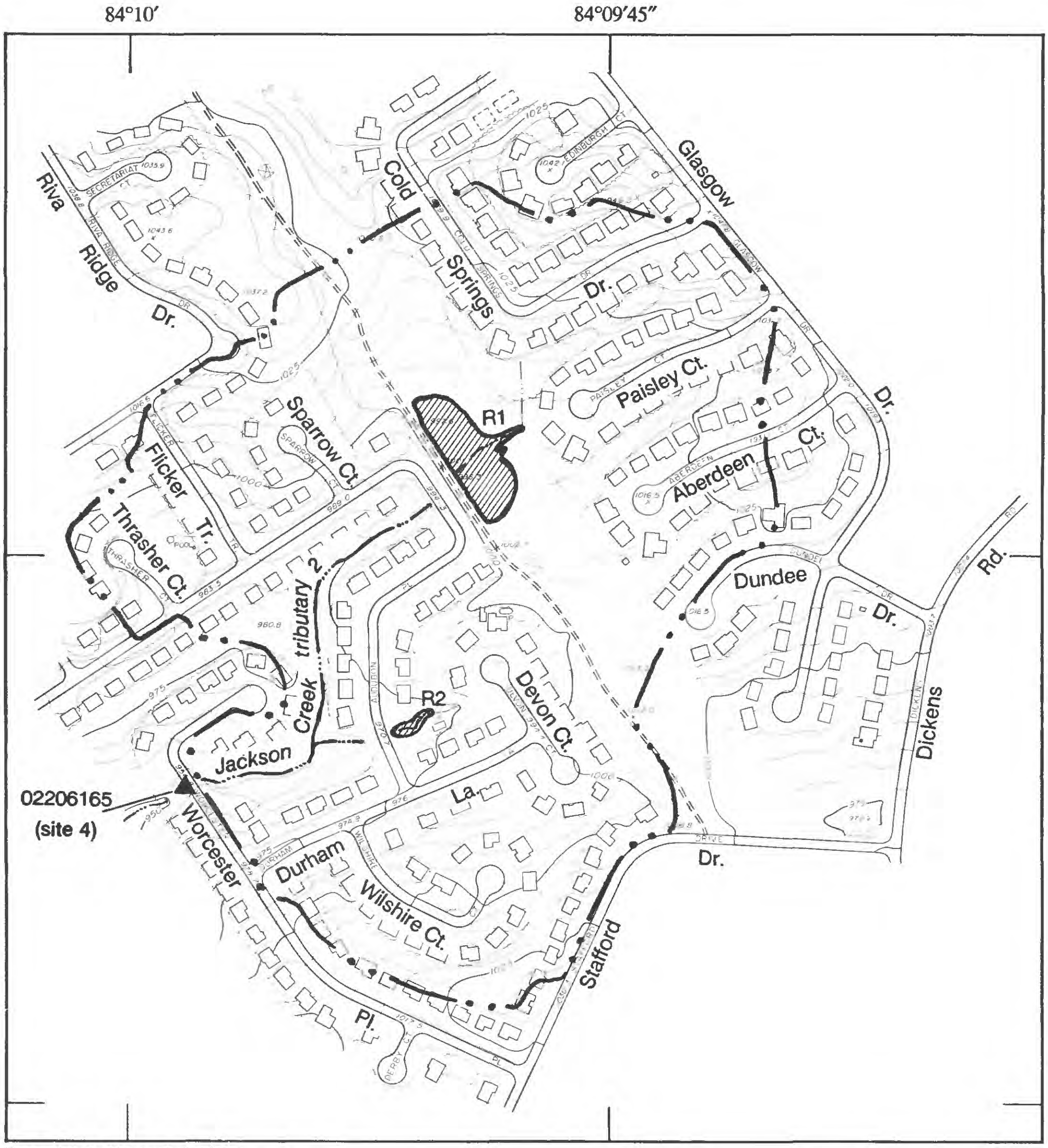


Figure 4.--Drainage basin and flood-detention reservoirs for site 3, Jackson Creek Tributary at Williams Road near Lilburn, Georgia.



Base printed with permission from
Abrams Aerial Survey Corporation
Worcester Place Site 1:2,400, 1989

0 200 400 600 800 1,000 FEET

0 50 100 150 200 250 300 METERS

EXPLANATION

R1 DETENTION RESERVOIR AND IDENTIFICATION NUMBER

DRAINAGE BASIN BOUNDARY

STREAM

02206165 (site 4) GAGING STATION AND IDENTIFICATION NUMBER

Figure 5.--Drainage basin and flood-detention reservoirs for site 4, Jackson Creek Tributary 2 at Worcester Place near Lilburn, Georgia.

