Flood-Frequency and Detention-Storage Characteristics of Bear Branch Watershed, Murfreesboro, Tennessee

By GEORGE S. OUTLAW

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CONVERSION FACTORS AND ABBREVIATIONS

Multiply	By	To obtain
inch (in)	25.4	millimeter
foot (ft)	0.3048	meter
square mile (mi^2)	259	hectare
square mile (mi^2)	2.59	square kilometer
cubic foot (ft^3)	0.02832	cubic meter
cubic foot (ft^3)	28.32	liter
cubic foot (ft^3)	28,320	cubic centimeter
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
foot per foot (ft/ft)	0.3048	meter per meter
inches per hour (in/h)	25.4	millimeters per hour
cubic feet per second - hour (ft^3/s-h)	0.02832	cubic meters per second - hour

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 the United States and Canada, formerly called the Sea Level Datum of 1929.

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Abstract

The U.S. Geological Survey's Distributed Routing Rainfall-Runoff Model [DR_3M] was applied to a 2.27-square-mile portion of Bear Branch watershed in northern Murfreesboro to demonstrate the application of this model to small urban watersheds in central Tennessee. Kinematic wave theory was used to route excess rainfall overland and through a branched system of stream channels. The model was calibrated with hyetographs from two rain gages, hydrographs from two streamflow gages, and peak-flood elevations from two crest-stage gages that were operated in the watershed from March 1989 through July 1992. Standard errors of estimate for peak discharge at Northfield Boulevard and Compton Road are 41.4 and 92.2 percent, respectively. Standard errors of estimate for runoff volumes at Northfield Boulevard and Compton Road are 53.5 and 97.6 percent, respectively.

The calibrated model was used to simulate flood hydrographs for 73 large storms occurring during the period 1901-90 and the simulated flood peaks were used to develop flood-frequency relations for present (1992) conditions in the watershed.

Flood discharges for the 100-year recurrence-interval storm were estimated as 350 cubic feet per second (ft^3/s) at Northfield Boulevard, 1,100 ft^3/s upstream of DeJarnett Lane, 610 ft^3/s downstream of DeJarnett Lane, 800 ft^3/s upstream of Osborne Lane, 790 ft^3/s downstream of Osborne Lane, and 1,000 ft^3/s at Compton Road. The effect of detention storage on flood hydrographs was simulated at several locations in the watershed. Detention storage upstream of DeJarnett Lane significantly reduces downstream flood peaks, whereas detention storage upstream of Osborne Lane has almost no effect. The results of this study indicate that the Distributed Routing Rainfall-Runoff Model could be an important tool for testing the effects of potential future development and flood storage alternatives on flooding in small urban watersheds throughout the area.

INTRODUCTION

Urban development in and surrounding Murfreesboro, Tennessee, has increased steadily during the last two decades. Urban development alters the runoff characteristics of a drainage basin, primarily through associated increases in impervious areas, stream-channel improvements, storm-sewer developments, and curb and gutter streets. These changes increase the velocity and in some cases the volume of runoff, which in turn affect increases in the size of flood peaks for a given storm event.

A calibrated rainfall-runoff model is a tool to predict the hydrologic effect of specific options in urban development, such as regulations controlling density of developments, amount of impervious surfaces, and site design. Hydrologic effect is commonly quantified by estimating changes in a basin's flood-frequency characteristics, which are in turn estimated from model-simulated data.

In addition to land-use planning, a model is also useful in design of drainage systems. The model can be used to estimate detention storage and the corresponding attenuation of flood peaks of existing and designed culverts, bridges, and stormwater detention basins.

The U.S. Geological Survey (USGS) Distributed Routing Rainfall-Runoff Model, DR_3M (Alley and Smith, 1982), has been applied for these purposes to many small urban watersheds throughout the country. The model was designed specifically to simulate a small watershed with a distributed pattern of runoff characteristics and a variety of detention storage structures. This model has not been applied, however, to an area with hydrologic characteristics similar to the Murfreesboro area. The Murfreesboro area lies within the inner part of the Central Basin of Tennessee, and is characterized by thin soil cover overlying dense limestone formations. As a result of this poor water-storing capacity, streams in this area are extremely flashy. DR_3M has not been tested for its ability to simulate these unusual hydrologic conditions, and consequently its usefulness as a tool in land-use planning and drainageway design in this and similar areas is not known.

In 1989, the USGS, in cooperation with the City of Murfreesboro, initiated a 3-year study of a small urban watershed, Bear Branch, in northern Murfreesboro. The objectives of this study were to:

1. Collect rainfall and streamflow data at Bear Branch watershed during the period 1989-92.
2. Calibrate DR_3M for present conditions at Bear Branch watershed using hydrologic data collected during the period March 1989 through July 1992.
3. Develop flood-frequency relations at Bear Branch using simulated peak streamflows for the periods 1901-70, 1979,1986, and 1990.
4. Use the calibrated model to quantify the effectiveness of detention storage at two locations in the watershed.

Experience gained from calibrating and applying the model in this watershed can be used to indicate validity of DR_3M for similar watersheds in the Central Basin of Tennessee.

Purpose and Scope

The purposes of this report are to:

- summarize the methods of data collection and analysis for this study;
- describe the calibrated rainfall-runoff model;
- present the flood-frequency relations for current (1992) conditions; and
- present information on detention-storage characteristics at two locations in the watershed.

Information presented in the report includes calibrated model parameters, model application techniques, watershed physical characteristics, observed hyetographs, observed and simulated hydrographs, and flood-frequency relations.

Description of the study area

The Murfreesboro area receives an average of 51 inches of rainfall each year. This amount is not evenly distributed throughout the year, however; the wettest month is March which receives an average of 5.35 inches of rain, the driest month is October which receives an average of 2.87 inches (Perrich, 1993). Some evaporation takes place during the whole year but most occurs during the warmer months. Roughly 66 percent of all evapotranspiration (evaporation plus transpiration) takes place during the months of May through August (Nave, 1961, p. 24).

Murfreesboro lies within the inner part of the Central Basin physiographic province of Tennessee. Soil cover in this area is thin, generally less than 4 feet, and overlies dense limestone. In some areas, the land surface is bare of all soil cover except in the joints (vertical cracks) between blocks of limestone. The thin soil cannot store much water, and the only space for water storage in the dense bedrock is in cracks and solution cavities that develop along the joints. Joints are probably the principal pathways by which water is discharged from rock formations into the streams. As a result of the relatively small capacity for water storage, the streams in the Murfreesboro area are extremely flashy—the discharge rate of the streams changes rapidly during storm runoff (Burchett and Moore, 1971, p. 8).

The larger streams in the Murfreesboro area have well-defined channels cut into the limestone bedrock. The smaller streams usually have poorly-defined channels and flow in broad, shallow depressions; in many cases, the depth of a channel is determined by the thickness of the soil cover (Burchett and Moore, 1971, p. 6).

Bear Branch flows in a northerly direction, joining with Dry Branch, a smaller tributary, just prior to emptying into East Fork Stones River at a point adjacent to the Alvin C. York Veterans Administration Hospital in northern Murfreesboro. The Distributed Routing Rainfall-Runoff Model was applied to a 2.27-square-mile part of the Bear Branch watershed (the study area). The study area extends about four miles in length from the headwaters of the watershed at Middle Tennessee State University to Compton Road, the northern limit of the study area (figure 1).

Land surfaces in the study area are composed of approximately 6 percent impervious surfaces including paved areas and buildings and 94 percent pervious surfaces including yards, forest, fields and pasture. Land surface elevations in the study area range from approximately 620 feet above sea level in the headwaters to about 540 feet above sea level at Compton Road. The average land slope in the study area is approximately 0.015 foot per foot. The Bear Branch stream channel, typical of most small streams in the Murfreesboro area, is poorly formed and heavily vegetated in many places. The average stream channel bed slope in the study area is about 0.0042 foot per foot.

The effect of detention storage was simulated at three sites in the study area. The area upstream of Northfield Boulevard, where the channel is poorly formed and heavily vegetated, was simulated as a site of detention storage. Also, the road beds and culverts at DeJarnett Lane and Osborne Lane were simulated as sites of detention storage.

Acknowledgments

The author wishes to acknowledge the City of Murfreesboro for their support and cooperation throughout the project, and for providing detailed topographic data of the watershed. The author also wishes to thank Anne B. Hoos for her comments and suggestions during the writing of this report.

DATA COLLECTION

Rainfall and streamflow data were collected at two sites, and crest-stage data at two additional sites (fig.1, table 1) in the Bear Branch watershed from March 1989 through July 1992. Data are presented in a separate report (Outlaw and others, 1992). Additional data required for model calibration and historical simulation are daily evaporation amounts, long-term daily rainfall amounts, and long-term unit (5-minute) rainfall amounts for selected storm periods. Daily evaporation amounts were obtained from the records of the National Weather Service (NWS) gages in Tennessee for the period 1901 through 1992. Daily rainfall amounts and unit rainfall for annual peak storms for the period 1901 through 1970 were obtained from the NWS gage at Nashville. In addition, unit rainfall for three large storms that occurred in Murfreesboro in 1979, 1986, and 1990 were obtained from the NWS gage in Murfreesboro (for 1979 and 1986 storms) and the USGS rain gages in Murfreesboro (for the 1990 storm).

CALIBRATION OF RAINFALL-RUNOFF MODEL, DR_3M

DR_3M is a computer model that can be calibrated to simulate the hydrologic characteristics of a specific watershed using known or measured physical characteristics and hydrologic data for the watershed. Application of DR_3M to Bear Branch watershed was accomplished with the aid of two additional computer

Table 1. Description of gages in Bear Branch watershed

Gage number on fig. 1	Station number	Station name	Location
		Rain gages and partial-record streamflow gages	
1	034277045	Bear Branch at Murfreesboro.	Gages located at right bank, 20 feet upstream of culvert on Northfield Boulevard.
4	03427707	Bear Branch near Lascassas.	Gages located at right bank, 40 feet upstream of culvert on Compton Road.
		Crest-stage gages	
2	03427705	Bear Branch near Murfreesboro.	Upstream gage located at left bank, 40 feet upstream of culvert on DeJarnett Lane. Downstream gage located at left downstream wingwall.
3	03427706	Bear Branch near Compton.	Upstream gage located at left bank, 20 feet upstream of culvert on Osborne Lane. Downstream gage located at left bank, 10 feet downstream of culvert.

