EFFECTS OF URBAN FLOOD-DETENTION RESERVOIRS ON PEAK DISCHARGES AND FLOOD FREQUENCIES, AND SIMULATION OF FLOOD-DETENTION RESERVOIR OUTFLOW HYDROGRAPHS IN TWO WATERSHEDS IN ALBANY, GEORGIA

By Glen W. Hess and Ernest J. Inman

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CONTENTS

ILLUSTRATIONS

Page

TABLES

Page

Table

CONVERSION FACTORS

Multiply	*By*	*To obtain*
	Length	
inch (in)	25.4	millimeter
foot (ft)	0.3048	meter
	Area	
square mile (mi^2)	2.59	square kilometer
	Volume	
cubic feet (ft^3)	0.02832	cubic meter
	Flow	
cubic feet per second (ft^3/s)	0.02832	cubic meter per second

ABBREVIATIONS

ASCS	U.S. Department of Agriculture, Agricultural Stabilization and Conservation Service
DR3M	Distributed Routing Rainfall-Runoff Model
IACWD	Interagency Advisory Committee on Water Data
NWS	U.S. Department of Commerce, National Weather Service
USGS	U.S. Geological Survey

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ABSTRACT

This report describes the effects of flood-detention reservoirs on downstream peak discharges of two urban tributaries to Kinchafoonee Creek in Albany, Georgia and presents simulated flood-detention reservoir outflow hydrographs. Six years of short-term rainfall-runoff data were collected at stations 02351210 and 02351220 in these two urban watersheds. The drainage basin for station 02351210 has a total area of 0.12 square miles, 23.8 percent of which consists of impervious surfaces, and contains two flood-detention reservoirs. The drainage basin for station 02351220 has an area of 0.09 square miles, 12.9 percent of which consists of impervious surfaces, and contains one flood-detention reservoir. The Distributed Routing Rainfall-Runoff Model (DR3M) was calibrated for the two stations using short-term rainfall-runoff data collected during this investigation (1987-92). DR3M was then used to simulate long-term (1906-33, 1941-73) peak discharges for these stations for conditions ranging from the existing condition with all flood-detention reservoirs in place to the condition of no flood-detention reservoirs. Flood-frequency relations based on the long-term peak discharges were developed for each simulation by fitting the logarithms of the annual peak discharge data to a Pearson type III distribution curve. The effect of selected flood-detention reservoirs on peak discharges at a station was determined by comparison of simulated flood-frequency peak discharges for conditions with and without the flood-detention reservoirs. The comparisons indicated that the removal of flood-detention reservoirs from station 02351210 would increase the 10-, 50-, and 100-year peak discharges by 164 to 204 percent. Removal of the flood-detention reservoir from station 02351220 would increase these discharges by about 145 percent.

Observed flood-detention reservoir outflow hydrographs were compared to flood-detention reservoir outflow hydrographs simulated using DR3M and reservoir-routing model A697 for a single flood at each station. The purpose of comparing the hydrographs was to determine if the simpler of the two models (reservoir routing) can be used in place of the more complex (rainfall runoff) model in future studies in the basin. These comparisons indicate that hydrographs simulated using either DR3M or A697 were in agreement with the observed outflow hydrographs and that A697 can be used to simulate flood-detention reservoir outflow hydrographs. Comparisons of the DR3M inflow and outflow hydrographs at station 02351210 indicated that flood-detention reservoir 1 has little effect on peak discharges at the station.

INTRODUCTION

Peak discharge characteristics of many streams in urban Albany, Georgia, are affected by flood-detention reservoirs. Developers are required by city ordinances to provide these flood-detention reservoirs so that post-development peak discharges do not exceed peak discharges of pre-development conditions. However, developers are not required to determine the effect of flood-detention reservoir outflows on receiving streams and the effectiveness of flood-detention reservoirs is largely unknown. To address this problem, the U.S. Geological Survey (USGS) and the city of Albany initiated a cooperative investigation in 1986 to quantify the effects of urban flood-detention reservoirs on peak discharges.

Purpose and Scope

This report presents the results of a study to evaluate and quantify the effects of flood-detention reservoirs on peak discharges and flood frequencies along downstream reaches in two urban watersheds in Albany, Georgia. The study involved the calibration of a rainfall-runoff model for each watershed using data collected over a 6-year period, 1987-92, and use of the model (DR3M) to estimate annual peak discharges from long-term rainfall records. These peak discharges were then used to develop flood-frequency relations. The effect of the flood-detention reservoirs on downstream peak discharges was assessed by simulating discharge hydrographs from historical rainfall records with and without the flood-detention reservoirs. Also, observed flood-detention reservoir outflow hydrographs were compared to flood-detention reservoir outflow hydrographs simulated using the rainfall-runoff model and a reservoir routing model. The purpose of comparing the hydrographs was to determine if the simpler of the two models (reservoir routing) can be used in place of the more complex (rainfall runoff) model in future studies in the basin.

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 could poorly represent the long-term distribution of floods at a station. Thus, a frequency analysis of data for such a short period 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; the model then was used to synthesize long-term peak discharges based on long-term rainfall data. These synthesized long-term peak discharges provide a more reliable prediction of the 50- and 100-year peak discharge than estimates of the 50- and 100-year peak discharges that could be developed using short-term observed peak discharges.

Previous Studies

Several previous studies have addressed flood simulation in urban areas of Georgia. Lumb (1975) in his report, "UROS4: Urban Flood Simulation Model, Part 1, Documentation and User's Manual", explained how the UROS4 model was used to simulate an annual series of flood peaks and to perform a flood-frequency analysis at a selected point. James and Lumb (1975) applied this model to eight watersheds in DeKalb County, Ga., with limited observed data for verification.

Preliminary flood-frequency relations for urban streams in Metropolitan Atlanta based on a technique developed by Sauer (1974) were presented by Golden (1977) using the natural flood-frequency and rainfall-frequency characteristics of local areas in Oklahoma. The Sauer method (1974) adjusted the natural rural flood-frequency relations to those of urban conditions by adjusting 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. Price (1979) used Sauer's (1974) technique to determine flood-frequency relations for urban streams on a statewide basis in Georgia. Simplified equations that can be used to estimate peak-flood flows for small watersheds, 200 acres or less, in DeKalb County, Georgia 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 was based on observed data from 19 stations, which was used to calibrate the USGS rainfall-runoff model (Dawdy and others, 1972) and DR3M (Alley and Smith, 1982). These models then were used to synthesize long-term flood discharges for the 19 stations. The 2- to 100-year peak discharges were determined for each basin from these synthetic, long-term peak discharges using the Pearson type III distribution curve. Multiple-regression analyses were used to define 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. 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 of estimating flood-frequency for urban watersheds were presented by Sauer and others (1983). These techniques were based on data collected on a nationwide basis including five basins from the Atlanta area. A technique for simulating flood hydrographs and estimating lag-time at ungaged stations statewide and in the Atlanta urban area were provided by Inman (1986). He used multiple-regression analysis to define relations between lag-time and certain physical characteristics. Drainage area and slope were found to be statistically significant in the rural parts of the state, both above and below the fall line. Drainage area, slope, and impervious area were found to be 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). In his study he used the USGS rainfall-runoff model (Dawdy and others, 1972) calibrated using data from 45 urban drainage basins in two urban areas in Georgia to synthesize long-term peak discharges. Flood-frequency relations were developed from these long-term peak discharges by fitting the logarithms of the annual peak discharge data to a Pearson type III distribution curve. Multiple-regression analysis was used to define relations between the flood-frequency station data and certain physical characteristics, of which drainage area, measured total impervious area, and equivalent rural discharge were determined to be statistically significant. These relations can be used to estimate the magnitude and frequency of floods at ungaged urban streams throughout Georgia.

Techniques 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 U.S. Department of Agriculture, Agricultural Stabilization and Conservation Service (ASCS), Salt Lake City, Utah, for providing aerial photography of the area.

DATA COLLECTION AND PROCESSING

Rainfall-runoff data were collected from 1987-92 at two stations in Albany, Georgia, for calibration of DR3M. Physical characteristics of the basin required by the model were determined from field surveys, topographic maps, and aerial photographs.

Basin Selection

The two basins used for analysis were selected following a field reconnaissance of 10 basins. The following factors were evaluated for the selection of the two basins:

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

Some basins were excluded because of their hydraulic characteristics or because they contained no suitable rain-gage location. Basins were roughly delineated on USGS 7 1/2-minute topographic maps, and approximate drainage areas were determined. Two basins then were selected for study from those basins deemed suitable. The basins selected had the best hydraulic characteristics for theoretical computations of peak discharges at the gaging station and detention reservoirs, and also the most suitable rain-gage locations. Final drainage areas were delineated from a USGS 7 1/2-minute topographic map and verified by field inspections. The two basins, upstream of stations 02351210 and 02351220, had drainage areas of 0.12 and 0.09 square miles (mi^2), and impervious areas of 23.8 and 12.9 percent respectively. The basin upstream of station 02351210 had two flood-detention reservoirs, (A and B), and the basin upstream of station 02351220 had one flood-detention reservoir. Locations of the gaging stations in these basins are listed in table 1 and shown in figure 1. Detailed maps of the two basins are shown in figures 2 and 3.