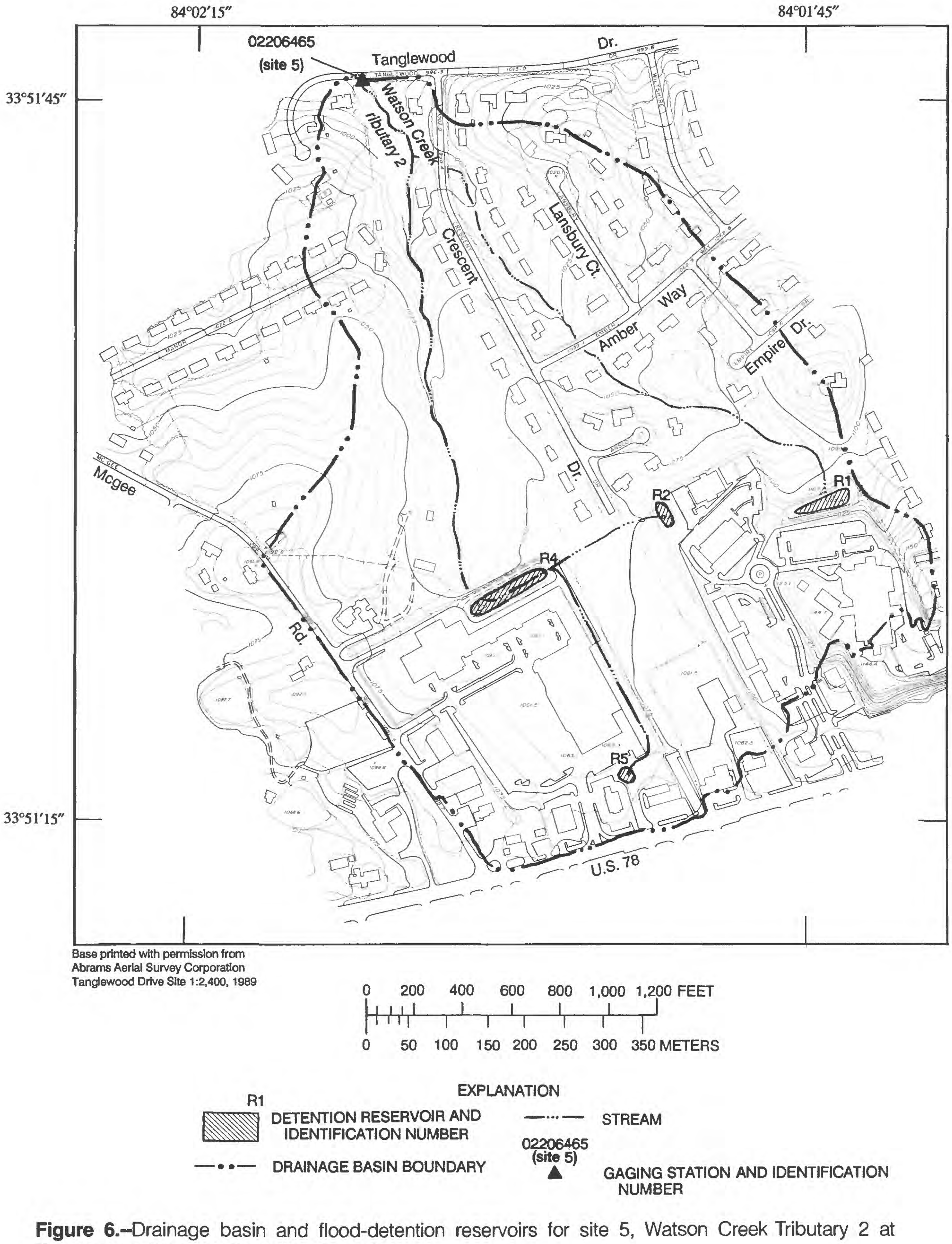


Figure 6.--Drainage basin and flood-detention reservoirs for site 5, Watson Creek Tributary 2 at Tanglewood Drive at Snellville, Georgia.

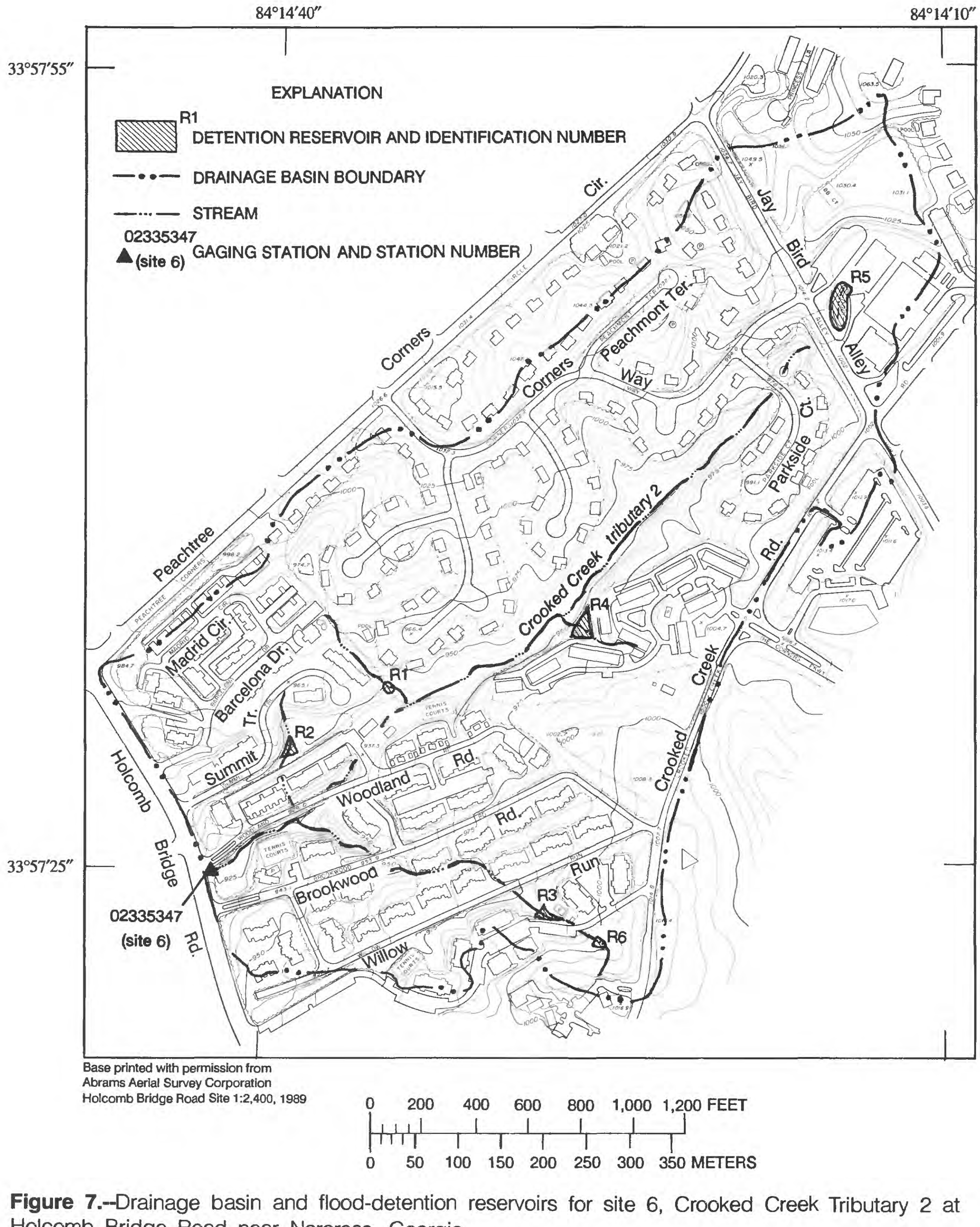


Figure 7.--Drainage basin and flood-detention reservoirs for site 6, Crooked Creek Tributary 2 at Holcomb Bridge Road near Norcross, Georgia.

Instrumentation and Data-Collection Techniques

Digital recorders equipped to record data at 5-minute intervals were used to collect stage and rainfall data at each gaging station. Most gages were located at culverts in the downstream end of each basin. The stage recorder for most basins was housed on top of an 18-in. vertical corrugated metal-pipe stilling well in the approach section upstream from a culvert. 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 recorders were housed on top of 3-in. galvanized pipes attached to the end of an upstream culvert wingwall. The stilling wells were flushed after every flood and intakes were cleaned during every inspection trip. Rainfall recorders were housed on top of 8-ft collector wells that were made from 3-in. galvanized pipe and designed to hold about 11 in. of rain. A drain plug near the bottom of the collector well was removed on each inspection trip to drain the accumulated rainfall.

Crest-stage gages also were installed at each station, at least one in the upstream approach section and one at the downstream end of the culvert. A plot of measured peak stages from the upstream and downstream crest-stage gages was established for each station and used to check for any changes in the hydraulic conditions at the station. 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 using methods described by Bodhaine (1968).