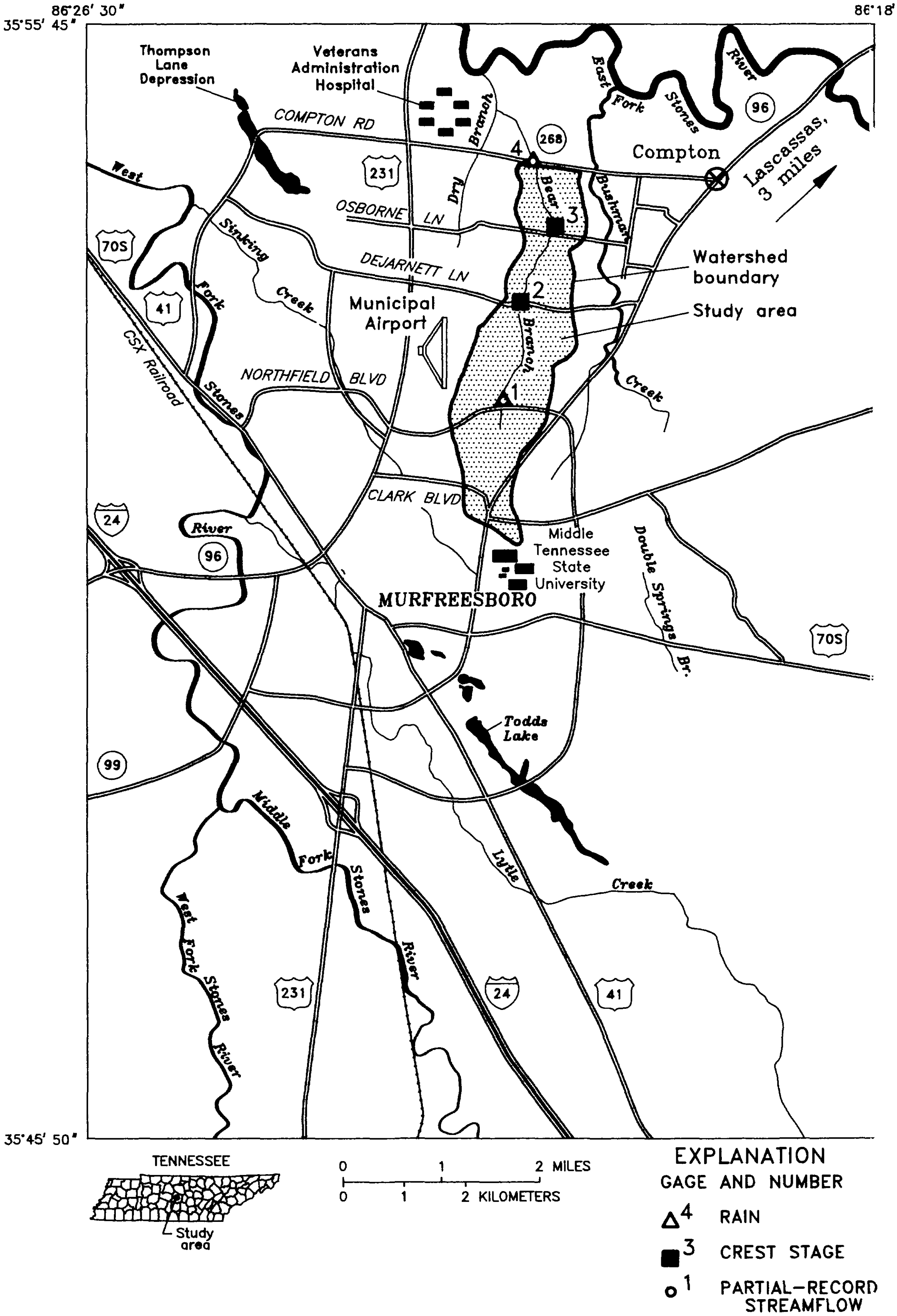

Figure 1. Location of watershed and data-collection points for Bear Branch.

programs: ANNIE, the USGS watershed data management file pre- and post-processing routines (Lumb and others, 1990); and WSPRO, the USGS water-surface profile computation model (Shearman, 1990). A flow chart of the method of analysis (fig. 2) illustrates the interaction between the models.

DR$_3$M opens a watershed data management (WDM) file to read observed rainfall, evaporation, and streamflow information, and to write simulated streamflow information. ANNIE software was used to create the WDM file and to add the observed hydrologic data. It was also used to produce rainfall-runoff plots and flood-frequency relations.

The water-surface profile model, WSPRO, was used to analyze discharge characteristics at two culverts on Bear Branch where the effects of detention storage were simulated by DR$_3$M. Discharge and backwater elevations provided by these hydraulic analyses were used with 2-foot contour interval topographic maps provided by the City of Murfreesboro to quantify detention storage at the culverts.

It is important to note that the results of this study apply specifically to Bear Branch. However, the techniques used are transferable to other small urban watersheds with a similar hydrologic setting where observed rainfall and streamflow data are available.

Model Construction

DR$_3$M requires information contained in two data files: a model control file, and a watershed data management (WDM) file. The model control file identifies storms to be simulated and defines the watershed in numeric terms. The WDM file contains observed hydrologic data for the period of model simulation. Examples of the input data files are shown in the DR$_3$M user's manual.

Watershed Data

Bear Branch watershed was defined as a set of overland segments, channel segments, and nodes. Bear Branch watershed was segmented to contain 26 overland segments, 14 channel segments, and 10 nodes. Detention storage was incorporated at three of these nodes. A schematic representation of the watershed (fig. 3) shows the location of the segments and nodes.

Overland segments carry uniformly distributed excess rainfall to channel segments. Overland segments can be conceptualized as rectangular planes with unique physical characteristics. These characteristics include rectangle dimensions (overland-flow length and channel length), surface slope, percentages of surface area representing pervious and impervious surfaces, and flow-resistance parameters applicable to pervious and impervious surfaces (table 2). Overland-segment characteristics for Bear Branch watershed reported in table 2 were determined using 2-foot contour interval topographic maps provided by the City of Murfreesboro and from field surveys. Evaporation, soil-moisture, and rainfall infiltration parameters (table 3) are used by DR$_3$M to estimate soil-moisture conditions between storm simulations and rainfall infiltration rates during storm simulations. Values for these parameters are determined by the model using an optimization technique described in a later section.

Channel segments receive uniform lateral inflow from overland segments and upstream inflow from other channel segments and detention-storage nodes. For Bear Branch watershed, channel segments are classified by the general shape of the channel cross section as natural or triangular. The natural channels have trapezoidal shaped cross sections. The triangular channels have equal side slopes.

Physical characteristics used to describe natural channels include length, slope, and kinematic wave parameters. Kinematic wave parameters, which are discussed in detail in the DR$_3$M user's manual (Alley and Smith, 1982), incorporate channel geometry, slope, and roughness. For natural channels, general expressions for the kinematic wave parameters, a and m, have been developed by rearranging the parameters in the Manning formula. To solve the equations for the kinematic wave parameters, a log-log relation must be developed between the wetted perimeter and flow area of a given channel cross section. The slope and y-intercept of the log-log relation can be substituted into the rearranged Manning formula to solve for the kinematic wave parameters. Physical characteristics used to describe triangular channels include length, slope, side slopes, and Manning's roughness coefficient. DR$_3$M computes kinematic wave parameters for triangular channels. Channel-segment characteristics for Bear Branch watershed (table 4) were determined using the 2-foot contour interval topographic maps.

Nodes are located at the downstream end of channel segments. DR$_3$M provides simulated discharge hydrographs at each node. Storage nodes are used to incorporate the effects of in-channel detention storage on flood characteristics and flood frequencies.

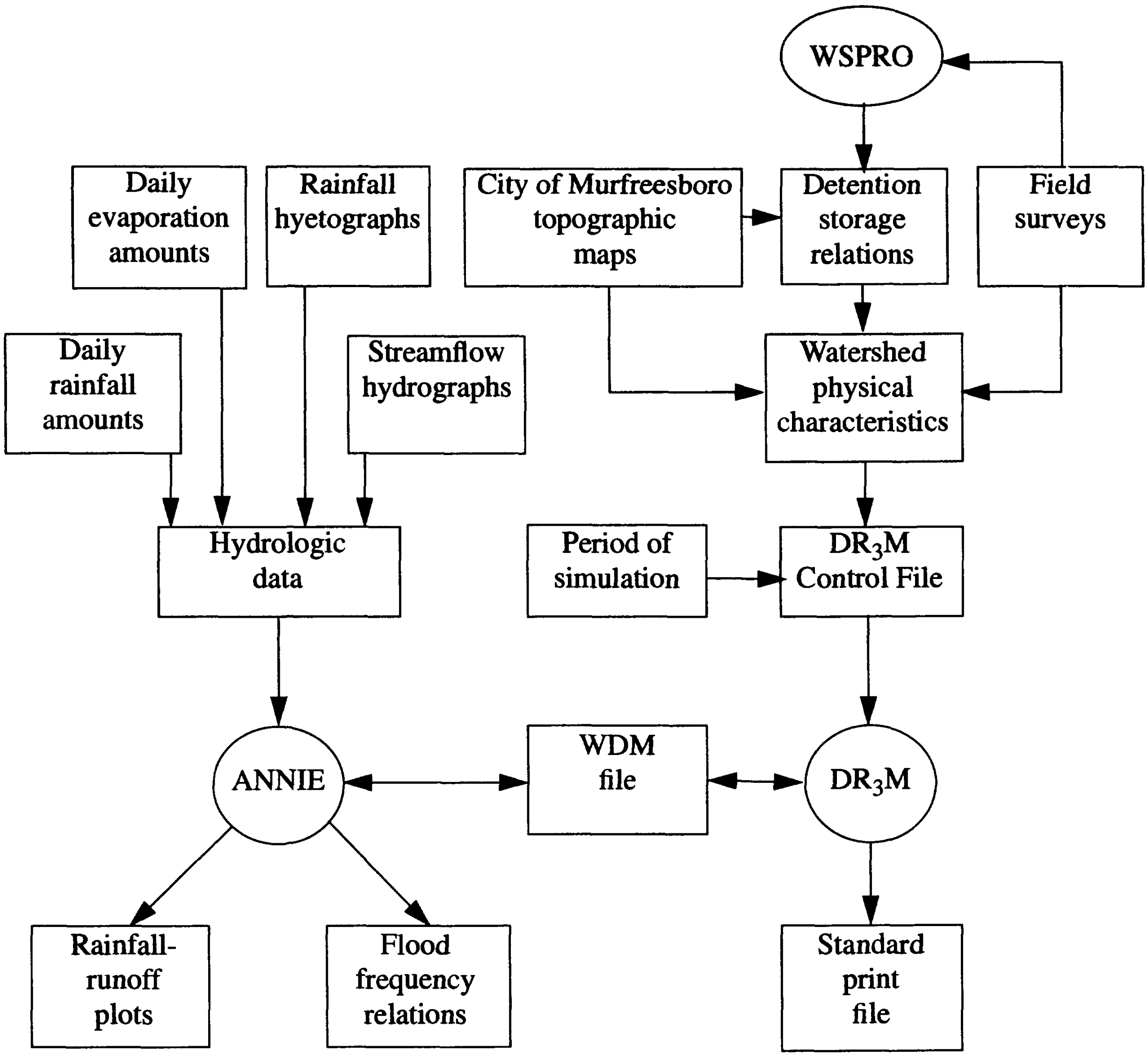

Figure 2. Flow chart of method of analysis.