Table 1. -- Gaging stations at which rainfall-runoff data were collected in Albany, Georgia.

Station number	Station name	Location
02351210	Kinchafoonee Creek tributary No. 1	Lat 31°36'23", long 84 °10'54" Dougherty County, at culvert on Hoover and Sharon Avenue at Albany
02351220	Kinchafoonee Creek tributary No. 2	Lat 31°37' 06", long 84°11' 25", Dougherty County, at culvert on Wilmar Lane at Albany

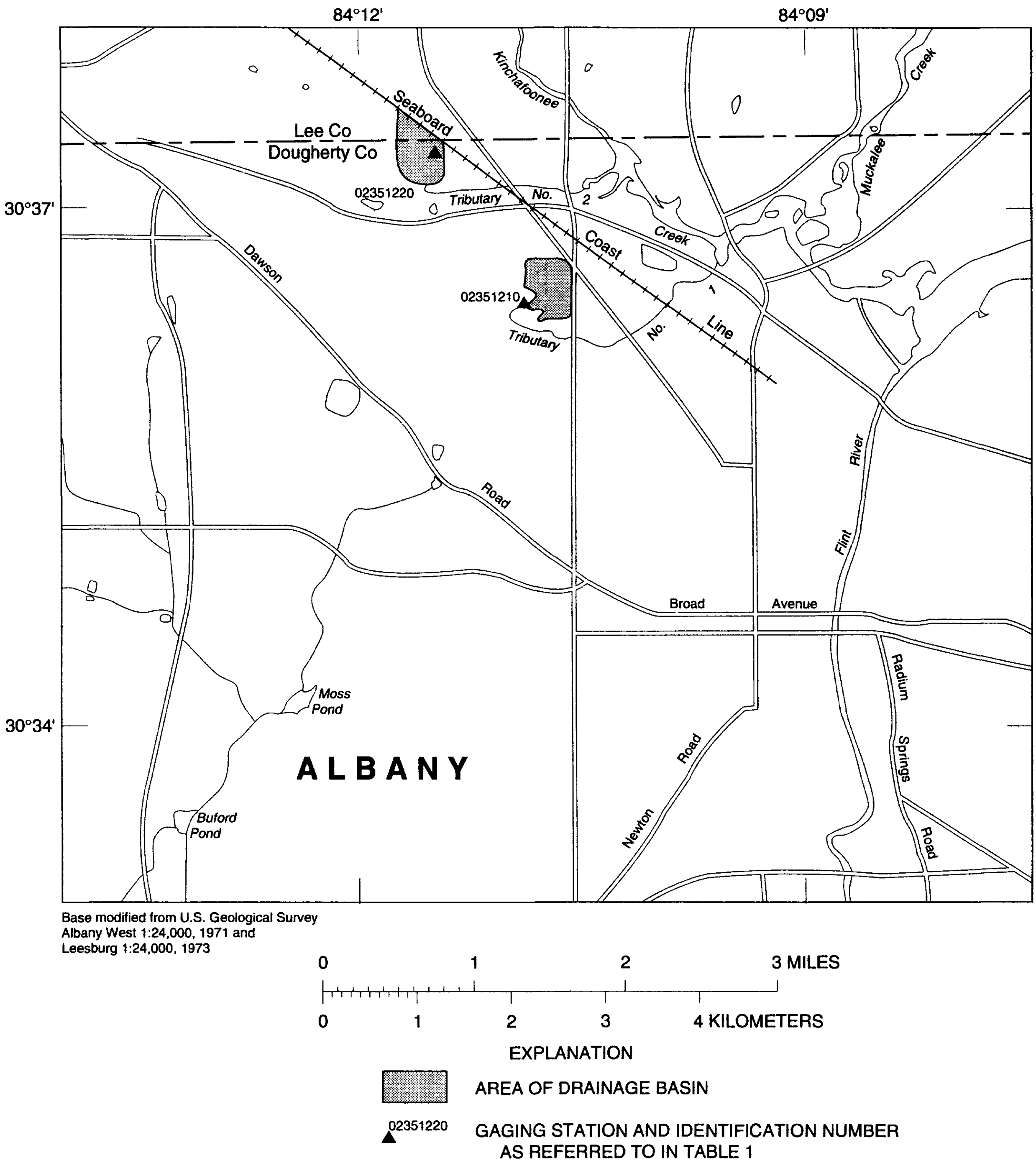

Figure 1. Locations of urban study basins in Albany, Georgia.

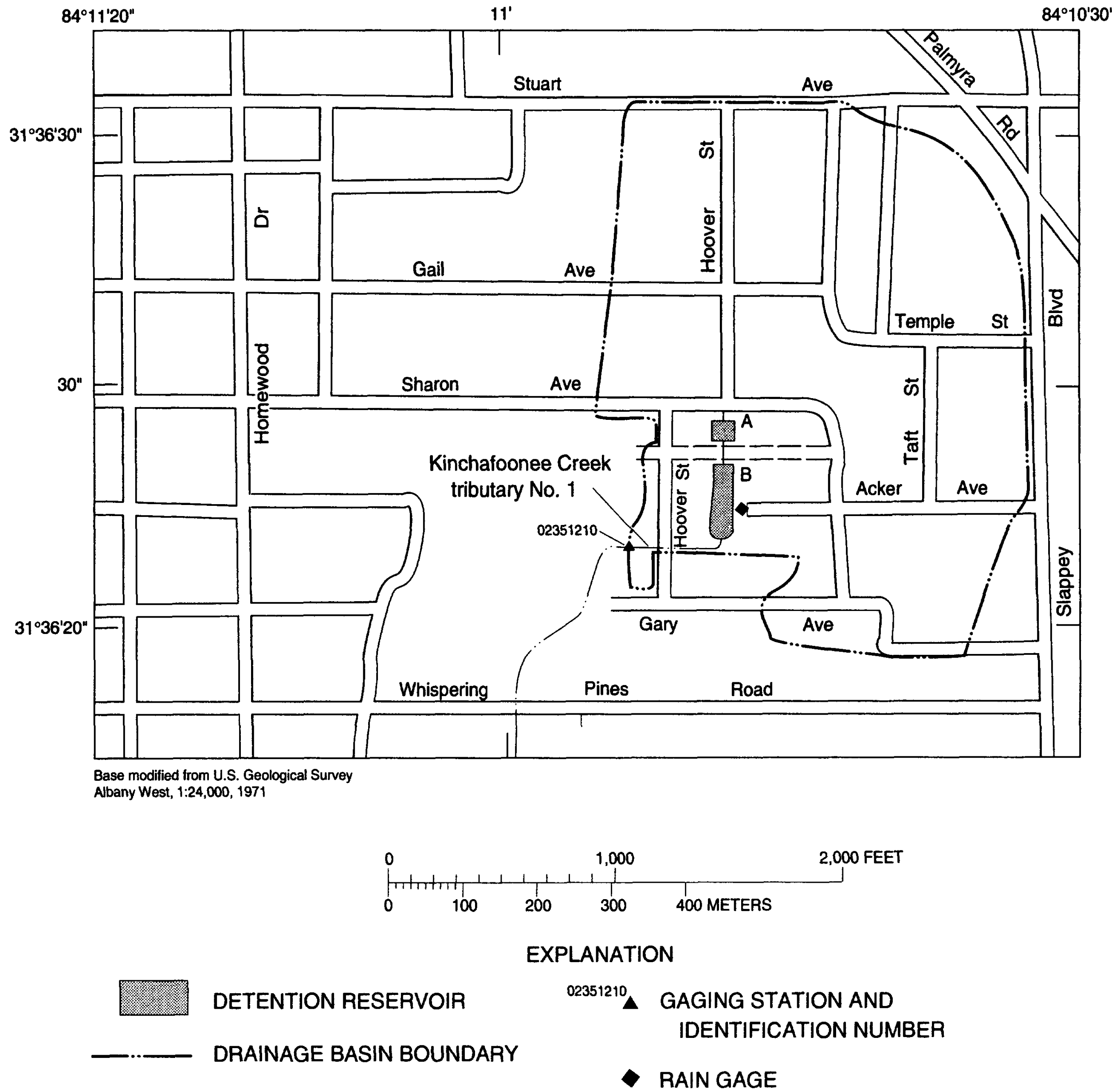

Figure 2. Drainage basin and detention reservoirs for Kinchafoonee Creek tributary No. 1 at culvert on Hoover and Sharon Streets, Albany, Georgia.