All theoretical stage-discharge relations were verified by current-meter measurements made at various stages during the operation of the gage. The relation between upstream and downstream water-surface elevations at gages in culverts having backwater control were monitored to detect accumulation of debris at a culvert entrance that could produce excessive fall, or a blockage downstream that could greatly reduce normal fall. For culverts having inlet or outlet control, the upstream-downstream water-surface relations are not consistent. Maintenance crews would often remove debris from culverts between gage-servicing trips. If debris were removed, plotting of the upstream and downstream water-surface elevations at crest-stage gages provided the only indication of blockage in the culvert.

At most stations, the stage at the recording gage was lower than the stage at the upstream crest-stage gage. This was a result of drawdown at the intakes rather than to intake lag, as can be demonstrated using the equation presented by Buchanan and Somers (1968, p. 13). A relation between stages at the upstream crest-stage gage and at the recorder was established to relate theoretical discharge computations, to the recorded stage so that digital-recorder records could be processed without having to make shift corrections in the stage-discharge relation. This relation was monitored to detect equipment problems or clogged intakes.

Aerial photographs and topographic maps of each basin and flood-detention reservoir and field surveying were used to define physical characteristics of the basin, such as channel slope, channel length, overland flow slope, and impervious area (see "Glossary" for definition of parameters). At each flood-detention reservoir, planimetered topographic maps were used to determine the contributing drainage area, maximum storage capacity (table 2), and the relation between stage and storage. From a detailed survey of each outflow structure, the stage-discharge relation was determined using theoretical discharge computations. Theoretical stage-discharge relations for outflow structures at several of the flood-detention reservoirs were verified by current-meter measurements.

Table 2 .--Gaging stations and flood-detention reservoirs in Gwinnett County, Georgia, used in this study
[R, reservoir]

Site number	Reservoir identification number (figs. 2-7)	Contributing drainage area (in square miles)	Maximum storage capacity (in cubic feet)
	Wolfcreek basin near Suwannee		
1	R2	0.0901	75,200
	R3	.0329	89,500
	R4	.0204	27,600
2	Jackson Creek basin near Lilburn		
	R1	0.0138	23,700
	R2	.0135	6,480
	R4	.0071	4,300
	R5	.0128	5,600
	R6	.0317	28,000
3	Jackson Creek tributary basin near Lilburn		
	R1	0.0181	13,500
	R2	.2182	28,200
	R3	.0344	120,000
	R4	.0781	20,100
	R5	.0085	3,930
4	Jackson Creek tributary 2 basin near Lilburn		
	R1	0.0307	166,000
	R2	.0253	6,730
5	Watson Creek tributary 2 near Snellville		
	R1	0.0051	23,900
	R2	.0104	29,900
	R4	.0650	254,000
	R5	.0015	2,740
6	Crooked Creek tributary 2 basin near Norcross		
	R1	0.0126	3,640
	R2	.0061	5,200
	R3	.0080	8,370
	R4	.0167	12,500
	R5	.0189	81,500
	R6	.0025	749

Current Data

Data for all floods for which complete rainfall and stage data were available and flow through the culvert was not obstructed were processed and loaded into USGS computer storage on a near-current basis. Generally, data for five to eight floods per year were processed for each station. Unit rainfall, unit discharge, and daily rainfall data were retrieved and the unit data were plotted against time. Unit data hydrographs were used to (1) visually edit data to identify erroneous data entries by the recorder or misread recorder output, (2) detect partially clogged rain-gage intakes or hanging floats, (3) serve as a basis for estimating a rising limb of a flood hydrograph if the stilling well intakes were above the stage at the beginning of a rise, and (4) estimate a falling limb of the hydrograph when the intakes became partly clogged with sediment on the recession. Data then were edited, estimations completed, and reloaded into USGS computer storage.

Daily evaporation data at a station near Athens, Georgia, for the period 1986-92 were obtained from the National Weather Service (NWS) (table 3). Evaporation maps presented by Kohler and others (1959) were used as a guide in selecting Athens as the evaporation station most representative of the study area.

Table 3.--National Weather Service rainfall and evaporation stations for which data were used in the Distributed Routing Rainfall Runoff Model

Type station	Location	Period of record (in water years)[1]	Station number
Rainfall	near Atlanta, Ga.	1898-1980	333900084260050
Evaporation	near Athens, Ga.	1940-80, 1986-92	335500083210050

[1]A water year extends from October 1 of one calendar year through September 30 of the next calendar year and is designated by the year in which it ends.

Long-Term Rainfall and Evaporation Data

Long-term rainfall and evaporation data are required for peak-discharge simulation. Daily rainfall records were obtained from the NWS station near Atlanta, Georgia, (U.S. Department of Commerce, 1948-80) (table 3). About four to eight storms per year were selected on the basis of total rainfall, rainfall intensity, and in some instances, hourly rainfall data. For periods before 1948, the unpublished daily rainfall-recorder charts were obtained from the NWS for all storms with rainfall totals of 1/2 in. or more per day, and selections of storms were made based on hydrologic judgement. As a check on the procedure for selecting storms, computer program E436 (Carrigan and others, 1977) was used to select five major rainfall storms from each water year (1898-1980).

Observed daily evaporation data (1940-80) were obtained for the NWS station near Athens, Georgia. For the period prior to 1940 (1898-1939), computer program H266 (Carrigan and others, 1977) was used to generate harmonic average evaporation data from the observed daily evaporation data collected during 1940-80.

DATA ANALYSIS

Peak-flood discharges for the six urban watersheds included in this study were estimated using a mathematical Distributed Routing Rainfall-Runoff Model (DR3M) developed by the USGS (Alley and Smith, 1982). The model was calibrated using short-term (1986-92) rainfall-runoff data, and then used to estimate long-term annual-peak discharges from long-term rainfall and evaporation data from the NWS. DR3M was used to simulate the effects of flood-detention reservoirs on peak discharges for each basin and to determine a long-term annual peak discharge for the existing condition with all flood-detention reservoirs in place. Subsequent simulations were made to simulate the effects of removing individual flood-detention reservoirs on peak discharges. In addition, a simulation of peak discharges for conditions with all flood-detention reservoirs removed was made for each basin. Flood-frequency relations at the station were computed for each simulation by fitting the logarithms of the long-term annual peak discharge data to a Pearson type III distribution curve. The effect of existing reservoirs for a stream system may be determined using this technique by comparing each simulation representing varying amounts of flood-detention to the existing condition to determine the change in flood-frequency estimates at the six gaging stations.

Description of the Distributed Rainfall-Runoff Model

The DR3M 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, pervious-area rainfall excess, and impervious-area rainfall excess. Model parameters are adjusted using optimization procedures discussed later (see "Glossary" for definition of parameters).

The soil-moisture-accounting component of DR3M determines the effect of antecedent conditions on infiltration. Rainfall excess is routed over pervious surfaces and two types of impervious surfaces (1) effective impervious areas--impervious areas draining directly into the channel's drainage system, and (2) noneffective impervious areas--impervious areas that drain into pervious areas.