[ANNIE, A computer program for interactive hydrologic analyses and data management; DR_3M, Distributed routing rainfall-runoff model; WDM, watershed data management; WSPRO, water-surface profile computation model]

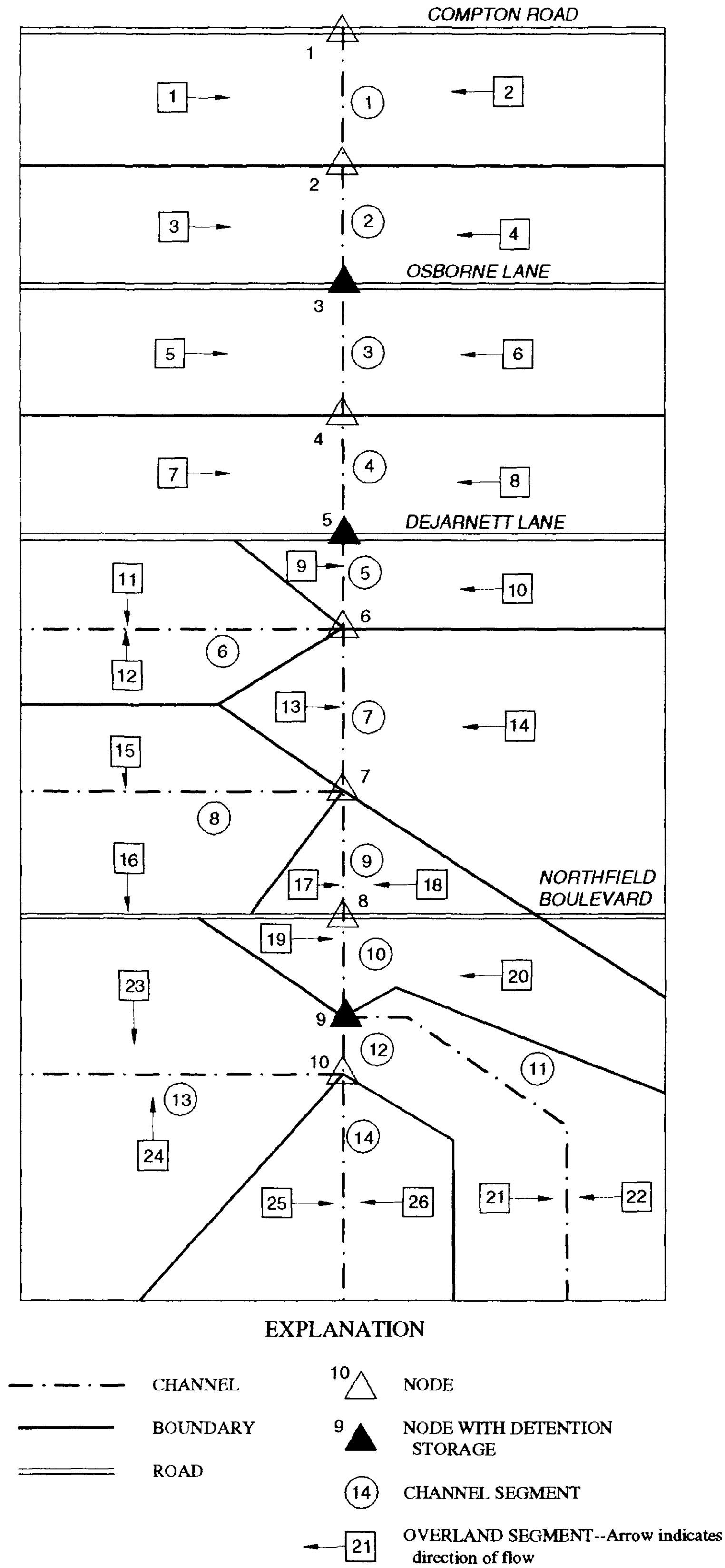

Figure 3. Schematic representation of Bear Branch watershed.

Table 2. Physical characteristics of overland segments

[mi^2, square miles; resistance parameters are discussed in the Distributed Routing Rainfall-Runoff Model user's manual (Alley and Smith, 1982); ft, feet; ft/ft, foot per foot]

Overland segment number	Area (mi^2)	Percent impervious area	Resistance parameters: Laminar flow coefficients	Resistance parameters: Turbulent flow Manning's n	Overland flow length (ft)	Average land slope (ft/ft)
1	0.090	2	4,000	0.03	1,932	0.020
2	.071	8	3,000	.03	1,519	.018
3	.163	2	4,000	.03	1,563	.019
4	.154	2	4,000	.03	1,485	.018
5	.123	2	4,000	.03	1,633	.021
6	.090	2	4,000	.03	1,195	.022
7	.103	2	4,000	.03	1,251	.022
8	.122	2	4,000	.03	1,483	.017
9	.012	2	4,000	.03	406	.022
10	.048	2	4,000	.03	1,565	.017
11	.115	4	4,000	.03	842	.018
12	.121	2	4,000	.03	891	.013
13	.137	2	4,000	.03	943	.021
14	.222	3	4,000	.03	1,528	.016
15	.144	3	4,000	.03	756	.014
16	.057	19	3,000	.03	302	.010
17	.004	10	3,000	.03	270	.010
18	.002	5	3,000	.03	170	.010
19	.020	5	3,000	.03	560	.012
20	.049	2	3,000	.03	1,360	.009
21	.166	22	3,000	.03	943	.006
22	.081	12	3,000	.03	460	.008
23	.061	24	4,000	.03	827	.007
24	.037	2	3,000	.03	503	.008
25	.034	8	4,000	.03	935	.008
26	.044	12	3,000	.03	1,236	.007

Table 3. Values for evaporation, soil-moisture, and rainfall infiltration parameters

[See Distributed Routing Rainfall-Runoff Model user's manual (Alley and Smith, 1982) for additional information]

Parameter code	Parameter value	Parameter definition
EVC	0.70	A coefficient to convert pan evaporation to potential evaporation. Typical value is 0.70.
RR	.90	Proportion of daily rainfall that infiltrates soil. Typical range is 0.70 to 0.95.
BMSN	3.1	Available soil water at field capacity, in inches. Typical range is 2 to 6.
KSAT	.06	Effective saturated value of hydraulic conductivity, in inches per hour. Typical range is 0.05 to 1.2.
RGF	10.8	Ratio of suction at the wetting front for soil moisture at wilting point to that at field capacity. Typical range is 5 to 20.
PSP	4.1	Suction at the wetting front for soil moisture at the field capacity, in inches. Typical range is 0.5 to 8.0.

Three sites where detention storage was a factor were simulated in Bear Branch watershed (fig. 3). The site upstream of Northfield Boulevard (node 9, fig. 3) represents the combined storage of a poorly formed, heavily vegetated channel with several ponds, sinks, and depressions. At this site, a storage constant of 1-hour (Alley and Smith, 1982, p. 15, 56) was used. This storage constant is used in the linear-storage routing equation S=KO, where S is the reservoir storage, O is outflow from the reservoir, and K is the storage constant. The storage constant of 1 hour was determined by trial and error during the calibration of the model. Detention storage at DeJarnett Lane (gage 2, fig. 1; node 5, fig. 3) and at Osborne Lane (gage 3, fig. 1; node 3, fig. 3) was used to account for ponding behind culverts. Outflow-storage relations (table 5) were developed for gages 2 and 3 using WSPRO (Shearman, 1990), 2-foot contour interval topographic maps and field surveys.

Hydrologic Data

DR_3M requires evaporation and rainfall information to perform streamflow simulations. Daily amounts of evaporation and rainfall are used by DR_3M to simulate antecedent soil-moisture conditions that control rainfall infiltration rates and the production of excess rainfall during simulated storms. Simulated storms are defined by 5-minute time-step rainfall hyetographs.

The model simulates streamflow, at 5-minute intervals, produced by storm runoff. These simulations do not include base flow. Observed streamflow data were used to check the accuracy of model-simulated streamflow.

During the 3 years of data collection, usable rainfall and runoff data were recorded for 21 storms. These 21 storms are referred to as the model calibration storms in this report. The model calibration storms include a storm occurring on February 3, 1990. This storm was the largest to occur in Murfreesboro during the period of this study (March 1989 through June 1992). Particular emphasis was placed on ensuring the calibrated computer model accurately simulated the magnitude and timing of runoff produced by the February 3, 1990 storm. Flood-frequency relations developed for Bear Branch during this study indicate that this particular storm has a recurrence interval of from 10 to 20 years.