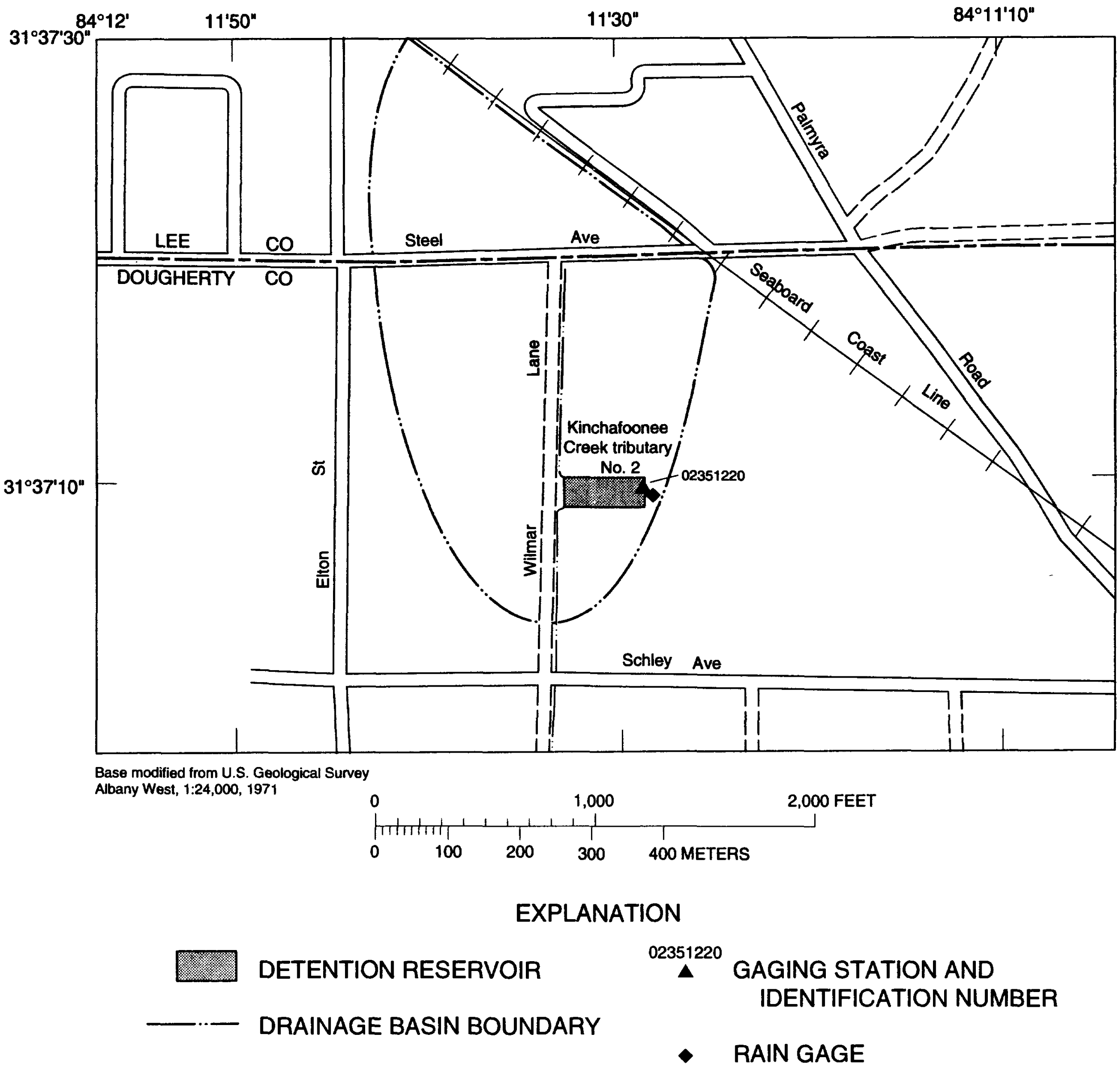

Figure 3. Drainage basin and detention reservoir for Kinchafoonee Creek tributary No. 2 at culvert on Wilmar Lane, Albany, Georgia.

Instrumentation and Data-Collection Techniques

Rainfall-runoff data were collected at both stations using digital recorders with 5-minute data-collection intervals. The stage recorders were housed on top of an 18-in. vertical corrugated metal-pipe stilling well near the outlet of the reservoirs. 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. The stilling wells were flushed after every flood, and intakes were rodded out during every inspection trip.

A rain gage was installed near the stage gage in each basin. Rain-gage recorders were housed on top of 8-ft collector wells made from 3-in. galvanized pipe designed to hold about 11 in. of rainfall. A drain plug near the bottom of the collector well was removed on each inspection trip to drain the accumulated rainfall.

Aerial photographs provided by ASCS, topographic maps provided by the City of Albany (1993), and USGS 7-1/2 minute topographic maps were used to delineate the basins and flood-detention reservoirs. Topographic maps of the reservoirs were planimetered to determine the relation between stage and storage for each flood-detention reservoir. Stage-discharge relations at the gaging stations were determined by current-meter measurements.

Data Processing

Data for all floods for which complete rainfall and stage data were available and for which rainfall totalled more than 0.5 in. and there were no blockages of inflow or outflow, were loaded into USGS computer storage on a near-current basis. Generally, 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, allowing for a bad punch by the recorder or a misread punch by the electronic-tape transmitter to be detected easily, (2) detect partially clogged rain-gage intakes or hanging floats, (3) estimate the rising limb of a flood hydrograph if the stilling-well intakes were above the stream stage at the beginning of the storm, and (4) estimate the falling limb of the hydrograph when the intakes became partially clogged with sediment on the recession. Once the data were reviewed and edited, the data were stored in the data base for further processing.

Daily Class A pan evaporation data (1987-92) were obtained for the National Weather Service (NWS) station near Tifton, Ga. (table 2). Evaporation maps presented by Kohler and others (1959) were used as a guide in selecting Tifton as the suitable evaporation station.

Long-Term Rainfall and Evaporation Data

Long-term rainfall and evaporation data are required for peak discharge simulation. Daily rainfall records for the period 1947-73 were obtained for the NWS station near Thomasville, Ga., (table 2). Four to eight storms per year were selected for study on the basis of daily rainfall totals from hourly data. For periods before 1947 (1906-33 and 1941-46), the unpublished daily charts were obtained from the NWS for all rainfall storms of 1/2-in. or more per day, and selections of storms were made. To ensure that no large storms were overlooked, computer program E436 (Carrigan and others, 1977) was used to select five major rainfall storms from each water year for the periods 1906-33, and 1941-73, and these storms were compared with those selected on the basis of daily rainfall totals.

Observed daily Class A pan evaporation data for the period 1947-73 were obtained for the NWS station near Tifton. To estimate daily evaporation for the period prior to 1947, the computer program H266 (Carrigan and others, 1977) was used to generate evaporation estimates based on a harmonic (sine-cosine) analysis of observed data (1947-73).

Table 2. --National Weather Service rainfall and evaporation stations used in this study of detention reservoirs at Albany, Georgia.

Type	Location	Periods of record (water years[1])	Station number
Rainfall	near Thomasville	1906-33, 1941-73	310100083520050
Evaporation	near Tifton	1947-73, 1987-92	312900083320050

1. Water years begin October 1 and end September 30, and are designated by the year in which they end.

MODEL DESCRIPTION AND CALIBRATION

For simulating flows in urban areas with flood-detention reservoirs, the USGS developed a deterministic Distributed Routing Rainfall-Runoff Model (DR3M) (Alley and Smith, 1982). This model was calibrated for this study using short-term (1987-92) rainfall-runoff data.

Description of the Distributed Routing Rainfall-Runoff Model

DR3M computes and routes rainfall excess through a branched system of pipes or natural channels using rainfall as input. DR3M 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. The soil-moisture-accounting component in 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 channels of the drainage system, and (2) noneffective impervious areas--impervious areas that drain into pervious areas. In the model, a fixed amount of rainfall on the effective impervious areas is held as impervious retention, (usually from 0.02 to 0.05 in.) to account for water retained by the impervious surfaces. Rainfall must exceed this amount before runoff from effective impervious areas can occur.

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 model parameter RAT, which is the ratio of the sum of the pervious and noneffective impervious areas to the pervious area. RAT is calculated as follows:

$$DA3 = (1 - MIA)\ DA \qquad (1)$$

$$RAT = (DA2 + DA3)/DA3 \qquad (2)$$

where DA3 is the area of the basin covered by pervious surfaces, which is calculated as the total drainage area, (DA) minus the measured impervious area ratio, (MIA), and DA2 is the area of the basin covered by noneffective impervious surfaces.

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 defined as:

$$\text{OBJECTIVE FUNCTION} = \Sigma\ (\log\ (\text{synthesized value})-\log\ (\text{observed values}))^2 \qquad (3)$$

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. For modeling purposes, the 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. DR3M can provide simulated discharges at each segment. The model will accept as many as 99 total segments, 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 and 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 can also receive up to four lateral inflows from overland-flow segments. Reservoir segments can be used to describe an on-channel flood-detention reservoir using stage-storage-discharge relations, or 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.

The assumptions behind the use of the kinematic wave equations for channel and overland-flow routing should be recognized by any potential user of DR3M. 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 overland flow 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 vice versa 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

Calibration is the process of determining a set of parameter values that will produce model simulations which best duplicate observed floods. Initially, an average of more than 40 floods per station were available for model calibration. On average 27 floods were used for the final calibrations (table 3). Some outliers were detected and were not used in the model calibration because of one or more of the following possible conditions:

- nonrepresentative rainfall could be determined based on comparison with nearby rain gages, usually associated with localized summer thunderstorms where the rainfall could vary widely over a small area;
- runoff exceeded rainfall;
- rainfall greatly exceeded runoff; and
- upstream crossings could have been clogged with debris, making the peak discharges at the gaging station artificially low.