The only rainfall on effective impervious areas that does not runoff to streams is that which is retained as impervious retention. Impervious retention generally is assumed to be a fixed amount, usually from 0.02 to 0.05 in. Impervious retention losses occur before runoff from effective impervious areas begins.

Rainfall on noneffective impervious areas is assumed to run off onto the surrounding pervious areas. DR3M assumes that the runoff occurs instantaneously and that the runoff volume is uniformly distributed over the pervious area. This volume, expressed in inches over the pervious area, is added to the rainfall 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 ratio of the sum of the pervious and noneffective impervious areas to the pervious area, as presented in the following equations:

$$DA3 = 1 - MIA. \tag{1}$$

$$\text{Then } RAT = (DA2 + DA3)/DA3, \tag{2}$$

where DA3 is the area of the basin covered by pervious surfaces, DA2 is the area of the basin covered by noneffective impervious surfaces, and MIA is the measured total impervious area.

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 peak discharges or flood-runoff volumes from the observed peak discharges or flood-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 uses the kinematic wave theory for routing flows over a given drainage basin. A basin is represented as a set of segments that collectively describe all subbasins in the total basin. The purpose of 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. DR3M will provide simulated discharges at each segment. 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. The channel segments are used to represent natural or man-made conveyances and may receive upstream inflow from as many as three other segments, including combinations of other channel segments, reservoir segments, and nodal segments. The channel segments also can receive lateral inflow from overland-flow segments. Reservoir segments can be used to describe an on-channel flood-detention reservoir using stage-storage-discharge relations and can be used to simulate culverts that detain water because of 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 flood. Topographic maps (AASC, 1989) were used to delineate and segment the six drainage basins in Gwinnett County included in this study.

The assumptions and 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 non-pressurized-flow capacity. In addition to the assumptions behind the kinematic wave routing, other major assumptions are listed below (Alley and Smith, 1982).

- rainfall excess is assumed to be uniformly distributed over an overland-flow segment;
- pervious and impervious parts of a segment are assumed to be uniformly distributed over the segment;
- 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);
- infiltration ceases when rainfall 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 from turbulent to laminar will not occur; and,
- rainfall on noneffective impervious areas is assumed to be instantaneously and uniformly distributed over the pervious area of the watershed.

Calibration

DR3M calibration is the process of determining a set of parameter values that will produce model simulations that best duplicate observed floods by calibrating for flood volumes and then calibrating for peak discharges. Initially, an average of more than 40 floods per station were available for model calibration. About 80 percent of these floods were used on the final calibrations. Some outliers were detected and were not used in the final model calibration because one or more of the following possible conditions:

- nonrepresentative rainfall as a result of localized summer thunderstorms;
- runoff exceeded rainfall;
- rainfall greatly exceeded runoff; and,
- upstream crossings clogged with debris, making peak discharges at the station artificially low.

The first step in calibrating DR3M for flood volumes was to optimize the soil-moisture accounting and infiltration parameters (see Glossary for definition of parameters) by using large floods and holding EAC constant. EVC was fixed at 0.75, as determined 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 in values for parameter KSAT of 0.05 to 0.40 was obtained from Chow (1964). Most of the soils of the six basins in this study were type B soils and 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, BMSN, RGF, and PSP were obtained from Golden and Price (1976) and Inman (1983).

The next step in calibrating the model was to optimize on effective impervious area using the parameter EAC with all the other parameters being held constant. Calibration was accomplished by using only small floods for which runoff was largely contributed from the effective impervious area of the watershed. This starting value of EAC was set at 1.0, with a lower limit of 0.80 and an upper limit of 1.15. DR3M assumes that any adjustment to effective impervious area (DR3MIA) 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. 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.

At least one cross-section was field-surveyed or obtained from topographic maps (AASC, 1989) for all channel segments. A stage-storage-discharge relation was prepared for upstream crossings that were determined to have storage potential. The discharge was obtained from a theoretical rating at the outlet of the reservoir, generally a culvert. Reservoir storage created by the road embankment was computed from field surveys or obtained by planimetry on 5-ft contour topographic maps (AASC, 1989). The discharge and storage were input to the DR3M at corresponding stages.

An explicit finite-difference scheme of routing was used in the calibration of peak discharges. The use of the explicit finite-difference scheme requires selection of a model time step to achieve accuracy. Two factors are important in selection of the model time step are: (1) the ability to accurately define the rainfall intensity, and (2) the ability to accurately define the hydrograph. Because the observed rainfall was collected at 5-minute time intervals and the six basins are small and have a fast response time to rainfall, a 2 1/2-minute model time step was chosen for calibration and simulation. Roughness values, ALPADJ, and NDX are the three parameters that were manually adjusted. Automatic techniques of optimization for routing, such as the Rosenbrock (1960) method, were not utilized. NDX is a model parameter that defines the number of length intervals for finite-difference routing, where an increase in NDX increases discharge and a decrease in NDX decreases discharge. ALPADJ is a factor used to adjust the combined effects of roughness, bed slope, and cross-sectional geometry, which make up α. These three routing parameters were manually adjusted so that simulated peak discharges were most representative of observed peak discharges. A scatter plot of simulated and observed peak discharges is shown on figure 8. The standard error of estimate of calibration, an indicator of the "goodness of fit" of the calibration in percent, was based on the mean square differences of logarithms of observed and synthesized peaks. The range in standard error of estimate for volumes was 19 to 40 percent, and for peak discharge, the range was 29 to 46 percent. The optimized DR3M parameter values and selected physical characteristics are listed in table 4. Simulated peak discharge and observed peak discharge on figure 8 illustrate the results of the DR3M calibrations at six drainage basins.