Model Simulation of Storms During May 1989–June 1992

DR_3M was calibrated for present conditions in the Bear Branch watershed using hydrologic data collected from 21 storms during the period March 1989 through July 1992 (table 6). Model-simulated streamflow hydrographs are compared with observed hydrographs (from which base flow has been removed) to

Table 4. Physical characteristics of channel segments

[ft, feet; ft/ft, foot per foot; kinematic wave parameters are given for natural channels and are discussed in the Distributed Routing Rainfall-Runoff Model user's manual (Alley and Smith, 1982); side slopes are given for triangular channels as the ratio of feet horizontal to feet vertical; --, no data]

Channel segment number	Channel shape	Length (ft)	Bed slope (ft/ft)	Kinematic wave paramters a	Kinematic wave paramters m	Side slopes	Manning's roughness coefficient
1	natural	1,300	0.0041	0.58	1.20	--	--
2	natural	2,900	.0046	.55	1.20	--	--
3	natural	2,100	.0040	.53	1.19	--	--
4	natural	2,300	.0026	.50	1.18	--	--
5	natural	850	.0011	.43	1.18	--	--
6	triangular	3,800	.0075	--	--	2.0	0.12
7	natural	4,050	.0050	.57	1.17	--	--
8	triangular	5,300	.0037	--	--	2.0	.11
9	natural	400	.0045	.60	1.17	--	--
10	triangular	1,000	.0040	--	--	3.0	.10
11	triangular	4,900	.0035	--	--	2.0	.10
12	triangular	300	.0040	--	--	3.0	.10
13	triangular	2,050	.0046	--	--	2.0	.05
14	triangular	1,000	.0060	--	--	2.0	.05

Table 5. Outflow-storage relations for culverts at gages 2 and 3

[Water-surface elevation is given in feet (ft) above sea level at upstream side of culvert; ft^3/s, cubic feet per second; ft^3/s-h, cubic feet per second-hours; gage 2, DeJarnett Lane; gage 3, Osborne Lane; gage locations are shown on figure 1]

Gage 2			Gage 3		
Water-surface elevation (ft)	Outflow (ft^3/s)	Storage (ft^3/s-h)	Water-surface elevation (ft)	Outflow (ft^3/s)	Storage (ft^3/s-h)
572.0	0	0	556.6	0	0
572.6	30	3	557.0	10	1
574.3	150	85	560.0	70	6
575.0	200	130	561.0	110	13
575.7	250	180	562.0	160	25
576.8	350	325	563.0	210	47
577.8	450	450	*565.0	300	135
580.0	600	900	565.5	1,000	175
581.0	750	1,400	566.0	1,250	220
584.0	900	2,800	567.0	1,500	350

* Road overflow begins at this elevation.

ensure that the model is accurately simulating streamflow (table 7). Rainfall-runoff plots for several of the calibration storms are provided in the Supplemental Information section of this report.

Rainfall data were collected at two separate locations (fig. 1) for the calibration storms. These data define variation in rainfall amount and intensity within the watershed (table 6) and allow for more accurate calibration of DR_3M.

Calibrating a rainfall-runoff model requires a systematic adjustment of model parameters that control excess rainfall production and streamflow routing. Specifically, excess rainfall production is adjusted by optimizing soil-moisture accounting and infiltration parameters using a modified Rosenbrock direct-search technique (Alley and Lumb, 1982, p.17, 31). Streamflow routing is controlled by adjusting the kinematic wave parameters as necessary to reproduce similar timing and peaks of simulated and observed streamflow hydrographs (Alley and Lumb, 1982, p. 31).

Simulated peak discharge and detention-storage information is provided for culverts at DeJarnett Lane (gage 2, fig. 1) and Osborne Lane (gage 3, fig. 1) (table 8). Information collected from storm no. 10, which occurred on February 3, 1990, was used to check model simulation of detention storage at these locations. During this storm, storage volumes of 461 cubic feet per second-hour (ft^3/s-h) and 153 ft^3/s-h were simulated at DeJarnett Lane and Osborne Lane, respectively. Outflow-storage relations developed for these locations (table 5) indicate that a storage volume of 461 ft^3/s-h at DeJarnett Lane produces a water-surface elevation of approximately 578.0 feet above sea level and a storage volume of 153 ft^3/s-h at Osborne Lane produces a water-surface elevation of about 565.3 feet above sea level. These simulated water-surface elevations agree with elevations recorded by crest-stage gages during this storm.

Reliability of Model Calibration

The differences between observed and simulated peak discharges and runoff volumes for the 21 calibration storms (table 7) can be used as an indication of the accuracy of the calibrated model. Graphical presentations of observed and model simulated peak discharges at Northfield Boulevard (gage 1) and Compton Road (gage 4) are provided as figures 4 and 5. The standard errors of estimate for peak discharge at gage 1 and gage 4 are 41.4 and 92.2 percent, respectively. Graphical presentations of observed and model simulated runoff volumes at gage 1 and gage 4 are provided as figures 6 and 7. The standard error of estimate for runoff volume at gages 1 and 4 are 53.5 and 97.6 percent, respectively.

Errors can be attributed to several sources. A large part of random model error is probably due to errors in measuring the rainfall over the watershed and the runoff in the creek. Other errors are probably attributable to the inability of the model algorithms to

Table 6. Period of simulation, base flow, and rainfall amount at gages 1 and 4 for calibration storms

[Time is given in hours and minutes on a 24-hour time scale; ft^3/s, cubic feet per second; in., inches; p, plot provided in Supplemental Information section; gage 1, Northfield Boulevard; gage 4, Compton Road; gage locations shown on figure 1; e, estimated]

Storm number	Period of simulation: Begin Date	Begin Time	End Date	End Time	Base flow, (ft^3/s) Gage 1	Base flow Gage 4	Rainfall amount, (in.) Gage 1	Rainfall amount Gage 4
1	890520	0155	890520	2400	0	0	1.99	2.60
2 p	890601	1405	800602	2400	0	0	2.34	3.38
3	890612	0550	890616	2400	4	9	3.44	3.16
4	890618	2055	890620	2400	5	13	2.23	1.90
5	890701	0620	890701	2400	5	14	2.31	4.61
6	890711	0955	890711	2400	2	0	1.72	2.24
7	890915	0720	890915	2400	0	0	2.20	1.99
8	890930	0120	891001	2400	0	0	4.38	3.74
9	891016	1210	891017	2400	2	4	1.30	1.78
10 p	900203	0135	900203	2400	0	0	4.85	5.57
11	900209	1625	900210	2400	4	10	1.09	1.63
12	901220	1050	901222	2400	4	11	5.11	5.18
13	901230	0005	901230	2400	6	16	.93	1.03
14	910217	0400	910219	2400	0	3	4.67	4.82
15	910322	0525	910323	2400	0	3	1.85	1.65
16	910327	1805	910330	2400	3	8	2.42	2.20
17	911130	0315	911203	0630	0	0	6.02	6.36
18	920102	0410	920103	2400	3	9	2.06	1.57
19	920223	0145	920226	2400	0	0	2.35	2.35
20	920309	1730	920310	2400	0	0	1.87	2.00
21	920618	0635	920619	0700	0	1	2.82	2.61e

Table 7. Observed and simulated peak discharge and runoff volume at gages 1 and 4 for calibration storms

[ft^3/s, cubic feet per second; in., inches; Obsv, observed; Simul, simulated; gage 1, Northfield Boulevard; gage 4, Compton Road; gage locations shown on figure 1]

Storm number	Gage 1: Peak discharge (ft^3/s) *Obsv	Peak discharge Simul	Runoff volume (in.) *Obsv	Runoff volume Simul	Gage 4: Peak discharge (ft^3/s) *Obsv	Peak discharge Simul	Runoff volume (in.) *Obsv	Runoff volume Simul
1	27	56	0.26	0.70	62	103	0.26	0.52
2	106	120	1.21	1.22	388	400	.88	.99
3	90	37	2.28	1.07	91	145	1.59	.86
4	35	14	1.07	.67	63	48	.82	.54
5	82	119	1.40	1.74	367	363	1.82	1.54
6	89	61	1.30	.80	149	149	.79	.66
7	43	33	.64	.54	17	75	.08	.40
8	186	83	4.18	2.03	335	266	2.41	1.87
9	19	22	.22	.40	23	51	.19	.28
10	195	208	3.57	3.35	871	847	2.67	3.09
11	22	11	.47	.41	38	33	.35	.30
12	48	46	3.03	2.06	115	122	1.95	1.73
13	16	12	.25	.19	42	32	.22	.11
14	50	31	3.03	1.81	99	95	2.00	1.51
15	59	67	.64	.87	77	192	.55	.81
16	19	16	.60	.70	44	61	.62	.55
17	79	70	4.91	2.64	312	187	3.06	2.27
18	23	10	.63	.43	57	39	.72	.30
19	14	12	.34	.56	40	27	.93	.39
20	33	31	.54	.63	66	91	.50	.50
21	135	127	1.41	1.27	320	278	1.34	1.14

* Base flow removed.

Table 8. Simulated peak discharge and detention-storage volume at gages 2 and 3 for calibration storms

[ft^3/s, cubic feet per second; ft^3/s-h, cubic feet per second-hours; US, upstream of culvert; DS, downstream of culvert; detention-storage volume is upstream of culvert; gage 2, DeJarnett Lane; gage 3, Osborne Lane; gage locations shown on figure 1]

Storm number	Gage 2 Peak discharge (ft^3/s) US	Gage 2 Peak discharge (ft^3/s) DS	Gage 2 Detention-storage volume (ft^3/s-h)	Gage 3 Peak discharge (ft^3/s) US	Gage 3 Peak discharge (ft^3/s) DS	Gage 3 Detention-storage volume (ft^3/s-h)
1	101	88	42	96	95	10
2	178	148	84	221	194	40
3	134	110	58	132	130	35
4	34	33	5	40	40	4
5	244	210	139	243	229	65
6	149	128	70	141	139	20
7	70	60	24	68	68	6
8	347	255	188	280	255	91
9	39	37	8	44	44	4
10	544	454	461	624	623	153
11	19	19	2	24	24	2
12	107	98	49	111	111	13
13	21	21	2	26	26	2
14	71	67	29	80	80	8
15	200	168	101	186	181	34
16	43	42	11	52	52	5
17	170	151	86	167	164	27
18	28	28	3	34	34	3
19	21	21	2	24	24	2
20	76	70	31	81	81	8
21	452	277	219	294	269	105

accurately imitate nature. DR_3M is most accurately applied to small, highly urban, non-karst drainage basins.

SIMULATION OF ANNUAL PEAK DISCHARGE AND VOLUME

Historical records of evaporation and rainfall were used by DR_3M to synthesize long-term record of peak streamflows and storage volumes at gaged locations on Bear Branch (table 9). Daily values of evaporation supplied to the model were obtained from evaporation records published by the NWS for Tennessee for the period 1901-92. Historical records of rainfall were obtained from the NWS gage at Nashville for the period 1901-70. Five-minute time-step rainfall hyetographs for significant storms occurring during this period were developed from rain gage strip-chart records. Additionally, large storms that occurred at Murfreesboro in September 1979, September 1986, and February 1990 were simulated by DR_3M.