The first step in calibrating the model was to optimize on effective impervious area using the parameter EAC (see "Glossary" for definition of parameters) with all the other parameters being held constant. Calibration was accomplished by using only data for small floods for which runoff was largely contributed from the effective impervious area of the watershed. The value of EAC initially was set at 1.0, but varied from 0.80 to 1.15 during calibration. 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 ratio values of each subbasin, and this sum then was divided by the total drainage area to obtain optimized impervious area for the basin. An adjustment to RAT is necessary after EAC has been optimized.

The next step in calibrating DR3M was to optimize the soil-moisture accounting and infiltration parameters KSAT, RR, BMSM, RGF and PSP (see "Glossary" for definition of parameters) by using large floods and holding EAC constant. The pan evaporation coefficient EVC was set at 0.75, in accordance with data published by the U. S. Department of Commerce (1959) and Kohler and others (1959). Because the model parameters EVC and RR are highly interactive, only RR was optimized.

A range in values for the infiltration parameter, KSAT, of 0.05 to 0.40 was obtained from Chow (1964). Most of the soils of the two 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, BMSM, RGF, and PSP were obtained from Golden and Price (1976) and Inman (1988).

A stage-storage-outlet discharge rating was prepared for each of the flood-detention reservoirs. The stage-discharge relation was obtained from a theoretical or measured rating at the outlet of the flood-detention reservoir. Reservoir storage was computed from field surveys or obtained by planimetry on 1-ft contour topographic maps (City of Albany, 1993). The outlet discharge and storage were input to the DR3M at corresponding stages.

An explicit finite-difference method is used by DR3M to solve the kinetic routing equations scheme. 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. These three routing parameters were manually adjusted so that simulated peak discharges best matched observed peak discharges.

The standard error of estimate of calibration, in percent, based on the mean square differences of logarithms of the observed and simulated values, was used to quantify improvements in the model made through optimization. The optimized DR3M parameter values and selected physical characteristics are listed in table 3. Simulated peak discharge is plotted against observed peak discharge in figure 4 to illustrate the results of the DR3M calibrations at the gaging stations in the two drainage basins.

Verification is the procedure in which estimates of the model parameters computed by the calibrated model are compared to observed data used for calibration. The use of part of the data for calibration and a different part of the data for verification is referred to as split sample testing. The observed period of record at the two stations, 1987-92, was generally characterized by low runoff. Thus a limited amount of storm events (average 27 events) was available for model calibration and verification. Due to the limited amount of storm events available for verification, average of 13 events, the number of events was considered too small for verification. Thus split-sample testing for verification was not possible.

The slope of the relation between observed and simulated peak discharges for the two stations is 0.98 and 1.00 respectively. However, the simulated peak discharges appear slightly under-predicted. This slight under-prediction bias may be due to 1) calibration of model parameters based on low peak discharge events or 2) inadequate total definition of stage-storage relations at reservoirs.

Table 3. --**Optimized Distributed Routing Rainfall-Runoff Model parameters[1] and selected physical characteristics of drainage basins of stations 02351210 and 02351220, Albany, Georgia.**
[in, inches; in/hr, inches per hour; mi^2, square miles; see "Glossary" for definition of parameters]

Station number	PSP (in.)	KSAT (in./hr.)	RGF	BMSM (in.)	RR	EAC	RAT	Number of segments	Standard error of estimate (in percent)	Drainage area (mi^2)	MIA (in percent)	DR3MIA (in percent)
02351210	2.52	0.22	20.0	8.25	0.77	.806	1.06	10	40	0.12	23.8	21
02351220	1.95	.15	9.9	2.07	.95	.852	1.02	11	36	.09	12.9	11

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

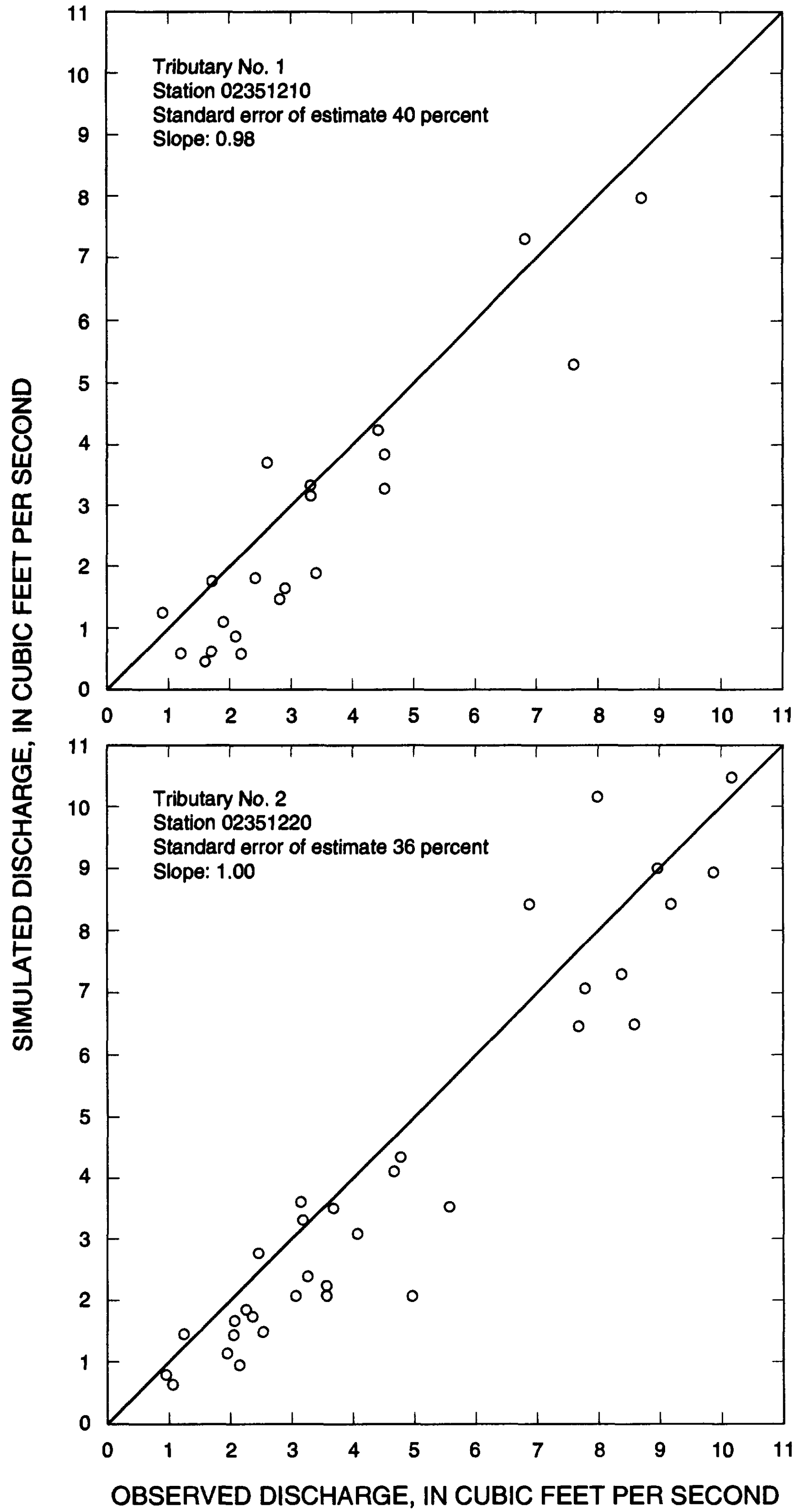

Figure 4. Relation between observed peak discharges and simulated peak discharges for the calibrated Distributed Routing Rainfall-Runoff Model at stations 02351210 on tributary No. 1, and 02351220 on tributary No. 2, Albany, Georgia.

EFFECTS OF URBAN FLOOD-DETENTION RESERVOIRS ON PEAK DISCHARGES AND FLOOD FREQUENCIES

Effects of urban flood-detention reservoirs on peak discharges and flood frequencies for the two tributaries to Kinchafoonee Creek were determined by simulating annual peak discharges for the 2-, 5-, 10-, 25-, 50-, and 100-year peak discharges for conditions with and without the flood-detention reservoirs. Annual peak discharges for the periods 1906-33 and 1941-73 for the two drainage basins were simulated using the final calibrated DR3M parameters, long-term unit and daily rainfall data, and long-term daily evaporation data. The first simulation was for existing conditions with flood-detention reservoirs in place. The model was also used to simulate conditions without flood-detention reservoirs for each basin to examine the cumulative effect of these reservoirs on peak discharges.

The logarithms of the long-term simulated annual peak discharge data for each station, with and without flood-detention basins, 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 two stations are listed in table 4.

The relation between peak magnitude and probability of exceedence (or recurrence interval) is referred to as the flood-frequency relation. Probability of exceedence is the probability that a flood will exceed a specific magnitude in any one year. Recurrence interval is the reciprocal of the probability of exceedence and is the average interval, in years, in which a given flood will be exceeded. For example, a flood having a probability of exceedence of 0.04 has a recurrence interval of 25 years. A flood discharge having a recurrence interval of 25 years does not mean that for each 25-year period the flood discharge will be exceeded, but that the flood discharge will be exceeded an average of once every 25 years over a long period of time, such as a few hundred years. Flood discharges having a 25-year recurrence interval can be exceeded in successive years or even twice in the same year, or might not be exceeded in some 25-year periods.