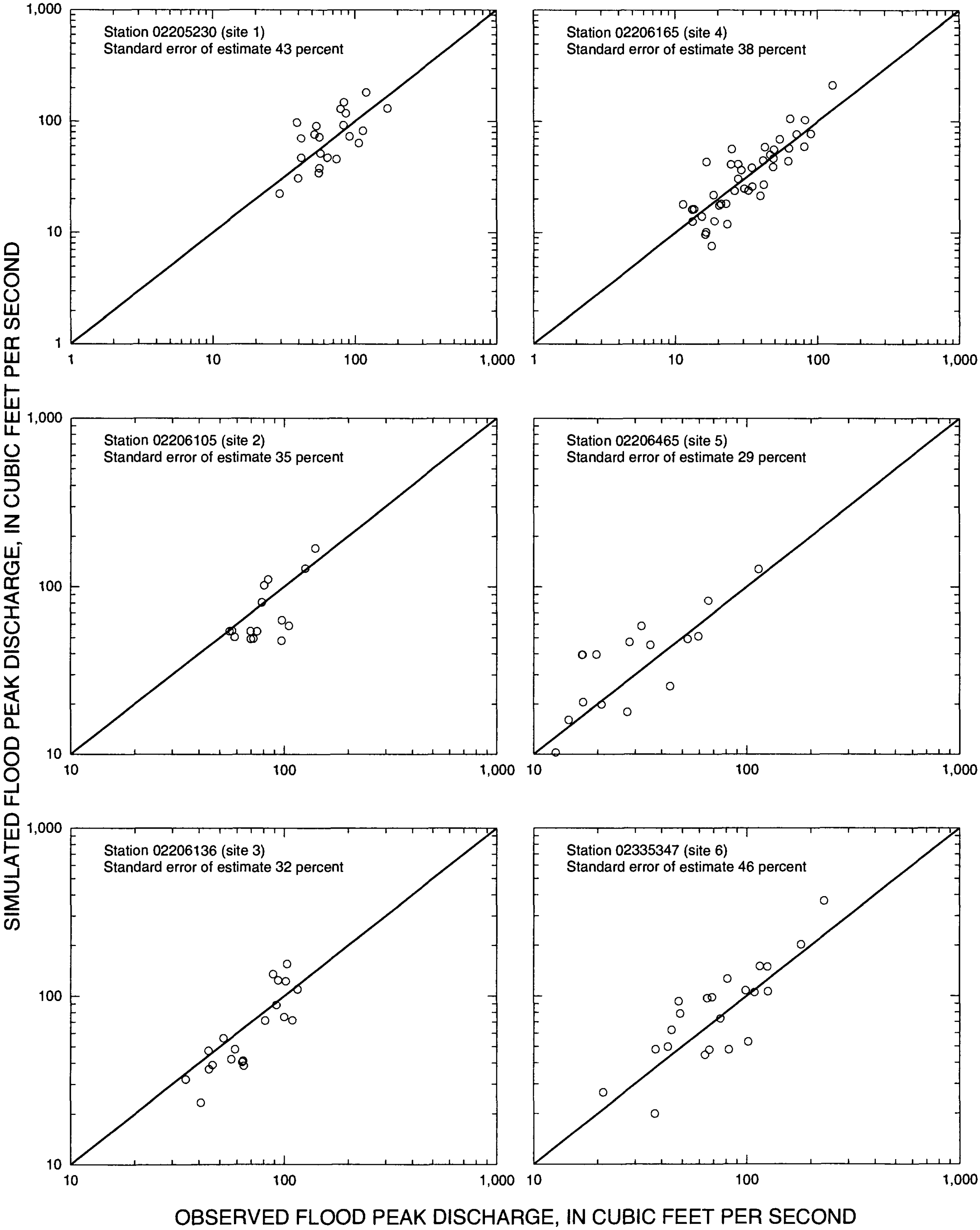


Figure 8.--Relation between observed peak discharges and simulated peak discharges at stations in the six study basins, Gwinnett County, Georgia.

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 considered acceptable (verified) if the mean square-error obtained during the verification process falls within preselected allowable 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. It is the primary basis for assessing the accuracy of the model for purposes of prediction.

The DR3M was verified at seven of the Atlanta area sites from an earlier study (Inman, 1983), by split-sample testing. Floods at each site were divided into two samples. The 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. The model was recalibrated by using only the 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 were all acceptable. No additional split-sample testing was deemed necessary, due to time and financial constraints.

Table 4. -- Optimized Distributed Routing Rainfall-Runoff Model parameters and physical characteristics of drainage basins[1] in Gwinnett County, Georgia

Site no.	PSP (inch)	KSAT (inch/ hour)	RGF	BMSM (inch)	RR	EAC	RAT	Number of segments	Standard error of estimate (in percent)	Drain-age area (mi^2)	MIA (in per-cent)	DR3MIA (in percent)
1	0.70	0.12	9.5	5.57	0.9	1.15	1.00	44	43	0.37	15	16
2	2.15	.23	31.5	2.88	.90	.86	1.07	29	35	.18	33	25
3	1.64	.12	22.2	2.32	.91	.81	1.08	53	32	.33	29	24
4	1.53	.16	19.5	3.25	.94	.86	1.08	12	38	.10	32	27
5	2.95	.17	25.5	4.71	.88	.86	1.06	29	29	.20	32	27
6	1.87	.17	20.0	2.75	.92	.81	1.08	35	46	.19	35	30

1. Parameter EVC was assigned a value of 0.75 and not optimized.

Simulation

Annual peak discharges for the period 1898-1980 for the six drainage basins were simulated using the final calibrated DR3M parameters, long-term unit and daily rainfall, and long-term daily evaporation. The first simulation was for existing conditions with all flood-detention reservoirs in place. Subsequent simulations involved eliminating individual reservoirs in sequence for each basin to determine the effect of a single reservoir on peak discharges for the entire basin. A final simulation for conditions without any flood-detention reservoirs was made for each basin to examine the cumulative effect of all reservoirs on peak discharges on that basin.

Flood-Frequency Data Analysis

The relation of flood-peak magnitude to probability of exceedance (or recurrence interval) is referred to as the flood-frequency relation. Probability of exceedance is the probability that a flood will exceed a specific magnitude in any one year. Recurrence interval is the reciprocal of the probability of exceedance times 100, and is the average interval, in years, in which a given flood will be exceeded. For example, a flood having a probability of exceedance of 0.04 has a recurrence interval of 25 years. A flood having a recurrence interval of 25 years might not occur in a given 25-year period, or might occur more than once in a 25-year period, but will be exceeded on average every 25 years over a long period of time, such as a few hundred years.

The logarithms of the long-term annual peak-discharge data for each simulation were fit to a Pearson type III distribution curve in accordance with "Guidelines for Determining Flood-Flow Frequency, Bulletin 17B" (Interagency Advisory Committee on Water Data (IACWD), 1982). The station-skew coefficients of the frequency curves were used as specified in IACWD (1982). Peak- discharge simulations for recurrence intervals of 2, 5, 10, 25, 50, and 100 years for the six stations are listed in table 5.

Table 5. -- Flood-frequency estimates from synthesized long-term peak-discharge data for existing and simulated conditions at stations in the six study basins, Gwinnett County, Georgia

Site number	Condition	Flood-discharge for indicated recurrence interval in years, (in cubic feet per second)					
		2	5	10	25	50	100
1	all reservoirs (existing)	182	303	388	501	586	673
	without R2	182	304	390	504	592	680
	without R3	192	328	426	557	658	761
	without R4	182	305	393	509	597	687
	detention-free	193	331	431	566	671	778
2	all reservoirs (existing)	72	128	172	235	288	344
	without R1	75	132	176	237	287	340
	without R2	72	129	173	238	292	351
	without R4	72	127	170	233	285	341
	without R5	75	131	175	238	290	346
	without R6	74	131	175	240	293	351
	detention-free	79	139	184	246	296	349
3	all reservoirs (existing)	113	194	256	344	414	489
	without R1	114	195	257	345	415	491
	without R2	116	200	264	354	427	505
	without R3	122	209	276	369	445	525
	without R4	114	195	257	345	416	491
	without R5	115	194	256	343	414	489
	detention-free	128	220	291	390	469	553
4	all reservoirs (existing)	89	143	181	228	264	300
	without R1	110	181	230	294	343	392