Historical simulations for the periods 1901-70, 1979, and 1986 were made using a single basin-wide rainfall hyetograph, because distributed information was not available for these storms. The historical simulation for the 1990 storm used the same distributed rainfall hyetograph, incorporating information from rain gages at Northfield Boulevard (gage 1) and Compton Road (gage 4), that was used during model calibration for this storm. Consequently, streamflow hydrographs from the historical simulation of this storm are identical to results obtained during model calibration. A rainfall-runoff plot for the storm occurring on September 3-4, 1986 (Supplemental Information section) is provided to illustrate the dynamic nature of a flood hydrograph on Bear Branch.

FLOOD-FREQUENCY CHARACTERISTICS

Flood-frequency relations were developed for Bear Branch using simulated annual peak flows for the periods 1901-70, 1979, 1986, and 1990, and using statistical methods described in Bulletin 17B of the Interagency Committee on Water Data of the Water

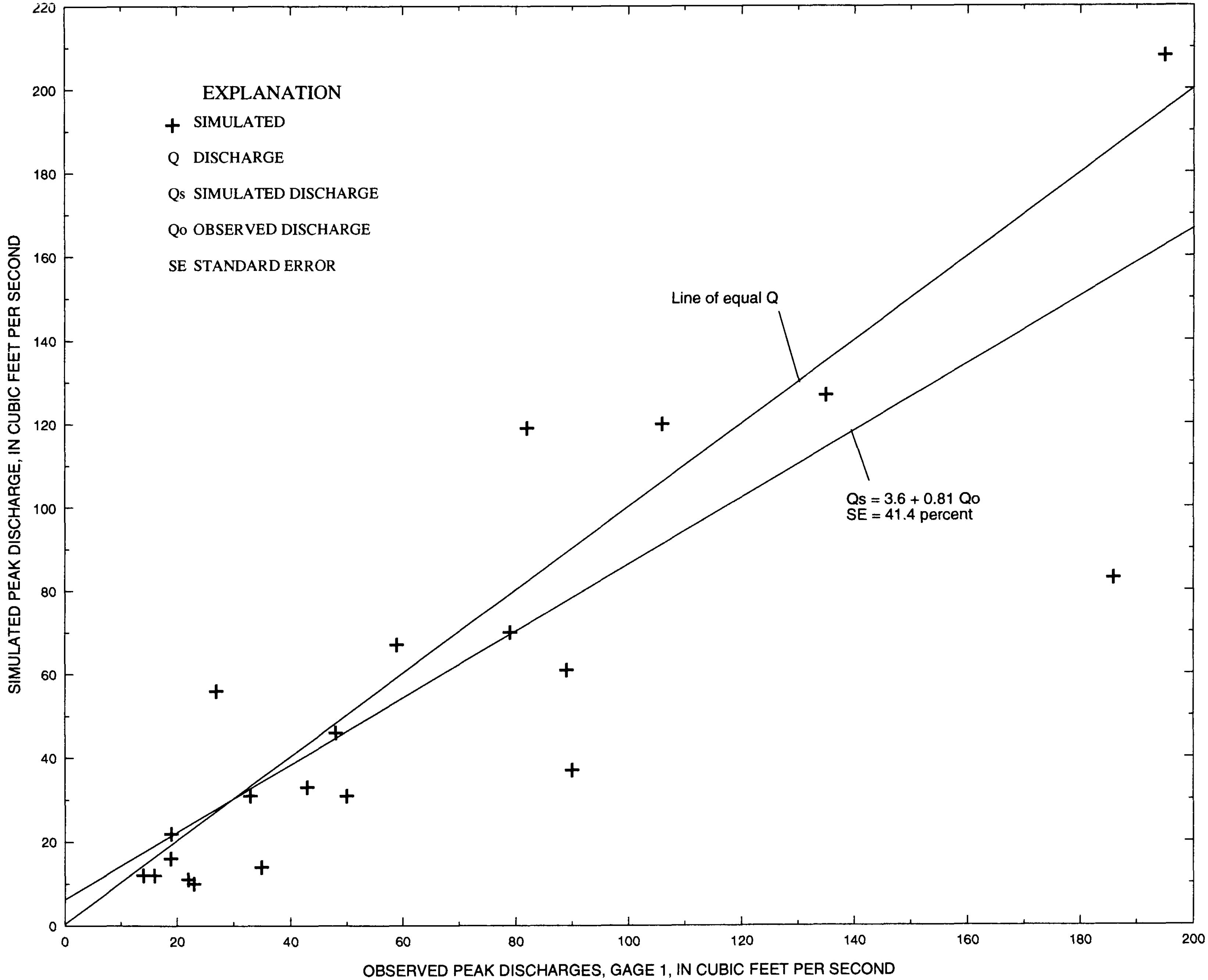

Figure 4. Observed and simulated peak discharge at gage 1 (Northfield Boulevard).

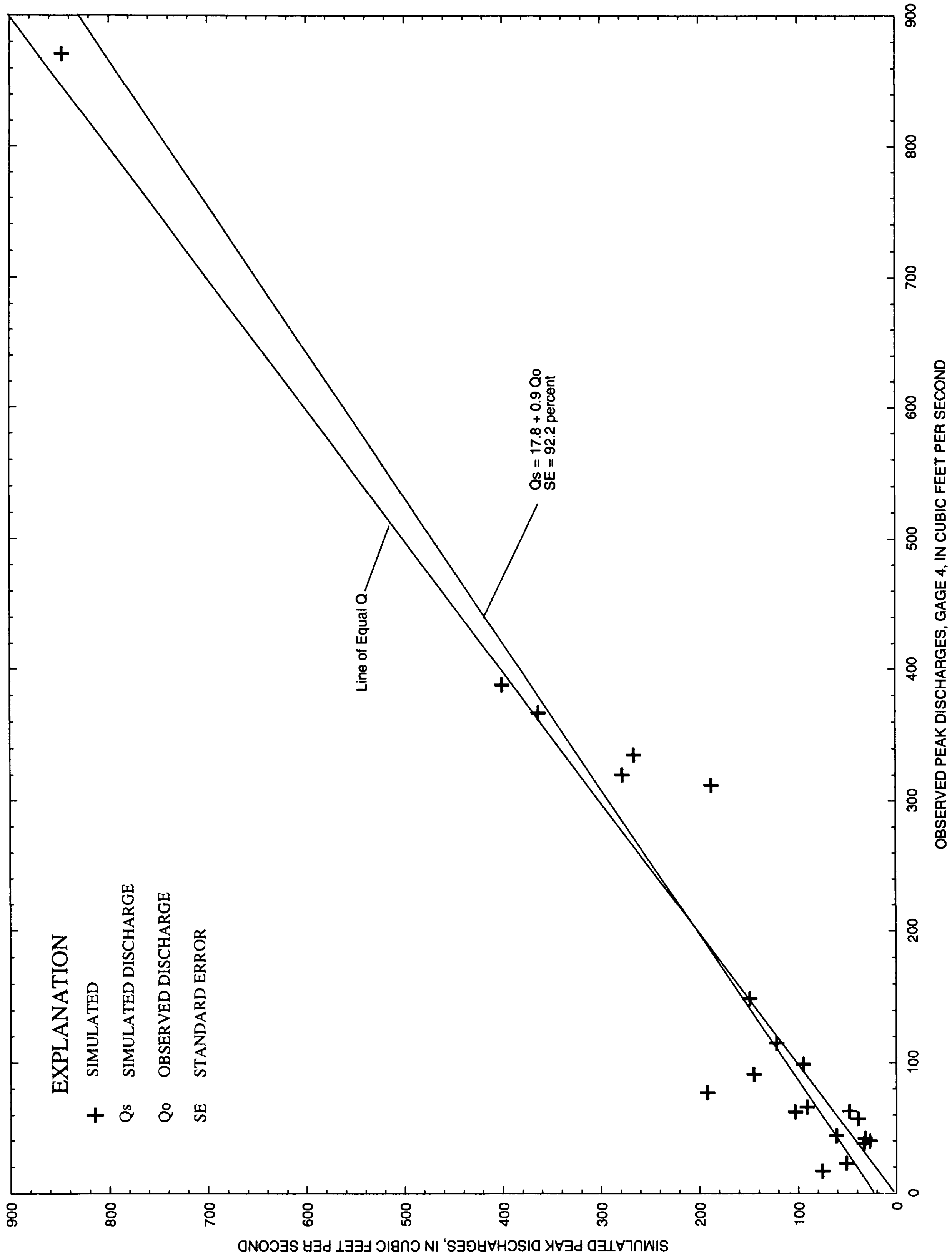

Figure 5. Observed and simulated peak discharge at gage 4 (Compton Road).