A comparison of the 10-, 50-, and 100-year peak discharges for conditions with existing reservoirs and without the reservoirs was used to determine the cumulative effect of all flood-detention reservoirs in a basin. The 10-, 50-, and 100-year peak discharges are presented because these peak discharges are specified in Albany design codes for hydraulic structures. The percent increase in the 10-, 50- and 100-year discharges that could be expected without flood-detention reservoirs for station 02351210 and station 02351220 are presented in table 5.

The data presented in table 5 indicate that the removal of the two flood-detention reservoirs A and B from the basin of station 02351210 (tributary No. 1) would increase the 10-, 50-, and 100-year peak discharges from 164 to 204 percent, and that removal of the flood-detention reservoir from the basin of station 02351220 (tributary No. 2) would increase the 10, 50-, and 100-year peak discharges about 145 percent. The magnitude of the percent increase in peak discharge for station 02351210 decreases from the 10-year peak discharge (204 percent) to the 100-year peak discharge (164 percent), which indicates that the effect of the two flood-detention reservoirs in the basin on peak discharges decreases with increasing recurrence intervals (increasing magnitude of peak discharges). However, the magnitude of the percent increase in peak discharge for station 02351220 is about the same for the 10-year peak discharge (148 percent) and the 100-year peak discharge (146 percent) which indicates the effect of the one flood-detention reservoir in the basin on peak discharges is about the same with increasing recurrence intervals. This difference in the effects of flood-detention reservoirs on peak discharges and flood-frequencies between station 02351210 and 02351220 can be attributed to the number of reservoirs in each basin and the stage-storage-outlet discharge relation at each reservoir.

The evaluation of model results for this study indicates the quality of the simulations produced using the calibrated model and model uncertainties. Indicators of model uncertainties are 1) lack of model verification, 2) standard error of estimate, 3) range and size of observed peak discharges, and 4) slight under-prediction bias of calibration.

DR3M has several parameters that must be selected by calibration. Verification data is used to verify that the parameters selected in the model are adequate. Because verification was not possible due to a small data set, the quality of the simulations produced from the calibrated model may be lessened. Expected errors in simulated peak discharges are indicated by the standard error of estimate for calibration at each station which was 40 and 36 percent respectively (table 3). These errors are typical of other urban studies in the area (Inman, 1983, 1988, and Bohman, 1992).

The range of observed peak discharges (table 3) indicated that during the period of record, 1987-92, the maximum peak discharge was about a 2-year peak discharge (table 5). The period of 1987-92 was generally characterized by low peak discharge events. Thus the lack of high peak discharge events during the observed period of record determined a calibration of model parameters based on low peak discharge events. The calibrated model parameters may or may not represent high peak discharge events. Thus the synthesized long-term peak discharges provided a less reliable prediction of the high peak discharges. As described above, a limited amount of storm events (average 27 events) was available for model calibration. A model calibrated with a small data set with a small range of discharge is probably not as robust as a model calibrated with a large data set with a larger range of discharges. The simulated peak discharges appear slightly under-predicted as shown in figure 4 although the slopes of the relations are 0.98 and 1.00, respectively. This slight under-prediction may indicate under-predicted synthesized long-term peak discharges.

Table 4. -- Peak discharges for floods of selected recurrence intervals at stations 02351210 and 02351220 with and without the existing flood-detention reservoirs, Albany, Georgia.

Station number	Simulated condition	Peak discharge for indicated recurrence interval in years, (cubic feet per second)					
		2	5	10	25	50	100
02351210	With existing reservoirs A and B	9	17	25	37	48	61
	Without detention reservoirs	31	56	76	106	132	161
02351220	With existing reservoir	13	22	29	38	45	52
	Without detention reservoir	34	56	72	93	110	128

Table 5. -- Percent increase in simulated peak discharges of floods having 10-, 50- and 100-year recurrence intervals at stations 02351210 and 02351220, when existing flood detention reservoirs are not included in the simulation near Albany, Georgia.

Station number	Percent increase in peak discharge for indicated recurrence interval in years		
	10	50	100
02351210	+204	+175	+164
02351220	+148	+144	+146

SIMULATION OF FLOOD-DETENTION RESERVOIR OUTFLOW HYDROGRAPHS

Observed flood-detention reservoir outflow hydrographs were compared to simulated flood-detention reservoir outflow hydrographs from DR3M and reservoir-routing model A697 (Jennings, 1977) for a flood at each station. Also, DR3M-simulated flood-detention reservoir outflow hydrographs were compared to A697-simulated flood-detention reservoir outflow hydrographs for a historical flood with a peak discharge having a recurrence interval of approximately 10-years. The purpose for the comparison of hydrographs was to determine if the simpler of the two models (reservoir routing) can be used in place of the more complex (rainfall runoff) model in future studies in the basin.

The Downstream-Upstream Reservoir Routing Model, A697, can be used to route an inflow hydrograph (downstream routing) through a reservoir to calculate an outflow hydrograph by the modified-Puls method (U.S. Department of Agriculture, 1972). This method solves the continuity equation with inflow rate, outflow rate, storage, and constant computation time interval as variables. Assumptions in the method include (1) the water surface of the reservoir is level and responds instantaneously to inflows and outflows, (2) the outflows are uniquely described by a storage-discharge relation on an uncontrolled reservoir and, (3) the computation time interval is smaller than twice the ratio of the instantaneous storage to instantaneous outflow at the end of an instantaneous time interval. A697 was used to route a flood-detention reservoir inflow hydrograph generated from DR3M through a flood-detention reservoir and calculate the outflow hydrograph.

The observed June 26, 1991 flood-detention reservoir B outflow hydrograph at station 02351210 was compared to simulated flood-detention reservoir outflow hydrographs from DR3M and A697 (fig. 5). A DR3M simulated inflow hydrograph for April 1, 1948, which had a peak discharge that approximates the 10-year recurrence interval inflow peak discharge was routed through flood-detention reservoir B using A697. The DR3M was also used to route the hydrograph through flood-detention reservoir B. The A697 detention reservoir outflow hydrograph and the DR3M detention reservoir outflow hydrographs are shown in figure 6.

Hydrographs having peak discharges that approximate the 10-year (April 1, 1948) and 25 year (June 19, 1945) recurrence interval discharges were routed through flood-detention reservoir A at station 02351210 using DR3M. The inflow and outflow hydrographs for the 10-year recurrence interval discharge are shown in figure 7 and those for the 25-year recurrence interval discharge are shown in figure 8. These plots indicate that flood-detention reservoir A has little effect on peak discharges at station 02351210 at both the 10- and 25-year recurrence interval. Dead storage (storage that exists at elevations below the normal pool elevation) constitutes most of the storage available in flood-detention reservoir A. Also, reservoir A spills over the dirt road that functions as a dam at relatively low stages. The reservoir overtops the road at a stage only about 1.7 ft above the dead pool elevation. This limits storage capacity of this reservoir to about 5,510 ft^3. Field inspection of the reservoir indicates that flow over the road occurs frequently.