Table 5. -- Flood-frequency estimates from synthesized long-term peak-discharge data for existing and simulated conditions at stations in the six study basins, Gwinnett County, Georgia--Continued

Site number	Condition	Flood-discharge for indicated recurrence interval in years, (in cubic feet per second)					
		2	5	10	25	50	100
	without R 2	92	146	183	228	262	295
	detention-free	111	182	232	296	344	393
5	all reservoirs (existing)	77	127	166	222	269	321
	without R1	79	133	174	234	284	339
	without R2	79	131	172	230	279	332
	without R4	96	157	204	268	319	373
	without R5	77	127	166	223	270	321
	detention-free	106	175	227	298	355	416
6	all reservoirs (existing)	135	233	306	406	485	569
	without R1	136	233	306	406	485	569
	without R2	135	232	306	408	490	575
	without R3	136	234	308	410	491	577
	without R4	134	233	307	410	493	580
	without R5	137	236	310	413	494	579
	without R6	135	233	306	406	486	570
	detention-free	137	235	310	414	496	584

EFFECTS OF URBAN FLOOD-DETENTION RESERVOIRS ON PEAK DISCHARGES

Effects of urban flood-detention reservoirs on peak discharges on receiving streams in Gwinnett County, Georgia, were determined by simulating removal of individual reservoirs as well as removal of all reservoirs in the six drainage basins. A comparison of the 2-, 10-, and 100-year simulated and existing peak discharges was used to determine the effect of a single detention reservoir and the cumulative effect of all detention reservoirs in a basin. The 2-, 10-, and 100-year peak discharges are specified in Gwinnett County design codes for hydraulic structures.

A comparison of flood-frequency estimates for site 1 (fig. 1, table 1) for the existing condition with all reservoirs (table 5, fig. 2) and for conditions representing removal of individual reservoirs R2, R3, and R4 in sequence indicates that the peak discharges increased from 0 to 5 percent at the 2-year recurrence interval, from 0 to 10 percent at the 10-year recurrence interval, and from 1 to 13 percent at the 100-year recurrence interval (table 6). The removal of reservoirs R2 and R4 at site l had little effect on the peak discharges, whereas, the removal of reservoir R3 caused the peak discharges to increase 10 percent at the 10-year recurrence interval. However, reservoirs R2 and R3 have similar maximum storage capacities. The difference in the increase in peak discharges between removal of reservoirs R2 and R3 is because of (1) differences in outflow-structure capacities, and (2) differences in the elevation to storage relations. Also, although reservoir R2 has a larger contributing drainage area than does reservoir R3, the removal of reservoir R3 had more effect on peak discharges than the removal of reservoir R2 because of differences in the outflow-structure capacity. A comparison of the existing condition with all reservoirs to the condition of no flood-detention reservoirs indicates that the cumulative effect of removing R2, R3, and R4 was an increase in peak discharge of the receiving stream of 6 percent at the 2-year recurrence interval, 11 percent at the 10-year recurrence interval, and 16 percent at the 100-year recurrence interval.The simulation of the removal of all flood-detention reservoirs increased peak discharges by about the cumulative percent change resulting from the removal of individual reservoirs.

Table 6. -- Percent change in peak discharges of 2-, 10- and 100-year recurrence intervals, from simulated existing to simulated detention-free condition at stations in the six study basins, Gwinnett County, Georgia

Site number	Simulated condition	Percent change in peak discharge for indicated recurrence interval in years		
		2	10	100
1	without R2	0	0	+1
	without R3	+5	+10	+13
	without R4	0	+1	+2
	detention-free	+6	+11	+16
2	without R1	+4	+2	-1
	without R2	0	+1	+2
	without R4	0	-1	-1
	without R5	+4	+2	+1
	without R6	+3	+2	+2
	detention-free	+10	+7	+2
3	without R1	+1	0	0
	without R2	+3	+3	+3
	without R3	+8	+8	+7
	without R4	+1	0	0
	without R5	+1	0	0
	detention-free	+13	+14	+13
4	without R1	+24	+27	+31
	without R2	+3	+2	-2
	detention-free	+25	+28	+31
5	without R1	+3	+5	+6
	without R2	+3	+4	+3
	without R4	+20	+23	+16
	without R5	0	0	0
	detention-free	+38	+37	+30
6	without R1	+1	0	0
	without R2	0	0	+1
	without R3	+1	+1	+1
	without R4	-1	0	+2
	without R5	+1	+1	+2
	without R6	0	0	0
	detention-free	+1	+1	+3

A comparison of the flood-frequency estimates for site 2 (fig. 1, table 1) for the existing condition with all reservoirs (table 5, fig. 3) and for the conditions representing removal of individual reservoirs R1, R2, R4, R5, and R6 in sequence indicates that the peak discharges increased from 0 to +4 percent at the 2-year recurrence interval, from -1 to +2 percent at the 10-year recurrence interval, and from -1 to +2 percent at the 100-year recurrence interval (table 6). Removal of reservoirs R1 and R6 individually increased peak discharges 2 percent at the 10-year recurrence interval; these reservoirs have similar contributing drainage areas, and maximum storage capacities. Reservoirs R2 and R5 have almost the same contributing drainage area and maximum storage capacity. The removal of reservoir R2 resulted in 0, 1, and 2 percent change in peak discharges for the 2-, 10-, and 100-year recurrence intervals, respectively. The removal of reservoir R5, however, resulted in increases of 4, 2, and 1 percent change in peak discharge. Removal of reservoir R4 caused peak discharges to decrease 1 percent at the 10-year recurrence interval, because the reservoir is located near the outlet of the basin where the timing and magnitude of the peaks at the outlet are influenced by reservoir outflow. A comparison of the existing condition with all reservoirs to the detention-free condition (no reservoirs) indicates that the cumulative effect of removing reservoirs R1, R2, R4, R5, and R6 would be an increase in peak discharges of 10 percent at the 2-year recurrence interval; 7 percent at the 10-year recurrence interval; and 2 percent at the 100-year recurrence interval.

A comparison of the flood-frequency estimates for site 3 (fig. 1, table 1) for the existing condition with all reservoirs (table 5, fig. 4) and for the condition with removal of individual reservoirs R1, R2, R3, R4, and R5 in sequence indicates that the peak discharges increased from 1 to 8 percent at the 2-year recurrence interval, from 0 to 8 percent at the 10-year recurrence interval, and from 0 to 7 percent at the 100-year recurrence interval (table 6). Removal of reservoirs R1, R4, and R5 have less than 1 percent change on peak discharges, but removal of reservoir R2, which has a larger contributing drainage area and is located on the main stem of the stream, resulted in a 3 percent change in peak discharges. Removal of reservoir R3 resulted in an 8 percent change in peak discharges because of a larger maximum storage capacity than reservoirs R1, R2, R4, and R5. A comparison of the existing condition with all reservoirs to the detention-free (no reservoir) condition indicates that the cumulative effect of removing reservoirs R1, R2, R3, R4, and R5 was an increase in peak discharges of the receiving stream of 13 percent at the 2-year recurrence interval, 14 percent at the 10-year recurrence interval, and 13 percent at the 100-year recurrence interval.