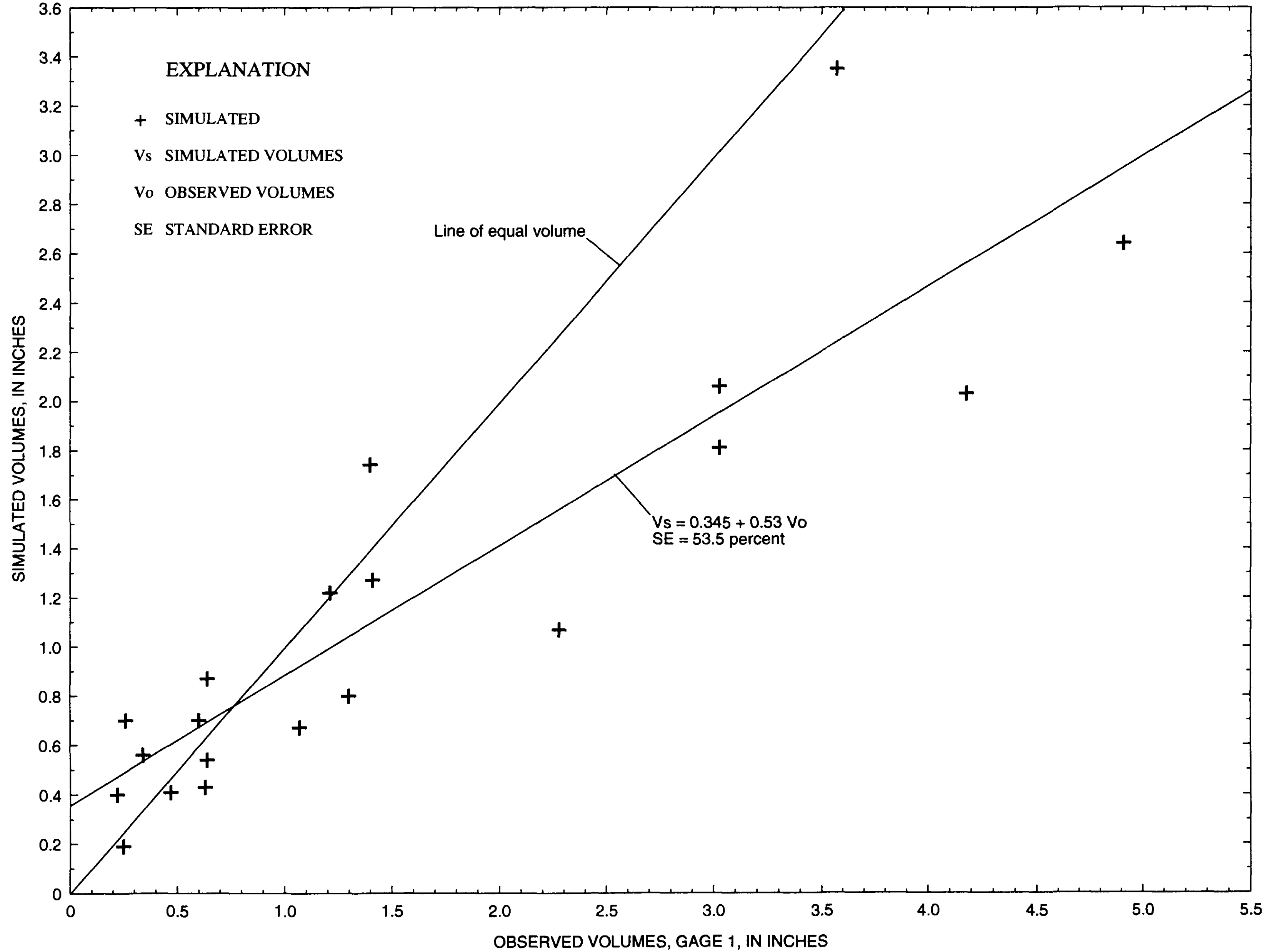

Figure 6. Observed and simulated runoff volumes at gage 1 (Northfield Boulevard).

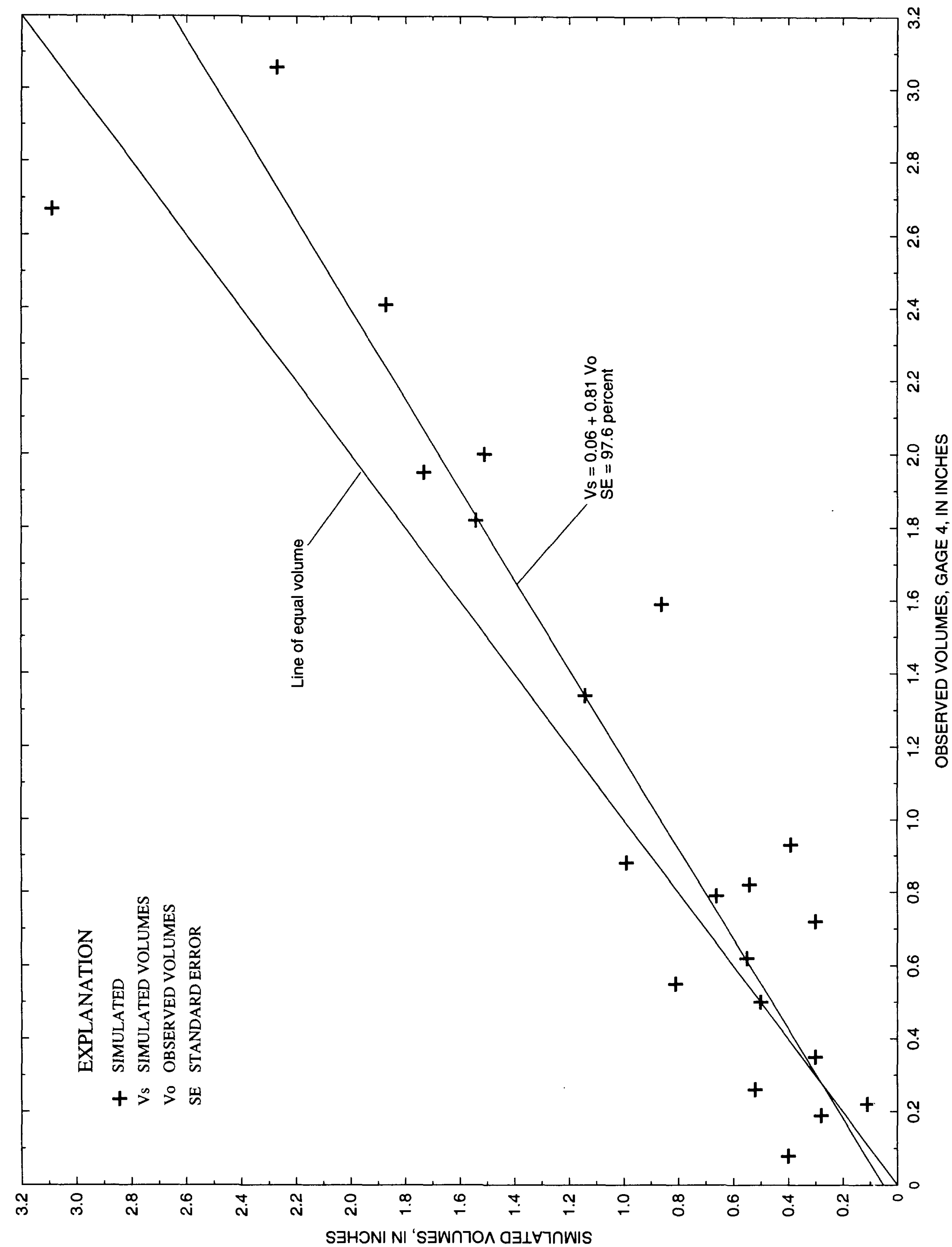

Figure 7. Observed and simulated runoff volumes at gage 4 (Compton Road).

Table 9. Simulated annual peak discharge at gages 1 through 4 and detention-storage volume at gages 2 and 3 for water years 1901-70, 1979, 1986, and 1990

[Water year 1901, October 1, 1900 to September 30, 1901; in., inches; ft³/s, cubic feet per second; ft³/s-h, cubic feet per second-hours; US, upstream of culvert; DS, downstream of culvert; detention-storage volume is upstream of the culvert; a plot of the 1986 storm is provided in the Supplemental Information section; gage 1, Northfield Boulevard; gage 2, DeJarnett Lane; gage 3, Osborne Lane; gage 4, Compton Road; gage locations shown on figure 1]

Water year	Rain-fall amount (in.)	Gage 1 Peak discharge (ft³/s)	Gage 2 Peak discharge (ft³/s)		Gage 2 Detention-storage volume (ft³/s-h)	Gage 3 Peak discharge (ft³/s)		Gage 3 Detention-storage volume (ft³/s-h)	Gage 4 Peak discharge (ft³/s)
			US	DS		US	DS		
1901	1.30	51	131	116	62	129	128	17	138
1902	3.65	70	186	169	102	220	216	53	280
1903	2.55	198	595	374	355	401	400	141	460
1904	1.65	70	174	157	91	182	177	33	193
1905	3.35	82	241	174	107	194	190	38	217
1906	2.45	136	408	286	232	313	291	126	330
1907	4.15	87	242	213	143	251	238	75	291
1908	1.25	34	92	86	41	97	97	11	106
1909	3.05	182	570	335	303	359	355	138	441
1910	3.30	157	512	300	253	320	296	131	375
1911	3.80	159	436	309	266	330	327	137	374
1912	4.00	158	479	357	334	513	511	147	632
1913	2.00	123	340	256	188	272	253	89	272
1914	2.65	121	335	247	177	263	245	81	275
1915	2.65	36	95	90	44	112	111	13	132
1916	4.05	76	204	189	120	226	221	58	283
1917	1.55	69	167	152	87	173	170	29	183
1918	2.95	75	204	188	119	248	234	70	288
1919	4.80	64	170	157	91	189	187	37	224
1920	3.40	176	512	335	303	397	397	141	441
1921	3.75	198	562	363	341	385	385	140	440
1922	2.60	87	234	210	140	249	239	75	279
1923	2.10	121	333	256	188	282	260	96	271
1924	2.30	158	532	313	271	333	322	136	412
1925	3.45	41	97	88	43	102	102	12	115
1926	2.00	79	242	182	114	199	193	40	224
1927	5.55	93	268	245	175	326	318	136	394
1928	4.25	205	665	461	483	673	669	156	788
1929	3.75	100	250	206	136	275	246	83	301
1930	3.94	124	322	250	180	269	248	84	258
1931	1.05	11	27	27	3	36	36	3	44
1932	1.80	82	208	181	113	202	196	41	211
1933	3.10	117	322	260	195	286	270	105	286
1934	3.10	80	220	178	111	197	192	39	204
1935	2.35	77	204	193	124	238	228	64	266
1936	2.80	165	464	338	308	364	364	139	382
1937	2.30	97	278	231	161	257	245	81	263
1938	1.60	40	108	102	52	118	118	15	137
1939	2.20	62	166	153	88	183	180	34	206
1940	1.60	45	125	112	59	126	125	16	135
1941	1.15	42	104	94	47	103	102	12	109
1942	2.50	110	294	222	152	239	225	62	242
1943	2.45	151	519	303	257	352	330	137	422
1944	2.90	66	180	159	94	183	180	34	195

Table 9. Simulated annual peak discharge at gages 1 through 4 and detention-storage volume at gages 2 and 3 for water years 1901-70, 1979, 1986, and 1990—Continued