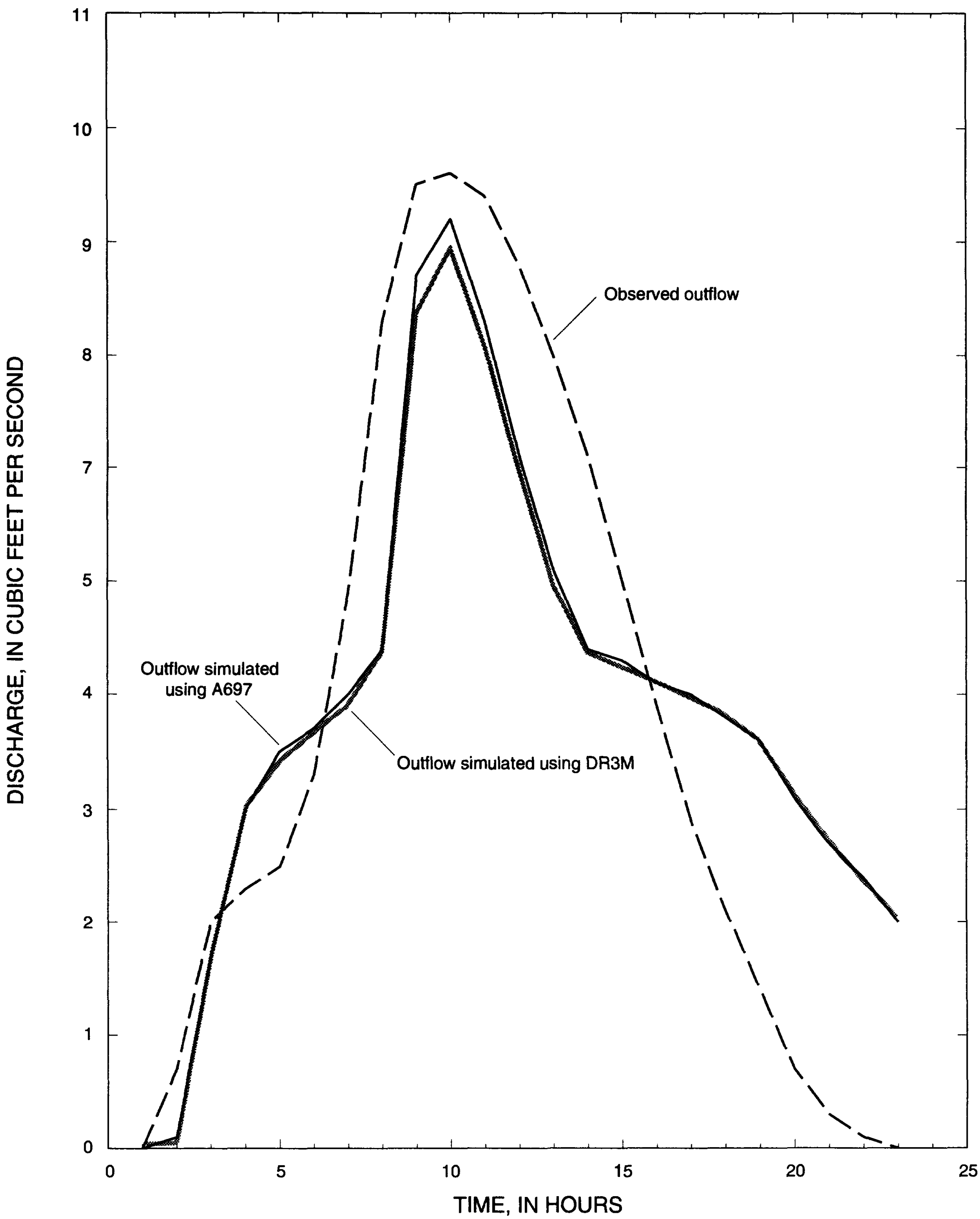

Figure 5. Observed outflow hydrograph and hydrographs simulated using DR3M, and A697 models at flood-detention reservoir B for flood of June 26, 1991, at station 02351210, at Albany, Georgia.

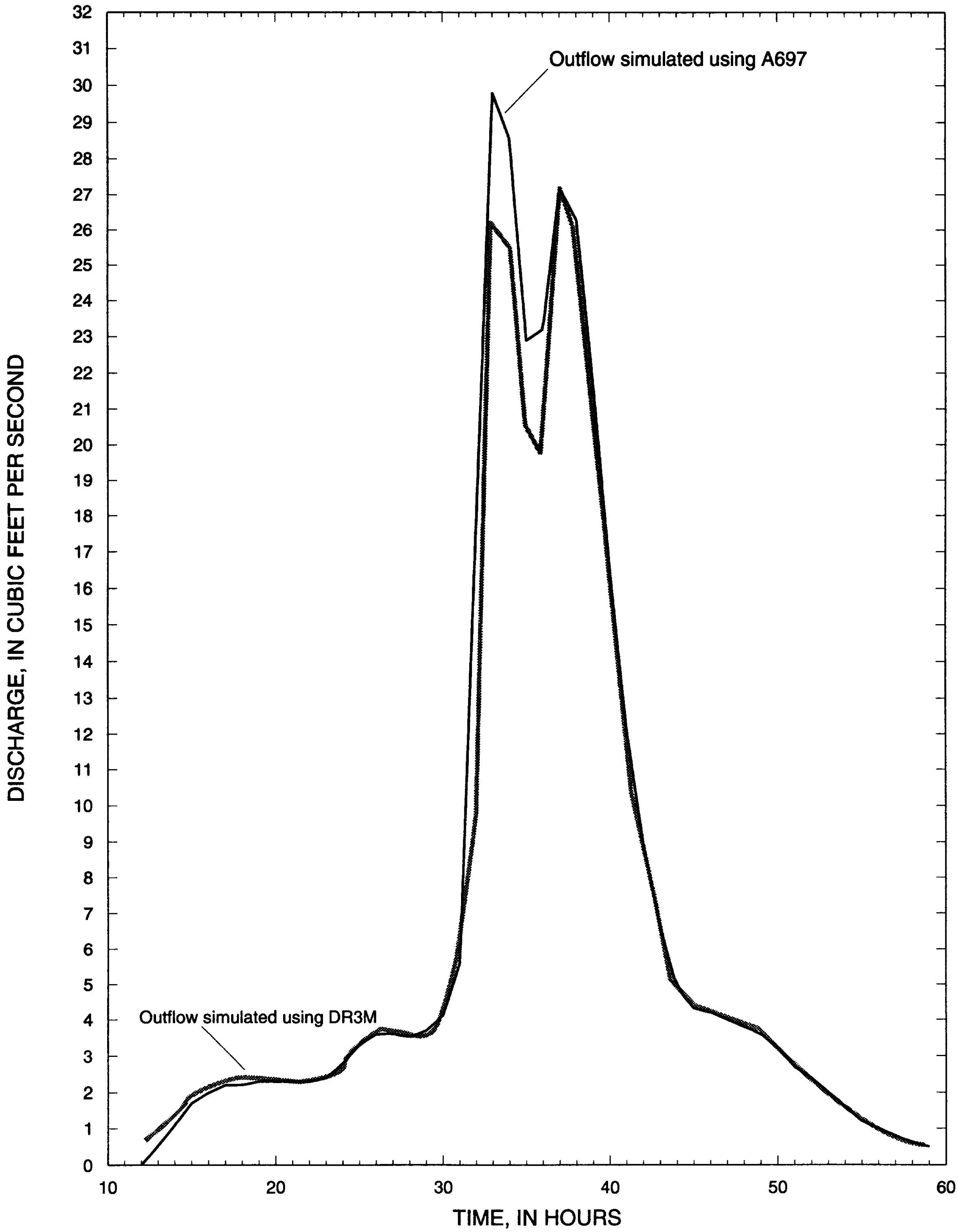

Figure 6. Outflow hydrographs simulated using DR3M and A697 models at flood-detention reservoir B for flood of April 1, 1948, at station 02351210 at Albany, Georgia.

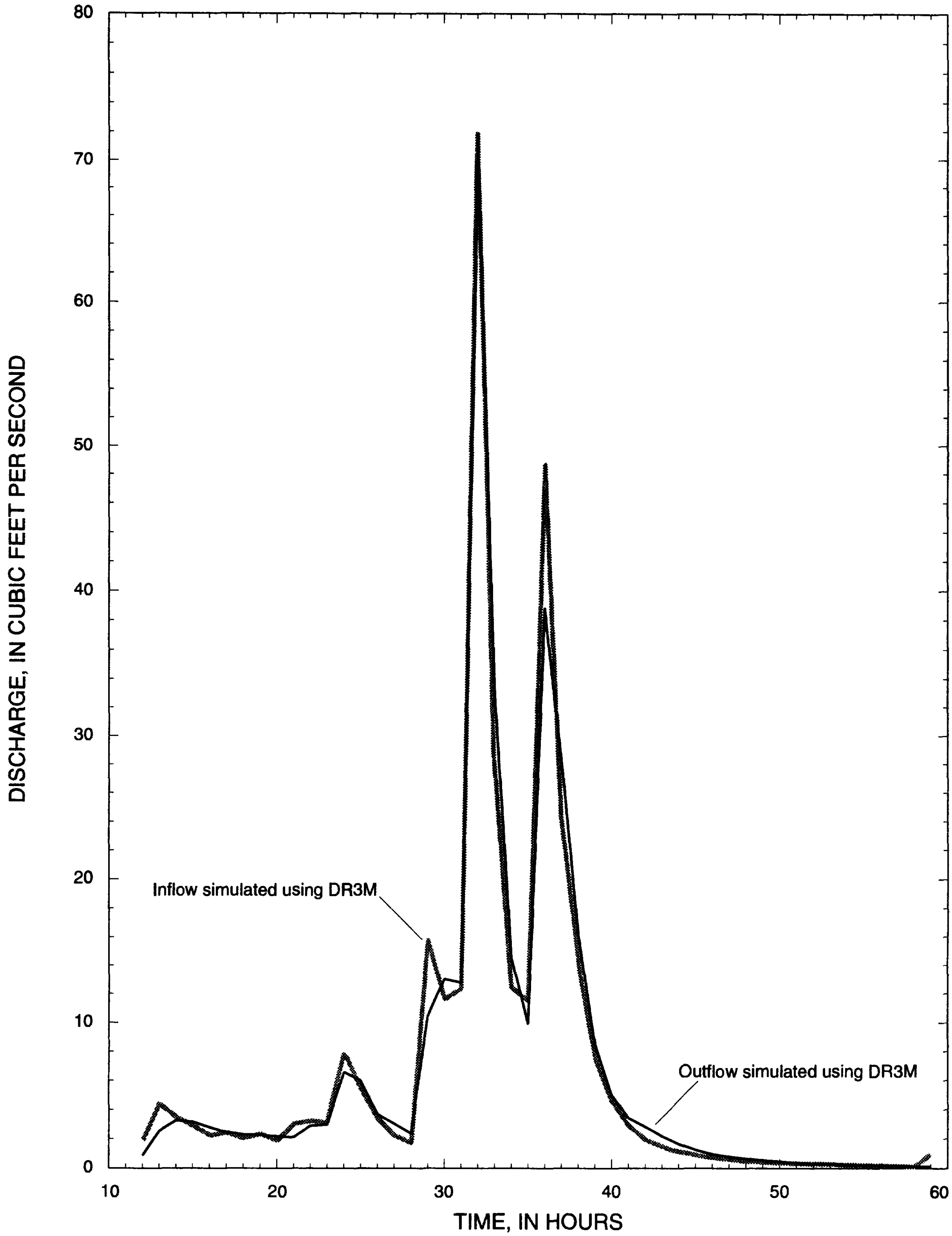

Figure 7. Inflow and outflow hydrographs simulated using DR3M at flood-detention reservoir A for flood of April 1, 1948, at station 02351210 at Albany, Georgia.