A comparison of the flood-frequency estimates for site 4 (fig. 1, table 1) for the existing condition with all reservoirs (table 5, fig. 5) and for the condition with removal of individual reservoirs R1 and R2 in sequence indicates that the peak discharges increased from 3 to 24 percent at the 2-year recurrence interval, from 2 to 27 percent at the 10-year recurrence interval, and from -2 to 31 percent at the 100-year recurrence interval (table 6). Removal of reservoir R2 resulted in little change in peak discharges, but removal of reservoir R1 resulted in an increase in peak discharges because reservoir R1 had a the much larger maximum storage capacity. Reservoirs R1 and R2 have about the same contributing drainage area. The removal of reservoir R2 caused the 100-year recurrence interval peak discharge to decrease 2 percent. This decrease in peak discharges is because the reservoir is located close to the outlet of the basin where the timing and magnitude of the peaks at the outlet are influenced by reservoir R2 outflow.

A comparison of the existing condition with all reservoirs to the detention-free (no reservoirs) condition indicates that the cumulative effect of removing reservoirs R1 and R2 on the receiving stream was peak discharge increases of 25 percent at the 2-year recurrence interval, 28 percent at the 10-year recurrence interval, and 31 percent at the 100-year recurrence interval. Although only two flood-detention reservoirs are located in this basin, the removal of these reservoirs increased peak discharges an average of 28 percent.

A comparison of the flood-frequency estimates for site 5 (fig. 1, table 1) for the existing condition with all reservoirs (table 5, fig. 6) and for the condition with removal of individual reservoirs R1, R2, R4, and R5 in sequence indicates that peak discharges increased from 0 to 20 percent at the 2-year recurrence interval, from 0 to 23 percent at the 10-year recurrence interval, and from 0 to 16 percent at the 100-year recurrence interval (table 6). Removal of reservoir R5 had no effect on peak discharges as a result of the small contributing drainage area, small maximum storage capacity, and location in the upper reaches of the basin. The removal of reservoirs R1 and R2 caused the peak discharges to increase about 3 percent at the 2-year recurrence interval. However, at the 100-year recurrence interval, the peak discharges increased 6 percent and 3 percent for the removal of reservoirs R1 and R2, respectively. The difference in the increase of peak discharges between removal of reservoirs R1 and R2 is because the capacity of the outflow structure of R1 is less than R2. The removal of reservoir R4 which has a relatively large maximum storage capacity and contributing drainage area, increased discharges on average of 20 percent. A comparison of the existing condition with all reservoirs to the detention-free (no reservoir) condition indicates that the cumulative effect on the receiving streams of removing reservoirs R1, R2, R4 and R5 was an increase in peak discharges of 38 percent at the 2-year recurrence interval, 37 percent at the 10-year recurrence interval, and 30 percent at the 100-year recurrence interval.

A comparison of flood-frequency estimates for site 6 (fig. 1, table 1) for the existing condition with all reservoirs (table 5, fig. 7) and for the condition with removal of individual reservoirs R1, R2, R3, R4, R5, and R6 in sequence indicates that peak discharges increased from -1 to 1 percent at the 2-year recurrence interval, from 0 to 1 percent at the 10-year recurrence interval, and from 0 to 2 percent at the 100-year recurrence interval (table 6). The removal of the reservoirs in this basin would have little effect on peak discharges because of their small contributing drainage areas and locations in the upper reaches of the basin. A comparison of the existing condition with all reservoirs to the detention-free (no reservoirs) condition indicates that the cumulative effect of the removal of reservoirs R1, R2, R3, R4, R5, and R6 on the receiving stream was an increase in peak discharges of 1 percent at the 2- and 10-year recurrence intervals, and 3 percent at the 100-year recurrence interval. Although six small flood-detention reservoirs are located in this basin, removal of all six reservoirs has little effect on peak discharges at the outlet of the basin.

Results from all 25 simulations of the removal of individual flood-detention reservoirs from all six basins indicate that the peak discharges increased from -1 to 24 percent at the 2-year recurrence interval, from -1 to 27 percent at the 10-year recurrence interval, and from -2 to 31 percent at the 100-year recurrence interval (table 6). Results from the simulation of the removal of all flood-detention reservoirs from all six basins indicate that the peak discharges increased from 1 to 38 percent at the 2-year recurrence interval, from 1 to 37 percent at the 10-year recurrence interval, and from 2 to 31 percent at the 100-year recurrence interval (table 6).

In this study of six basins, several characteristics were determined to influence the effect of flood-detention reservoirs on peak discharges. The contributing drainage area, the maximum storage capacity, the outflow-structure capacity, and the elevation-to-storage relation of the flood-detention reservoir influenced peak discharges in several basins. The location and number of flood-detention reservoirs in the drainage basin also influenced the effect of flood-detention reservoirs on peak discharges in several basins.

SUMMARY

The effect of flood-detention reservoirs on peak discharges along downstream reaches of streams in six small urban drainage basins in Gwinnett County, Georgia, were studied during 1986-93 using the U.S. Geological Survey's Distributed Routing Rainfall-Runoff Model (DR3M). Short-term rainfall-runoff data were collected at selected stations in these drainage basins which range in size from 0.10 to 0.37 square mile, and contain from 15 to 35 percent impervious area. The six basins each contain from two to six flood-detention reservoirs. The rainfall runoff data collected in these basins were used to calibrate the model for each basin. The models were then used to simulate long-term (1898-1980) peak discharges at the gaging stations in each basin from historical rainfall data. The models also were used to simulate peak discharges with and without the various flood-detention reservoirs. Flood frequency relations based on the long-term annual peak discharges were developed for each simulated condition by fitting the logarithms of the annual-peak discharge data to a Pearson type III distribution curve. The effect of individual flood-detention reservoirs on peak discharges in downstream reaches of the streams was determined by comparison of flood-frequency estimates for simulations with and without reservoirs. The cumulative effect of all reservoirs in a basin on downstream peak discharges was determined by comparison of detention-free (no reservoir) conditions to existing conditions. Results from 25 simulations representing the removal of individual reservoir from the six basins indicate that reservoir removal increased peak discharges from -1 to 24 percent at the 2-year recurrence interval, from -1 to 27 percent at the 10-year recurrence, and from -2 to 31 percent at the 100-year recurrence interval. Simulation results indicate that the cumulative effect of all the reservoirs in each of the six basins was to reduce peak discharges from 1 to 38 percent at the 2-year recurrence interval, from 1 to 37 percent at the 10-year recurrence interval, and from 2 to 31 percent at the 100-year recurrence interval.