Water year	Rainfall amount (in.)	Gage 1 Peak discharge (ft^3/s)	Gage 2 Peak discharge (ft^3/s) US	Gage 2 Peak discharge (ft^3/s) DS	Gage 2 Detention-storage volume (ft^3/s-h)	Gage 3 Peak discharge (ft^3/s) US	Gage 3 Peak discharge (ft^3/s) DS	Gage 3 Detention-storage volume (ft^3/s-h)	Gage 4 Peak discharge (ft^3/s)
1945	2.40	84	242	192	123	213	207	46	233
1946	6.20	122	381	293	243	398	387	140	444
1947	2.00	89	239	206	243	236	224	60	239
1948	2.65	107	283	244	174	325	287	122	330
1949	2.40	132	399	268	206	289	268	104	313
1950	3.35	222	670	387	372	414	413	141	507
1951	2.30	116	289	221	151	235	222	58	230
1952	4.00	121	386	256	189	294	257	93	411
1953	1.25	38	99	90	44	101	101	11	109
1954	3.30	58	144	132	73	148	147	22	159
1955	4.70	157	464	330	296	383	382	140	441
1956	3.80	100	284	235	164	282	268	103	320
1957	2.85	204	642	372	353	393	393	140	488
1958	3.05	114	314	227	157	241	227	63	242
1959	1.95	16	36	34	6	40	40	3	45
1960	4.45	212	604	409	398	442	441	143	536
1961	1.55	19	51	47	15	54	54	5	60
1962	5.00	328	985	540	721	659	653	155	793
1963	5.15	134	377	281	225	315	310	136	345
1964	1.75	35	95	88	43	102	102	12	114
1965	1.80	101	253	211	141	232	220	57	233
1966	1.80	21	34	32	4	37	37	3	41
1967	4.10	61	152	136	76	152	150	23	163
1968	2.15	53	143	125	68	142	141	20	154
1969	3.65	67	177	171	104	221	215	52	272
1970	3.20	100	268	218	148	257	243	79	280
1979	6.61	314	1,005	577	831	839	835	166	1,032
1986	8.28	312	979	588	863	968	964	173	1,220
1990	5.21	208	544	454	461	624	623	153	847

Resources Council (WRC) (U.S. Geological Survey, 1982). Flood-frequency estimates (tables 10 and 11) were developed at the culvert at Northfield Boulevard (gage 1), upstream of the culvert at Compton Road (gage 4), and upstream and downstream of the culverts at DeJarnett Lane (gage 2) and Osborne Lane (gage 3). Ninety-five percent confidence limits are provided for the estimates.

Flood discharges for the 100-year recurrence-interval storm were estimated as 350 cubic feet per second (ft^3/s) at Northfield Boulevard, 1,100 ft^3/s upstream of DeJarnett Lane, 610 ft^3/s downstream of DeJarnett Lane, 800 ft^3/s upstream of Osborne Lane, 790 ft^3/s downstream of Osborne Lane, and 1,000 ft^3/s at Compton Road.

DETENTION-STORAGE CHARACTERISTICS

Results from the historical simulation and flood-frequency analysis were used to characterize detention storage at existing culverts at Compton Road and DeJarnett Lane. Simulated peak values of detention-storage volumes for the period 1901-90 at DeJarnett Lane and Osborne Lane were 850 and 170 ft^3/s-h, respectively. The water-surface elevations corresponding with these peak volumes are about 580.0 and 565.5 feet above sea level, respectively, computed from outflow-storage relations at DeJarnett Lane (gage 2) and Osborne Lane (gage 3) (table 5).

Comparison of upstream to downstream estimates for a given annual exceedence probability

Table 10. Annual exceedance probability for flood discharge at gages 1 and 4

[ft^3/s, cubic feet per second; estimates obtained using methods recommended by the Water Resources Council, 1982; pct, percent; --, insufficient data to estimate; gage 1, Northfield Boulevard; gage 4, Compton Road; gage locations shown on figure 1]

Annual exceedance probability	Gage 1			Gage 4		
	Flood estimate (ft^3/s)	95-pct confidence limits		Flood estimate (ft^3/s)	95-pct confidence limits	
		Lower	Upper		Lower	Upper
0.995	--	--	--	37	26	48
.990	--	--	--	46	33	58
.950	30	24	36	81	64	97
.900	39	33	46	110	88	130
.800	55	47	63	150	130	170
.500	96	85	110	270	240	310
.200	160	140	190	460	400	540
.100	210	180	250	590	500	710
.040	260	220	330	760	640	960
.020	310	250	390	890	730	1,100
.010	350	290	450	1,000	830	1,300
.005	390	320	510	1,100	920	1,500
.002	450	360	590	1,300	1,000	1,800

Table 11. Annual exceedance probability for flood discharge at gages 2 and 3

[ft^3/s, cubic feet per second; estimates obtained using methods recommended by the Water Resources Council, 1982; pct, percent; --, insufficient data to estimate; gage 2, DeJarnett Lane; gage 3, Osborne Lane; gage locations shown on figure 1]

Annual exceedance probability	Gage 2			Gage 3		
	Flood estimate (ft^3/s)	95-pct confidence limits		Flood estimate (ft^3/s)	95-pct confidence limits	
		Lower	Upper		Lower	Upper
			Upstream of culvert			
0.995	--	--	--	32	23	42
.990	--	--	--	40	30	51
.950	72	56	88	73	58	88
.900	99	80	120	98	81	120
.800	140	120	170	140	120	160
.500	270	230	310	250	220	280
.200	470	410	570	400	350	470
.100	620	520	760	500	440	600
.040	820	670	1,000	630	530	770
.020	960	780	1,200	720	600	900
.010	1,100	890	1,500	800	660	1,000
.005	1,300	1,000	1,700	880	720	1,100
.002	1,500	1,100	2,000	980	800	1,300
			Downstream of culvert			
.995	--	--	--	33	23	42
.990	--	--	--	41	30	52
.950	69	56	81	72	58	87
.900	91	76	110	96	80	110
.800	130	110	140	130	110	150
.500	210	190	240	240	210	270
.200	340	300	390	390	340	450
.100	410	360	490	490	420	590
.040	500	430	600	620	520	760
.020	560	470	680	710	590	880
.010	610	520	760	790	650	1,000
.005	660	550	830	880	720	1,100
.002	720	600	920	980	800	1,300

(table 11) shows that detention storage upstream of DeJarnett Lane (gage 2) significantly reduces downstream flood peaks. Detention storage upstream of Osborne Lane (gage 3), however, has almost no effect, particularly for events with lower annual exceedence probability (longer recurrence interval). Outflow-storage relations (table 5) developed for the culverts at DeJarnett Lane and Osborne Lane clearly show that for large storms, where inflow to a culvert exceeds outflow, the available storage volume upstream of DeJarnett Lane can readily hold excess runoff without allowing DeJarnett Lane to overtop, whereas, the small storage volume upstream of Osborne Lane quickly fills allowing excess runoff to flow over Osborne Lane.

SUMMARY

The Distributed Routing Rainfall-Runoff Model, DR$_3$M, was applied to a 2.27-square-mile part of Bear Branch watershed in northern Murfreesboro, Tennessee. Kinematic wave theory was used to route excess rainfall through a system of 26 overland segments and 14 channel segments. On-channel detention storage was simulated at three locations: the heavily vegetated lowland upstream of Northfield Boulevard, the culvert at DeJarnett Lane, and the culvert at Osborne Lane.

DR$_3$M was calibrated for present conditions with observed rainfall and streamflow data collected at the watershed for 21 storms occurring during the period May 1989 through June 1992. The standard errors of estimate for peak discharge at Northfield Boulevard (gage 1) and Compton Road (gage 4) are 41.4 and 92.2 percent, respectively. The standard error of estimate for runoff volume at gages 1 and 4 are 53.5 and 97.6 percent, respectively.

Errors can be attributed to several sources. A large part of random model error is probably due to errors in measuring the rainfall over the watershed and the runoff in the creek. Other errors are probably attributable to the inability of the model algorithms to accurately imitate nature. DR$_3$M is most accurately applied to small, highly urban, non-karst drainage basins.

The calibrated model was used to simulate streamflow produced by the largest storm from each year during the periods 1901-70, 1979, 1986, and 1990. Flood-frequency relations were developed from these simulations, using methods developed by the Water Resources Council (U.S. Geological Survey, 1982), at Northfield Boulevard, upstream and downstream of DeJarnett Lane, upstream and downstream of Osborne Lane, and upstream of Compton Road. Flood discharges for the 100-year recurrence-interval storm were estimated as 350 (ft^3/s) at Northfield Boulevard, 1,100 ft^3/s upstream of DeJarnett Lane, 610 ft^3/s downstream of DeJarnett Lane, 800 ft^3/s upstream of Osborne Lane, 790 ft^3/s downstream of Osborne Lane, and 1,000 ft^3/s at Compton Road. Detention storage upstream of DeJarnett Lane significantly reduces downstream flood peaks, whereas, detention storage upstream of Osborne Lane has almost no effect.