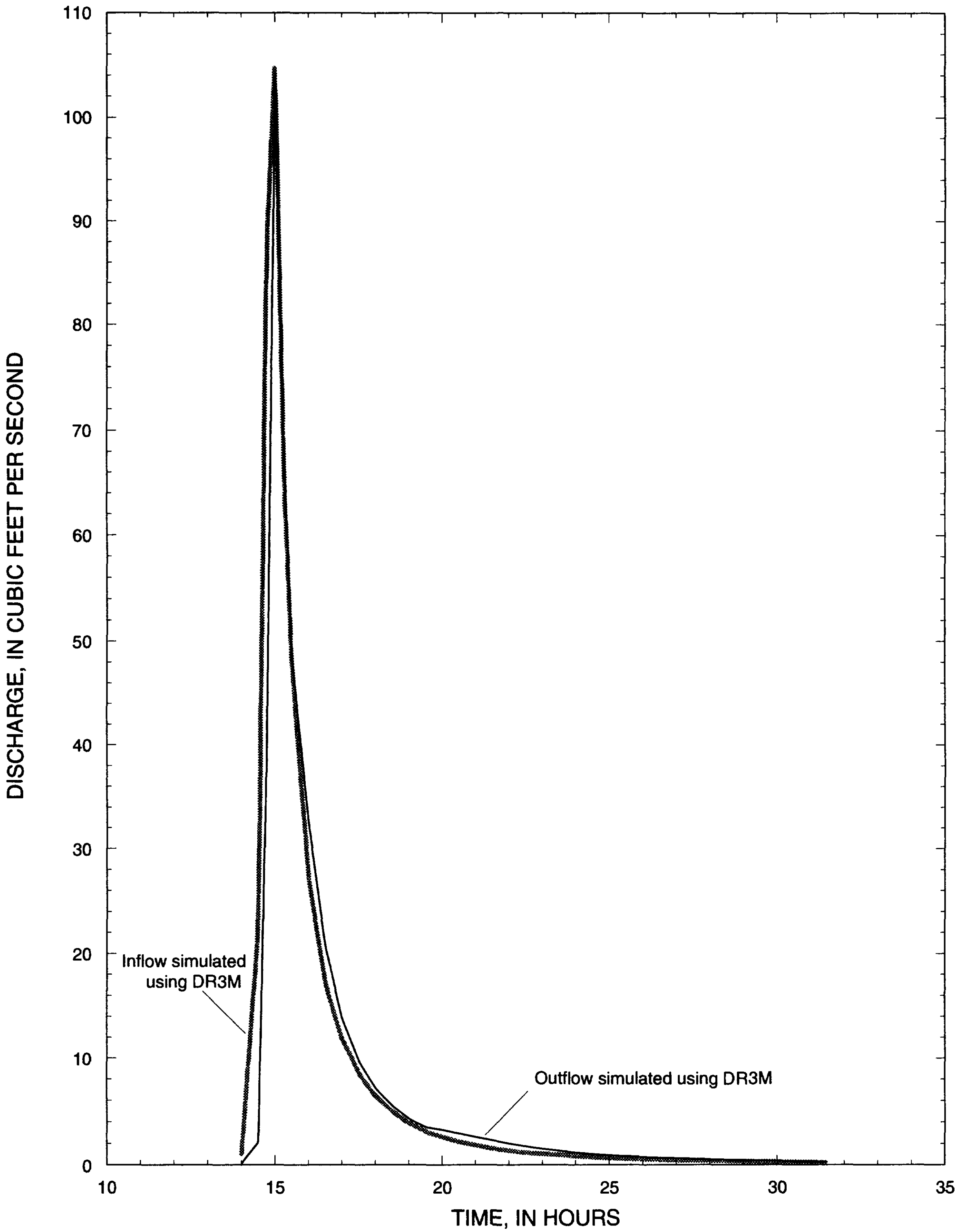

Figure 8. Inflow and outflow hydrographs simulated using DR3M at flood-detention reservoir A for flood of June 19, 1945, at station 02351210 at Albany, Georgia.

The suitability of DR3M and A697 for simulating flood-detention reservoir outflow hydrographs was evaluated by comparison of observed and simulated outflow hydrographs. The observed June 26, 1991 flood-detention reservoir outflow hydrograph at station 02351220 is shown along with flood-detention reservoir outflow hydrographs simulated using DR3M and A697 in figure 9. A comparison of observed and simulated hydrographs in this figure indicates that outflow hydrographs for this station simulated using DR3M and A697 are in good agreement with the observed outflow hydrographs. Outflow hydrographs for this station also were simulated using DR3M and A697 and an inflow hydrograph for a much larger flood on April 1, 1948 which had a peak discharge hydrograph that approximates the 10-year recurrence interval peak discharge (fig. 10). A comparison of the DR3M and A697 outflow hydrographs in figure 10 indicates that DR3M and A697 produced similar flood-detention reservoir outflow hydrographs at this station.

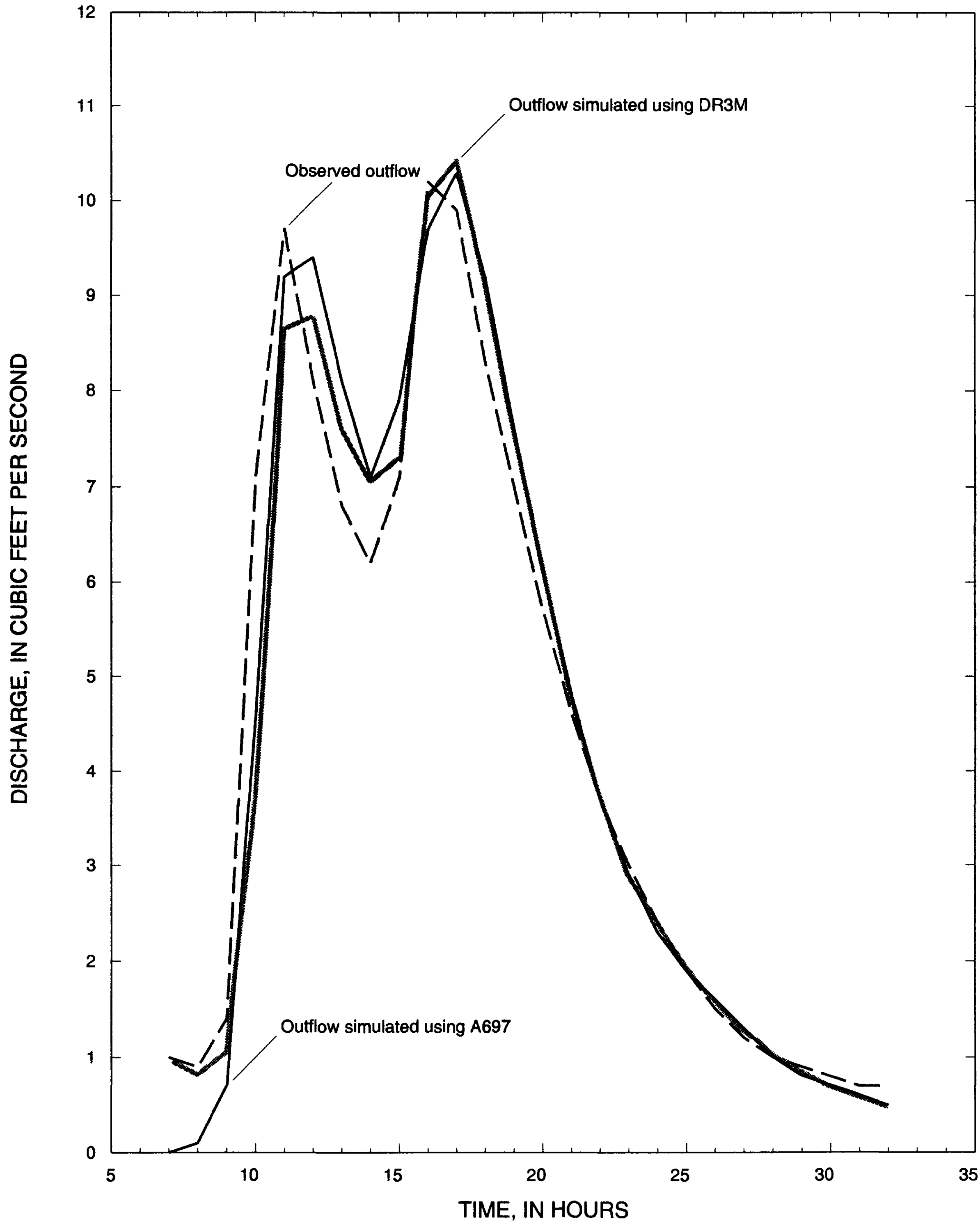

Figure 9. Observed outflow hydrograph and hydrographs simulated using DR3M, and A697 models for flood of June 26, 1991, at station 02351220, at Albany, Georgia.