In this study of six basins, several characteristics were determined to influence the effect of flood-detention reservoirs on peak discharges. The contributing drainage area, the maximum storage capacity, the outflow-structure capacity, and the elevation-to-storage relation of the flood-detention reservoir influenced peak discharges in several basins. The location of the flood-detention reservoirs in the drainage basin and number of flood reservoirs in the basin also influenced peak discharges in several basins.

REFERENCES

Alley, W.M., and Smith, P.E., 1982, User's guide for distributed routing rainfall-runoff model version II: U.S. Geological Survey Open-File Report 82-344, NSTL Station, Mississippi, 201 p.

Bodhaine, G.L., 1968, Measurement of peak discharge at culverts by indirect methods: U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A3, 60 p.

Bohman, L.R., 1992, Determination of flood hydrographs for streams in South Carolina: Volume 2. Estimation of peak-discharge frequency, runoff volumes, and flood hydrographs for urban watersheds: U.S. Geological Survey Water-Resources Investigations Report 92-4040, 79 p.

Buchanan, T.J., and Somers,W.P., 1968, Stage measurement at gaging stations: U.S. Geological Survey Techniques of Water-Resources Investigations, book 3, chapter A7, 28 p.

Carrigan, P.H., Jr., Dempster, G.R., Jr., and Bower, D.E., 1977, User's guide for U.S. Geological Survey rainfall-runoff models--Revision of Open-File Report 74-33: U.S. Geological Survey Open-File Report 77-884, 260 p.

Chow, V.T., 1964, Handbook of applied hydrology: New York, John Wiley, p. 12-1--12-30.

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

Dawdy, D. R., Lichty, R.W., and Bergmann, J.M.,1972, A rainfall-runoff simulation model for estimation of flood peaks for small drainage basins: U.S. Geological Survey Professional Paper 506-B, 28 p.

Golden, H.G., 1977, Preliminary flood-frequency relations for urban streams in Metropolitan Atlanta, Georgia: U. S. Geological Survey Open-File Report 77-57, 16 p.

Golden, H.G., and Price, McGlone, 1976, Flood-frequency analysis for small natural streams in Georgia: U.S. Geological Survey Open-File Report 76-511, 75 p.

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

----- 1986, Simulation of flood hydrographs for Georgia streams: U.S. Geological Survey Water-Resources Investigations Report 86-4004, 41 p.

----- 1988, Flood-frequency relations for urban streams in Georgia: U.S. Geological Survey Water-Resources Investigations Report 88-4085, 36 p.

Interagency Advisory Committee on Water Data, 1982, Guidelines for determining flood-flow frequency: Bulletin 17B, 183 p.

James, L.D., and Lumb, A.M., 1975, UROS4: Urban flood simulation model, Part 2. Application to selected DeKalb County Watersheds: Atlanta, Ga., School of Civil Engineering, Georgia Institute of Technology, 223 p.

Jones, K.R., 1978, Determination of the effects of urbanization on expected peak flows from small watersheds in DeKalb County, Georgia: Atlanta, Georgia, Masters of Science in Civil Engineering thesis, School of Civil Engineering, Georgia Institute of Technology, 132 p.

Kohler, M.A., Nordenson, T. J., and Baker, D.R., 1959, Evaporation maps for the United States: U.S. Weather Bureau Technical Paper 37, 13 p.

LeClerc, Guy, and Schaake, J.C., Jr., 1973, Methodology for assessing the potential impact of urban development on urban runoff and the relative efficiency of runoff control alternatives: Boston, Mass., Ralph M. Parsons Laboratory Report No. 167, Massachusetts Institute of Technology, 257 p.

Lumb, A.M., 1975, URSO4: Urban flood simulation model, Part 1. Documentation and user's manual: Atlanta, Ga., School of Civil Engineering, Georgia Institute of Technology, 214 p.

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

Rosenbrock, H.H., 1960, An automatic method of finding the greatest or least value of a function: Computer Journal, v. 3, p. 175-184.

Sauer, V.B., 1974, An approach to estimating flood-frequency for urban areas in Oklahoma: U.S. Geological Survey Water-Resources Investigations Report 23-74, 10 p.

Sauer, V.B., Thomas, W.O., Jr., Stricker, V.A., and Wilson, K.V., 1983, Flood characteristics of urban watersheds in the United States--Techniques for estimating magnitude and frequency of urban floods: U.S. Geological Survey Water-Supply Paper 2207, 103 p.

U.S. Department of Commerce, 1948-80, Climatological data, monthly publications for Georgia: Asheville, N. C., National Oceanic and Atmospheric Administration, National Climatic Data Center.

GLOSSARY

Some of the technical terms used in this report are defined for convenience and clarification. The reader is referred to Alley and Smith (1982) for additional information regarding Distributed Routing Rainfall-Runoff Model (DR3M) parameters.

ALPADJ: The DR3M factor used to adjust the combined effects of roughness, bed slope, and cross-sectional geometry.
BMSM: The soil-moisture storage at field capacity, in inches, used as a DR3M soil-moisture accounting parameter.
Optimized effective impervious area, DR3M - (DR3MIA): The optimized value of effective impervious area used as a parameter in DR3M.
Drainage area: The drainage area of a basin, in mi^2, planimetered from topographic maps and basin boundaries were field checked.
DA2: The DR3M parameter describing the area of basin covered by noneffective impervious surfaces.
DA3: The DR3M parameter describing the area of the basin covered by pervious surfaces.
E436: The computer program E436 (Carrigan and others, 1977) that selects five events for each water year from daily rainfall data.
EAC: The DR3M factor by which the value of effective impervious area is multiplied.
EVC: The pan coefficient for converting measured pan evaporation to potential evapotranspiration used as a, DR3M soil-moisture accounting parameter.
H266: The computer program H266 (Carrigan and others, 1977) used to generate synthetic evaporation data.
KSAT: The effective saturated value of hydraulic conductivity, in inches per hour, used as a DR3M infiltration parameter.
Measured total impervious area (MIA): The percentage of drainage area that is impervious to infiltration of rainfall. This parameter was determined by a grid-overlay method using aerial photographs. 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 delineated and the results averaged for the final value of measured total impervious.
NDX: The DR3M model parameter that defines the number of length intervals for finite-difference routing.
PSP: The DR3M infiltration parameter describing suction at wetting front for soil moisture at field capacity, in inches.
RAT: The DR3M parameter describing the ratio of the sum of the pervious and noneffective impervious areas to the pervious area.
RGF: The DR3M infiltration parameter describing the ratio of suction at the wetting front for soil moisture at wilting point to that at field capacity.
RR: The DR3M soil-moisture accounting parameter describing proportion of daily rainfall that infiltrates into the soil for the period of simulation, excluding unit days.

☆ U.S. GOVERNMENT PRINTING OFFICE: 1994—505-687