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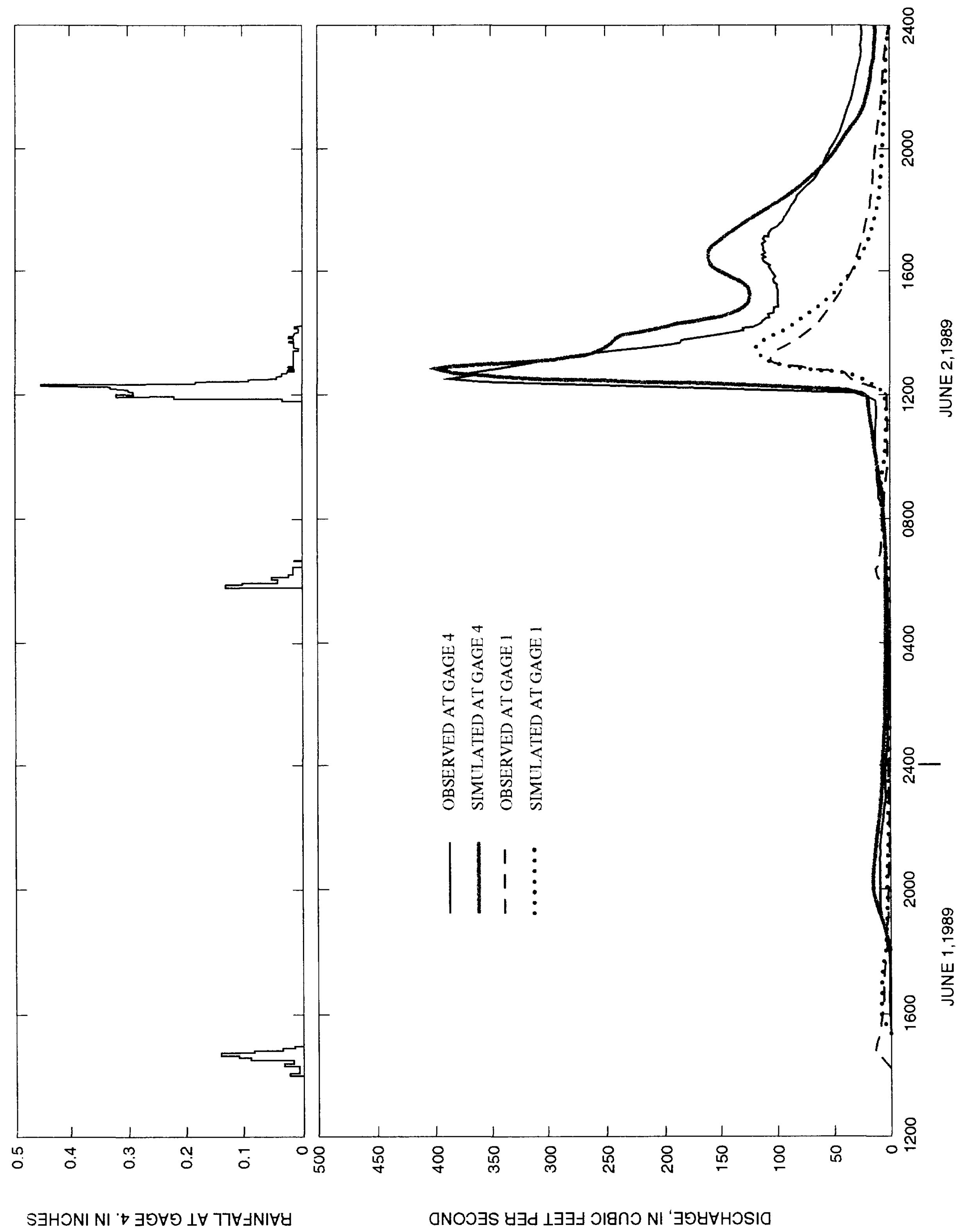
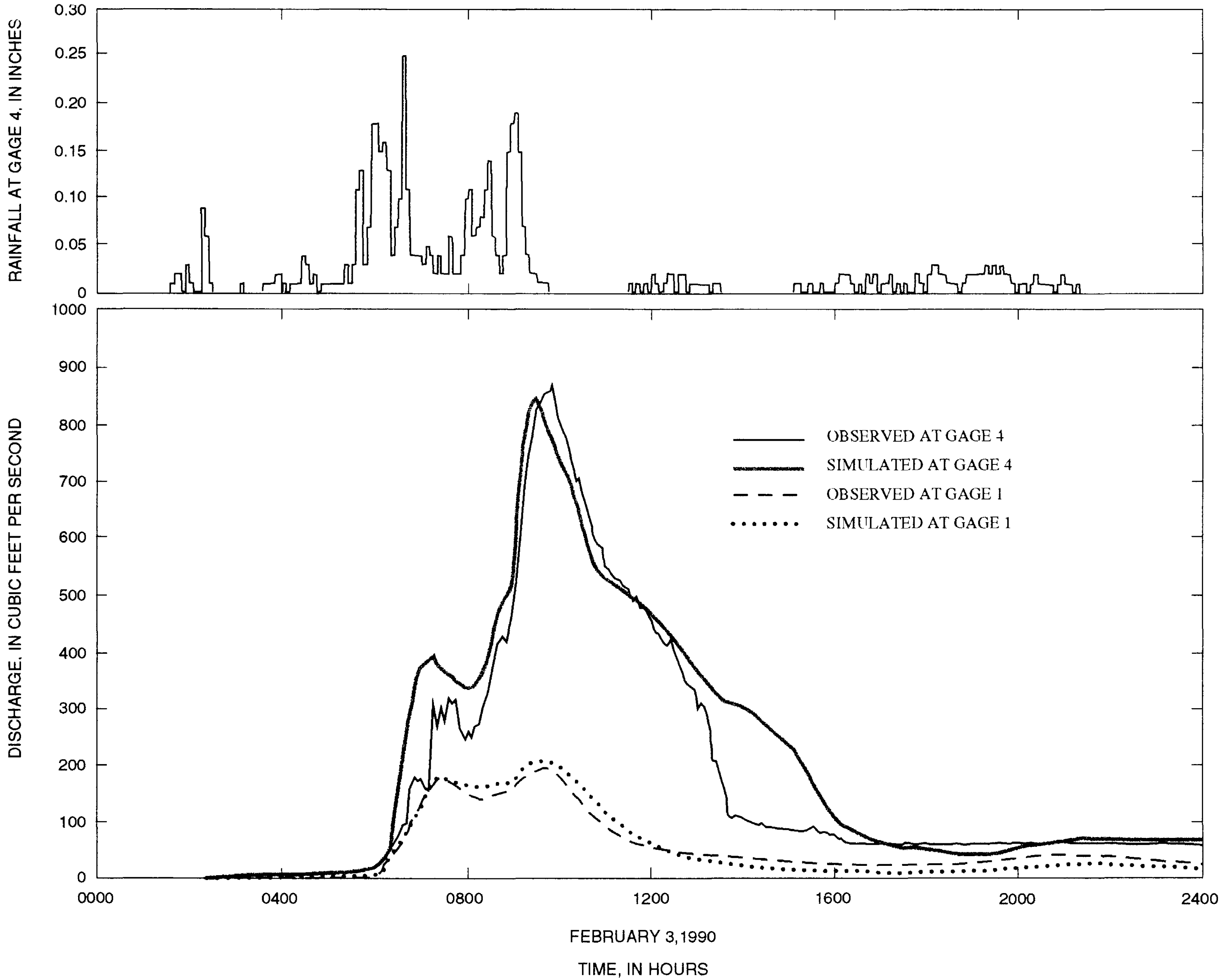
Supplemental Information

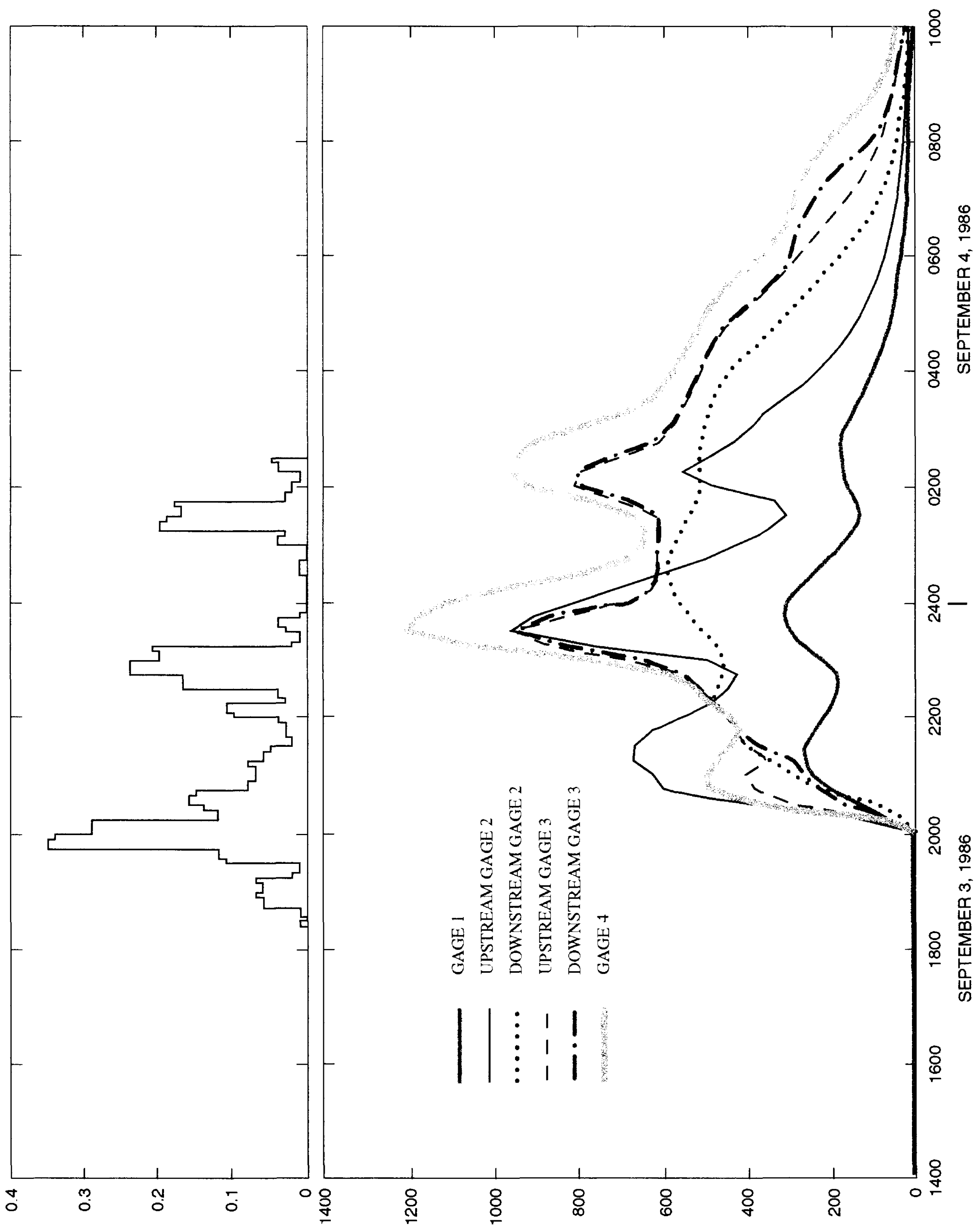
Selected rainfall-runoff plots from the model calibration
and simulation at Bear Branch watershed



Calibration plot for June 1-2, 1989 (storm 2) for gages 1 and 4. (See figure 1 and table 1 for gage location information.)



Calibration plot for February 3, 1990 (storm 10) for gages 1 and 4. (See figure 1 and table 1 for gage location information.)



Hydrograph simulation for September 3-4, 1986 for gages 1-4. (See figure 1 and table 1 for gage location information.)