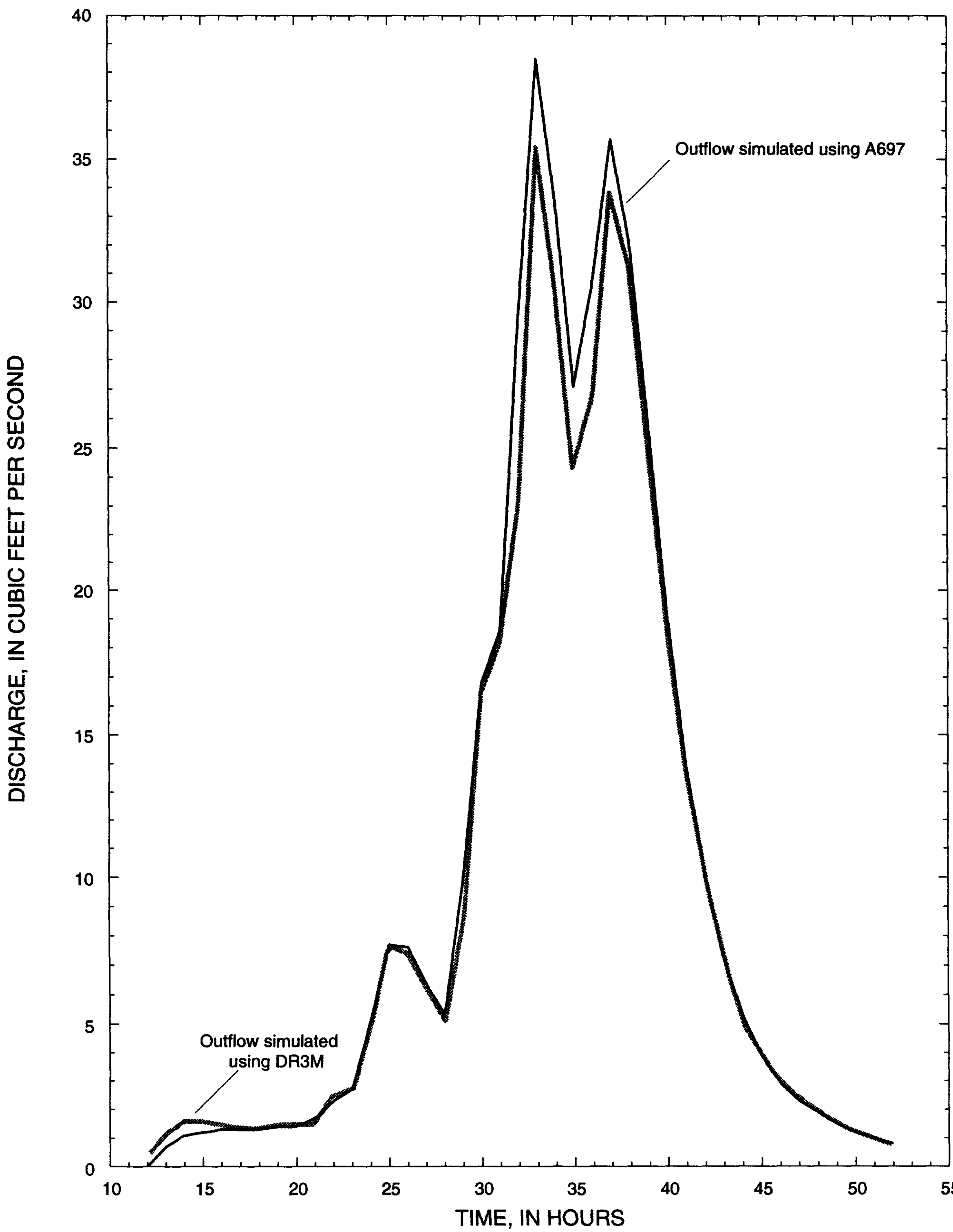

Figure 10. Outflow hydrographs simulated using DR3M and A697 models for flood of April 1, 1948 at station 02351220 at Albany, Georgia.

SUMMARY

This report describes the effect of flood-detention reservoirs on peak discharges with recurrence intervals of as much as 100 years along downstream reaches in two urban tributaries to Kinchafoonee Creek at Albany, Georgia. The report also describes and compares flood-detention reservoir outflow hydrographs simulated using a Distributed Rainfall Routing Model (DR3M) and a reservoir routing model A697. The purpose of comparing the hydrographs was to determine if the simpler of the two models (reservoir routing) can be used in place of the more complex (rainfall runoff) model in future studies in the basin.

The two urban basins, upstream of stations 02351210 and 02351220 used in this study, have drainage areas of 0.12 and 0.09 square miles, and impervious area within these basins total 23.8 and 12.9 percent, respectively. The basin upstream from station 02351210 contains two flood-detention reservoirs and the basin upstream from station 02351220 contains one reservoir. The DR3M was calibrated with observed short-term rainfall-runoff data collected during 1987-92 for each of the two basins. The calibrated DR3M and historical rainfall data were then used to simulate long-term (1903-33, 1941-73) annual peak discharges for existing conditions with the flood-detention reservoirs in place and for conditions without the reservoirs. Also, flood-frequency relations were developed using the simulated long-term annual peak discharges for conditions with and without the detention reservoirs by fitting the logarithms of the annual peak discharge data to a Pearson type III distribution curve. The effect of the flood-detention reservoirs on peak discharges at each station was determined by comparison of simulated discharges having recurrence intervals of 2-, 5-, 10-, 25-, 50-, and 100-years for conditions with and without the reservoirs. The comparisons indicate that without the two upstream flood-detention reservoirs the 10-, 50-, and 100-year peak discharges at station 02351210 would increase 164 to 204 percent, and without the flood-detention reservoir upstream from station 02351220 the 10-, 50-, and 100-year discharges at this station would increase about 145 percent. The differences in effect of flood-detention reservoirs on peak discharges and flood-frequencies between stations 02351210 and 02351220 can be attributed to differences in the number of reservoirs in the basins and the stage-storage-outlet discharge relation at each reservoir.

Observed flood-detention reservoir outflow hydrographs were compared to flood-detention reservoir outflow hydrographs simulated using DR3M and reservoir-routing model A697 for a single flood at each station. These comparisons indicate that simulated outflow hydrographs from both DR3M and A697 were in good agreement with the observed outflow hydrographs. A comparison of a DR3M-simulated flood-detention reservoir outflow hydrograph and an A697-simulated flood-detention reservoir outflow hydrograph at each station for a historical flood that had a peak discharge with a recurrence interval of approximately 10-years indicates that A697 can also be used to simulate flood-detention reservoir outflow hydrographs. Comparisons of the DR3M inflow and outflow hydrographs for reservoir A upstream from station 02351210 indicate that reservoir A has little effect on peak discharges at that station.

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.

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.

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, Georgia, School of Civil Engineering, Georgia Institute of Technology, 223 p.

Jennings, M.E., 1977, Downstream-upstream reservoir routing: U.S. Geological Survey Computer Contribution, NSTL Station, Miss., 42 p.

REFERENCES--continued

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, Georgia, 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 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 Agriculture, 1972, National engineering handbook, section 4, hydrology in flood-routing: Soil Conservation Service, Chap. 17, p. 1-93.

U.S. Department of Commerce, 1948-1980, Climatological data, monthly publications for Georgia: Asheville, N. C. National Oceanic and Atmospheric Administration, Weather Bureau, National Climatic Data Center, variously paged.

U.S. Department of Commerce, 1959, Evaporation maps for the United States: Washington D.C., National Oceanic and Atmospheric Administration, Weather Bureau, Technical Paper No. 37, 13 p.

GLOSSARY

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

A697: The Downstream-upstream Reservoir Routing Model (Jennings, 1977) is used to route hydrographs through an uncontrolled reservoir.

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 is used in DR3M as a soil-moisture accounting parameter.

DR3MIA (Optimized effective impervious area, DR3M): The optimized value of effective impervious area as used in DR3M.

Drainage area (DA): The drainage area of a basin, in mi^2, at a specified location is that area planfmetered from topographic maps and the basin boundaries are field checked.

DA2: The DR3M parameter used to define the area of basin covered by noneffective impervious surfaces.

DA3: The DR3M parameter used to define the area of the basin covered by pervious surfaces.

E436: The computer program E436 (Carrigan and others, 1977) selects five events for each water year from daily rainfall data.

EAC: The DR3M parameter used to define the factor by which the value of effective impervious area is multiplied.

EVC: The pan coefficient for converting measured pan evaporation to potential evapotranspiration used in DR3M as a 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 in DR3M as an infiltration parameter.

MIA (Measured total impervious area): The percentage (ratio) of drainage area that is impervious to infiltration of rainfall 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 that defines the suction at wetting front for soil moisture at field capacity, in inches.

RAT: The DR3M parameter which defines the ratio of the sum of the pervious and noneffective impervious areas to the pervious area.

RGF: The DR3M infiltration parameter which defines 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 which defines the proportion of daily rainfall that infiltrates into the soil for the period of simulation, excluding unit days.